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1.0 INTRODUCTION AND GENERAL DESCRIPTION OF PLANT

1.1 INTRODUCTION

This Final Safety Analysis Report (FSAR) complies with the "Standard Format and Content of Safety Analysis Reports," (Rev 1) issued by the Atomic Energy Commission (AEC) in October 1972. Conformance with applicable Regulatory Guides is discussed in appendix A to the FSAR.

A discussion of the format of the FSAR is presented in subsection 1.1.8, Organization of Contents.

1.1.1 LICENSE REQUESTED

This FSAR was originally submitted in support of the application by the Georgia Power Company (GPC), the Ogelthorpe Power Corporation (OPC), the Municipal Electric Authority of Georgia, and the city of Dalton, Georgia, for a facility operating license for the Edwin I. Hatch Nuclear Plant-Unit 2 (HNP-2) at a core thermal power level of 2436 MWt, the power level equivalent to 100% of the rated steam flow. (The design steam flow is 105% of the rated steam flow and occurs at a core thermal power level of ~ 2537 MWt.) Pursuant to an application dated September 18, 1992, the NRC issued an operating license amendment on March 17, 1997, effective March 22, 1997, designating Southern Nuclear Operating Company (SNC) as the exclusive operating licensee of HNP. SNC has no ownership interest in HNP. The application was made under section 103(b) of the Atomic Energy Act of 1954, as amended, and the regulations of the Nuclear Regulatory Commission (NRC) set forth in Title 10 Code of Federal Regulations (CFR) Part 50.

In Amendment 155 to the Technical Specifications, the HNP-2 facility operating license NPF-5 was revised to increase the maximum power level from 2558 MWt to 2763 MWt. In Amendment 214 to the Technical Specifications, the HNP-1 operating license was also revised to 2763 MWt. In Amendment 180 to the Technical Specifications, the HNP-2 operating license was revised to increase the maximum power level from 2763 MWt to 2804 MWt. In Amendment 238 to the Technical Specifications, the HNP-1 operating license to the Technical Specifications was also revised to 2804 MWt.

1.1.2 NUMBER OF PLANT UNITS

HNP-2 is located adjacent to the HNP-1. The application for an operating license for HNP-1 was submitted on NRC Docket No. 50-321. HNP-1 operating license no. DPR-57 was granted August 6, 1974. The application for HNP-2 was made separately on NRC Docket No. 50-366. The HNP-2 operating license no. NPF-5 was granted on June 13, 1978. Renewed operating license no. NPF-5 for HNP-2 was granted by the NRC on January 15, 2002, in accordance with the provisions of 10 CFR 54.

1.1.3 PLANT LOCATION

The plant is located on the south side of the Altamaha River, southeast of the intersection of the river with U.S. Hwy No.1 in the northwestern sector of Appling County, Georgia. The site is ~ 11 miles north of Baxley, Georgia.

1.1.4 NUCLEAR STEAM SUPPLY SYSTEM

The HNP-2 NSSS is a BWR 4 boiling water reactor--1967 product line, 218-in. vessel--designed and supplied by General Electric Company (GE).

1.1.5 CONTAINMENT STRUCTURE

The HNP-2 containment is the Mark I BWR Containment incorporating the drywell/pressure suppression concept. The reinforced concrete secondary containment structure was designed by Bechtel Power Corporation.

1.1.6 POWER OUTPUT

The design operating thermal power level is 2804 MWt which corresponds to 100% of rated steam flow. This will yield a gross electrical output of ~ 940 MWe, a net electrical output of ~ 905 MWe.

1.1.7 SCHEDULE FOR COMPLETION AND COMMERCIAL OPERATION

Initial fuel loading of HNP-2 began in April 1978. Commercial operation of HNP-2 began on September 5, 1979.

1.1.8 ORGANIZATION OF CONTENTS

1.1.8.1 <u>Subdivisions</u>

The FSAR is organized into 17 chapters, each of which consists of a number of sections that are numerically identified by two numerals separated by a decimal; e.g., 3.4 is the fourth section of chapter 3. Further subdivisions are referred to as subsections and then as paragraphs.

1.1.8.2 Standard Format

The HNP-2 FSAR is written to comply with the "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants, Regulatory Guide (RG) 1.70, Revision 1, issued by the AEC in October 1972. The FSAR uses the same chapter, section, subsection, and paragraph headings as those used in the standard format, except in cases where this format was not applicable to plant design. Where appropriate, the FSAR is subdivided beyond the extent of the standard format to isolate all information specifically requested in that document. Where information is presented that is not specifically requested by the standard format and is identified numerically (chapter, section, subsection, or paragraph), this information is presented under the appropriate general headings.

In unique instances, selected HNP-2-FSAR information has been determined to apply to both HNP-1 and HNP-2. Where this condition exists, the term "(HNP-1 and HNP-2)" may be denoted, following the applicable subsection/paragraph title, to assist the FSAR reader in understanding that the subject textual information is applicable to both HNP units.

1.1.8.3 References (HNP-1 and HNP-2)

In accordance with the guidelines of Nuclear Energy Institute (NEI) 98-03, "Guidelines for Updating Final Safety Analysis Reports," Revision 1, Section A4.3, the FSAR contains two types of references:

- General References (alternatively references), and
- ***Documents Incorporated by Reference into the FSAR***

General references (alternatively references) are not considered part of the FSAR, but provide background information or additional detail on particular material presented in the FSAR. General references (alternatively references) are not subject to the FSAR update requirements of 10 CFR 50.71(e) or the change control requirements of 10 CFR 50.59.

Documents Incorporated by Reference into the FSAR are part of the FSAR and, therefore, subject to the update requirements of 10 CFR 50.71 (e) and the change control requirements of 10 CFR 50.59. When referenced in the text, ***documents incorporated by reference into the FSAR*** appear in bold, italicized type and are listed on the references page at the end of the section.

Drawings ***incorporated by reference into the FSAR*** utilize the standard FSAR font type when referenced in text and are listed in HNP-2-FSAR table 1.1-1.

1.1.8.4 Tables, Figures, and Drawings (HNP-1 and HNP-2)

Tables are identified by the section number, followed by a number according to its order of mention in the section; e.g., table 3.3-5 is the fifth table of section 3.3. Tables are located at the

end of the applicable section. Drawings, sketches, curves, graphs, and engineering diagrams identified as figures are numbered according to the order of mention in the section; e.g., figure 3.4-2 is the second figure of section 3.4. Figures are located at the end of the applicable section.

Selected HNP-2-FSAR tables and figures have been determined to apply to both HNP-1 and HNP-2. Where this unique condition exists, the term "(HNP-1 and HNP-2)" is denoted on the applicable HNP-2-FSAR tables and figures to identify information applicability to both HNP units.

Some drawings and engineering diagrams are included, in conjunction with specific system descriptions, by reference to their drawing identification number in lieu of inclusion as figures. Table 1.1-1 lists the drawings referenced in HNP-1-FSAR and/or HNP-2-FSAR, and is provided as an aid for identifying such referenced drawings (with their applicable sheet numbers). The drawings listed in table 1.1-1 are considered ***Incorporated by Reference into the FSAR*** as discussed in HNP-2-FSAR paragraph 1.1.8.3 (i.e., the drawings are still part of the FSAR and subject to the update requirements of 10 CFR 50.71 (e) and the change control requirements of 10 CFR 50.59).

1.1.8.5 Numbering of Pages (HNP-1 and HNP-2)

Pages are numbered sequentially within each section; e.g., page 1.1-2 is the second page of section 1.1.

1.1.8.6 Amending the FSAR (HNP-1 and HNP-2)

The FSAR is amended on an annual basis in accordance with 10 CFR 50.71e.

1.1.8.7 Historical Information (HNP-1 and HNP-2)

Selected information contained in the FSAR is designated *Historical* in accordance with the guidelines of NEI 98-03, Revision 1, Section A3. Historical information is generally not expected to require updating in accordance with the requirements of 10 CFR 50.71 (e). However, based upon the particular subject content of the historical information, future updating may be required; therefore, FSAR information designated *Historical* cannot be completely ignored for possible future FSAR updating purposes. Changes to designated Historical information are still subject to the change control requirements of 10 CFR 50.59.

Historical information is identified by use of the following italicized font:

14A.17 PROCESS LIQUID RADIATION MONITOR SYSTEM (2D11-3530)

A. Test objective - The process liquid radiation monitor will be shown to be operable in calibration, to have correct trip settings, and to perform its required annunciator functions.

For FSAR tables, historical information is designated by either the above unique italicized font type or by the term *Historical* located directly to the right of the table title. For FSAR figures, the term *Historical* appears directly above the figure title block.

TABLE 1.1-1 (SHEET 1 OF 19)

Drawing No.	Sheet No.	Title
A-21603	1 thru 18	Nameplate Engraving List for Panel 2H11-P603
A-21725	1, 2	Nameplate Engraving List for Panel 2H21-P173
A-21726	1 thru 8	Nameplate Engraving List for Panel 2C82-P001
A-47039	1	E. I. Hatch Nuclear Plant – Unit No. 1 / 2 License Renewal Aging Management Review Summary
D-11001	1	Service Water Piping at Intake Structure (P41) P&ID
D-11004	1	RHRSW System (E11) Outside Bldg P&ID
E-10173	1	General Bldg Site Plan
H-10167	1	Refueling Floor Heavy Load Paths
H-11018	1	Main and Auxiliary Steam Systems (N36) P&ID
H-11019	1	Condensate and Feedwater System (N21) P&ID
H-11020	1	Moisture Separator and Heater Drain System (N22) P&ID
H-11024	1	Service Water Piping (P41) P&ID
H-11025	1	Condenser Vacuum and Offgas System (N62) P&ID
H-11036	1 thru 3	Circulating Water System (N71/N61) P&ID
H-11037	1	Fuel Oil and Diesel Oil System (R43) P&ID
H-11039	1	Service and Instrument Air at High Pressure Air Compressors (P51) P&ID
H-11126	1	Piping - Circulating Water System
H-11142	1	Piping - Service Water at Intake Structure
H-11335	1	Piping - Chlorine System Outside and at Chlorine Bldg
H-11353	1	General Arrangement - Outside Piping (E51+00 to E55+00 and N52+00 to N57+50)
H-11354	1	General Arrangement - Outside Piping (E47+00 to E51+00 and N54+50 to N60+25)
H-11355	1	General Arrangement - Outside Piping to River (N57+50 to N60+00, and E51+00 to E54+70, N60+40, and E49+20)
H-11600	1	Service Water at Diesel Generator Bldg (P41) P&ID
H-11601	1	Main (N11) and Auxiliary Steam Systems P&ID
H-11602	1	Main (N11) and Auxiliary Steam Systems P&ID
H-11603	1	Condensate and Feedwater System (N21) P&ID

TABLE 1.1-1 (SHEET 2 OF 19)

Drawing No.	Sheet No.	Title
H-11604	1	Condensate and Feedwater System (N21) P&ID
H-11605	1	Condensate and Feedwater System (N21) P&ID
H-11606	1	Moisture Separator and Heater Drain System (N22) P&ID
H-11607	1	Moisture Separator and Heater Drain System (N22) P&ID
H-11608	1	Moisture Separator and Heater Drain System (N22) P&ID
H-11609	1	Service Water Piping (P41) P&ID
H-11610	1	Service Water Piping (P41) P&ID
H-11611	1	Service Water Piping (P41) P&ID
H-11612	1	Condenser Vacuum and Offgas Systems (N62) P&ID
H-11613	1	Condenser Vacuum and Offgas Systems (N22/N33) P&ID
H-11631	1, 2	Diesel Generators 1A and 1C (R43) P&ID
H-11638	1, 2	Diesel Generator 1B (R43) P&ID
H-11641	1	Instrument Air at High Pressure Air Compressors (P52) P&ID
H-11646	1	Moisture Separator and Heater Drain System (N22) P & ID
H-11730	1	Hypochlorination Piping System Plan - Chlorine Bldg el 128 ft 4 in.
H-11802	1	Fire Hazards Analysis General Building Site Plan
H-11807	1	Fire Hazards Analysis Turbine Building el 164 ft 0 in.
H-11817	1	Fire Hazards Analysis Control Building Below el 164 ft 0 in.
H-11823	1	Fire Hazards Analysis Turbine Building el 164 ft 0 in.
H-11826	1	Fire Hazards Analysis Reactor Building Below el 130 ft 0 in.
H-11836	1	Fire Hazards Analysis Reactor Building el 164 ft 0 in.
H-11850	1	Fire Hazards Analysis Transformer Switch Yard
H-11860	1	Fire Hazards Analysis Reactor Building Below el 130 ft 0 in.
H-11982	1	Water Treatment System (W23) P&ID
H-12192	1	Outdoor Concrete Intake Structure - General Arrangement (W35)
H-12320	1	Diesel Generator Bldg Concrete Base Slab - General Arrangement
H-12405	1	Control Building Concrete - General Arrangement - Section A-A

TABLE 1.1-1 (SHEET 3 OF 19)

Drawing No.	Sheet No.	Title
H-12406	1	Control Building Concrete - General Arrangement - Sections BB & CC
H-12523	1	General Arrangement - Plant Site Outdoor Benchmarks
H-12619	1	Architectural - Diesel Generator Bldg Heating and Ventilation System (X22) - General Arrangement and Parts Nos.
H-12626	1	Architectural - Turbine and Control Bldgs General Plan at el 112 ft 0 in.
H-12627	1	Architectural - Control Bldg Partial Plan at el 112 ft 0 in.
H-12628	1	Architectural - Turbine and Control Bldgs General Plan (U22/Z22) at el 130 ft 0 in.
H-12629	1	Architectural - Control Bldg Partial Plan at el 112 ft 0 in.
H-12631	1	Architectural - Control Building - Detailed Floor Plan at el 147 ft 0 in.
H-12632	1	Turbine and Control Bldgs General Plan (T22/Z22) at el 164 ft 0 in.
H-12678	1	Architectural - Chlorine Bldg Floor Plan and Elevations (Y64)
H-13350	1	Master Single-Line Diagram
H-13369	1,2	120/208-V Essential ac System MPLS (1R25-S064) Single-Line Diagram
H-13370	1	125/250-V-dc Station Service Division I (R22/R25) Single-Line Diagram
H-13370	2	125/250-V-dc Station Service Division II (R25) Single-Line Diagram
H-13371	1, 2	125-V-dc Emergency Station Service (R25) Single-Line Diagram
H-13635	1	120-V Vital ac and 24/48-V-dc System (R25/R42) Single Line Diagram
H-13850	1	General Arrangement - Outdoor Switchyard
H-13867	1	Plant Switchyard and Transmission Line Connections
H-15006	1	Reactor Bldg Drywell - Interior Vessel Protection Plans (T22)
H-15650	1	Main Stack Plans and Elevations
H-15851	1	Architectural Floor Plan at el 87 ft 0 in.
H-15852	1	Architectural Floor Plan at els 130 ft 0 in. and 132 ft 4 in.
H-15854	1	General Arrangement Plant Building and Equipment Plan el 158 ft and 164 ft

TABLE 1.1-1 (SHEET 4 OF 19)

Drawing No.	Sheet No.	Title
H-15874	1	Architectural - Main Stack
H-15903	1	Shielding - Waste Gas Treatment Bldg
H-16000	1	Nitrogen Inerting System (T48) P&ID
H-16002	1	FPCC System (G41) P&ID
H-16003	1	Fuel Pool Filter/Demineralizer System (G41) P&ID
H-16005	1	Reactor Bldg Ventilation System (T41) P&ID
H-16007	1	Drywell Cooling System (T47) P&ID
H-16008	1	Radwaste Bldg Ventilation System (V41) P&ID
H-16009	1	RBCCW System (P42) P&ID
H-16011	1	Reactor Bldg Service Water System (P41) P&ID
H-16014	1	Reactor Bldg Refueling Floor Ventilation System (T41) P&ID
H-16020	1	SGTS (T46) P&ID
H-16022	1	Piping and Equipment Code Classification Diagram
H-16023	1	Safeguard Equipment Cooling (T41) P&ID
H-16024	1	Primary Containment Purge and Inerting System (T48) P&ID
H-16027	1	Equipment Locations - Reactor and Radwaste Bldgs Above el 144 ft 4 in.
H-16029	1	Equipment Locations - Reactor and Radwaste Bldgs at el 185 ft 0 in.
H-16030	1	Equipment Locations - Reactor Bldg at el 203 ft 0 in.
H-16031	1	Equipment Locations - Reactor Bldg at el 228 ft 0 in.
H-16032	1	Equipment Locations - Reactor Bldg Section A-A
H-16033	1	Equipment Locations - Reactor Bldg Section B-B
H-16034	1	Control Bldg Radiochemical Lab and Health Physics Area Air-Conditioning (Z41/Z45) P&ID
H-16035	1	Control Bldg Computer and Water Analysis Rooms Air-Conditioning (Z41) P&ID
H-16036	1	Equipment Locations - Radwaste Bldg Addition
H-16037	1	Turbine Bldg Ventilation System (U41) P&ID
H-16040	1	Control Bldg Ventilation System (Z41) P&ID
H-16041	1	Control Bldg Ventilation System (Z41) PFD

TABLE 1.1-1 (SHEET 5 OF 19)

Drawing No.	Sheet No.	Title
H-16042	1	Control Bldg Control and Cable Spreading Rooms Air-Conditioning Temperature Control (Z41) Diagram
H-16061	1	SLCS (C41) P&ID
H-16062	1	Nuclear Boiler System (B21) P&ID
H-16063	1	Nuclear Boiler System (B21) P&ID
H-16064	1	CRD System (C11) P&ID
H-16065	1	CRD System (C11) P&ID
H-16066	1	RRS (B31)
H-16110	1	Unit No. 1&2 Types of Penetration Seals for Pipe
H-16145	1	Nuclear Boiler System (B21) P&ID
H-16147	1	Reactor Bldg Nitrogen Inerting System (T48) Below el 130 ft 0 in.
H-16173	1	Fission Products Monitoring System (D11) P&ID
H-16174	1	SGTS (T46) P&ID
H-16176	1	Radwaste System (G11) P&ID
H-16177	1	Radwaste System (G11) P&ID
H-16178	1	Radwaste System (G11) P&ID
H-16179	1	Radwaste System (G11) P&ID
H-16180	1	Radwaste System (G11) P&ID
H-16181	1	Radwaste System (G11) P&ID
H-16182	1	Radwaste System (G11) P&ID
H-16188	1	RWC System (G31) P&ID
H-16189	1	RWC System (G31) P&ID
H-16239	1 thru 9	Reactor Bldg Instrument Air System (P52) P&ID
H-16249	1	MCR Instrument and Primary Point Locations (Z99)
H-16274	1	Fission Products Monitoring System (D11) P&ID
H-16276	1	Primary Containment Atmosphere H_2&O_2 Analyzer P&ID Sh 1
H-16280	1	Primary Containment Atmosphere H_2&O_2 Analyzer P&ID Sh 2
H-16286	1	Drywell Pneumatic System (P70) P&ID
H-16299	1	Drywell Pneumatic System (P70) P&ID
H-16326	1, 2	Turbine Bldg Chilled Water System (P63) P&ID

TABLE 1.1-1 (SHEET 6 OF 19)

Drawing No.	Sheet No.	Title
H-16327	1	Turbine Bldg Chilled Water System (P63) P&ID
H-16329	1	RHR System (E11) P&ID
H-16330	1	RHR System (E11) P&ID
H-16331	1	CS System P&ID
H-16332	1, 2	HPCI System (E41) P&ID
H-16333	1	HPCI System (E41) P&ID
H-16334	1	RCIC System (E51) P&ID
H-16335	1	RCIC System (E51) P&ID
H-16339	1	ERF/SPDS (X75) Block Diagram
H-16512	1	Radwaste Bldg Addition Ventilation System (V41) P&ID
H-16517	1	Radwaste Bldg Addition Support Systems (G11) P&ID and PFD
H-16519	1	General Arrangement – Offgas Recombiner Bldg
H-16532	1, 2	Off-gas System (N62) P&ID
H-16536	1	General Arrangement - Waste Gas Treatment Bldg
H-16549	1	Waste Gas Treatment Bldg Air-Conditioning System (N62) P&ID
H-16560	1	PRNM System (C51) IED
H-16561	1	PRNM System (C51)
H-16564	1	Process Radiation Monitoring System (D11) P&ID
H-16567	1	Feedwater Control System Turbine-Driven Feed Pumps (C32) IED
H-16568	1	RPS (C71) P&ID
H-17012	1	Reactor Building 600 V MCC “1E-A” & “1E-B” MPL R24-S018A & R24-S018B Single-Line Diagram
H-17791	1	RPS (C71) Elementary Diagram
H-17792	1	RPS (C71) Elementary Diagram
H-19900	1	Logic Diagrams Legend and General Notes
H-19901	1	Nuclear Boiler System (B21) Logic Diagram
H-19902	1	Nuclear Boiler System (B21) Logic Diagram
H-19903	1	Nuclear Boiler System (B21) Logic Diagram
H-19904	1	Nuclear Boiler System (B21) Logic Diagram

TABLE 1.1-1 (SHEET 7 OF 19)

Drawing No.	Sheet No.	Title
H-19905	1	Nuclear Boiler System (B21) Logic Diagram
H-19906	1	Nuclear Boiler System (B21) Logic Diagram
H-19907	1	Nuclear Boiler System (B21) Logic Diagram
H-19908	1	Nuclear Boiler System (B21) Logic Diagram
H-19909	1	Nuclear Boiler System (B21) Logic Diagram
H-19910	1	Nuclear Boiler System (B21) Logic Diagram
H-19911	1	Nuclear Boiler System (B21) Logic Diagram
H-19912	1	Nuclear Boiler System (B21) Logic Diagram
H-19913	1	RRS (B31) Logic Diagram
H-19914	1	RRS (B31) Logic Diagram
H-19915	1	RRS (B31) Logic Diagram
H-19916	1	RRS (B31) Logic Diagram
H-19917	1	RRS (B31) Logic Diagram
H-19918	1	CRDHS (C11) Logic Diagram
H-19919	1	CRDHS (C11) Logic Diagram
H-19920	1	CRDHS (C11) Logic Diagram
H-19921	1	CRDHS (C11) Logic Diagram
H-19922	1	CRDHS (C11) Logic Diagram
H-19923	1	CRDHS (C11) Logic Diagram
H-19924	1	CRDHS (C11) Logic Diagram
H-19925	1	CRDHS (C11) Logic Diagram
H-19926	1	SLCS (C41) Logic Diagram
H-19927	1	NMS (C51) Logic Diagram
H-19928	1	NMS (C51) Logic Diagram
H-19929	1	NMS (C51) Logic Diagram
H-19930	1	NMS (C51) Logic Diagram
H-19931	1	NMS (C51) Logic Diagram
H-19932	1	NMS (C51) Logic Diagram
H-19933	1	RPS (C71) Logic Diagram
H-19934	1	RPS (C71) Logic Diagram

TABLE 1.1-1 (SHEET 8 OF 19)

Drawing No.	Sheet No.	Title
H-19935	1	RPS (C71) Logic Diagram
H-19936	1	RPS (C71) Logic Diagram
H-19937	1	RHR System (E11) Logic Diagram
H-19938	1	RHR System (E11) Logic Diagram
H-19939	1	RHR System (E11) Logic Diagram
H-19940	1	RHR System (E11) Logic Diagram
H-19941	1	RHR System (E11) Logic Diagram
H-19942	1	RHR System (E11) Logic Diagram
H-19943	1	RHR System (E11) Logic Diagram
H-19944	1	CS System (E21) Logic Diagram
H-19945	1	CS System (E21) Logic Diagram
H-19946	1	CS System (E21) Logic Diagram
H-19947	1	HPCI System (E41) Logic Diagram
H-19948	1	HPCI System (E41) Logic Diagram
H-19949	1	HPCI System (E41) Logic Diagram
H-19950	1	HPCI System (E41) Logic Diagram
H-19951	1	HPCI System (E41) Logic Diagram
H-19952	1	HPCI System (E41) Logic Diagram
H-19953	1	HPCI System (E41) Logic Diagram
H-19954	1	HPCI System (E41) Logic Diagram
H-19955	1	RCIC System (E51) Logic Diagram
H-19956	1	RCIC System (E51) Logic Diagram
H-19957	1	RCIC System (E51) Logic Diagram
H-19958	1	RCIC System (E51) Logic Diagram
H-19959	1	RCIC System (E51) Logic Diagram
H-19960	1	RCIC System (E51) Logic Diagram
H-19961	1	RCIC System (E51) Logic Diagram
H-19962	1	RCIC System (E51) Logic Diagram
H-19963	1	RWC System (G31) Logic Diagram

TABLE 1.1-1 (SHEET 9 OF 19)

Drawing No.	Sheet No.	Title
H-19964	1	RWC System (G31) Logic Diagram
H-19965	1	NMS (C51) Logic Diagram
H-19966	1	NMS (C51) Logic Diagram
H-19967	1	CRDHS (C11) Logic Diagram
H-21001	1	General Arrangement - Turbine Room Base Slab at el 112 ft 0 in.
H-21002	1	General Arrangement - Turbine Room Intermediate Floor at el 130 ft 0 in.
H-21003	1	General Arrangement - Turbine Room Floor Slab at el 147 ft 0 in.
H-21004	1	General Arrangement - Turbine Room Operating Floor at el 164 ft 0 in.
H-21006	1	General Arrangement - Turbine Bldg Section A-A
H-21007	1	General Arrangement - Turbine Bldg Section B-B
H-21012	1	Main Steam System P&ID
H-21018	1	Condensate Polishing and Demineralizer System (N21) P&ID
H-21026	1	Turbine Bldg Circulating Water System (W23/N22/N71) P&ID
H-21028	1	Control Bldg Service Air System (P51) P&ID
H-21030	1	Turbine Bldg Condenser Vacuum and Gland Seal System (N22/N33) P&ID
H-21033	1	Turbine Bldg Service Water System (P41) P&ID
H-21037	1 thru 5	Turbine Bldg Condensate and Feedwater System (N21) P&ID
H-21038	1 thru 3	Turbine Bldg Condensate and Feedwater System (N21) P&ID
H-21039	1	RHRSW System (E11) P&ID
H-21056	1	SJAE System (N22) P&ID
H-21061	1	Turbine Bldg Floor, Equipment, and Roof Drains (U45/U55) P&ID
H-21062	1	Turbine Bldg Floor and Equipment Drains P&ID
H-21063	1	Control Bldg Floor and Equipment Drains (Z45) P&ID
H-21074	1	Diesel Engine and Fuel Oil System (R43) P&ID
H-21077	1	Turbine Bldg Instrument Air System (P52) P&ID
H-21102	1	Piping - Service Water at Intake Structure

TABLE 1.1-1 (SHEET 10 OF 19)

Drawing No.	Sheet No.	Title
H-21114	1	Piping - Circulating Water System
H-22250	1	Control Bldg. Concrete - General Arrangement - el 112 ft 0 in., 130 ft 0 in., 147 ft 0 in., and 164 ft 0 in.
H-22802	1	Architectural Turbine Building – Floor Plan – el 112 ft 0 in.
H-23390	1	125/250-V DC Station Service Division 1 2R42A MPL's 2R25-S001 and 2R25-S002 Single-Line Diagram
H-23635	1	24/48-V-dc System (2R42E) Single Line Diagram
H-23754	1	Communications Turbine Building el 164 ft 0 in.
H-23761	1	Communications-Wiring Diagram Turbine and Control Building el 147 ft 0 in. and 164 ft 0 in.
H-23768	1	Communications – Typical Wiring Diagram for Communications Stations
H-23769	1	Turbine and Control Building Communications Interconnection Diagram
H-24700	1	Logic Diagram Legends and General Notes
H-24701	1	Nuclear Boiler System (B21) Logic Diagram
H-24702	1	Nuclear Boiler System (B21) Logic Diagram
H-24703	1	Nuclear Boiler System (B21) Logic Diagram
H-24704	1	Nuclear Boiler System (B21) Logic Diagram
H-24705	1	Nuclear Boiler System (B21) Logic Diagram
H-24706	1	Nuclear Boiler System (B21) Logic Diagram
H-24707	1	Nuclear Boiler System (B21) Logic Diagram
H-24708	1	Nuclear Boiler System (B21) Logic Diagram
H-24709	1	Nuclear Boiler System (B21) Logic Diagram
H-24710	1	Nuclear Boiler System (B21) Logic Diagram
H-24711	1	Nuclear Boiler System (B21) Logic Diagram
H-24712	1	Nuclear Boiler System (B21) Logic Diagram
H-24713	1	RRS (B31) Logic Diagram
H-24714	1	RRS (B31) Logic Diagram
H-24715	1	RRS (B31) Logic Diagram
H-24716	1	RRS (B31) Logic Diagram

TABLE 1.1-1 (SHEET 11 OF 19)

Drawing No.	Sheet No.	Title
H-24717	1	CRDHS (C11) Logic Diagram
H-24718	1	CRDHS (C11) Logic Diagram
H-24719	1	CRDHS (C11) Logic Diagram
H-24720	1	CRDHS (C11) Logic Diagram
H-24721	1	SLCS Logic (C41) Diagram
H-24722	1	NMS (C51) Logic Diagram
H-24723	1	NMS (C51) Logic Diagram
H-24724	1	NMS (C51) Logic Diagram
H-24725	1	NMS (C51) Logic Diagram
H-24726	1	NMS (C51) Logic Diagram
H-24727	1	NMS (C51) Logic Diagram
H-24728	1	RPS (C71) Logic Diagram
H-24729	1	RPS (C71) Logic Diagram
H-24730	1	RPS (C71) Logic Diagram
H-24731	1	RPS (C71) Logic Diagram
H-24732	1	RHR System (E11) Logic Diagram
H-24733	1	RHR System (E11) Logic Diagram
H-24734	1	RHR System (E11) Logic Diagram
H-24735	1	RHR System (E11) Logic Diagram
H-24736	1	RHR System (E11) Logic Diagram
H-24737	1	RHR System (E11) Logic Diagram
H-24738	1	RHR System (E11) Logic Diagram
H-24739	1	CS System (E21) Logic Diagram
H-24740	1	CS System (E21) Logic Diagram
H-24741	1	CS System (E21) Logic Diagram
H-24742	1	HPCI System (E41) Logic Diagram
H-24743	1	HPCI System (E41) Logic Diagram
H-24744	1	HPCI System (E41) Logic Diagram
H-24745	1	HPCI System (E41) Logic Diagram
H-24746	1	HPCI System (E41) Logic Diagram

TABLE 1.1-1 (SHEET 12 OF 19)

Drawing No.	Sheet No.	Title
H-24747	1	HPCI System (E41) Logic Diagram
H-24748	1	HPCI System (E41) Logic Diagram
H-24749	1	HPCI System (E41) Logic Diagram
H-24750	1	RCIC System (E51) Logic Diagram
H-24751	1	RCIC System (E51) Logic Diagram
H-24752	1	RCIC System (E51) Logic Diagram
H-24753	1	RCIC System (E51) Logic Diagram
H-24754	1	RCIC System (E51) Logic Diagram
H-24755	1	RCIC System (E51) Logic Diagram
H-24756	1	RCIC System (E51) Logic Diagram
H-24757	1	RCIC System (E51) Logic Diagram
H-24758	1	RWC System (G31) Logic Diagram
H-24759	1	RWC System (G31) Logic Diagram
H-24760	1	RRS (B31) Logic Diagram
H-24781	1	CRDHS (C11) Logic Diagram
H-24782	1	CRDHS (C11) Logic Diagram
H-24784	1	CRDHS (C11) Logic Diagram
H-24785	1	NMS (C51) Logic Diagram
H-24786	1	NMS (C51) Logic Diagram
H-24787	1	CRDHS (C11) Logic Diagram
H-25000	1	Reactor Bldg Containment Vessel Requirements - Drywell Plans and Sections
H-25004	1	Reactor Bldg RPV Pedestal Development Elevation (Inside and Out)
H-25005	1	Reactor Bldg RPV Pedestal Sectional Plans and Sections
H-25993	1	Shielding - Floor Plan at el 87 ft 0 in.
H-25994	1	Shielding - Floor Plan at el 130 ft 0 in.
H-25995	1	Shielding - Floor Plan at el 147 ft 0 in.
H-25996	1	Shielding - Floor Plan at el 158 ft 0 in.
H-25997	1	Shielding - Floor Plan at el 185 ft 0 in.

TABLE 1.1-1 (SHEET 13 OF 19)

Drawing No.	Sheet No.	Title
H-25998	1	Shielding - Floor Plan at el 203 ft 0 in.
H-25999	1	Shielding - Floor Plan at el 228 ft 0 in.
H-26000	1	Nuclear Boiler System (B21) P&ID
H-26001	1	Nuclear Boiler System (B21) P&ID
H-26002	1	TSC HVAC System (X75) P&ID and PFD
H-26003	1	RRS (B31) P&ID
H-26006	1	CRD System (C11) P&ID
H-26007	1	CRD System (C11) P&ID
H-26008	1	Reactor and Radwaste Bldgs Chilled Water System (P65) P&ID and PFD
H-26009	1	SLCS (C41) P&ID
H-26010	1	Area Radiation Monitoring System (D21) IED
H-26011	1	Process Radiation Monitoring System (D11) P&ID
H-26012	1	Process Radiation Monitoring System (D11) IED
H-26013	1	Process Radiation Monitoring System (D11) IED
H-26014	1	RHR System (E11) P&ID
H-26015	1	RHR System (E11) P&ID
H-26016	1	Fission Products Monitoring (D11) System P&ID
H-26017	1	Fission Products and Post-LOCA Monitoring Systems (D11) P&ID
H-26018	1	CS System (E21) P&ID
H-26019	1	Jockey Pump System P&ID and PFD for RHR and CS Systems
H-26020	1	HPCI System (E41) P&ID
H-26021	1	HPCI System (E41) P&ID
H-26023	1	RCIC System (E51) P&ID
H-26024	1	RCIC System (E51) P&ID
H-26025	1	Reactor and Radwaste Bldgs Chilled Water System (P65) P&ID and PFD
H-26026	1	Radwaste System (G11) P&ID
H-26027	1	Radwaste System (G11) P&ID
H-26028	1	Radwaste System (G11) P&ID and PFD

TABLE 1.1-1 (SHEET 14 OF 19)

Drawing No.	Sheet No.	Title
H-26029	1	Radwaste System (G11) P&ID
H-26030	1	Radwaste System (G11) P&ID
H-26031	1	Radwaste System (G11) P&ID
H-26032	1	Radwaste System (G11) P&ID
H-26035	1	Radwaste Bldg Support Systems (P&ID)
H-26036	1	RWC System (G31) P&ID
H-26037	1	RWC System (G31) P&ID
H-26039	1	FPCC System (G41) P&ID
H-26040	1	Fuel Pool Filter/Demineralizer System (G41) P&ID
H-26042	1	Torus Drainage and Purification System (G51) P&ID and PFD
H-26045	1	Offgas System (N62) P&ID
H-26046	1	Reactor and Radwaste Bldgs Condensate Storage and Transfer System (P11) Diagram
H-26048	1	Primary Containment Atmosphere H_2 and 0_2 Analyzer System (P33) P&ID
H-26049	1	Primary Containment Atmosphere H_2 and 0_2 Analyzer System (P33) P&ID
H-26050	1	Reactor Bldg PSW System (P41) P&ID
H-26051	1	Reactor Bldg PSW System (P41) P&ID
H-26054	1	RBCCW System (P42) P&ID
H-26055	1	RBCCW System (P42) P&ID
H-26063	1	Reactor, Radwaste, and Turbine Bldgs Auxiliary Steam System (P61) P&ID
H-26064	1	Reactor Bldg South Side Noninterruptable Instrument Air System (P52) P&ID
H-26066	1	Drywell Pneumatic System (P70) P&ID
H-26067	1	Reactor Zone Ventilation System (T41) P&ID
H-26070	1	Reactor Bldg North Side Noninterruptable Instrument Air System (P52) P&ID
H-26071	1	Safeguards Equipment Emergency Cooling System (T41) P&ID
H-26072	1	Reactor Bldg Refueling Floor Ventilation System (T41) P&ID
H-26074	1	Primary Containment Cooling System (T47) P&ID and PFD

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Drawing No.	Sheet No.	Title
H-26075	1	Reactor Bldg Floor Equipment and Roof Drainage System (T45) Diagram
H-26076	1	Leak Detection System Instrument and Drainage Sumps (T45) P&ID
H-26078	1	SGTS (T46) P&ID
H-26080	1	Primary Containment Chilled Water System (P64) P&ID and PFD
H-26081	1	Primary Containment Chilled Water System (P64) P&ID and PFD
H-26083	1	Nitrogen Inerting System (T48) P&ID
H-26084	1	Primary Containment Purge and Inerting System (T48) P&ID
H-26086	1	Turbine Bldg Ventilation System (U41) P&ID
H-26088	1	Turbine Bldg Chilled Water System P&ID Sheet 1
H-26089	1	Turbine Bldg Chilled Water System P&ID Sheet 2
H-26090	1	Radwaste Bldg Ventilation System (V41) P&ID
H-26092	1	Radwaste Bldg Floor and Equipment Drainage System (V45) Diagram
H-26093	1	Control Bldg Ventilation System (Z41) P&ID and PFD
H-26094	1	Control Bldg - MCR Air Conditioning System (Z41) PFD
H-26095	1	Piping and Valve Code Classification Diagram
H-26096	1	General Arrangement - Reactor Bldg Below el 130 ft
H-26097	1	General Arrangement - Radwaste Bldg at el 103 ft 0 in.
H-26098	1	General Arrangement - Reactor Bldg at el 130 ft 0 in. and Radwaste Bldg at el 132 ft 4 in.
H-26099	1	General Arrangement - Drywell Area at el 148 ft 3-1/2 in., Radwaste Bldg at el 148 ft 0 in., and Hot Machine Shop at el 130 ft 0 in.
H-26100	1	General Arrangement - Reactor Bldg at el 158 ft 0 in. and Radwaste Bldg at el 164 ft 0 in.
H-26101	1	General Arrangement - Reactor Bldg at el 185 ft 0 in. and Radwaste Bldg Roof at el 178 ft 0 in.
H-26102	1	General Arrangement - Reactor Bldg Plan at el 203 ft 0 in.
H-26103	1	General Arrangement - Reactor Bldg Plan at el 228 ft 0 in.

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Drawing No.	Sheet No.	Title
H-26104	1	General Arrangement - Reactor Bldg Section A-A
H-26105	1	General Arrangement - Reactor Bldg Section B-B
H-26106	1	General Arrangement - Radwaste Bldg Section C-C
H-26116	1	MCR Shift Supervisor's Area HVAC (Z41) P&ID and PFD
H-26128	1	HPCI System Plan View - HPCI Room and Reactor Bldg East
H-26130	1	HPCI System Sections
H-26142	1	Torus Drainage and Purification System
H-26189	1	Nuclear Boiler System (B21) P&ID
H-26191	1	ERF/SPDS (X75) Block Diagram
H-26202	1	Drywell Floor and Equipment Drainage System at el 114 ft 6 in.
H-26237	1	Reactor Bldg Ventilation System at el 203 ft 0 in. East
H-26238	1	Reactor Bldg Ventilation System at el 203 ft 0 in. West
H-26240	1	Reactor Bldg Ventilation System at el 228 ft 0 in. West
H-26260	1	Reactor Bldg Instrument Air System (P52) Plan and Sections Below el 130 ft 0 in.
H-26261	1	Reactor Bldg Instrument Air System (P52) Plan and Sections at el 130 ft 0 in.
H-26279	1	RCIC System - Reactor Bldg Below el 130 ft 0 in. West
H-26280	1	RCIC System - Reactor Bldg Below el 130 ft 0 in. East
H-26384	1	Post Accident Reactor Coolant and Containment Atmosphere Sampling System (P33) P&ID
H-26391	1	Reactor Bldg and Drywell Sumps Discharge Piping
H-26424	1	Instrument and Primary Point Locations Section A-A
H-26991	1	Feedwater Control System (C32) - Turbine-Driven Feed Pumps IED
H-26993	1	NMS IED
H-27021	1	Reactor Building 600V AC Essential MC 2E-A & MCC 2E-B MPL 2R24-S018A & 2R24-S018B Single Line Diagram
H-27057	1	Emergency Response Facility and Process Mini Computer Single-Line Diagram
H-27450	1	Unit 2 Nuclear Steam Supply Shutoff System 2A71 Elementary Diagram Sheet 1

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Drawing No.	Sheet No.	Title
H-27460	1	Unit 2 Nuclear Steam Supply Shutoff System 2A71 Elementary Diagram Sheet 11
H-27461	1	Unit 2 Nuclear Steam Supply Shutoff System 2A71 Elementary Diagram Sheet 12
H-27612	1	RPS (C71) Elementary Diagram
H-27613	1	RPS (C71) Elementary Diagram
H-28023	1	Drywell Pneumatic System (P70) P&ID
H-28135	1	Post Accident Sampling Chemical Analysis System (P33) P&ID
H-29000	1	Reactor Bldg ISI Doors N1A and N1B
H-29026	1	Reactor Building Pipe Whip Restraint Details
H-40056	1	Control Bldg Cold Lab HVAC P&ID at el 112 ft 0 in.
H-40429	1	Architectural - Control Bldg Partial Plan at el 130 ft 0 in.
H-40430	1	Architectural - Control Bldg Partial Plan at el 130 ft 0 in.
H-43801	1	Water Treatment System (HNP-1/HNP-2) P&ID
H-44073	1	River Intake Structure HVAC P&ID
H-45458	1	Dry Cask Spent-Fuel Storage Crawler Travel Path From RR Airlock to ISFSI
H-50563	1	Reactor and Radwaste Bldgs Chilled Water System PFD
H-51178	1	Control Bldg Chilled Water System P&ID and PFD
H-51179	1	Control Bldg Chilled Water Cooling Units P&ID
H-51307	1	Panel 2H11-P603 Reactor Control Demarcation and Layout
H-51357	1	Panel 2C82-P001 Remote Shutdown Demarcation and Layout
H-51358	1	Panel 2H21-P173 Remote Shutdown Instrument Demarcation and Layout
H-53210	1	Backup Air/Nitrogen Supply to Hardened Containment Vent System Air Operated Valves P&ID
H-53296	1	Backup Air/Nitrogen Supply to Hardened Containment Vent System Air Operated Valves P&ID
S-15051		Information Document Piping and Instrumentation Symbols
S-15059		CRDHS PFD
S-15062		RPV Nozzle Details - GE VPF-1983-52
S-15066		RCIC System PFD

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S-15070		NMS Arrangement
S-15117		CS System PFD
S-15213		RPV Assembly - GE VPF-1983-63-8
S-15227		Nozzle Details for 218" I.D. BWR
S-15247		MSIV Primary Steam Piping
S-15265		General Arrangement - Suppression Chamber Field Assembly
S-15290		General Plan
S-15304		RHR System PFD
S-15305		RHR System PFD
S-15329		Earthquake Ties
S-15422		Shell Stretchout
S-15520		Suppression Chamber Penetration Schedule and Orientation
S-15523		General Arrangement - RPV Elevation - GE VPF-1983-115-4
S-15524		General Arrangement - RPV Plan - GE VPF-1983-114-2
S-15584		NMS - PRNM Unit
S-15591		SRM/IRM Unit Purchase Parts
S-15665		Penetration Schedule Orientation Below Equator
S-15666		Penetration Schedule Orientation Above Equator
S-15667		Penetration Schedule Orientation Above Equator
S-16122		HPCI System PFD
S-25124		NMS - PRNM Unit Parts List
S-25140		RHR System PFD
S-25141		RHR System PFD
S-25171		RCIC System PFD
S-25176		HPCI System PFD
S-25178		CS System PFD
S-25285		RWC System PFD
S-25311		CRDHS PFD
S-25312		Process Data for CRDHS
S-25562		SRM/IRM Unit Purchase Part

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Drawing No.	Sheet No.	Title
S-26583		General Arrangement - Personnel Airlock
S-26984		Arrangement - Reactor Control BB
S-26986		Arrangement - Reactor Control BB
S-27793		Water Seal Assembly Inside Drywell Cylinder
S-28220		Reactor Assembly
S-28221		Reactor Assembly
S-28222		Reactor Assembly
S-28223		Reactor Assembly
S-28224		Reactor Assembly
S-28225		Reactor Assembly
S-28345		Barrier Plates
S-40969		HDFSS Module Assembly Details - 13-in. Wide
S-55894		Traveling Water Screen
S-60192		Condensate Polisher Body Feed System Piping and Instrumentation
SX-16121		Reactor Assembly - Section 1
SX-16122		Reactor Assembly - Section 2
SX-16123		Reactor Assembly - Section 3
SX-28760		Information Document Piping Instrument Symbols

1.2 GENERAL PLANT DESCRIPTION

1.2.1 SITE CHARACTERISTICS (HNP-1 AND HNP-2)

1.2.1.1 Location (HNP-1 and HNP-2)

The plant site is located in Appling County near Baxley, Georgia. The nearest site boundary to the reactor building is 4300 ft or 1310 m for HNP-2 and 4400 ft for HNP-1. The nearest site boundary to the main plant stack is 4100 ft or 1250 m. The nearest public road is US Hwy No. 1 located about 3500 ft west of the plant. The plant is bordered on the north by the Altamaha River. A plot plan is shown on drawing no. E-10173, and the plant property plan is shown in figure 1.2-1.

1.2.1.2 Site Ownership

The site is jointly owned by the Georgia Power Company (GPC), the Oglethorpe Power Corporation (OPC), the Municipal Electric Authority of Georgia, and the city of Dalton, Georgia.

1.2.1.3 Access to the Site

Access to the plant site and all activities thereon are under the control of Southern Nuclear Operating Company.

1.2.1.4 Site Environs

The area surrounding the site is primarily wooded, with a small amount of land devoted to farming. The nearest development community and the location of the nearest industry is Baxley, located about 11 miles south of the site, with an estimated population of 4800. The nearest major city with a population approaching 25,000 is Waycross, Georgia, with a population of ~ 21,200. Waycross is 48 miles south of the plant site.

1.2.1.5 Geology

The site is within the Atlantic Coastal Plain Province and is adjacent to the Altamaha River flood plain. The geologic strata consist of Holocene to Miocene age gravel, sand, silt, and clay to a depth of about 500 ft. Below these deposits are Tertiary and Cretaceous age sedimentary rocks with a consistency primarily of limestone and sandstone ~ 3500 ft thick. These strata overlie basaltic basement rock of pre-Cretaceous age, which is over 4000 ft beneath the site. The coastal plain sediments were eroded from the older Appalachian and Piedmont provinces to the west and deposited along coastal areas in progressively seaward belts. The sediments generally dip and thicken seaward, forming a wedge-shaped deposit. The surface of the basement rock on which the coastal plain strata rest is believed to be continuous with the surface of the Piedmont Province located above the Fall Line where the overlapping coastal plain sediments wedge out.

The major structural features of the Georgia Coastal Plain are a gentle southeastward dip of the coastal plain strata and the Southeast Georgia embayment. The southeastward dip is regional in character and ranges from 5 to 50 ft/mile. The dip increases with depth as a result of seaward thickening of the coastal plain deposits. The southeast Georgia embayment is a depositional basin recessed into the Atlantic Coast between Savannah, Georgia and Jacksonville, Florida. The basin received relatively thick sequences of Cretaceous-through-Miocene age material from higher areas to the north and south. Regional cross-sections through the embayment indicate that subsidence of the basin had ceased by the end of Miocene time.

1.2.1.6 Seismology and Design Response Spectra

The seismicity of the site was evaluated on the basis of historical earthquakes, damage from earthquake shocks, and the regional and local geologic structure. No active or recent faulting has been mapped in the area of the proposed site. The area is not seismically active; however, the effects of earthquakes from distant sources have been experienced at the site. The Charleston, South Carolina earthquake of 1886, with the epicenter located ~ 150 miles from the site, is the type which would be felt at the site. The maximum epicentral intensity probably decreased to MM-VII at Savannah, located ~ 70 miles from the site, and the area of greatest damage was within 100 miles of the plant site. The intensity decreased to about MM-VI in the site area. This value is considered conservative and corresponds to a peak horizontal acceleration of 0.08 g for plant design. The plant has a capability for safe shutdown if subjected to a peak horizontal acceleration of 0.15 g. Design spectra consistent with these accelerations were used for the analysis of Category I structures and equipment.

1.2.1.7 Hydrology

The site natural grade varies between 70 ft above msl at the river to 147 ft msl at the visitors' building. The natural grade for the various plant structures varies from 118 to 147 ft msl. The maximum river level established from the flood record was ~ el 91.3 ft msl. This is based on records from a station 18.8 miles upstream. Flooding of the plant site is unlikely. Seismic Category I plant structures and equipment located on the flood plain, such as discharge and intake structures necessary for long-term safe conditions at the plant, are flood protected and designed to withstand the design flood. The design flood stage at the plant is el 105.0 ft msl.

The extrapolated hypothetical natural minimum low flow, without reservoir supplementation, is 900 ft^3/s with a corresponding river elevation of 61.9 ft msl. The minimum instantaneous flow and stage of record are ~ 1200 ft^3/s and el 62.4 ft msl.

The ground water in the plant vicinity exists under free and confined conditions. Field observations show that the free ground water flow is toward the river. There is an impermeable deposit overlying the deep artesian aquifers. This aquiclude in conjunction with the upward gradient of the artesian zone precludes radioactive contamination of the deep artesian zone by downward percolation. The possibility of affecting the surface well supplies is remote, since the gradient of the perched and free water is toward the river. The ground water characteristics of the site are very favorable for the construction and operation of a nuclear power plant.

1.2.1.8 Meteorology

The climate at the site is typical of that in the southern Atlantic Coastal Plan; i.e., hot and humid in the summer and mild in the winter. Maximum rainfall in a 30-year period of record at Glennville, located 24 miles east of the site, was ~ 15 in. in a 24-h period. The maximum average monthly rainfall was about 7.6 in. Prevailing winds are from the WNW, but wind directions are well distributed. Maximum windspeeds at Savannah, located about 70 miles east on the coast, in a 40-year period of record were 90 mph. Equivalent maximum speeds at the site are expected to be less since the site is inland. During 44 years of record, 24 hurricanes or post-hurricane path center lines passes within 100 miles of the site. Seven tornadoes have occurred in a 10-year period of record (1956-1965) within a 25-mile radius. This is probably typical of tornado frequency in the site region. An onsite meteorological measurement program was initiated in 1970 to provide data to assess limits to be set later on radioactive gas releases.

1.2.1.9 Environmental Radiation Monitoring

An environmental radiation monitoring program was conducted before the startup of HNP-1. The environmental radiation monitoring program is discussed in section 11.6.

1.2.2 FACILITY ARRANGEMENT (HNP-1 AND HNP-2)

The two units are arranged on the site so that the turbine-generator axes are oriented on a north-south azimuth approximately perpendicular to the Altamaha River. The reactor buildings are located on the east side of the turbine building. The main control room is shared by both units and is physically located at the north end of the HNP-2 turbine building. The radwaste equipment for each unit is housed in separate buildings adjacent to both the turbine and the reactor buildings. The administration building and the machine shop are located west of the turbine building. The diesel generator building is located on the north side of the HNP-1 turbine building.

The main plant stack is located ~ 620 ft east of the north-south centerline of the HNP-2 reactor building. The cooling towers are located east of the main plant. The independent spent fuel storage installation (ISFSI) is located south of the main plant adjacent to the main rail line serving the plant.

The plant is located a minimum distance of 4300 ft from the nearest site boundary. The nearest site boundary to the main plant stack is 4100 ft. Figures 1.2-1 and 1.2-4 show the relationship between the principal plant structures and the site boundaries.

1.2.3 NUCLEAR SYSTEM (HNP-1 AND HNP-2)

The nuclear systems use single-cycle, forced-circulation, General Electric boiling water reactors (GE-BWRs) producing steam for direct use in the steam turbine. A heat balance showing the major parameters of the nuclear system for the rated power condition is shown for HNP-2 in figure 1.2-2 and for HNP-1 in figure 1.2-3. Both units were originally rated at 2436 MWt and designed for a power level corresponding to ~ 2537 MWt. Both units are now licensed for 2804 MWt.

Table 1.2-2 compares key reactor conditions at the original rated power, uprated power, extended uprated power, thermal optimization uprated power, and reactor operating pressure increase uprated power.

1.2.3.1 Reactor Primary Vessel and Internals

Each reactor primary vessel contains the reactor core and support structure; steam separators and dryers; jet pumps; control rod guide tubes; distribution lines for the feedwater, core spray (CS), and standby liquid control (SLC) systems; and the incore instrumentation and other components. The main connections to the vessel include the steam lines, the reactor coolant recirculation lines, the reactor feedwater lines, the emergency core cooling lines, and the control rod drive (CRD) housings.

The reactor primary vessel is designed and fabricated in accordance with the applicable codes for a pressure of 1250 psig. The nominal operating pressure is 1045 psig at the steam dome. The vessel is fabricated of carbon steel, and the cylindrical portion and bottom head are clad internally with stainless steel.

The reactor core is cooled by light water which enters the lower portion of the core and boils as it flows upward around the fuel rods. The steam leaving the core is dried by steam separators and dryers located in the upper portion of the reactor primary vessel. The steam is then directed to the turbine through the main steam lines. Each main steam line is provided with two isolation valves in series, one on each side of the primary containment vessel wall.

1.2.3.2 Reactor Core and Control Rods

The fuel for the reactor core consists of slightly enriched uranium dioxide pellets contained in sealed Zircaloy-2 tubes. These fuel rods are assembled into individual fuel assemblies. The number of fuel assemblies in the complete core is 560. The fuel assembly configurations are described in ***NEDE-24011-P-A, "GESTAR II - General Electric Standard Application for Reactor Fuel" (incorporated by reference into the FSAR).***

Gross control of the core is achieved by movable, bottom-entry control rods supplemented by gadolinia-urania burnable poison rods in the initial fuel load. The control rods are of cruciform shape and are dispersed throughout the lattice of fuel assemblies. The rods are controlled by individual hydraulic drives.

1.2.3.3 Reactor Recirculation System (RRS)

Reactor power level control is augmented by controlling the reactor coolant recirculation system flowrate through the reactor core. This is accomplished by two recirculation loops external to the reactor vessel but inside the primary containment. Each loop has one motor-driven recirculation pump. The speed of the recirculation pump can be varied to allow some control of reactor power level through the effects of coolant flowrate on moderator void content.

1.2.3.4 Residual Heat Removal (RHR) System

The RHR system is a system of pumps, heat exchangers, and piping that fulfills the following functions:

A. Removal of decay and sensible heat from the reactor core during and after shutdown.

B. Removal of stored and decay heat from the reactor core following a design basis loss-of-coolant accident (LOCA). This system is discussed in subsection 1.2.7.

C. Removal of heat from the primary containment following a LOCA in order to limit the increase in primary containment pressure. This is accomplished by cooling and recirculating the water inside the primary containment. The redundancy of the equipment provided for containment cooling is further extended by a separate part of the RHR system which sprays cooling water into the containment. This latter capability is discussed in paragraph 1.2.7.12.

1.2.3.5 Reactor Water Cleanup (RWC) System

A RWC system, which includes a filter-demineralizer arrangement, is provided to clean up the reactor water and to reduce the amounts of activated corrosion products in the water.

1.2.4 POWER CONVERSION SYSTEMS (HNP-1 AND HNP-2)

To produce electrical power, each unit utilizes a power conversion system which includes a turbine-generator, a turbine bypass system, a main condenser, air ejectors, air ejector condensers, a turbine gland-seal condenser, condensate demineralizers, and a feedwater heating and pumping system. The steam comes from the reactor, drives the turbine-generator, and is exhausted to the condenser. The deaerated condensate is demineralized prior to regenerative heating necessary for its return as feedwater to the reactor. The heat rejected in the main condenser is removed by a closed-loop circulating water system utilizing cooling towers as a heat sink. Figures 1.2-5 and 1.2-7 show the turbine-generator heat balance at rated load for HNP-2 and HNP-1, respectively.

1.2.4.1 Turbine-Generator (HNP-1 and HNP-2)

The turbine is a GE 1800-rpm, tandem-compound, 4-flow reheat unit with 43-in. last-stage buckets. It has a double-flow high-pressure section and two double-flow low-pressure sections. Exhaust steam from the high-pressure section passes through moisture separators and a two-stage reheater before entering the low-pressure sections of the unit. The turbine has six extraction stages for reactor feedwater system heating. Steam to drive the reactor feed pump turbines is taken from the steam path immediately after reheating but prior to passing through the low-pressure sections of the main turbine. The turbine controls include a speed governor, steam admission (control) valves, emergency stop valves, combined intermediate valves, and redundant initial pressure regulators.

The generator is a direct coupled, 60-Hz, 24,000-V synchronous unit with a water-cooled stator and a hydrogen-cooled rotor (field).

1.2.4.2 Turbine Bypass System (HNP-1 and HNP-2)

A bypass system is provided to allow passing of steam from the reactor directly to the main condenser under control of the initial pressure regulator. Steam is bypassed to the condenser whenever the steam generation rate exceeds the flowrate corresponding to the load connected to the turbine-generator. The bypass system would, for example, be used during generator synchronization or rejection of a large electrical load. The system has the capacity to pass ~ 21% for HNP-1 and ~ 20% for HNP-2 of the 100% RTP steam flowrate.

1.2.4.3 Main Condenser (HNP-1 and HNP-2)

The main condenser is of the single-pass, divided-water-box, deaerating type. It consists of two shells, one for each low- pressure turbine section. The Unit 2 hotwell is designed to provide a minimum condensate retention time of ~ 2 min., permitting decay of short-lived radioactive isotopes. Deaeration is provided in the condenser for removal of the dissolved gases from the condensate.

1.2.4.4 Main Condenser Air Ejector (HNP-1 and HNP-2)

Two twin, three-stage steam jet air ejectors (SJAEs) of 100% percent redundant capacity, complete with inter- and after-condensers, are provided for evacuating gases from the turbine and the main condenser during normal operation. One mechanical vacuum pump is provided to remove gases from the main condenser during startup and shutdown when steam is not available for the SJAEs.

1.2.4.5 Turbine Steam Sealing System (HNP-1 and HNP-2)

The turbine sealing system provides steam to the seals on the turbine valve packings and the turbine shaft packings at a pressure slightly above atmospheric. This system collects and

condenses the sealing steam and discharges air leakage to the gland holdup system. The holdup system serves mainly to allow short half-life radioactive gases to decay before being discharged to the main stack.

1.2.4.6 Circulating Water System (HNP-1 and HNP-2)

Two vertical, one-half capacity, removable-element, circulating water pumps located in the circulating water pump structure provide a continuous supply of condenser cooling water. The water is pumped from and returned to the mechanical draft cooling towers where the temperature of the water is reduced before being recirculated through the main condenser. Evaporation, drift, and blowdown losses are compensated for by makeup water taken from the Altamaha River.

1.2.4.7 Condensate Demineralizer System (HNP-1 and HNP-2)

The condensate demineralizer system consists of seven filter-demineralizers, including one spare, connected with associated piping and valving for parallel operation. Instrumentation and controls are provided to ensure proper operation and to protect against malfunction of the equipment. The system is designed to maintain a high purity reactor feedwater quality by removing ionic and particulate materials from the feedwater system.

1.2.4.8 Condensate and Reactor Feedwater Systems (HNP-1 and HNP-2)

The condensate and reactor feedwater system takes condensate from the main condenser and after six stages of heating delivers it to the reactor.

Condensate is pumped by three motor-driven vertical pumps through the steam jet air ejector inter- and after-condensers and the turbine gland-seal condenser. After leaving the turbine gland-seal condenser, the condensate passes through a full flow condensate demineralizer and the off-gas condenser. The purified condensate is then boosted in pressure by three motor-driven condensate booster pumps. The condensate is then divided into two parallel streams, each with five stages (four for HNP-1) of low-pressure feedwater heating. The feedwater is then boosted in pressure by two turbine-driven reactor feed pumps. The flow from the two reactor feed pumps is then divided into two parallel streams, each with one stage of high-pressure feedwater heating. The feedwater then flows in two streams to the reactor.

1.2.4.9 CROSSFLOW^{TM} System (HNP-1 and HNP-2)

The reactor feedwater systems are equipped with high accuracy, ultrasonic flow measurement devices that provide a correction to the feedwater flow venturi measurement used in the reactor heat balance and core thermal power computation. The accuracy of the CROSSFLOW^{TM} system reduces the overall power measurement uncertainty to within 0.5% and supports a 1.5% power uprate from 2763 MWt to the new rated power level of 2804 MWt. A more detailed description of the application of this system is provided in FSAR paragraph 7.6.8.3.7.3.

1.2.5 ELECTRICAL POWER SYSTEM (HNP-1 and HNP-2)

The generator for HNP-2 is connected to the 24-526-kV main step-up transformer with an isolated phase bus. The transformer is connected to the plant switchyard, which has 500-kV and 230-kV system connections and a 500/230-kV autotransformer. The HNP-1 generator is connected to the 22.8-230-kV main step-up transformer bus. This transformer is connected to the plant switchyard which has 230-kV system connections.

1.2.6 RADIOACTIVE WASTE SYSTEMS (HNP-1 and HNP-2)

The radioactive waste systems are designed to control the release of station-produced radioactive material to within the limits specified in 10 CFR 20.1001 - 20.2402. This is done by various methods such as collection, filtration, holdup for decay, and dilution. The methods employed for the controlled release of these contaminants are dependent primarily upon the state of the material: liquid, solid, or gaseous.

1.2.6.1 Liquid Radwaste System

The liquid radioactive waste control system collects, treats, stores, and disposes of all radioactive liquid wastes. These wastes are collected in sumps and drain tanks at various locations throughout the plant and then transferred to the appropriate collection tanks in the radwaste building for treatment, storage, dilution, and disposal, as necessary. Wastes are processed on a batch basis. Processed liquid wastes may be returned to the condensate system or discharged to the river by way of the discharge pipe. The liquid wastes in the discharge pipe are diluted with water from the cooling tower blowdown and with that portion of the service water in excess of normal cooling tower makeup in order to achieve a permissible concentration at the site boundary.

Equipment is selected, arranged, and shielded to permit operation, inspection, and maintenance with minimum personnel exposure. For example, tanks and processing equipment which are expected to contain significant radiation sources are located behind shielding; and similarly, sumps, pumps, instruments, and valves are located in controlled-access rooms or shielded spaces. Processing equipment is designed to require a minimum of maintenance.
Protection against accidental discharge of liquid radioactive waste is provided by an automatic shutoff valve via a high-radiation signal from a radiation monitor.

1.2.6.2 Solid Radwaste System

With the solid radwaste system, solid radioactive wastes are collected, processed, and packaged for storage. Generally, these wastes are stored onsite until the short half-lived activities are insignificant so as to permit economical shielding and shipping. Process solid wastes are collected, dewatered, and otherwise prepared for storage in containers for offsite shipment. Examples of these solid wastes are:

- Filter residue.
- Spent resins.
- Paper.
- Air filter elements.
- Used clothing.

Solid wastes from equipment originating in the nuclear system, such as control rods, fuel channels, incore detectors, etc., are stored for radioactive decay in the fuel storage pool and later prepared for offsite shipment or for storage at the onsite low-level radwaste (LLRW) storage facility.

1.2.6.3 Gaseous Radwaste System (GRS)

The GRS collects, processes, and delivers gases from the main condenser air ejector, startup vacuum pump, and gland-seal condenser to the main stack for elevated release to the atmosphere. Noncondensable radioactive gases are removed from the main condenser by the air ejector. The normal condenser offgas system uses a high-temperature catalytic recombiner to recombine radiolytically disassociated hydrogen and oxygen from the air ejector system. After cooling to strip the condensables and reduce the volume, the remaining noncondensables are delayed in the 30-min holdup system. Essentially, the 30-min decay time allows the noble gases with short half-lives to decay completely to solid daughters. The biologically significant decay chains containing Sr-89, Sr-90, Ba-140, and Cs-137 decay to these solids, which are removed by the high-efficiency off-gas filters. As a result of these process actions and the fact that halogens remain principally in the reactor coolant and are removed by the reactor cleanup demineralizer system, radioactive particles and halogens are not released from the off-gas system in significant amounts. Holdup in this system provides ample time to prevent release of fission product gases in excess of the permissible main stack release rate limits.

During startup, air is removed from the main condenser by the mechanical vacuum pump and is discharged to the main stack via the gland-seal holdup system.

The gland-seal condenser is exhausted by a blower into shielded piping which provides 1.75-min holdup to reduce the activity of short-lived radioactive gases (N-16 and O-19), which are then discharged to the main stack.

1.2.7 NUCLEAR SAFETY SYSTEMS AND ENGINEERED SAFEGUARDS (HNP-1 AND HNP-2)

1.2.7.1 Reactor Protection System (RPS) (HNP-1 and HNP-2)

The RPS initiates a rapid, automatic shutdown (scram) of the reactor. This action is taken in time to prevent fuel-cladding damage and any nuclear system process barrier damage following abnormal operational transients. The RPS overrides all operator actions and process controls.

1.2.7.2 Neutron Monitoring System (NMS) (HNP-1 and HNP-2)

Those portions of the NMS that provide high neutron flux signals to the RPS qualify as a nuclear safety system. The intermediate range monitors (IRMs) and average power range monitors (APRMs), which monitor neutron flux via incore detectors, signal the RPS to scram in time to prevent excessive fuel cladding damage as a result of overpower transients. The oscillation power range monitor (OPRM) detects thermal-hydraulic oscillations and provides indication and annunciation in the MCR.

1.2.7.3 Control Rod Drive System (HNP-1 and HNP-2)

When a scram is initiated by the RPS, it is the CRD system that inserts the negative reactivity necessary to shut down the reactor. Each control rod is controlled individually by a hydraulic control unit. When a scram signal is received, high-pressure water from an accumulator for each rod rapidly forces each control rod into the core.

1.2.7.4 Pressure Relief System (HNP-1 and HNP-2)

A pressure relief system consisting of safety relief valves mounted on the main steam lines is provided to prevent excessive pressure inside the nuclear system following either normal operations, abnormal operational transients, or accidents.

1.2.7.5 Reactor Core Isolation Cooling (RCIC) System (HNP-1 and HNP-2)

The RCIC system provides makeup water to the reactor vessel whenever the vessel is isolated. The RCIC system uses a steam-driven turbine-pump unit and operates automatically in time and with sufficient coolant flow to maintain adequate reactor vessel water level.

1.2.7.6 Primary Containment (HNP-1 and HNP-2)

A pressure suppression primary containment houses the reactor vessel, the reactor coolant recirculating loops, and other branch connections of the reactor primary system. The primary containment system consists of the following:

- A drywell.
- A pressure suppression chamber which stores a large volume of water.
- A connecting vent system between the drywell and the pressure suppression pool.
- A vacuum relief system.
- Isolation valves.
- Containment cooling systems.
- Other service equipment.

In the event of a process system piping failure within the drywell, reactor water and steam are released into the drywell air space. The resulting increased drywell pressure then forces a mixture of air, steam, and water through the vent system into the pool of water which is stored in the suppression chamber. The steam condenses in the suppression pool, resulting in a rapid pressure reduction in the drywell. Air which is transferred to the suppression chamber pressurizes the suppression chamber and is subsequently vented to the drywell to equalize the pressure between the two vessels. Cooling systems are provided to remove heat from the reactor core, from the drywell, and from the water in the suppression chamber, thus providing continuous cooling of the primary containment under accident conditions.

Appropriate isolation valves are actuated during this period to ensure containment of radioactive materials within the primary containment which otherwise might be released from the reactor during the course of the accident.

1.2.7.7 <u>Primary Containment and Reactor Vessel Isolation Control Systems</u> (HNP-1 and HNP-2)

The primary containment and reactor vessel isolation control systems automatically initiate the closure of isolation valves to close off all potential leakage paths for radioactive material to the environs. This action is taken upon indication of a potential breach in the nuclear system process barrier.

1.2.7.8 Secondary Containment (HNP-1 and HNP-2)

The reactor building is designed as a low in-leakage, elevated-release, secondary containment system which houses the primary containment system, refueling facilities, and most of the components of the nuclear steam supply system. The secondary containment system provides secondary containment when the primary containment system is closed and in service; it also provides primary containment when the primary containment system is open, as in refueling. The secondary containment system consists of the reactor building, standby gas treatment system (SGTS), reactor building isolation control system, and main stack.

In the event of a postulated pipe break inside the drywell or a fuel-handling accident, the reactor building is isolated by the reactor building isolation control system to provide a low leakage barrier. The SGTS is initiated by the same conditions that isolate the reactor building. The SGTS exhausts air from the reactor building to maintain a reduced pressure within the reactor building relative to the outside atmosphere. The system also treats the air to remove particulates and iodines and releases the air through the elevated release point, the main stack.

1.2.7.9 Main Steam Line Isolation Valves (MSIVs) (HNP-1 and HNP-2)

Although all pipelines which both penetrate the primary containment and offer a potential release path for radioactive material are provided with redundant isolation capabilities, the main steam lines, because of their large size and large mass flowrates, are given special isolation consideration. Two automatic isolation valves, each powered by both air pressure and spring force, are provided in each main steam line. These valves fulfill the following objectives:

A. They prevent excessive damage to the fuel barrier by limiting the loss of reactor coolant from the reactor vessel resulting from either a major leak from the steam piping outside the primary containment or from a malfunction of the pressure control system resulting in excessive steam flow from the reactor vessel.

B. They limit the release of radioactive materials by closing the nuclear system process barrier in case of a gross release of radioactive materials from the fuel to the reactor cooling water and steam.

C. They limit the release of radioactive material by closing the primary containment barrier in case of a major leak from the nuclear system inside the primary containment.

1.2.7.10 Main Steam Line Flow Restrictors (HNP-1 and HNP-2)

A venturi-type flow restrictor is installed in each steam line close to the reactor vessel. These devices limit the loss of coolant from the reactor vessel before the MSIVs are closed in case of a main steam line break outside the primary containment and prevent uncovering of the core.

1.2.7.11 Emergency Core Cooling System (ECCS) (HNP-1 and HNP-2)

Four ECCS subsystems are provided to prevent fuel cladding melting in the event of a breach in the nuclear system process barrier that results in a loss of reactor coolant. The four ECCS subsystems are:

- High-pressure coolant injection (HPCI).
- Automatic depressurization.
- CS.
- Low-pressure coolant injection (LPCI) - an operating mode of the RHR system.

The ECCS initiation and control instrumentation is divided in two parts: the incident detection circuitry (IDC) and the control instrumentation. The IDC includes those channels which detect a need for core cooling systems operation and the corresponding trip systems which initiate the proper ECCS response.

1.2.7.11.1 High-Pressure Coolant Injection (HPCI) System (HNP-1 and HNP-2)

The HPCI system provides and maintains an adequate coolant inventory inside the reactor vessel to prevent fuel cladding melting as a result of postulated small breaks in the nuclear system process barrier, with the exception of the HPCI steam line. In the case of breaks in the HPCI steam line, other systems are available for accident mitigation. A high-pressure system is needed for such breaks because the reactor vessel depressurizes slowly, preventing low-pressure systems from injecting coolant. The HPCI system includes a turbine-pump powered by reactor steam.

1.2.7.11.2 Automatic Depressurization System (HNP-1 and HNP-2)

The ADS acts to rapidly reduce reactor vessel pressure during postulated accident situations in which the HPCI system fails to automatically maintain reactor vessel water level. The depressurization provided by the system enables the LPCI and CS systems to deliver cooling water to the reactor vessel. The ADS uses some of the safety relief valves which are part of the nuclear system pressure relief system. The ADS is nitiated automatically upon conditions of high drywell pressure and low reactor vessel water level, provided either the CS system or the LPCI cooling mode of RHR is available for core cooling. In the event the reactor pressure vessel level 1 signal is present for 13 min without a concurrent high drywell pressure signal, the high drywell pressure is bypassed and the ADS timer sequence starts.

1.2.7.11.3 Core Spray System (HNP-1 and HNP-2)

The CS system consists of two independent pump loops that deliver cooling water to spray spargers over the core. The system is actuated by conditions indicating that a breach exists in the nuclear system process barrier, but water is delivered to the core only after reactor vessel pressure is reduced. This system provides the capability to cool the fuel by spraying water onto the core. Either CS loop is capable of preventing fuel cladding melting following a LOCA.

1.2.7.11.4 Low-Pressure Coolant Injection (HNP-1 and HNP-2)

The LPCI is an operating mode of the RHR system but is discussed here because the LPCI mode acts as an engineered safeguard in conjunction with the other standby cooling systems. LPCI uses the pump loops of the RHR system to inject cooling water at low pressure into a reactor recirculation loop. LPCI is actuated by conditions indicating a breach in the nuclear system process barrier, but water is delivered to the core only after reactor vessel pressure is reduced. LPCI operation, together with the core shroud and jet pump arrangement, provides the capability of core reflooding following a LOCA in time to prevent fuel clad melting.

1.2.7.12 Containment Spray and Suppression Pool Cooling (HNP-1 and HNP-2)

The suppression pool cooling subsystem of RHR is placed in operation to limit the temperature of the water in the suppression pool following a design basis LOCA. In the suppression pool cooling mode of operation, the RHR main system pumps take suction from the suppression pool and pump the water through the RHR heat exchangers where cooling takes place by transferring heat to the station cooling systems. The fluid is then discharged back to the suppression pool.

Another portion of the RHR system is provided to spray water into the containment as an augmented means of removing energy from the containment following a LOCA. This capability is in excess of the required energy removal capability and can be placed into service at the discretion of the operator.

1.2.7.13 (Deleted)

1.2.7.14 Control Rod Velocity Limiter (HNP-1 and HNP-2)

A control rod velocity limiter is attached to each control rod to limit the velocity at which a control rod can fall out of the core should it become detached from its CRD. The rate of reactivity insertion resulting from a rod drop accident is limited by this action. The limiters contain no moving parts.

1.2.7.15 <u>Control Rod Drive Housing Supports</u> (HNP-1 and HNP-2)

CRD housing supports are located beneath the reactor vessel near the control rod housings. The supports limit the travel of a control rod in the event a control rod housing is ruptured. The supports prevent a nuclear excursion as a result of a housing failure, thus protecting the fuel barrier.

1.2.7.16 <u>Standby Electric Power Systems</u> (HNP-1 and HNP-2)

The plant is designed to shut down safely and maintain a safe condition on complete loss of offsite electrical power. Standby power is supplied by diesel generators after shutdown to provide auxiliary cooling, lighting, and miscellaneous services to permit communication and access to all plant areas and also to ensure continued removal of decay heat.

1.2.7.17 <u>dc Power Supply</u> (HNP-1 and HNP-2)

In the event that normal plant power sources become unavailable, the plant battery system provides power for all controls vital to plant safety and for those functions required for a safe shutdown, such as:

- Closing of isolation valves.
- Operation of valves required for core cooling.
- Providing minimum required lighting.
- Providing minimum instrumentation.
    - Control rod position indicators.
    - Neutron channel to monitor the core during shutdown.

1.2.7.18 <u>Plant Service Water (PSW) System</u> (HNP-1 and HNP-2)

Each PSW system consists of four, one-third-capacity wet pit service water pumps which are located in the river intake structure and of distribution piping and controls. There is also one diesel standby service water pump. Portions of the system, including the pumps which are required for emergency cooling, are designed as a Seismic Category I system and meet the single-failure criteria.

1.2.7.19 Residual Heat Removal Service Water (RHRSW) System (HNP-1 and HNP-2)

Each RHRSW system supplies river water to the RHR heat exchangers to remove heat during both normal and accident conditions. The system consists of two independent loops, each with two pumps, and of the associated valves and piping. The RHRSW system pumps are located near the river in the Seismic Category I intake structure. A steel barrier exists between division 1 and division 2 of the RHRSW system pumps to provide protection from jet impingement to the RHRSW pump motors and associated equipment.

1.2.7.20 Main Steam Line Radiation Monitoring System (HNP-1 and HNP-2)

Each main steam line radiation monitoring system consists of gamma radiation monitors located externally to the main steam lines just outside the primary containment. The monitors are designed to detect a release of fission products from the fuel.

1.2.7.21 Reactor Building Isolation and Control System (HNP-1 and HNP-2)

Each reactor building isolation and control system consists of a number of radiation monitors arranged in the reactor building exhaust duct to monitor the activity level of the ventilation exhaust from the reactor building. The monitors are designed to detect release of fission products from the nuclear system. Upon detection of high radiation, the trip signals generated by the monitors are used to isolate the reactor building and to actuate the SGTS.

1.2.7.22 Containment Atmospheric Dilution (CAD) System (HNP-1 and HNP-2)

A CAD system is used to control the concentration of hydrogen and oxygen that may be generated in the drywell and torus following a postulated LOCA. The CAD system controls the combustible gases within the containment by diluting the combustible gases.

1.2.7.23 Main Control Room Environmental Control (MCREC) System (HNP-1 and HNP-2)

The MCREC system supplies heating, ventilation, and air-conditioning (HVAC) for the main control room during normal operating and accident conditions. The system consists of three 50% capacity air handling units with electric heaters, cooling coils and fans, two exhaust air fans, and two high-efficiency air filtration units complete with charcoal adsorbers. Signals from two monitors located in the outside air intake duct and inside the main control room automatically terminate the supply of outside air when conditions of high radiation exist. The main control room air is then recirculated through the air filtration ducts. The operator can manually bring outside air back into the main control room through the filter units.

1.2.7.24 Equipment Area Cooling Systems (HNP-1 and HNP-2)

Equipment area cooling systems are provided to maintain the local environment of specific areas at temperatures within normal operating temperatures for electrical components located within these areas. The system consists of a number of fan-coil coolers. The PSW system supplies water to the cooling coils and serves as the heat sink for the equipment area cooling systems.

1.2.7.25 Low-Low Set (LLS) Relief Logic System (HNP-2)

The LLS relief logic system extends the time between subsequent safety relief valve (SRV) actuations during a small- or intermediate-break LOCA. This action is taken to mitigate the postulated thrust loads and the effects of high-frequency loads on the torus shell caused by subsequent SRV actuations.

1.2.8 SPECIAL SAFETY SYSTEMS (HNP-1 AND HNP-2)

1.2.8.1 Standby Liquid Control System (HNP-1 and HNP-2)

Although not intended to provide prompt reactor shutdown, the SLCS provides a redundant, independent, and different way for the control rods to bring the nuclear fission reaction to subcriticality and to maintain subcriticality as the reactor cools. The system makes possible an orderly and safe shutdown in the event that not enough control rods can be inserted into the reactor core to accomplish shutdown in the normal manner. The system is sized to counteract the positive reactivity effect from full power to the cold shutdown condition.

As part of the implementation of an alternative source term (AST) (reference subsection 15.1.11), a new design function was added for SLCS to buffer the suppression pool by injection of a sufficient amount of sodium pentaborate solution to the suppression pool to prevent iodine re-evolution following a LOCA.

1.2.8.2 Shutdown Capability Outside the Main Control Room (HNP-1 and HNP-2)

Sufficient equipment and controls are available outside the main control room to enable an operator to shut down the reactor and maintain it in a safe condition if access to the control room is lost.

1.2.9 PROCESS CONTROL AND INSTRUMENTATION (HNP-1 AND HNP-2)

1.2.9.1 Nuclear System Process Control and Instrumentation

1.2.9.1.1 Reactor Manual Control System (RMCS) (HNP-1 and HNP-2)

The RMCS provides the means by which control rods are manipulated from the control room for gross power control. The system controls valves in the control rod drive hydraulic system. Only one control rod can be manipulated at a time. The RMCS includes the controls that restrict control rod movement--rod block--under certain conditions as a backup to procedural controls.

1.2.9.1.2 Recirculation Flow Control System (RFCS) (HNP-1 and HNP-2)

The RFCS controls the speed of the reactor recirculation pumps. Adjusting the pump speed changes the coolant flowrate through the core. This effects changes in core power level. The system is designed to allow manual adjustment of reactor power output to the load demand by adjusting the frequency of the electrical power supply for the reactor recirculation pumps.

1.2.9.1.3 Neutron Monitoring System (HNP-1 and HNP-2)

The NMS is a system of incore neutron detectors and out-of-core electronic equipment. The system provides indication of neutron flux which can be correlated to thermal power level for the entire range of flux conditions that may exist in the core. The source range monitors (SRMs) and the IRMs provide flux level indications during reactor startup and low-power operation. The local power range monitors (LPRMs) and the APRMs allow assessment of local and overall flux conditions during power range operation. Rod block monitors are provided to prevent rod withdrawal when reactor power should not be increased at the existing reactor coolant flowrate. The traversing incore probe subsystem provides a means to calibrate the LPRM system.

1.2.9.1.4 Refueling Interlocks (HNP-1 and HNP-2)

A system of interlocks that restricts the movements of refueling equipment and control rods when the reactor is in the refueling mode is provided to prevent an inadvertent criticality during refueling operations. The interlocks back up procedural controls that have the same objective. The interlocks affect the following:

- Refueling platform.
- Refueling platform hoists.
- Fuel grapple.

- Control rods.

1.2.9.1.5 Reactor Vessel Instrumentation (HNP-1 and HNP-2)

In addition to instrumentation provided for the nuclear safety systems and engineered safeguards, instrumentation is provided to monitor and transmit information that can be used to assess conditions existing inside the reactor vessel and the physical condition of the vessel itself. The provided instrumentation monitors the following:

- Reactor vessel pressure.
- Water level.
- Surface temperature.
- Internal differential pressures.
- Coolant flowrates.
- Top head flange leakage.

1.2.9.1.6 Process Computer System (HNP-1 and HNP-2)

An online process computer is provided to monitor and log process variables and to make certain analytical computations.

1.2.9.1.7 Rod Worth Minimizer (HNP-1 and HNP-2)

The rod worth minimizer prevents rod withdrawal under low-power conditions if the rod to be withdrawn is not in accordance with a preplanned pattern. The effect of the rod block is to limit the reactivity worth of the control rods by enforcing adherence to the preplanned rod pattern. The nuclear measurement analysis and control rod worth minimizer (NUMAC-RWM) enhanced rod position indicating system (RPIS) application is described in GE Topical Report NEDO-31146.

1.2.9.2 <u>Power Conversion Systems Process Control and Instrumentation</u> (HNP-1 and HNP-2)

1.2.9.2.1 Pressure Regulator and Turbine Control

The pressure regulator controls both the turbine admission control valves and the turbine bypass valves and maintains constant reactor pressure. Pressure regulation is coordinated with

the turbine speed and the load control systems. The turbine control utilizes an electrohydraulic control system arranged for remote operation.

1.2.9.2.2 Feedwater Control System

A three-element controller is used to regulate the feedwater system so that proper water level is maintained in the reactor vessel. The controller uses main steam flowrate, reactor vessel water level, and feedwater flowrate signals. The feedwater control signal is used to regulate the speed of the turbine-driven reactor feed pumps to adjust flow.

1.2.9.3 Electrical Power System Process Control (HNP-1 and HNP-2)

To prevent loss of system integrity due to an electrical failure of a component, high-speed automatic protective relaying is utilized to isolate the faulted component from the remainder of the system.

1.2.9.4 Radiation Monitoring and Control (HNP-1 and HNP-2)

1.2.9.4.1 Process Radiation Monitoring (HNP-1 and HNP-2)

Radiation monitors are provided on various lines to monitor either for radioactive materials released to the environs via process liquids and gases or for process system malfunctions. Monitors are provided in the following subsystems:

- Air ejector off-gas.
- Main stack.
- Reactor building ventilation.
- Main steam line.
- Process and cooling liquids.
    - Radwaste.
    - Service water.
    - RBCCW.
    - HVAC systems.

1.2.9.4.2 Area Radiation Monitors

A number of radiation monitors are provided to monitor for abnormal radiation at various locations in the reactor building, the turbine building, the radwaste building, and the main control room. These monitors annunciate alarms when abnormal radiation levels are detected.

1.2.9.4.3 Fission Products Monitoring System

The fission products monitoring system is installed to monitor samples from the primary containment atmosphere for radiation from air particulates, radioactive iodine, and noble gases. A three-channel monitor is provided for each function. The activity from each is displayed on a log rate meter located in the control room.

1.2.9.4.4 Liquid Radwaste System Control (HNP-1 and HNP-2)

The liquid radwaste system collects, treats, and stores liquid radioactive wastes on a batch basis with protection against accidental discharge provided by the design and supplemented by the procedural controls. Liquid wastes are discharged on a batch basis at a controlled rate after sampling and laboratory analysis. Instrumentation is provided with alarms to detect abnormal radioactivity concentration in the liquid radwaste discharges and to automatically isolate the liquid radwaste discharge.

1.2.9.4.5 Solid Radwaste Control (HNP-1 and HNP-2)

The solid radwaste system collects, treats, and stores solid radioactive wastes for offsite shipment. Wastes are handled on a batch basis. Radiation levels of the various batches are controlled by the operator.

1.2.9.4.6 Gaseous Radwaste System Control (HNP-1 and HNP-2)

The GRS is continuously monitored by the main stack radiation monitor and the air ejector off-gas radiation monitor. A high level signal from the air ejector off-gas radiation monitoring system, after an appropriate time delay, automatically isolates the off-gas system by closing the isolation valves between the air ejector system and the main stack.

1.2.10 AUXILIARY SYSTEMS (HNP-1 AND HNP-2)

1.2.10.1 Auxiliary ac Power (HNP-1 and HNP-2)

The unit auxiliary transformers are located outside the turbine building and are connected to the generator bus utilizing isolated phase bus. The plant startup auxiliary transformers are located

outside the turbine building and are connected to the transmission system via the 230-kV main switchyard.

The auxiliary ac power system utilizes 4160-V metal-enclosed switchgear, 600-V metal-enclosed switchgear and motor control centers, and 120/208-V metal-enclosed switchgear and distribution cabinets. All switchgear and vital, essential distribution cabinets have two separate sources of supply.

1.2.10.2 Plant Service Water System (HNP-1 and HNP-2)

Each PSW system consists of four, one-third capacity vertical wet pit service water pumps located in the river intake structure, distribution piping, and controls. Three service water pumps are required for normal operation. Three service pumps are required for plant startup, while only one pump is required for shutdown and emergency shutdown. The pumps are controlled so that if the operating pumps cannot maintain the required system pressure the standby pump or pumps start automatically.

1.2.10.3 Fire Protection System (FPS) (HNP-1 and HNP-2)

The FPS is designed to provide an adequate supply of water or chemicals to points throughout the plant area where fire fighting may be required. The water for the system is taken from two 300,000-gal tanks which are replenished automatically from deep wells. In addition to the tanks, the system consists of one electric-motor driven pump, two diesel engine pumps, one jockey-booster-pump and associated valves, piping, and hydrants. The necessary instrumentation and controls are provided to ensure proper operation of the water FPS. This system is shared by both units.

Chemical fire fighting systems are also provided as additions to or in lieu of the water fire fighting system.

1.2.10.4 Drywell Pneumatic System (HNP-1 and HNP-2)

The drywell pneumatic system provides gas of suitable quality and pressure to supply the equipment within the drywell requiring motive gas. The gas receiver storage capacity is adequate to supply equipment with gas for a minimum period of 10 min in the event that none of the supplies of nitrogen to the drywell pneumatic system are available.

1.2.10.5 Torus Drainage and Purification System (HNP-1 and HNP-2)

The torus drainage and purification system provides capability for the cleanup of suppression pool waters through a process of filtration and demineralization. The torus drainage and cleanup system can also be used in reducing or completely draining the suppression pool waters to allow inspection of the torus interior surface coating. See subsection 9.3.7 for system

design and description, and subsection 18.3.3 for a description of the protective coatings program.

1.2.10.6 Heating, Ventilation, and Air-Conditioning Systems (HNP-1 and HNP-2)

The station HVAC systems provide appropriate ambient temperature and environmental conditions for station operating personnel and equipment. Normal airflow is routed from lesser to progressively greater areas of contamination potential prior to final exhaust. The arrangement is ensured by a positive air pressure in the clean areas and a negative air pressure in the potentially contaminated areas.

1.2.10.7 New and Spent-Fuel Storage (HNP-1 and HNP-2)

New fuel is stored in a dry storage vault located adjacent to the spent-fuel pool area in the reactor building. Transport of spent fuel and irradiated channels during refueling is handled under water. Spent fuel is stored under water in the spent-fuel pool in the reactor building and at the ISFSI in dry storage casks until prepared for shipment from the site.

1.2.10.8 Fuel Pool Cooling and Filtering System (HNP-1 and HNP-2)

The spent-fuel pool cooling and demineralizer system is provided to clean the pool water and remove decay heat from the spent fuel stored in the spent fuel storage pool.

1.2.10.9 Service and Instrument Air Systems (HNP-1 and HNP-2)

The service and instrument air systems are supplied oil-free air by one 700-sf^3/min and two 500-sf^3/min compressors connected in parallel. Except for the drywell pneumatic system, diesel starting air, and low pressure service air, the 700-sf^3/min compressor normally supplies all compressed air requirements for service air and for the pneumatic instruments and controls throughout the plant. One of the 500-sf^3/min compressors is an automatic standby, and the other 500-sf^3/min compressor serves as the backup. Two low-pressure air blowers provide oil-free air for other low-pressure service requirements. A separate air system is provided to start the emergency diesel generators. Another independent system provides instrument air inside the drywell. The drywell pneumatic system provides instrument motive gas inside the drywell and is normally isolated from the instrument air system.

1.2.10.10 Makeup Water Treatment System (HNP-1 and HNP-2)

The makeup water treatment system is designed to maintain a supply of treated water suitable as makeup for the station and the reactor coolant cycles and as makeup for other demineralized water requirements. Well water is processed through a filter-demineralizer and stored in a 100,000-gal demineralized water storage tank for use as needed. Other components of the system include:

- Two 100% capacity pumps.
- Valves.
- Piping.
- Necessary instrumentation and controls to ensure proper operation of the equipment.

This system is shared by both units.

1.2.10.11 Potable and Sanitary Water System (HNP-1 and HNP-2)

The potable and sanitary water system provides water for drinking and sanitary purposes. Well water is filtered and treated to meet all applicable drinking water standards. Shower and lavatory waste water that does not contain radioactive material is directed to a sewage treatment system. This system is shared by both units.

1.2.10.12 Plant Equipment and Floor Drainage System (HNP-1 and HNP-2)

The plant equipment and floor drainage system is provided to collect and remove waste liquids from their points of origin and carry them to a suitable area for cleanup and disposal. Wastes are collected in the building sumps and pumped to the radwaste system for cleanup and eventual reuse or discharge. Section 10.13 of the HNP-1 FSAR provides further information related to the plant equipment and floor drainage systems.

1.2.10.13 Reactor Building Closed Cooling Water System (HNP-1 and HNP-2)

The RBCCW system is provided to supply a self-contained coolant to the reactor auxiliary system's equipment and accessories for heat removal during normal operating and shutdown conditions. The system consists of the following:

- Cooling loop containing three 50% capacity pumps.
- Two 100% capacity heat exchangers.
- Chemical addition tank.
- Surge tank.
- Associated valves and piping.

The RBCCW system is monitored continuously for radioactivity by the process radiation monitoring system.

1.2.10.14 Process Sampling System (HNP-1 and HNP-2)

The plant process sampling system is provided to monitor the quality of plant process flows. Information required for making operational decisions is obtained from analysis of samples from pertinent system streams.

1.2.10.15 Plant Communication System (HNP-1 and HNP-2)

Plant communications are provided through three independent systems: a public address system, a dial telephone system, and a two-way radio system. The public address and dial telephone systems are designed so that power can be provided by emergency diesel generators on loss of normal ac power.

1.2.10.16 Plant Lighting System (HNP-1 and HNP-2)

The plant lighting system consists of normal ac lighting equipment and emergency ac lighting equipment. In the event that ac power is lost, the emergency lighting equipment is automatically transferred to the station battery. This transfer to a dc power source ensures lighting continuity in the critical areas of the plant.

1.2.11 SHIELDING (HNP-1 AND HNP-2)

Shielding implemented by occupancy requirements in the various areas of the station is provided to meet the limits of applicable regulations.

1.2.12 IMPLEMENTATION OF LOADING CRITERIA (HNP-1 AND HNP-2)

Structures and equipment are designed to resist structural and mechanical damage due to loads produced by environmental and thermal forces. For the purpose of categorizing mechanical strength designs for these loads, the following definitions are established:

- Seismic Category I.
- Other than Seismic Category I.

1.2.12.1 Seismic Category I (HNP-1 and HNP-2)

This class includes those structures, pieces of equipment, and components whose failure or malfunction might cause or increase the severity of an accident which would endanger public health and safety. This category includes those structures, pieces of equipment, and components which are required for safe shutdown and isolation of the reactor.

1.2.12.2 Other Than Seismic Category I (HNP-1 and HNP-2)

This category includes those structures, pieces of equipment, and components which are important to reactor operation but are not essential for the mitigation of the consequences of these accidents. This category does not degrade the integrity of any item designated as Seismic Category I.

<u>DOCUMENTS INCORPORATED BY REFERENCE INTO THE FSAR</u>

"GESTAR II - General Electric Standard Application for Reactor Fuel," <u>NEDE-24011-P-A</u>.

TABLE 1.2-2 (SHEET 1 OF 2)

ORIGINAL AND UPRATED REACTOR OPERATING CONDITIONS

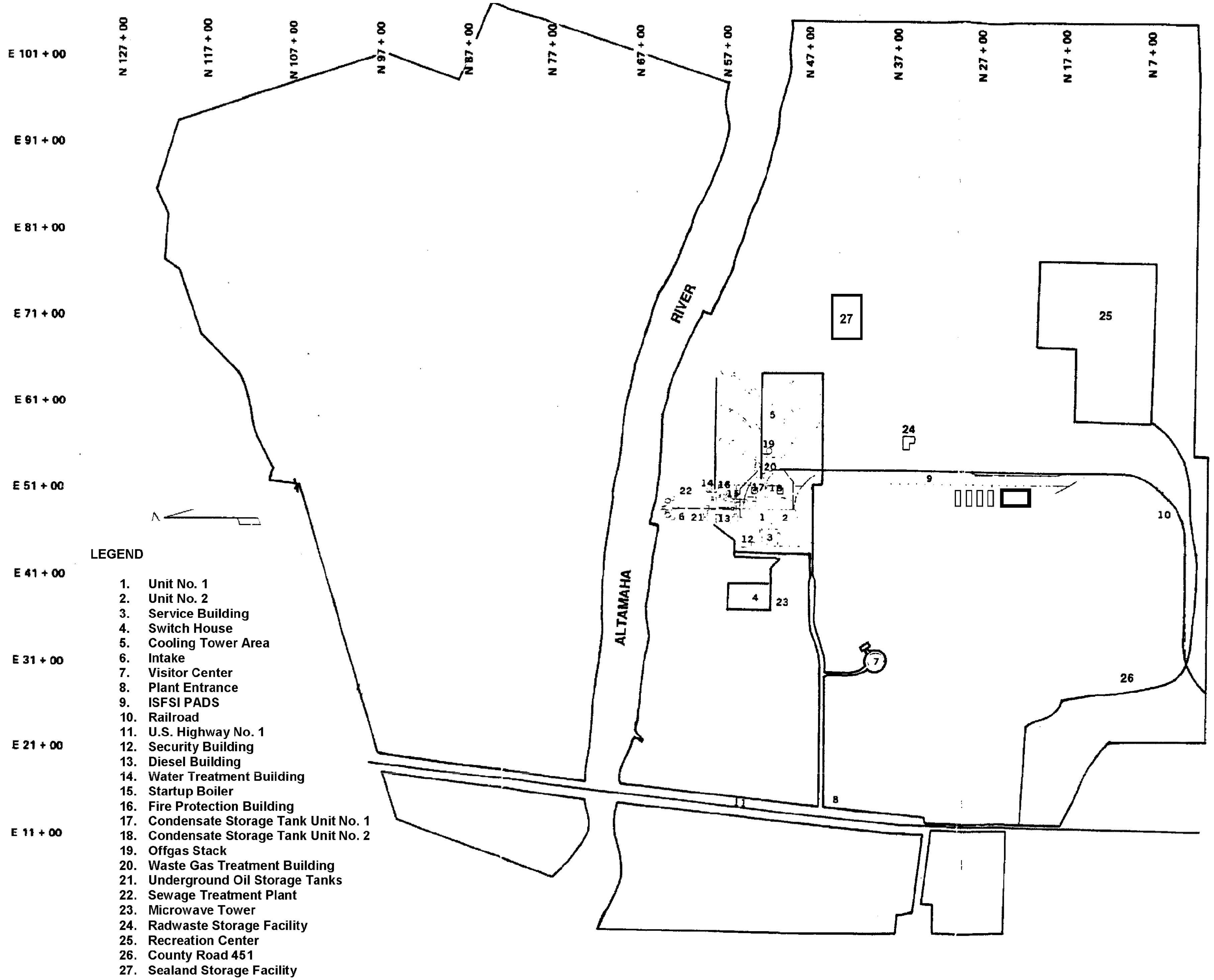
Parameter	HNP-1 Original Rated Power	HNP-1 Uprated Power	HNP-1 Extended Uprated Power	HNP-1 TPO Uprated Power	HNP-1 ROPI Uprated Power
Thermal power (MWt)	2436	2558	2763	2804	2804
Vessel steam flow (Mlbm/h)	10.0	10.6	11.5	11.6	11.6
Full power core flow range (% of rated)	87-105	87-105	91-105	93-105	93-105
Dome pressure (psia)	1020	1050	1050	1050	1060
Dome temperature (°F)	547	551	551	551	552
Turbine inlet pressure (psia)	950	985	1000	1002[b]	1012[b]
Feedwater flow[a] (Mlbm/h)	10.1	10.7	11.6	11.7	11.7
Feedwater temperature[a] (°F)	388	393	398	393	393
Core inlet enthalpy (Btu/lbm)	523.7	527.1	525.6	524.6	525.9

a. Includes RWC system flow.
b. Upstream side of TSV

TABLE 1.2-2 (SHEET 2 OF 2)

Parameter	HNP-2 Original Rated Power	HNP-2 Uprated Power	HNP-2 Extended Uprated Power	HNP-2 TPO UpRrated Power	HNP-2 ROPI Uprated Power
Thermal power (MWt)	2436	2558	2763	2804	2804
Vessel steam flow (Mlbm/h)	10.5	11.1	12.0	12.2	12.2
Full power core flow range (% of rated)	87-105	87-105	91-105	93-105	93-105
Dome pressure (psia)	1020	1050	1050	1050	1060
Dome temperature (°F)	547	551	551	551	552
Turbine inlet pressure (psia)	950	985	1000	993[b]	1003[b]
Feedwater flow[a] (Mlbm/h)	10.5	11.1	12.1	12.2	12.2
Feedwater temperature[a] (°F)	420	424	425	426	426
Core inlet enthalpy (Btu/lbm)	526.9	530.3	528.7	528.4	529.7

a. Includes RWC system flow.
b. Upstream side of TSV



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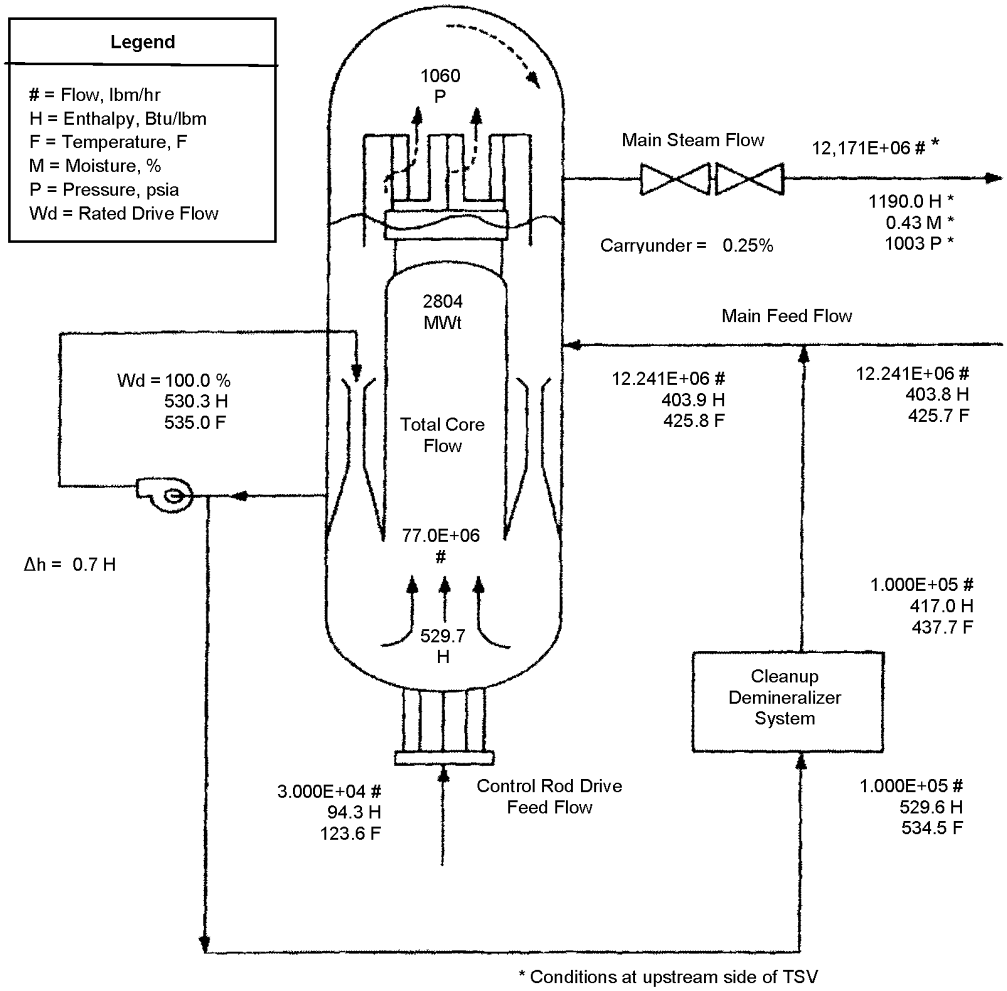
SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

PLANT PROPERTY PLAN

FIGURE 1.2-1



Core Thermal Power	2804.0
Pump Heating	7.5
Cleanup Losses	- 3.3
Other System Losses	- 1.1
Turbine Cycle Use	2807.1 MWt

REV 22 9/04

SOUTHERN COMPANY
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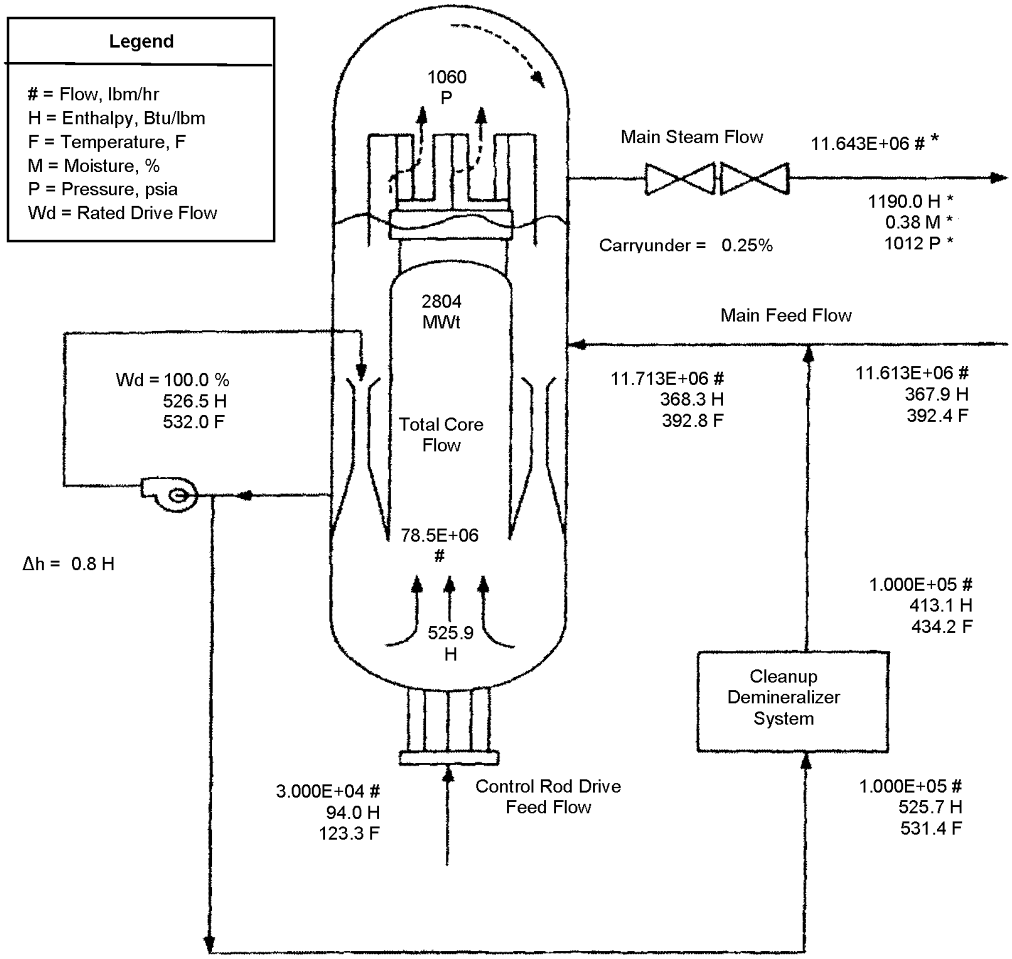
SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

REACTOR SYSTEM HEAT BALANCE FOR 100% RATED CONDITION OR 2804 MWt (HNP-2)

FIGURE 1.2-2



* Conditions at upstream side of TSV

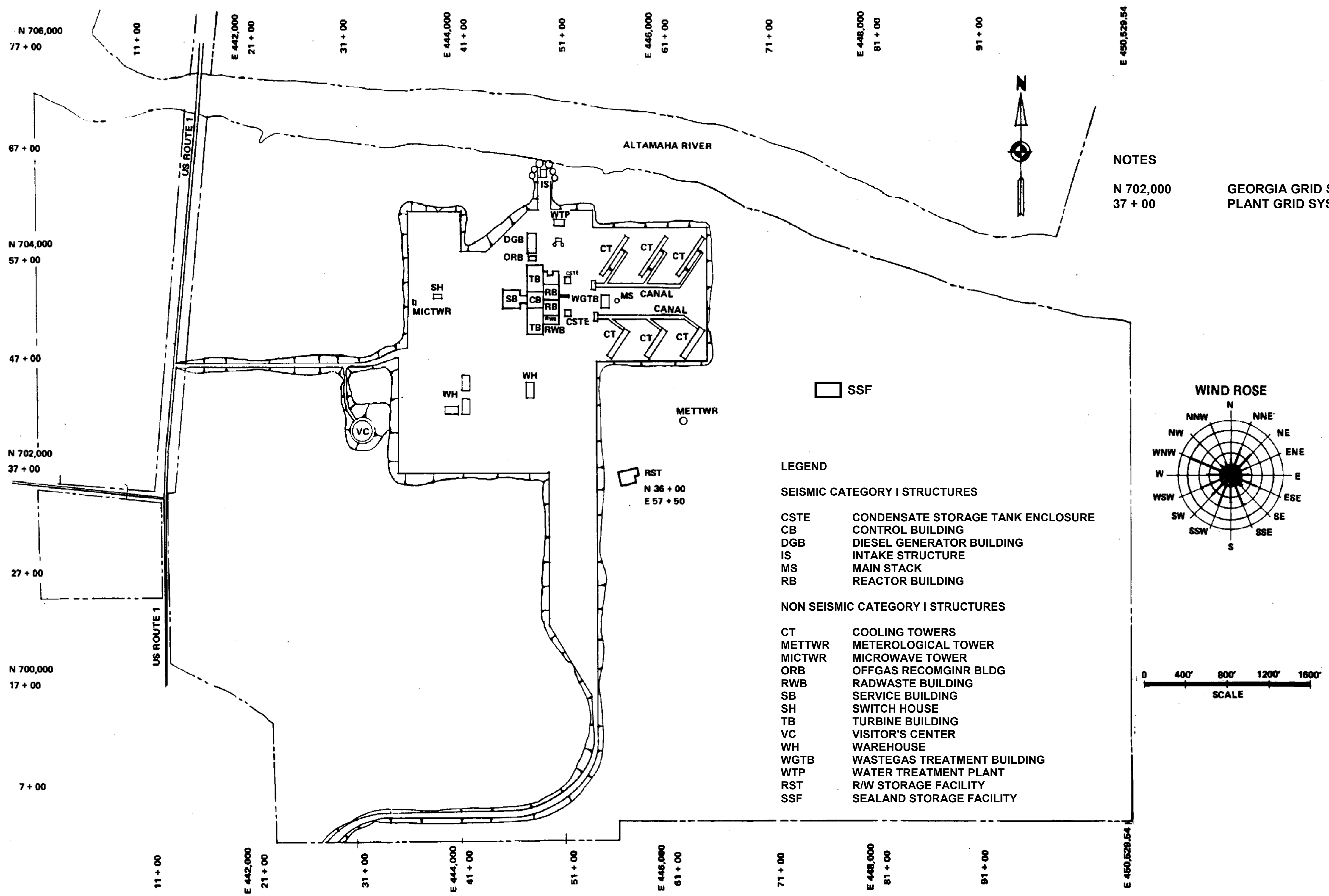
Core Thermal Power	2804.0
Pump Heating	7.8
Cleanup Losses	- 3.3
Other System Losses	- 1.1
Turbine Cycle Use	2807.4 MWt

REV 22 9/04

SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

REACTOR SYSTEM HEAT BALANCE FOR 100% RATED CONDITION OR 2804 MWt (HNP-1)

FIGURE 1.2-3



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REV 26 9/08

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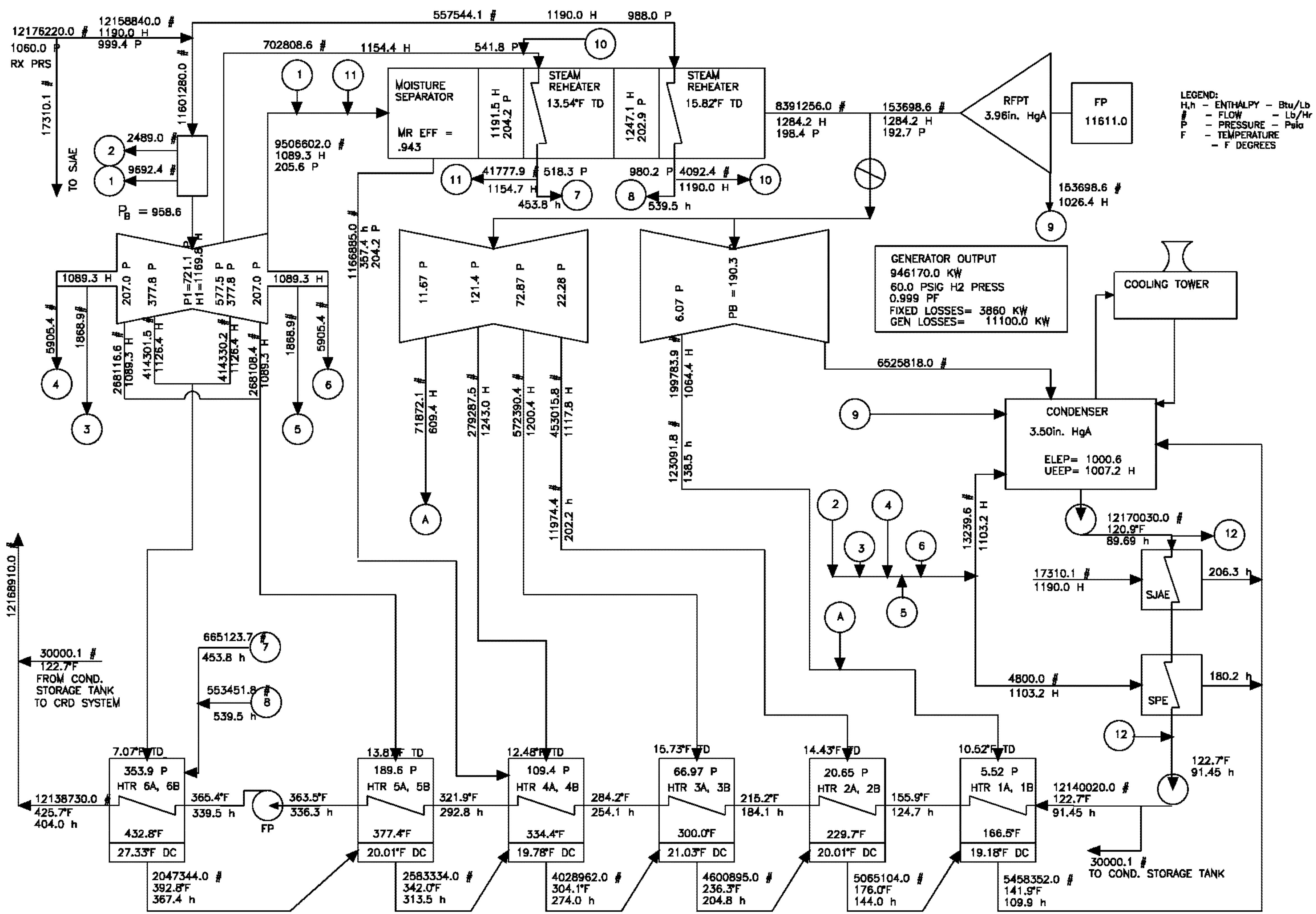
SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

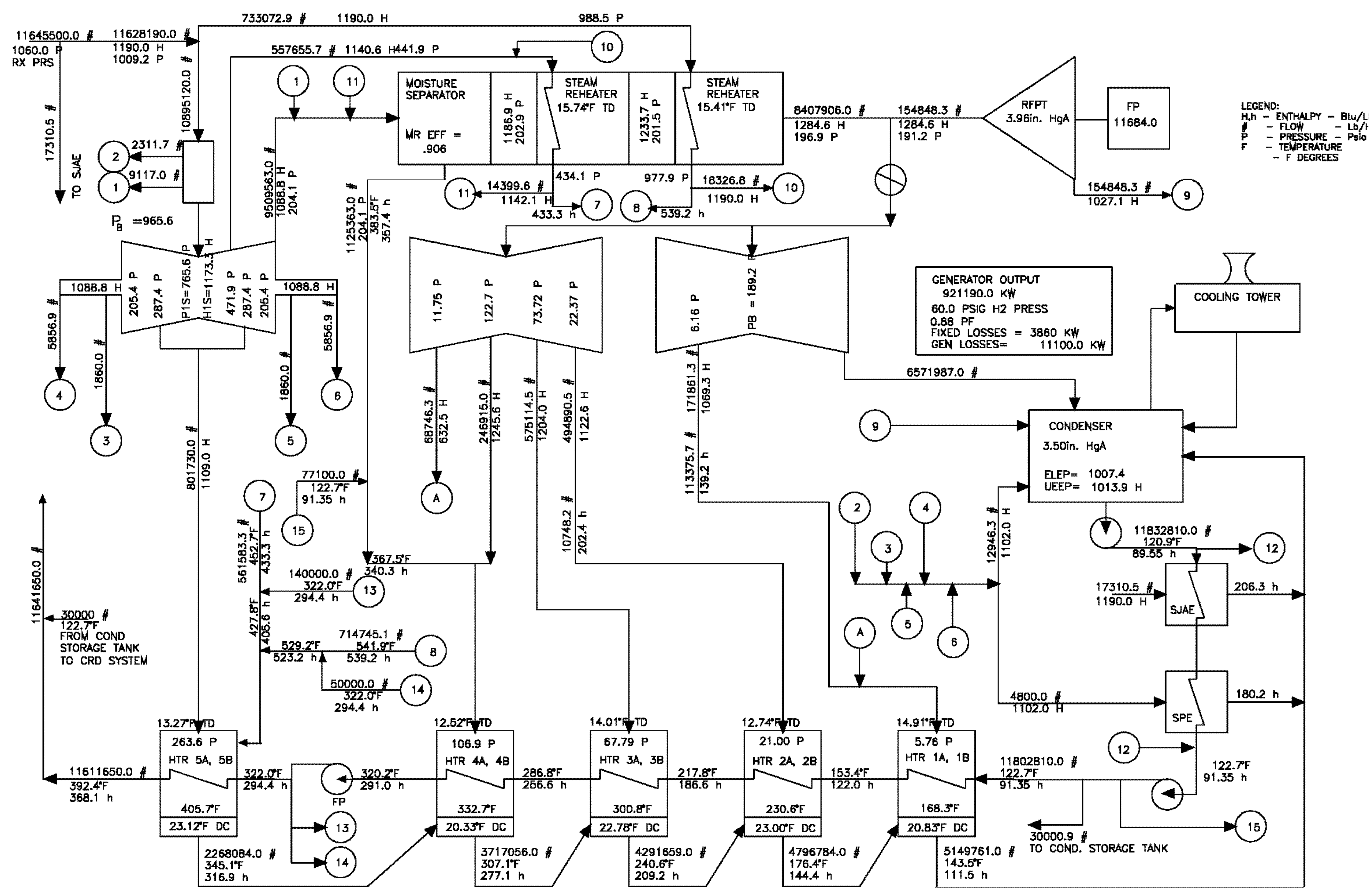
PROPERTY PLAN AND PRINCIPAL STRUCTURES

FIGURE 1.2-4



REV 22 9/04

SOUTHERN COMPANY Energy to Serve Your World®	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	RATED TURBINE GENERATOR HEAT BALANCE (HNP-2)
		FIGURE 1.2-5



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SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

RATED TURBINE GENERATOR HEAT BALANCE (HNP-1)

FIGURE 1.2-7

1.3 COMPARISON TABLES

1.3.1 COMPARISON WITH SIMILAR FACILITY DESIGNS (HNP-1 AND HNP-2)

This section highlights the principal design features of the plant and compares the major features with those of other boiling water reactor (BWR) facilities. Table l.3-1 summarizes the initial plant design characteristics for Edwin I. Hatch Nuclear Plant-Units 1 and 2, Brunswick Steam Electric Plant-Unit 2, and Cooper Nuclear Station..

The design of these facilities is based upon proven technology attained during the development, design, construction, and operation of BWRs of similar or identical types. However, any of the data on this plant or the other plants are subject to revisions.

Table 1.3-1 does not reflect the license change to increase the rated thermal power to 2804 MWt. Key parameter differences between the original rated power of 2436 MWt and the current rated power of 2804 MWt are provided in section 1.2. Current fuel design parameters are provided in section 4.3.

TABLE 1.3-1 (SHEET 1 OF 11)

NUCLEAR PLANTS
PRINCIPAL PLANT DESIGN FEATURES COMPARISON

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
A. Site				
1. Location	Brunswick County, NC	Nemaha County, Nebr	Appling County, Ga	Appling County, Ga
2. Size of site (acres)	1200	1090	2100	2100
3. Site ownership	Carolina Light and Power Co	NPPD	(a)	(a)
4. Plant ownership	CP & L	NPPD	(a)	(a)
5. Number of units onsite	2	1	2	2
B. Plant				
1. Reactor warranted conditions				
a. Net electrical output (MWe)	821	778	786	795
b. Gross electrical output (MWe)	849	801	813	822
c. Net heat rate (Btu/kW-h)	10,120	10,190	10,490	10,120
d. Gross heat rate (Btu/kW-h)	9788	10,142	10,218	9959
e. Feedwater temperature (°F)	420	367	387.4	424
C. Reactor Primary Vessel				
1. Inside diameter (ft-in.)	18-2(b)	18-2(b)	18-2(b)	18-2(b)
2. Overall length inside (ft-in.)	69-4(b)	69-4(b)	69-4(b)	69-4(b)
3. Design pressure (psig)	1250(b)	1250(b)	1250(b)	1250 (b)
4. Wall thickness (in.)	5-17/32(b)	5-17/32(b)	5-17/32(b)	5-17/32(b)

TABLE 1.3-1 (SHEET 2 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
D. Reactor Coolant Recirculation Loops				
1. Location of recirculation loops	Primary containment system drywell structure	Primary containment system drywell structure	Primary containment system drywell structure	Primary containment system drywell structure
2. Number of recirculation loops	2	2	2	2
3. Pipe size (in.)	28	28	28	28
4. Pump capacity, each gal/min	45,200	45,200	45,200	45,200
5. Number of jet pumps	20	20	20	20
6. Location of jet pumps	Inside reactor pressure vessel	Inside reactor pressure vessel	Inside reactor pressure vessel	Inside reactor pressure vessel
E. Reactor				
1. Reactor warranted conditions				
a. Thermal output (MWt)	2436	2381	2436	2436
b. Reactor operating pressure (psig)	1005	1005	1005	1005
c. Total reactor core flowrate (lb/h)	78.5×10^6	74.5×10^6	78.5×10^6	78.5×10^6
d. Main steam flowrate (lb/h)	10.03×10^6	9.81×10^6	10.03×10^6	10.47×10^6
2. Reactor core description (initial core)				
a. Lattice	7 x 7	7 x 7, 8 x 8	7 x 7, 8 x 8	8 x 8
b. Control rod pitch (in.)	12.0	12.0	12.0	12.0
c. Number of fuel assemblies	560	548	560	560
d. Number of control rods	137	137	137	137

TABLE 1.3-1 (SHEET 3 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
2. Reactor core description (initial core) (cont)				
e. Number of instrument tubes	43	43	43	43
f. Effective active fuel length (in.)	144	144	144	146
g. Equivalent reactor core diameter (in.)	160.2	158.5	160.2	160.2
h. Circumscribed reactor core diameter (in.)	170.5	170.5	170.5	170.5
i. Total weight UO_2 (lb)	272,850	257,350	272,850	260,570
3. Reactor fuel description (initial core)				
a. Fuel material	UO_2	UO_2	UO_2	UO_2
b. Fuel density (lb/ft^3) @ 98.3% theoretical	639	639	639	647
c. Fuel pellet diameter(in.)	0.487	0.487	0.487	0.416
d. Fuel rod cladding material	Zircaloy-2	Zircaloy-2	Zircaloy-2	Zircaloy-2
e. Fuel rod cladding thickness (in.)	0.082	0.082	0.082	0.082
f. Fuel rod cladding process	Free standing loaded tubes	Free standing loaded tubes	Free standing loaded tubes	Free standing loaded tubes
g. Fuel rod outside diameter (in.)	0.562	0.562	0.562	0.493
h. Length of gas plenum (in.)	16.0	16.0	16.0	14.0
i. Fuel rod pitch (in.)	0.737	0.737	0.737	0.640
j. Fuel assembly channel material	Zircaloy-4	Zircaloy-4	Zircaloy-4	Zircaloy-4

TABLE 1.3-1 (SHEET 4 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
4. Reactor control				
a. Control rods				
1. Number	137	137	137	137
2. Shape	Cruciform	Cruciform	Cruciform	Cruciform
3. Material	B_4C granules compacted in SS tubes	B_4C granules compacted in SS tubes	B_4C granules compacted in SS tubes	B_4C granules compacted in SS tubes
4. Pitch (in.)	12.0	12.0	12.0	12.0
5. Poison length (in.)	143.0	143.0	143.0	143.0
6. Blade span (in.)	9.75	9.75	9.75	9.75
7. Number of control material tubes for rod	84	84	84	84
8. Tube dimensions(in.)	0.1830Dx0.025-wall	0.1830Dx0.025-wall	0.1830Dx0.025-wall	0.1830Dx0.025-wall
9. Stroke (in.)	144.0	144.0	144.0	144.0
b. Supplementary reactivity control	Gadolinia burnable poison	Gadolinia burnable poison	Gadolinia burnable poison	Gadolinia burnable poison
5. Thermal hydraulic data (initial core)				
a. Heat transfer area per assembly (ft^2)	86.513	86.513	86.513	98.93
b. Reactor core heat transfer area (ft^2)	48,451	47,409	48,451	55,394
c. Maximum heat flux[(c)] (BTU/h ft^2)	428,308	428,308	428,308	346,600
d. Average heat flux[(c)] (BTU/h ft)	164,740	164,740	164,740	142,600

TABLE 1.3-1 (SHEET 5 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
5. Thermal hydraulic data (initial core) (cont)				
e. Maximum power per fuel[c] rod unit length (kW/ft)	18.5	18.5	18.5	13.4
f. Average power per fuel rod unit length (kW/ft)[c]	7.11	7.11	7.11	5.4
g. Maximum fuel temperature (°F)	3290	4380	4380	3290
h. Minimum critical heat flux ratio	1.9	1.9	1.9	1.9
i. Total heat generated in fuel (%)	95.0	95.0	95.0	95.0
j. Core average exit quality	13.6	13.2	13.0	14.0
6. Power distribution - peaking factors (peak/average) (initial core)				
a. Axial	1.50	1.50	1.50	1.40
b. Relative assembly	1.40	1.40	1.40	1.40
c. Local (within assembly)	1.24	1.24	1.24	1.24
d. Gross (1) x (2)	2.10	2.10	2.10	1.96
7. Nuclear design data (initial core)				
a. Average discharge exposure-1st core	19,000 MWD/short ton U	19,000 MWD/short ton U	19,000 MWD/short ton U	15,777 MWD/short ton U
b. Moderator to fuel volume ratio at total core H_2O/UO_2 cold	2.41	2.41	2.41	2.45

TABLE 1.3-1 (SHEET 6 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
8. Incore instrumentation				
a. Number of power range (in core) monitoring assemblies (fixed)	31	31	31	31
b. Number of intermediate range monitoring chambers	8	8	8	8
c. Number of startup range monitoring counters	4	4	4	4
d. Number of startup sources	5	5	5	5
e. Number of reactor primary vessel penetrations	43	43	43	43
9. Reactivity control (initial core)				
a. Reactivity of core with all control rods in (cold)	0.95 k_{eff}	0.95 k_{eff}	0.95 k_{eff}	0.95 k_{eff}
b. Reactivity of core with strongest control rod cut (cold)	0.99 k_{eff}	0.99 k_{eff}	0.99 k_{eff}	0.99 k_{eff}
c. Typical moderator temperature coefficient ($\Delta k/k$°F)				
• Cold	-5.0×10^{-5}	-5.0×10^{-5}	-1.0×10^{-5}	-5.0×10^{-5}
• Hot (no voids)	-39.0×10^{-5}	-39.0×10^{-5}	-39.0×10^{-5}	-39.0×10^{-5}
• Operating				

TABLE 1.3-1 (SHEET 7 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
9. Reactivity control (cont)				
d. Typical moderator void coefficient (Δk/k% void)				
• Cold				
• Hot (no voids)	-1.0×10^{-3}	-1.0×10^{-3}	-1.0×10^{-3}	-1.0×10^{-3}
• Operating	-1.5×10^{-3}	-1.5×10^{-3}	-1.5×10^{-3}	-1.5×10^{-3}
e. Typical fuel temperature (Doppler) coefficient				
• Cold	-1.3×10^{-3}	-1.3×10^{-5}	-1.3×10^{-5}	-1.3×10^{-5}
• Hot (no voids)	-1.2×10^{-5}	-1.2×10^{-5}	-1.2×10^{-5}	-1.2×10^{-5}
• Operating	-1.3×10^{-5}	-1.3×10^{-5}	-1.3×10^{-5}	-1.3×10^{-5}
F. Containment Systems				
1. Primary containment				
a. Type	Pressure suppression	Pressure suppression	Pressure suppression	Pressure suppression
b. Construction				
• Drywell	Conical and cylindrical steel-lined concrete vessel	Light bulb/steel vessel	Light bulb/steel vessel	Light bulb/steel vessel
• Pressure suppression chamber	Torus/steel-lined concrete vessel	Torus/steel vessel	Torus/steel vessel	Torus/steel vessel
c. Pressure suppression chamber - internal design pressure (psig)	+62	+56	+56	+56
d. Pressure suppression chamber - external design pressure (psig)	+2	+2	+2	+2

TABLE 1.3-1 (SHEET 8 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
1. Primary containment (cont)				
e. Drywell - internal design pressure (psig)	+62	+56	+56	+56
f. Drywell - external design pressure (psig)	+2	+2	+2	+2
g. Drywell free volume (ft^3) (including vent system)	164,100	145,430	146,010	146,266
h. Pressure suppression chamber free volume (ft^3)(minimum) @ high water level	124,000	109,810	112,900	109,800
i. Pressure suppression pool water volume (ft^3)(minimum)	87,600	87,660	85,112	87,420
j. Submergence of vent pipe below pressure pool surface (ft)	4	4	3 ft 8 in.	4 ft 8 in.
k. Design temperature of drywell (°F)	281	281	281	340
l. Design temperature of pressure suppression chamber (°F)	281	281	281	340
m. Downcomer vent pressure loss factor	6.21	6.21	6.18	4.4
n. Break area/total vent area	0.019	0.019	0.0194	0.0202
o. Drywell free volume/pressure suppression chamber free volume	1.32	1.32	1.293	1.34
p. Primary system volume/pressure suppression pool volume	0.214	0.194	0.191	0.155

TABLE 1.3-1 (SHEET 9 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
1. Primary containment (cont)				
q. Drywell free volume/primary system volume	8.8	8.58	8.667	8.68
r. Calculated maximum pressure after blowdown with no prepurge				
• Drywell (psig)	46	46	46.5	57.51
• Pressure suppression chamber (psig)	28	28	28	26.61
s. Initial pressure suppression chamber temperature rise (°F)	< 50	50	50	45
t. Leakage rate (percent free volume per day)	0.50	0.50	1.2	1.2
2. Secondary containment				
a. Type	Controlled leakage evaluated release	Controlled leakage evaluated release	Controlled leakage evaluated release	Controlled leakage evaluated release
b. Construction				
Lower levels	Reinforced concrete	Reinforced concrete	Reinforced concrete	Reinforced concrete
Upper levels	Steel superstructure and siding panels	Steel superstructure and siding panels	Steel superstructure and precast concrete	Steel superstructure and precast concrete
Roof	Steel sheeting	Steel sheeting	Steel sheeting and reinforced concrete slabs	Steel sheeting and reinforced concrete slabs
c. Internal design pressure (psig)	0.25	0.25	0.25	0.25
d. Design inleakage rate (percent free volume/day at 0.25 in. H_2O)	100	100	100	100

TABLE 1.3-1 (SHEET 10 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
3. Elevated release point				
a. Type	Stack	Stack	Stack	Stack
b. Construction	Steel	Steel	Reinforced concrete	Reinforced concrete
c. Height (above ground)	100 m	100 m	120 m	120 m
G. Plant Auxiliary Systems				
1. Emergency core cooling system (number)[(d)]				
a. Reactor core spray cooling system	2 loops	2 loops	2 loops	2 loops
b. Reactor core high pressure coolant injection system	1 pump	1 pump	1 pump	1 pump
c. Auto-relief system	1	1	1	1
d. Reactor core residual heat removal system				
Low pressure coolant injection sub-system	4 pumps	4 pumps	4 pumps	4 pumps
Primary containment spray/cooling sub-system	1	1	1	1
Reactor shutdown cooling sub-system	1	1	1	1
2. Reactor auxiliary system (number)				
a. Spent fuel pool cooling and demineralizing system	1	1	2[(f)]	1
b. Reactor cleanup demineralization system	1	1	1	1
c. Reactor core isolation cooling system	1	1	1	1

TABLE 1.3-1 (SHEET 11 OF 11)

	Brunswick Steam Electric Plant-Unit 2 (CP&L)	Cooper Nuclear Station (NPPD)	Edwin I. Hatch Nuclear Plant-Unit 1 (SNC)	Edwin I. Hatch Nuclear Plant-Unit 2 (SNC)
H. Plant Electrical Power Systems				
1. Transmission system				
a. Outgoing lines	7-230 kV	4-345 kV	4-230 kV	2-500 kV[e]
2. Auxiliary power systems				
a. Incoming lines	7-230 kV	1-69 kV	4-230 kV	2-500 kV[e]
b. Onsite sources				
Auxiliary transformers	2	2	2	2
Startup transformers	2	1	2	2
Shutdown transformers	0	1	0	0
3. Standby diesel generator system				
a. Number of diesel generators				
Generators	4	3 or 4	3	2[e]

a. See HNP-2 subsection 1.4.1.
b. The values shown are nominal and may vary slightly with vessel manufacturers.
c. These items are shown at design limits rather than design point.
d. The design capacities of the systems listed are the same for all four plants listed.
e. One of the HNP-1 units is shared with HNP-2.

1.4 IDENTIFICATION OF AGENTS AND CONTRACTORS

1.4.1 INTRODUCTION

Georgia Power Company (GPC) was the general contractor for the construction of the Edwin I. Hatch Nuclear Plant (HNP) and is the co-owner with Oglethorpe Power Corporation (OPC), the Municipal Electric Authority of Georgia (MEAG), and the city of Dalton, Georgia. GPC was the sole operator of the facility. Effective March 22, 1997, Southern Nuclear Operating Company (SNC) is the exclusive operating licensee. Southern Company Services, Inc. (SCS) was responsible for the design which was subcontracted to Bechtel Power Corporation. The nuclear steam supply system (NSSS) and the turbine-generator are designed and supplied by General Electric (GE).

GPC, a co-owner of the HNP, was responsible for the design, construction, and operation of the plant through March 21, 1997. Since March 22, 1997, as the exclusive operating Licensee, SNC is responsible for the planning, design, licensing, operation, maintenance, repair, modification, addition of, license renewal, and retirement and decommissioning of HNP pursuant to a Nuclear Operating Agreement between SNC and GPC.

1.4.2 APPLICANTS (HNP-1 AND HNP-2)

1.4.2.1 Georgia Power Company

1.4.2.1.1 General

GPC, a wholly owned subsidiary of The Southern Company, is a Co-Applicant. GPC is a public utility incorporated under the laws of the State of Georgia with its principal offices located in Atlanta, Georgia. A description of GPC, including its qualifications and history, is located in the license application.

GPC acted as the general contractor during construction of HNP and is a co-owner.

GPC has a traditional relationship with SCS within The Southern electric system in the design and construction of power generating facilities. This relationship is based on company contracts between the two companies which delegate certain design and engineering responsibilities to SCS. GPC was responsible for construction and operation. Effective March 22, 1997, SNC is the exclusive operating licensee. SNC has inputs to the design and the procurement activities to ensure that the plant concept, capacity, layout, and operating features include desired provisions and arrangements for constructibility and operability. Certain documents, such as arrangement drawings, purchase inquiries, and recommendations, are submitted to SNC for concurrence to verify that these features have been included in accordance with their requirements.

1.4.2.1.2 Technical Qualifications

GPC has participated in the development of nuclear power for more than 20 years, beginning as a member of Atomic Power Development Associates, Inc. (APDA) and Power Reactor Development Company, the designers and operators of the Enrico Fermi Atomic Power Plant-Unit 1. The participation has consisted of both financial contributions and assignment of personnel. Originally, GPC participated in the design, construction, and operation of HNP-1 and HNP-2. Effective March 22, 1997, SNC is the exclusive operating licensee.

Employees of GPC received inservice nuclear training through various courses, such as the Introduction to Nuclear Power course developed by Nuclear Utility Services. GPC employees also participated in the licensing, design, construction, and operation of HNP-1 and HNP-2. Effective March 22, 1997, SNC assumed the technical qualifications of GPC in all aspects.

The technical qualifications of SNC are further delineated in section 13.1, Organizational Structure.

1.4.2.2 Oglethorpe Power Corporation

OPC, incorporated in August 1974, is composed of the rural electric membership corporations within the state of Georgia that purchase wholesale electric energy from GPC. OPC owns a 30% undivided interest of HNP-1 and HNP-2.

1.4.2.3 Municipal Electric Authority of Georgia

MEAG was created by the 1975 Georgia General Assembly to provide electric energy to local municipal government-owned electric distribution systems within the state of Georgia. MEAG owns a 17.7% undivided interest of HNP-1 and HNP-2.

1.4.2.4 City of Dalton, Georgia

Dalton, an incorporated municipality, owns a 2.2% undivided interest of HNP-1 and HNP-2.

1.4.2.5 Georgia Power Company

GPC has the authority to make available to OPC, MEAG, and Dalton their prorated shares of the net capacity and net electric energy output.

1.4.2.6 Southern Nuclear Operating Company

Southern Nuclear Operating Company (SNC), a wholly-owned subsidiary of Southern Company, is the exclusive operating licensee of HNP and is responsible to GPC for the operation of HNP pursuant to a Nuclear Operating Agreement.

1.4.3 ARCHITECT/ENGINEER (HNP-1 AND HNP-2)

1.4.3.1 General

SCS, as the service company to the Southern Company, was responsible to SNC for certain engineering and design requirements of HNP-1 and HNP-2. As a result of the consolidation of SCS and SNC nuclear expertise, and in addition to being the licensee, SNC serves as its own architect/engineer and performs the functions previously performed by SCS.

Bechtel was engaged by SCS to perform the engineering and design of HNP-2 and also assisted SCS in the engineering and design of HNP-1.

1.4.3.2 Technical Qualifications

SCS provided engineering, design, financial, and other management services at cost to the operating companies of the Southern Company. In this capacity, SCS had extensive experience in the design of thermal, hydroelectric, and nuclear generating plants.

SCS participated in the development of nuclear power for more than 20 years, beginning as a member of APDA and Power Reactor Development Company. Personnel from SCS worked full time at APDA's facilities in Detroit, Michigan on the research and engineering for the Enrico Fermi Atomic Power Plant-Unit 1. SCS had a representative on APDA's technical and engineering committee during the entire term of its membership.

SCS was responsible for the design of HNP-1 and HNP-2 and the Alabama Power Company Joseph M. Farley Nuclear Plant, which was designed by SCS and Bechtel.

Bechtel has extensive experience in the design and construction of thermal generating units, including such domestic units as:

- *Monticello-Unit 1.*
- *Pilgrim-Unit 1.*
- *Peach Bottom-Units 2 and 3.*
- *Duane Arnold-Unit 1.*
- *Limerick-Units 1 and 2.*
- *HNP-Units 1 and 2.*

1.4.4 NUCLEAR STEAM SUPPLY SYSTEM SUPPLIER (HNP-1 AND HNP-2)

1.4.4.1 General

GE was responsible for the design, fabrication, and delivery of the direct-cycle boiling water NSSS, the fabrication of the first core and reloads of nuclear fuel, and the provision of technical direction for installation and startup of this equipment.

1.4.4.2 Technical Qualifications

GE has been engaged in the development, design, construction, and operation of boiling water reactors (BWRs) since 1955. Thus, GE has substantial experience, knowledge, and capability to design, manufacture, and furnish technical assistance for the installation and startup of BWRs.

1.4.5 DIVISION OF RESPONSIBILITY

1.4.5.1 Design Stage

GE and Bechtel were delegated the responsibility for design of the NSSS and the balance of the plant, respectively. For preparation of the FSAR, GPC, SCS, Bechtel, and GE were involved in the preparation and review of design bases and philosophies of both systems and structures. The intent of this review was to allow as much expertise as possible to contribute to the plant design.

1.4.5.2 Procurement Stage

1.4.5.2.1 General Electric Scope of Supply

All items within the GE scope of supply were the sole responsibility of GE.

1.4.5.2.2 Bechtel Scope of Supply

For the equipment under the Bechtel scope of supply, procurement procedures were established to require GPC, SCS, and Bechtel participation. Bechtel prepared the inquiries and transmitted them to GPC and SCS for approval, allowing for review to ascertain that sufficient information was contained to inform the bidders of all requirements for the supplied equipment including, but not limited to, material, documentation, and shipping requirements. From this point, Bechtel had the responsibility for sending the inquiry out for bids in accordance with a bidder's list supplied by GPC. After Bechtel reviewed the bids, prepared the requisition, and obtained approval by GPC, the purchase order was prepared by GPC.

1.4.5.2.3 Construction Stage

All construction activities at the site were under the supervision of GPC, with independent testing agencies being contracted, as necessary, to perform special testing and provide expertise in the interpretation of results.

1.4.5.2.4 Operation Stage

GPC initially had the sole responsibility for the operation of HNP-2. Effective March 22, 1997, SNC is the exclusive operating licensee of HNP-2.

1.5 (Deleted)

1.6 REFERENCED TOPICAL REPORTS

Table 1.6-1 lists the topical reports referenced in the original HNP-2 FSAR in support to the initial license application. The reports listed in table 1.6-1 are on file with the Nuclear Regulatory Commission.

Topical reports that are relevant to the current plant design and operation are referenced in the specific HNP-2-FSAR section.

TABLE 1.6-1 (SHEET 1 OF 4)

REFERENCE TOPICAL REPORTS

A. General Electric Company Reports

Report No.	*Title*
APED 4827	*Maximum Two-Phase Vessel Blowdown from Pipes (April 1965)*
APED 5450	*Design Provisions for In-Service Inspection (April 1967)*
APED 5458	*Effectiveness of Core Standby Cooling Systems for General Electric Boiling*
APED 5460	*Design and Performance of General Electric Boiling Water Reactor Jet Pumps (September 1968)*
APED 5499	*Control Rod Worth Minimizer (March 1967, revision in progress)*
APED 5555	*Impact Testing on Collet Assembly for Control Rod Drive Mechanism 7RDB144A (November 1967)*
APED 5640	*Xenon Considerations in Design of Large Boiling Water Reactors (June 1968)*
APED 5706	*In-Core Neutron Monitoring System for General Electric Boiling Water Reactor (November 1968; revised April 1969)*
APED 5736	*Guidelines for Determining Safe Test Intervals and Repair Times for Engineered Safeguards (April 1969)*
APED 5750	*Design and Performance of General Electric Boiling Water Reactor Main Steam Line Isolation Valves (March 1969)*
APED 5756	*Analytical Methods for Evaluating the Radiological Aspects of the General Electric Boiling Water Reactor (March 1969)*
GEAP 4059	*Vibration in Fuel Rods in Parallel Flow (July 1962)*
GEAP 4616	*Two-Phase Pressure Drop in Straight Pipes and Channels: Water-Steam Mixtures at 600 to 1400 psia (May 1964)*
GEAP 4966	*Vibration of SEFOR Fuel Rods in Parallel Flow (September 1965)*
GEAP 5620	*Failure Behavior in ASTM A106B Pipes Containing Axial Through-Wall Flaws (April 1968)*
NEDE 21156	*Supplemental Information for Plant Modification to Eliminate Significant In-Core Vibration, Class III (January 1976)*

TABLE 1.6-1 (SHEET 2 OF 4)

Report No.	*Title*
NEDO 11146	*Design Basis for New Gas Systems (July 1971) (Proprietary)*
NEDM 10735	*Fuel Densification Effects on General Electric Boiling Water Reactor Fuel, Supplements 6, 7, and 8 (August 1973)*
NEDO 10029	*An Analytical Study on Brittle Fracture of GE-BWR Vessel Subject to the Design Basis Accident (May 1969)*
NEDO 10139	*Compliance of Protection Systems to Industry Criteria: General Electric BWR Nuclear Steam Supply System (June 1970)*
NEDO 10173	*Current State of Knowledge, High Performance BWR Zircaloy-Clad UO_2 Fuel (May 1970)*
NEDO 10174	*Consequences of a Postulated Flow Blockage Incident in a BWR (May 1970)*
NEDO 10299	*Core Flow Distribution in a Modern Boiling Water Reactor as Measured in Monticello (January 1971)*
NEDO 10320	*The General Electric Pressure Suppression Containment Analytical Model (April 1971)*
NEDO 10329	*Loss-of-Coolant Accident and Emergency Core Cooling Models for General Electric Boiling Water Reactor (April 1971), Supplement 1 (April 1971), Addenda (May 1971)*
NEDO 10505	*Experience with BWR Fuel Through September 1971 (May 1972)*
NEDO 10527	*Rod Drop Accident Analysis for Large Boiling Water Reactors (March 1972), Supplement 1 (July 1972), Supplement 2 (January 1973)*
NEDO 10541	*Visual and Photographic Examination of Dresden-1 High Exposure Control Rod B87 (April 1972)*
NEDO 10585	*Behavior of Iodine in Reactor Water During Plant Shutdown and Startup (August 1972)*
NEDO 10602	*Testing of Improved Jet Pumps for the BWR/6 Nuclear System (June 1972)*
NEDO 10734	*A General Justification for Classification of Effluent Treatment System Equipment As Group D (February 1973)*
NEDM 10735	*Densification Considerations in BWR Fuel Design and Performance (December 1972)*

TABLE 1.6-1 (SHEET 3 OF 4)

Report No.	*Title*
NEDO 10751	*Experimental and Operational Confirmation of Off-Gas System Design Parameters (January 1973) (Proprietary)*
NEDO 10802	*Analytical Methods of Plant Transient Evaluations for the General Electric Boiling Water Reactor (April 1973)*
NEDO 10871	*Technical Derivation of BWR 1971 Design Basis Radioactive Material Source Terms (March 1973)*
NEDO 10899	*Chlorine Control in BWR Coolants (June 1973)*
NEDO 10958 and NEDE 10958	*General Electric BWR Thermal Analysis Basis (GETAB) Data, Correlation and Design Application (November 1973)*
NEDO 12037	*Summary of Gamma and Beta Energy and Intensity Data (1970)*
GEAP 13112	*Thermal Response and Cladding Performance of an Internally Pressurized, Zircaloy-Clad, Simulated BWR Fuel Bundle Cooled by Spray Under Loss of Coolant Conditions (April 1971)*
NEDO 20340	*Process Computer Performance, Evaluation Accuracy (December 1974)*
NEDO 20360	*General Electric Boiling Water Reactor Generic Reload Application for 8x8 Fuel, Rev. 1, Supplement 3 (May 1975)*
NEDO 20360-1P	*General Electric Boiling Water Reactor Generic Reload Application for 8x8 Fuel, Rev. 4 (March 25, 1976)*
NEDO 20377	*8x8 Fuel Bundle Development Support (February 1975)*
NEDE 20386	*Fuel Channel Deflections (May 1974)*
NEDO 20566 (Draft)	*General Electric Company Analytical Model for-Loss-of-Coolant Analysis in Accordance with 10CFR50, Appendix K*
NEDO 20922	*Experience with BWR Fuel Through September 1974 (June 1975)*
NEDO 20939	*Lattice Physics Methods Verification (June 1976)*
NEDO 20944	*BWR/4 and BWR/5 Fuel Design, Rev. 1, 76NED35 (October 1976)*
NEDO 20945	*3D BWR Simulator (August 1976)*
NEDO 20946	*BWR Simulator Methods Verification (August 1976)*

TABLE 1.6-1 (SHEET 4 OF 4)

Report No.	*Title*
NEDO 20948-P	*BWR/6 Fuel Design (June 1976)*
NEDO 20964	*Generation of Void and Doppler Reactivity Feedback for Application to BWR Plant Transient Analysis (August 1975)*
NEDC 20989-P	*Mark I Containment Evaluation Short Term Program Final Report, Addendum 2, "Loads and Their Application for Torus Support System Evaluation," June 1976*
NEDO 21291	*Group Notch Mode of the Rod Sequence Control System for Cooper Nuclear Station (June 1976)*

B. Bechtel Power Corporation Reports

Report No.	*Title*
BC-TOP-9A	*Design of Structures for Missile Impact Rev. 2 (September 1974)*
BN-TOP-1	*Testing Criteria for Integrated Leak Rate Testing of Primary Containment Structures for Nuclear Power Plants, Rev. 1 (November 1972)*
BN-TOP-2	*Design for Pipe Rupture Effects, Rev. 2 (May 1974)*
BP-TOP-1	*Seismic Analysis Piping System, Rev. 1 (February 1974)*

2.0 SITE CHARACTERISTICS

2.1 GEOGRAPHY AND DEMOGRAPHY (HNP-1 AND HNP-2)

2.1.1 SITE LOCATION AND DESCRIPTION

2.1.1.1 Specification of Location

The Hatch Nuclear Plant (HNP) site is located in Appling and Toombs Counties, Georgia, at the intersection of the Altamaha River with U.S. Hwy No. 1, as shown in figures 2.1-1 and 2.1-2. This location is ~ 98 miles southeast of Macon, Georgia, and ~ 73 miles northwest of Brunswick, Georgia. The Universal Transverse Mercator coordinates of the HNP-2 reactor, to the nearest 100 m, are Zone 17R LF 3,533,700 m N and 372,900 m E. These coordinates correspond to 82°20'39" W long. and 31°56'2" N lat. The HNP-1 reactor is located 149 ft 3 in. due north of the HNP-2 reactor.

2.1.1.2 Site Area Map

Figure 2.1-3 is a map of the HNP site area which includes 2244 acres. The site boundary and exclusion area boundary for practical purposes, coincide with the plant property line. The minimum distance from the HNP-2 reactor to the exclusion area boundary is ~ 4300 ft to the southwest. The exclusion area boundary, as determined per Title 10 Code of Federal Regulations (CFR) Part 100, for HNP is that area falling within 1250 m from the center of the plant site.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

The property lines shown in figure 2.1-3 are the boundaries for determining effluent release limits. Effluent releases at the boundary do not exceed the limits specified in the Offsite Dose Calculation Manual.

As indicated in figure 2.1-3, the minimum distances to the boundary from the main plant stack and the HNP-2 reactor building vent are 4100 ft and 4300 ft, respectively. The Altamaha River traverses the site north of the HNP-1 and the HNP-2 complexes. The distance from the HNP-1 reactor building to the nearest river bank is ~ 850 ft.

Use of the wildlife refuge area and the Boy Scout camp area is allowed only with prior permission from and notification of Southern Nuclear Operating Company (SNC). Under normal circumstances, use of the highway, County Road 451, wayside park, river, recreation center, and visitor center is not controlled. (See paragraph 2.1.2.2.) In the event of emergency conditions at the plant, the Emergency Plan provides for control of these areas. Control over access to the owner-controlled area and the protected area is maintained through implementation of the Security Plan described in section 13.7 and the Emergency Plan

described in section 13.3. Such access is monitored and controlled by the plant security force, the plant staff, and the implementation of administrative procedures.

2.1.2 EXCLUSION AREA AUTHORITY AND CONTROL

2.1.2.1 Authority

Georgia Power Company (GPC) owns the entire plant exclusion area in fee simple. Pursuant to the Nuclear Operating Agreement, GPC, for itself and as agent for the co-owners, has delegated to SNC complete authority to regulate any and all access and activity within the entire plant exclusion area. Minimum distance to the exclusion area boundary is discussed in paragraph 2.1.1.2.

2.1.2.2 Control of Activities Unrelated to Plant Operation

The following areas located within the exclusion area are those in which activities unrelated to the plant operation occur:

- U.S. Hwy No. 1.
- County Road 451.
- Wayside park adjacent to U.S. Hwy No. 1.
- Altamaha River.
- Wildlife refuge area.
- Boy Scout camp area.
- Visitor center.
- Recreation center.

The locations of these areas within the exclusion area are shown in figure 2.1-3. The exclusion area outside the controlled area fence is posted and, except for the highway, County Road 451, wayside park, river, and visitor center, is closed to persons not having received permission to enter the property.

Although the Emergency Plan provides for execution of passage control if emergency plant conditions occur, GPC does not normally control passage along the portion of U.S. Hwy No.1 and County Road 451 that lies within the exclusion area. The wayside park provides simple recreational facilities for public use, in addition to parking and picnicking facilities to simultaneously accommodate ~ 10 families. Limitations are not normally imposed upon park

use, although the Emergency Plan does provide for limitations in the event of plant emergency conditions. If emergency conditions occur, the plant security force, in conjunction with local law enforcement agencies, notify any persons within the park of the proper action to take.

GPC does not generally control passage or use of the Altamaha River within the exclusion area. The Emergency Plan provides for control over use of the river if emergency conditions occur. In the event of emergency conditions, the plant security force, in conjunction with local law enforcement agencies, notify any persons on the river of the proper action to take.

An estimate of river usage believed to be representative of peak daily usage on a given summer day was determined for Deen's Landing.[a] Usage of that portion of the river located within the exclusion area should not differ substantially from river usage at Deen's Landing.

Persons attempting to enter the wildlife refuge area without permission are considered to be trespassing; however, this area is not in use at the present time, although efforts were made to interest ecological groups in the area for the purpose of conducting ecological studies. If such use commences in the future, groups are anticipated to be small and to remain in the refuge area for short periods of time only.

A lease agreement between GPC and the Area Council of the Boy Scouts of America allows scouting groups to use the Boy Scout camp area. The lease agreement requires that all instructions given by GPC and, specifically, plans for evacuation are promptly adhered to and obeyed. The leader of each group using the area is given a set of emergency instructions to follow in the event of plant emergency conditions. Such notification would be made by the plant security force, in conjunction with local law enforcement agencies. In the past, the area has been used on weekends by Scouts, with the number of Scouts simultaneously using the area varying between 25 and 50. These visits, which are for weekends only, are expected to continue. In the future, the Area Council of Boy Scouts may possibly hold camporees involving 400 to 500 Scouts at the Boy Scout camp area on weekends only.

The recreation center is accessed from County Road 451, which originates at U.S. Hwy No. 1. Persons using the recreation center may occupy the center, its immediate area, and the parking lot immediately adjacent to the center. In the event of emergency conditions, the Emergency Plan specifies control of access to the recreation center, and the plant security force is responsible for notifying persons in the recreation center of the proper action to take.

The visitor center is accessed from the main plant access road that originates at U.S. Hwy No. 1. Persons visiting the center may occupy the center, its immediate area, and the parking lot immediately adjacent to the center. If a plant emergency condition occurs, the procedure for proper notification of visitors, in addition to the procedures to be executed by the visitor center director or designated alternate, is provided in the Emergency Plan. For the period of

a. Of 373 recreational manhours spent at Deen's Landing, 144 were boating hours.

August 1971 to January 1975, the center accommodated ~ 56,000 visitors, with the peak number of daily visitors being 860. The peak number of visitors does not exceed 1000 daily and 52,000 annually.

2.1.2.3 Arrangements for Traffic Control

SNC has arranged with the law enforcement agencies of Appling and Toombs Counties and with the Georgia Highway Patrol for control of traffic on U.S. Hwy No. 1 and County Road 451 in the event of an emergency. Because of the remote site location, plant personnel control the traffic in an emergency until officers of the aforementioned agencies arrive.

2.1.3 POPULATION DISTRIBUTION

At the time of submittal of the FSAR to support the license application, the information on population projections was based on the 1970 Census data. For the most current information regarding the population, schools, and recreational and public areas, as well as population density within the 16 meteorological zones, consult the Emergency Plan and the Annual Radiological Environmental Operating Report. For the most current information regarding operational dose estimates, consult the Annual Radiological Environmental Operating Report and the Annual Effluent Release Report.

2.1.3.1 Population Within 10 Miles

Figure 2.1-4 shows projected populations from the center of the plant site outward to a 10-mile radius.

The population projections for the area within a 50-mile radius of the proposed plant site were based on the 1970 Census data and the county population projections developed by the Georgia Social Sciences Advisory Committee.[1]

The total populations and the rural populations of the counties in question were obtained from the 1970 Census Report, and the rural population percentage was calculated. In the linear approximation developed, it is assumed that this percentage remained unchanged over the time interval of the study. The rural populations for the years 1980, 1990, 2000, and 2020 were determined by multiplying the rural population percentage of each county by the projected county population indicated in the aforementioned publications. Using the same formula, the rural population densities were calculated for each of these years. The rural densities for 1972, 1982, 1992, and 2012 were then determined by linear interpolation. Rural populations for each sector for these same years were determined by multiplying the rural densities for the counties involved by the appropriate area of each county falling within 1 of the 16 sectors. Total sector populations were found by adding the rural population to the projected city populations within a sector. Projected city populations were found by linearly projecting each city's 1970 population at its county's projected rate of growth. It was assumed that the growth rate for a given county would be a reasonable approximation of the growth rates of the cities within that county. Cities

were selected from the 1970 Census listing of all incorporated places and unincorporated places of 1000 or more.

After comparing the results of this method of projection for the HNP vicinity (50-mile radius) with the results of the ratio method of projection (the population ratio between the HNP vicinity and the whole U.S. population is multiplied by the projected U.S. population figures calculated by the U.S. Census Bureau), it was found that the method described in the above paragraph rendered a higher, more conservative projection.

2.1.3.2 Population Between 10 and 50 Miles

Figure 2.1-5 shows the projected population distribution between 10 and 50 miles from the plant site.

2.1.3.3 Transient Population

Within a 5-mile radius of the plant, the greatest population shifts on a daily basis should come from HNP and the Altamaha School, located ~ 4 miles SSE of the plant. The permanent HNP operating staff totals ~ 970 people, divided among 3 daily shifts. According to the Statistical Services Division of the Georgia Department of Education, the average daily attendance for the Altamaha School during the 1973-1974 school year was 313 students and 17 teachers. Because of hunting and fishing activities, some seasonal population fluctuations in this area occur; but these variations are likely to be insignificant.

2.1.3.4 Low-Population Zone

The low-population zone (LPZ), as determined per 10 CFR 100, for HNP is that area falling within 1250 meters from the center of the plant site. Figure 2.1-4 shows that this area is expected to remain sparsely populated during the anticipated life of the plant. For practical purposes, the Technical Specifications state that the LPZ coincides with the site and exclusion area boundaries.

In June 1973, GPC conducted a population survey over a 5-mile radius, the center originating at the main stack. The survey results showed a population of 1465 permanent residents within the 5-mile radius as compared to a population of ~ 840 permanent residents in 1970. The bulk of this population increase is believed to stem from the influx of construction workers, many residing in trailer parks near the site. Figure 2.1-6 shows the population breakdown per mile radius and directional sector. Also shown are the locations of the major trailer parks.

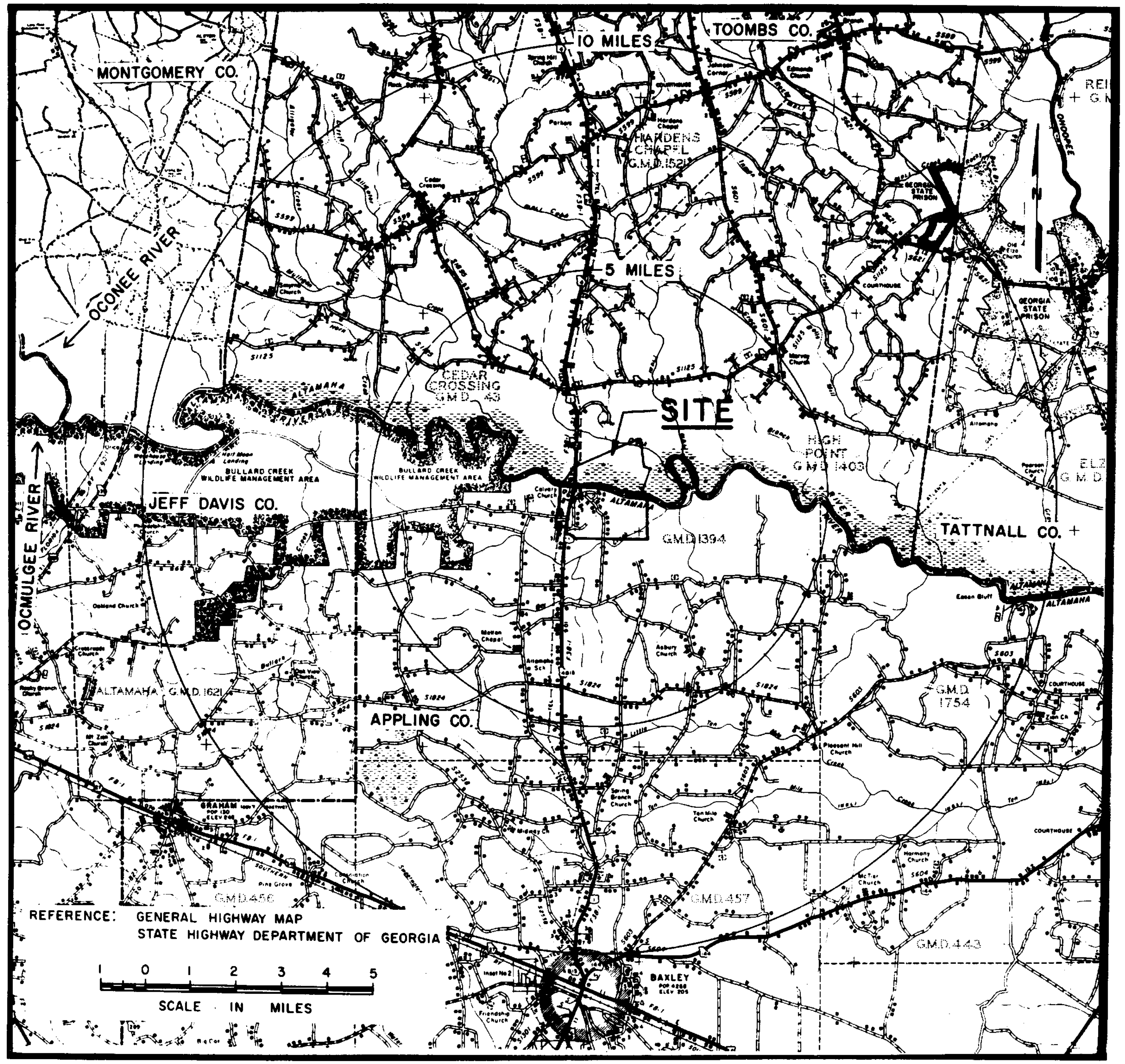
2.1.3.5 Population Center

The nearest population center, as defined in 10 CFR 100, is Savannah, Georgia, located ~ 67 miles ENE of HNP. In 1970, Savannah's population was 118,349; however, Savannah is located beyond the 50-mile radius of the HNP population study. In the period from 1960 to

1970, Appling County experienced a population decrease of ~ 4%, while Toombs County increased in population by almost 14%. Appling County's urban population decreased by 18% during this same period, but the rural population grew nearly 3%. At the same time, the Toombs County rural population declined slightly over 2%, but its urban population climbed by almost 23%. Recent experience seems to indicate that, in coming years, urban populations will increase at the expense of rural areas.

REFERENCES

1. Lyle, C. V., Chief Economist, Southeastern Region, Federal Water Pollution Control Administration, U. S. Department of the Interior, Georgia County Population Projections as Developed by the Georgia Social Sciences Advisory Committee, Georgia Social Sciences Advisory Committee, Atlanta, Georgia, February 1968.



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EDWIN I. HATCH NUCLEAR PLANT SITE

FIGURE 2.1-1

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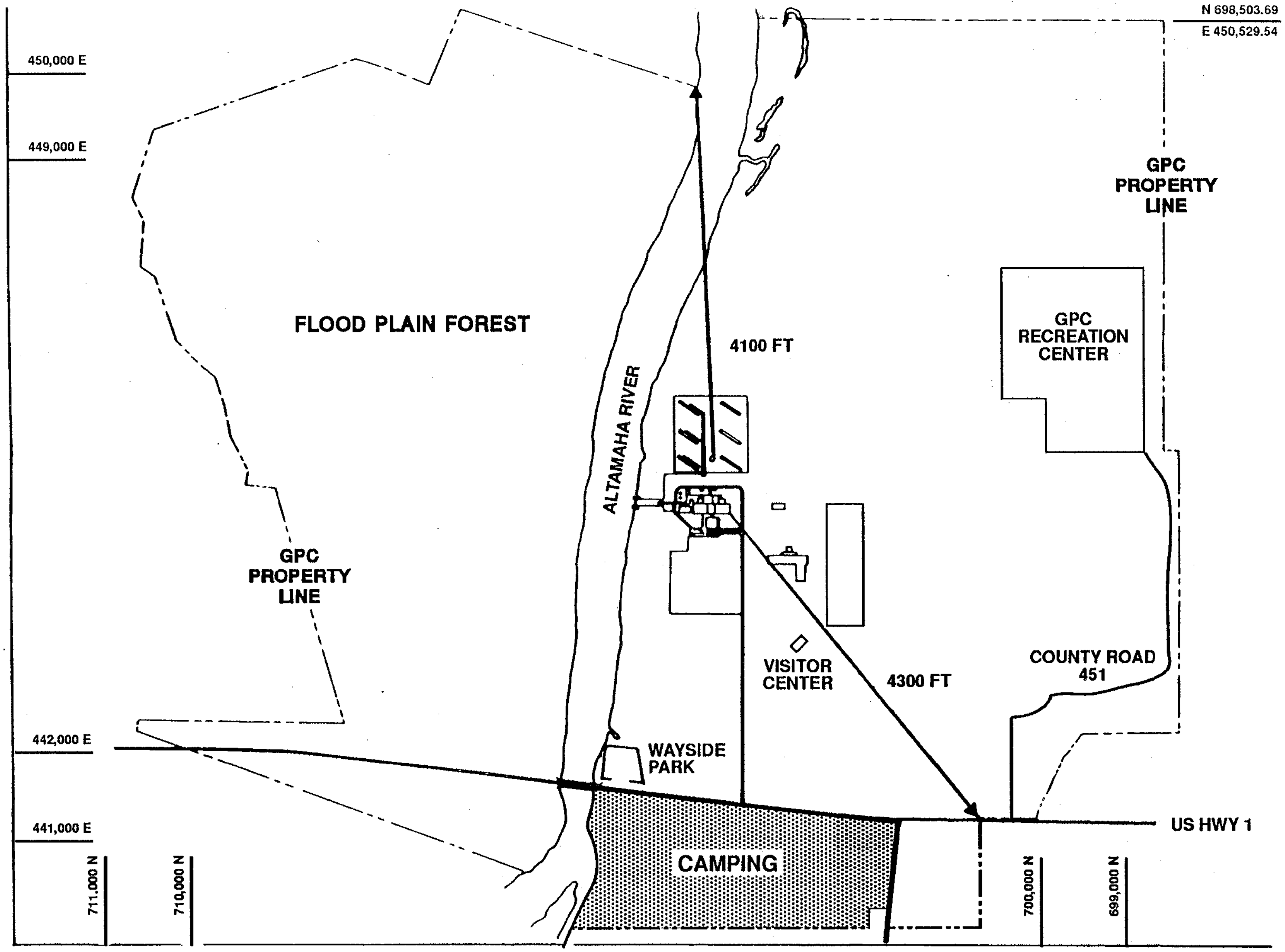
LOCAL SITE ENVIRONS

FIGURE 2.1-2



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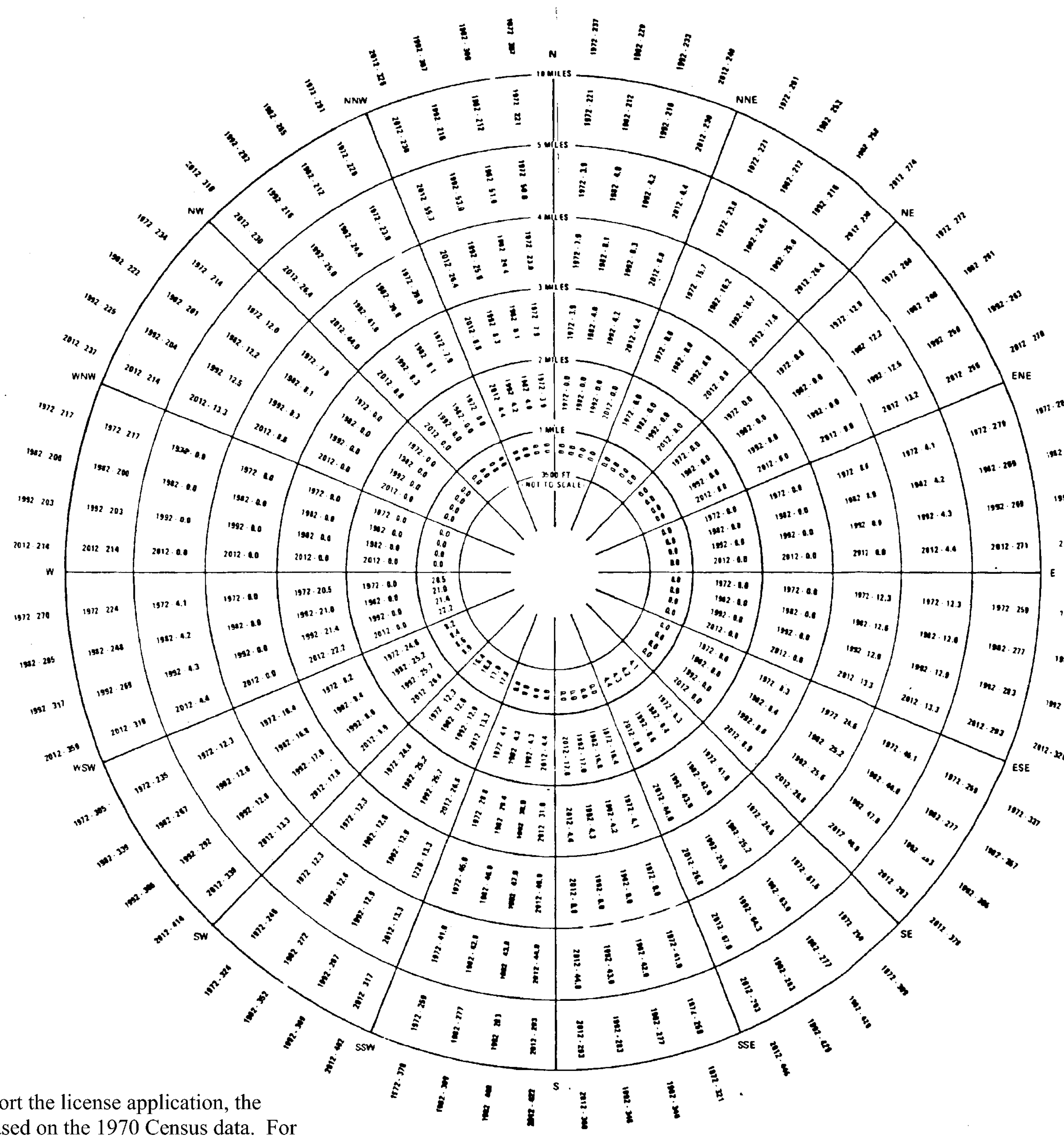
SITE PROPERTY PLAN

FIGURE 2.1-3



NOTE:
At the time of submittal of the FSAR to support the license application, the information on population projections was based on the 1970 Census data. For the most current information regarding the population, schools, and recreational and public areas, as well as population density within the 16 meteorological zones, consult the Emergency Plan and the Annual Radiological Environmental Operating Report.

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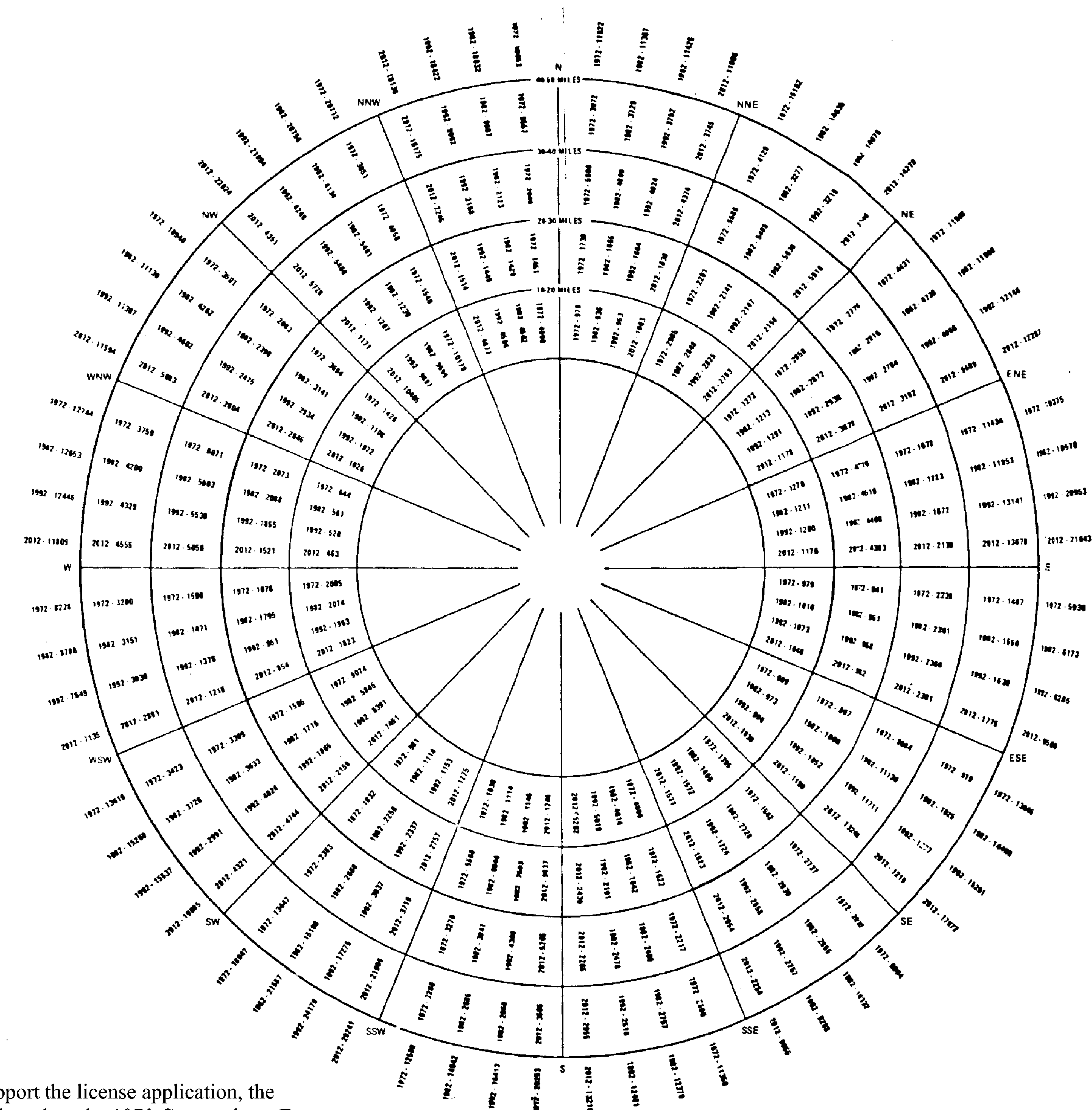
POPULATION DISTRIBUTION
(0-10 MILES)

FIGURE 2.1-4



NOTE:
At the time of submittal of the FSAR to support the license application, the information on population projections was based on the 1970 Census data. For the most current information regarding the population, schools, and recreational and public areas, as well as population density within the 16 meteorological zones, consult the Emergency Plan and the Annual Radiological Environmental Operating Report.

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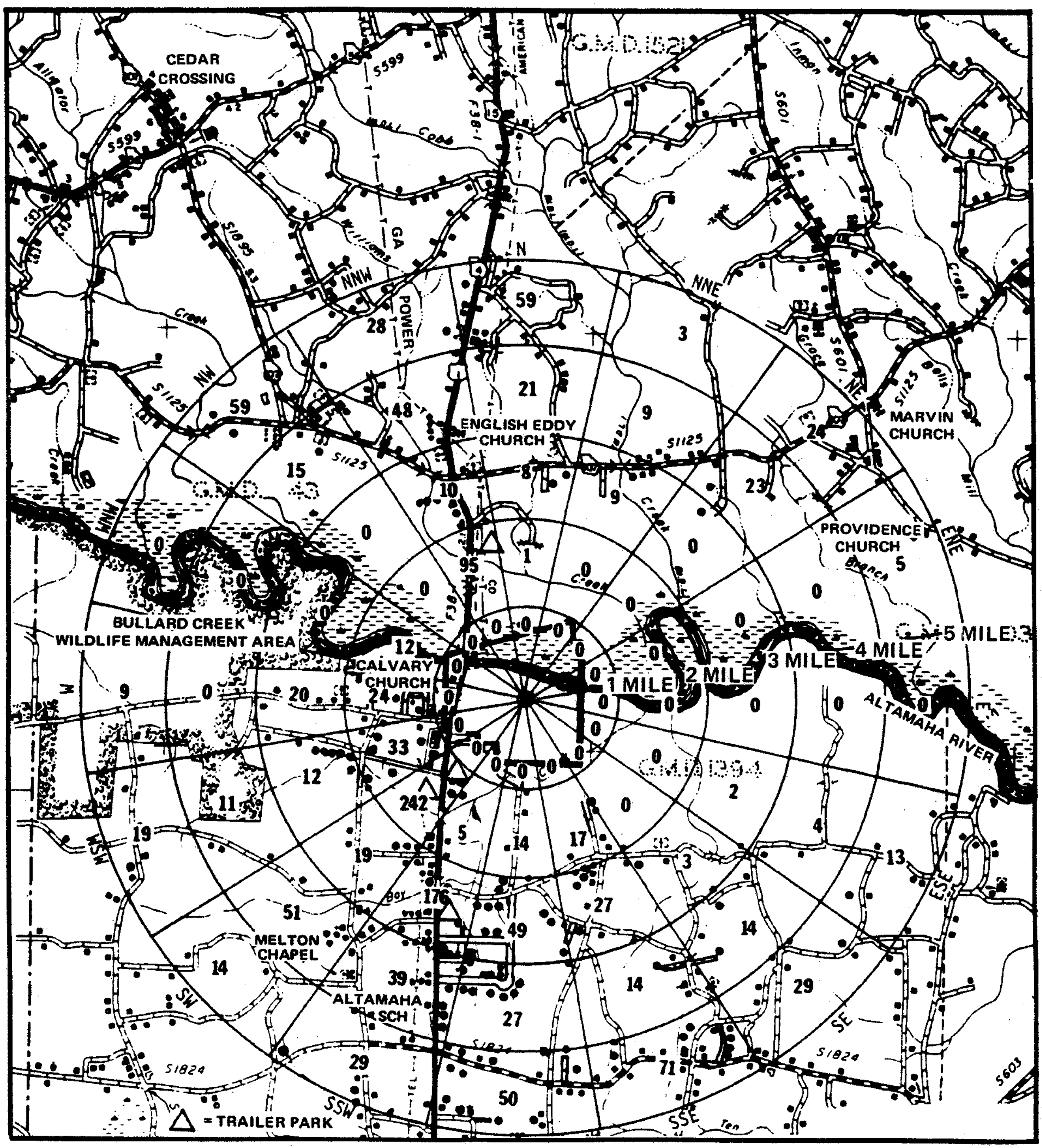
POPULATION DISTRIBUTION
(10-50 MILES)

FIGURE 2.1-5



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POPULATION DISTRIBUTION
(0-5 MILES)

FIGURE 2.1-6

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

2.2.1 LOCATIONS AND ROUTES

Figure 2.2-1 is a map of the site area showing the location of transportation routes and a pipeline.

2.2.2 DESCRIPTIONS

2.2.2.1 Description of Facilities

Within a 5-mile radius of Hatch Nuclear Plant-Unit 2 (HNP-2), there are no manufacturing plants, chemical plants, refineries, storage facilities, mining and quarrying operations, military bases, missile sites, transportation facilities, oil and gas wells, or underground gas storage facilities. Also, there are no known military firing or bombing ranges or aircraft low-level flight holding or landing patterns near the site area. There is truck traffic on U.S. Highway No. 1, which passes about 3500 ft west of the plant buildings. The nearest railroad passes about 10 miles southwest of the site. A spur line has been constructed to the site.

2.2.2.2 Description of Products and Materials

The cargo most frequently transported near the plant is longleaf and slash pine logs harvested from managed forest areas for pulpwood. There are no records available from either state or federal sources concerning the nature and quantities of potentially hazardous and/or explosive material that might be transported along U.S. Highway No. 1 in the vicinity of the plant site. Also, there are no apparent factors that should cause shipments along this route to differ significantly from shipments along any other federal highway. Since U.S. Highway No. 1 is a federal highway, it would be reasonable to assume that shipments of hazardous and/or explosive materials along it would conform to applicable federal and state regulations.

2.2.2.3 Pipelines

A Southern Natural Gas Company pipeline is located within ~ 4 1/2 miles of HNP-2 as shown on figure 2.2-1. The pipeline, which was designed for 1200-psi operation, carries natural gas at an operating pressure of 820 psi. The 12 3/4-in.-OD pipe ranges in wall thickness from 0.219 in. to 0.500 in. and in minimum yield strength from 35,000 psi to 52,000 psi. The pipeline was constructed in 1964 and is buried at a minimum depth of 30 in. Figure 2.2-1 shows the location with respect to HNP-2 of ASA 6OO No. M and J M3 12-in. gate valves that can be used as isolation valves in the pipeline. The pipeline is not used for storage of gas at higher than normal pressure. The Southern Natural Gas Company does not anticipate using the pipeline to carry a product other than natural gas.

2.2.2.4 Waterways

There is no commercial traffic on the Altamaha River in the site region. Deen's Landing, a commercial launching facility for small boats, is located slightly over a mile upstream from the plant (figure 2.2-1).

The only barge traffic on the Altamaha River in the vicinity of the HNP site is the snagging barge operated by the Corps of Engineers. It is estimated that this barge passes the site perhaps twice a year (once going upstream and once going downstream); however, it probably passes on a less frequent schedule. Since the intake structure is located on a straight portion of the river the barge would not be involved in any maneuvers that require it to move toward the intake structure.

2.2.2.5 Airports

The nearest airport with scheduled passenger service is in Savannah, Georgia, about 67 miles northeast of HNP-2. There are small municipal fields not used for scheduled commercial service at Baxley, about 13 miles south; Hazlehurst, about 16 miles southwest; Vidalia, about 20 miles north; and Alma, about 28 miles south.

2.2.2.6 Projections of Industrial Growth

The area within 5 miles of HNP-2 is largely rural, with most of the land being used for either residential or agricultural purposes. Much of the north-south vehicular traffic that traveled U.S. Highway No. 1 in years past now moves along federal interstate highways. Other than the development of several trailer parks to accommodate the influx of construction workers associated with HNP, this area has remained relatively stable over the last several years and shows no tendencies toward any drastic changes in the foreseeable future.

2.2.3 EVALUATION OF POTENTIAL ACCIDENTS

2.2.3.1 Determination of Design Basis Events

The accident categories discussed below consider the potential for accidents at other facilities or transportation routes affecting HNP-2.

A. Explosions

   There are no known facilities or activities within a 5-mile radius of HNP where the process, storage, or use of high explosives, munitions, chemicals, or liquid and gaseous fuels creates the potential for accidental detonations posing a threat to HNP-2.

Transportation routes within the 5-mile radius include the Altamaha River, a Southern Natural Gas Company pipeline, and the road system, principally U.S. Highway No. 1. Traffic on the Altamaha River in the vicinity of HNP is not of a nature that creates the potential for accidental detonations posing a threat to HNP-2. The Southern Natural Gas Company pipeline is ~ 4 1/2 miles from HNP-2 and is sufficiently distant that potential detonations would not affect HNP-2. U.S. Highway No. 1 passes ~ 3400 ft west of the HNP-2 plant structures. Accidents involving detonation of materials or cargoes in transit on the highway would be sufficiently distant that HNP-2 would not be affected.

B. Flammable Vapor Clouds (Delayed Ignition)

Accidental releases of flammable liquids or vapors that result in the formation of unconfined vapor clouds from locations outside the 5-mile radius of HNP should be sufficiently dispersed, even under the most adverse meteorological conditions, so that the concentration, by the time the cloud reaches HNP-2, is below the flammable point. The natural gas pipeline, likewise, is sufficiently distant that the resulting cloud should be dispersed below the flammable concentration. The distance of U.S. Highway No. 1 from HNP-2 and the comparative size of shipments that travel along the highway result in an exceedingly low probability of a cloud having a flammable concentration reaching HNP-2.

C. Toxic Chemicals

Transportation of toxic chemicals along U.S. Highway No. 1 is sufficiently distant from HNP-2 that the probability of a toxic concentration resulting from a potential release reaching HNP-2 is exceedingly low.

There are no known storage or transportation facilities within a 5-mile radius of HNP-2 that pose a threat to HNP-2.

The following chemicals are stored on site in bulk quantities: acid and caustic (used for makeup water demineralization) and sodium hypochlorite (used for treatment of circulating water, sanitary water, and plant service water). The capability to store sodium bromide, a corrosion inhibitor, and a silt dispersant (for treatment of service water systems) is also provided. In normal operation, fumes from these chemicals are not toxic. However, if mixed together, sodium hypochlorite and acid could generate and release molecular chlorine gas. Precautions are, therefore, taken to make certain that only the required chemical can be put into the respective storage tank. Administrative controls have been established to ensure that chemical delivery trucks are escorted on site and are sampled prior to unloading to ensure the correct chemical is being supplied to the tank. Tank fill connection valves are kept locked closed, and the chemistry department personnel (who perform the sampling) control the keys. In addition, in the same way that caustic connections are designed, the fill connections on the water treatment chemical storage tanks are a type that is incompatible with the acid truck discharge hose connection.

D. Fires

There are no nearby industrial, chemical, or storage facilities from which effects of fires pose a threat to HNP-2. The Southern Natural Gas pipeline is sufficiently distant that a fire associated with the pipeline should not affect HNP-2. Likewise, fires associated with transportation accidents are sufficiently distant as not to affect HNP-2. The terrain and ground cover surrounding HNP-2 are of nature that is not conducive to forest or brush fires that might otherwise affect HNP-2.

If, however, a fire in the site area causes smoke to drift to the main control room air intake, control room personnel can manually isolate the control room and initiate the recirculation mode (subsections 6.4.1 and 15.4.4). No mechanical or electrical smoke detection apparatus is provided at the intake to warn the control room operators of smoke being drawn into the intake. It is expected that an operator on duty will detect the condition long before the room becomes uninhabitable. Further it is unlikely that control room personnel would not be aware of the existence and location of a fire of sufficient magnitude to engulf the plant with smoke. Smoke particles that enter the room prior to manual isolation will probably settle as dust. If the concentration in the main control room becomes heavy, portable breathing apparatus already available within the room may be deployed during the interval preceding and immediately following manual isolation, while the smoke particles settle out or are captured by the recirculation filter system. When conditions permit, and if desired, the purge mode to remove lingering odor within the main control room can be initiated manually.

E. Collisions With Intake Structure

There is no commercial barge traffic on the Altamaha River in the vicinity of HNP-2 at present and no future traffic is anticipated.

Barge traffic on the river is required to have a permit from the U.S. Army Corps of Engineers; a permit is not required of rafting or other movement of logs under Title 33 USC Sections 554 and 555. At present, there are no permits or applications for permits from the Corps of Engineers for any barge traffic on the river. The major companies with forestry operations in the vicinity of the river upstream of the plant site do not use barge logs. Only one of these companies has used barges to move logs down the river in the past. This particular company discontinued the use of barges prior to 1972 and has since disposed of all barges, tugboats, and other equipment that was used in its barging operations. The company has no plans to use barges on the river in the future.

The Savannah District Corps of Engineers removes snags and fallen trees from the river during a period of 4 to 6 months each year. The material removed by the Corps is placed on the river bank. For this operation, the Corps uses a barge 110 ft by 30 ft with a 7-ft draft which displaces 126 long tons and a towboat 61 ft by 21 ft with a 6-ft draft which displaces 80 long tons. Maximum speed of the barge and towboat is 5 mph. The towboat and barge pass the plant site at most once moving upstream and once downstream per year and possibly as seldom as once

every 2 or 3 years. However, as the towboat and barge pass the plant site they engage in snagging operations and move, at most, at less than maximum speed. Based on accident statistics for the years 1968 through 1973 collected by the U.S. Coast Guard,[1] the national average accident frequency involving all barges where damage was in excess of $1500 was found to be 0.42 accidents per million miles.[2] The frequency of all types of barge accidents is therefore 0.42×10^{-6} per mile. Runaway barge accidents form a small subset of all accidents since most barge accidents are caused by impact with bridges, weirs, spillways, piers, other barges, etc., but not due to runaways. A very conservative estimate of the number of runaways per total number of accidents is estimated at 0.1.[3] In this report, runaway barges are classified as being due to material failure (e.g., broken towline) and are found to represent 4% of all barge accidents; a conservative estimate of 10% is used. Assuming that the barge can run away in any direction with equal probability results in a probability of 0.5 that it will strike the side of the river on which the intake structure is stationed. The probability that a barge would run away and strike the side of the river on which the intake structure is located, within a mile of shoreline which contains the intake structure, then becomes:

$$(0.42 \times 10^{-6}) \times (0.1) \times (0.5) = 0.21 \times 10^{-7}$$

Assuming that the barge runs away and hits the side of the river that has the intake structure, the probability of striking the intake structure is ~ equal to the ratio of the intake structure width to the width of the shoreline that the barge is assumed to strike (in this case 1 mile) which is equivalent to the intake structure width in miles (including the width of the sheet piles) or 0.03 miles. Therefore the probability that a barge passing the intake structure would run away and strike the intake structure becomes:

$$(0.21 \times 10^{-7}) \times (0.03) = 0.63 \times 10^{-9}$$

It is concluded therefore that, even if there were as many as 100 barges per year (in reality there are less than two) passing the HNP site, the probability of a barge running away and striking the intake structure is very remote (i.e., $< 10^{-7}$ occurrences per year).

The intake structure is protected, however, by sheet pile cells from a direct hit by river traffic or debris moving in the direction of the river flow. The cells are comprised of soil-filled sheet piling and are 63 ft in diameter, extend from elevation 22 ft to elevation 105 ft and weigh ~ 14,500 tons.

F. Liquid Spills

There is no commercial barge traffic on the Altamaha River in the vicinity of HNP-2 at present. The nearest industrial plant upstream of HNP-2 is located near Macon, Georgia. If any appreciable amount of corrosive, cryogenic, or coagulant oil or liquid is released into the river from an upstream location, the material should be diluted substantially before reaching the intake structure.

Heat transfer areas of the heat exchanges might be affected initially. However, conservative sizing of heat transfer surfaces and continuous flushing of service water flow negates the effect of such materials on the heat exchangers.

2.2.3.2 Effects of Design Basis Events

Potential accidents considered above should have a negligible effect on HNP-2.

REFERENCES

1. U.S. Coast Guard Headquarters, Computer File on All Accidents Involving Damage in Excess of $1500, Washington, D.C., 1974.

2. U.S. Department of Commerce, "A Model Economic and Safety Analysis of the Transportation of Hazardous Materials in Bulk," Report to Office of Domestic Shipping by Arthur D. Little, Inc., Cambridge, Massachusetts, July 1974.

3. U.S. Coast Guard, "Statistical Summary of Casualties to Commercial Vessels on Western Rivers," November 1973.

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INDUSTRIAL AND MILITARY FACILITIES, TRANSPORTATION AND PIPELINE ROUTINGS IN SITE ENVIRONS

FIGURE 2.2-1

2.3 METEOROLOGY (HNP-1 AND HNP-2)

2.3.1 REGIONAL CLIMATOLOGY

2.3.1.1 Data Sources

Climatic references used to evaluate the site meteorology are listed at the end of this section. (See references 1 through 8.)

2.3.1.2 General

The site is located in the middle coastal plain region of Georgia, which is characterized by mild, short winters; long periods of mild, sunny weather in the autumn; and somewhat more windy but mild weather in the spring. Summers are warm and humid, being affected by maritime air from the Atlantic; but long periods of extremely hot weather over 100°F are unusual.

The climate of the site area is characterized by the mean and extreme temperatures and precipitation of Glennville and Lumber City, Georgia. Glennville is located about 24 miles east of the site and Lumber City about 20 miles west of the site. Since there is close agreement in the data from these two stations and the Glennville Station has a longer period of record, only the Glennville data are shown on figures 2.3-1 and 2.3-2.

Also used in the climatic study were the first-order stations of Savannah, Georgia, 75 miles east of the site, and Macon, Georgia, 98 miles northwest of the site. Data from these cities are shown in tables 2.3-1 and 2.3-2.[(1)] These stations are the closest first-order stations and were used to provide more complete data not available at the other two stations for the site region.

During the 46-year period 1920 to 1965, for which 39 years of record are available, there have been 10 tornadoes within a 25-mile radius of the site. During the period 1956 through 1965 inclusive, there were seven tornadoes reported within this radius. The higher frequency in the latter period is probably due to improved reporting.[(9)(10)]

During the 49 years of record 1915 to 1965,[(11)(12)(13)] there were 29 hurricane or post-hurricane paths which passed within 100 miles of the site. Since the plant is ~ 80 miles from the coast, the hurricane windspeeds are generally lower than those further to the east or south. It is expected that the analogous windspeed at the site would have been less. High winds are discussed in paragraph 2.3.1.3.

Snow and ice storms are very rare in the region. The average annual snowfall for the region is < 1/2 in. The maximum snowfall in a 30-year period of record at Glennville was 4 in. in 1973.[(14)]

The area is subject to a relatively high incidence of slow-moving anticyclones associated with high air pollution potential (paragraph 2.3.1.3).

2.3.1.3 Severe Weather

A. Heavy Precipitation

The heaviest precipitation of several hours' duration usually occurs with tropical storms in the late summer and fall and with coastal storms in the winter. Heavy rains of short duration occur in thunderstorms, which average about 2 out of every 5 days from June through August.

Rainfall frequencies from 30 min to 24 h for return periods of from 1 to 100 years are shown in the following table. The figures were interpolated from maps in reference 2.

Amount of Rainfall (in.) in a Given Period

Recurrence Interval	30 min	1 h	6 h	24 h
1 year	1.4	1.8	2.5	3.4
5 years	2.0	2.5	3.8	5.4
10 years	2.2	2.8	4.5	6.2
25 years	2.5	3.2	5.2	7.0
50 years	2.8	3.5	5.8	7.8
100 years	3.0	3.8	6.5	8.8

Maximum recorded rainfall has been tabulated below for Macon and Savannah, which are the nearest first-order stations most representative of the site for periods ranging from 5 min to 24 h.[3] The amount is an average of the Macon and Savannah amounts for the time period.

1906-1961		1899-1961	
(min)	(in.)	(h)	(in.)
5	0.77	2	5.75
10	1.23	3	6.00
15	1.65	6	6.60
30	2.55	12	8.50
60	3.95	24	10.00

The values of rainfall return amounts in the first table, paragraph 2.3.1.3A, were interpolated directly from reference 2 for the Hatch site area. The amounts in reference 2 for 1- to 10-year periods were derived using a partial-duration series, which takes the highest rainfall values for a station, regardless of year, to find frequency of occurrence. These types of data stations were used in the study: the first-order Weather Bureau stations, the recording gage hydrologic network, and the nonrecording gage data with daily observations. From 1 to 10 years, the data curves are based entirely on empirical calculation of the partial-duration series.

For periods longer than 20 years, the Gumbel procedure was used for fitting annual series data (which uses the highest rainfall amount for each year) to the Fisher-Tippett type I distribution.[15]

Peak rainfall amounts reported in the second table, paragraph 2.3.1.3A, are an average of the two maximum rainfall amounts from the two specific recording stations (Macon and Savannah)[3] for each time period. As expected, the resulting peaks are generally higher than for the longest return period (100 years) reported in the first table, paragraph 2.3.1.3A. However, it is not considered irregular for values in the return-period table to exceed peak values because of the statistical methods used to estimate return period values.

B. Hail

Heavy hail (greater than 3/4 in. in diameter) occurs in this area ~ 3 times in 13 years, or about once in 4 years, for a 1-degree (latitude and longitude) "square" (figure 2.3-3). For a 2-degree square, the total reports were about five for the 13-year period, which is consistent with those for the 1-degree square[4] (figure 2.3-4).

C. Ice Storms

Freezing rain, resulting in occasional heavy loading, is a very rare occurrence in the site area. Based on a 9-year study, it is estimated that one storm will occur about every 9 years.[5] Maximum accumulation of between a trace and 0.25 in. can be expected.

D. Thunderstorms

The number of thunderstorms in the site area is related to other weather phenomena, including strong winds (paragraph 2.3.1.3H), heavy precipitation (paragraph 2.3.1.3A), and lightning (paragraph 2.3.1.3E). Based on a 25-year period of record at Macon[1] and a 23-year period of record at Savannah,[1] the annual number of days in which thunderstorms occur for the site region is about 60 days per year (tables 2.3-1 and 2.3-2). About 40 of the thunderstorms occur during the summer months associated with the warm, humid, subtropical climate of the region. The remainder of the thunderstorms are scattered throughout the year with a minimum in the winter.

E. Lightning

The probability of lightning striking a particular point on the ground is extremely small. However, during a thunderstorm it is not uncommon for lightning to strike the ground. For example, based on National Weather Service records[16] from Macon, Georgia, during the summer months of June, July, and August 1975, lightning was estimated to have struck the ground at some time during 55% of the thunderstorms. In about 16% of these thunderstorms, cloud-to-ground lightning

was coded as "frequent." As stated in paragraph 2.3.1.3D, about 60 thunderstorms occur annually.

F. Tornadoes

The probability of a particular point being affected by a tornado is a function of the average number of tornadoes occurring in a given area and the average area covered by a tornado. Based on a 13-year study[4] from 1955 through 1967 reported in figure 2.3-5, the average number of tornadoes is about 13 (or about 1 per year) for the 1-degree square. The total number of tornadoes for a 13-year period[4] for 2-degree squares is shown in figure 2.3-6.

The area encompassed by a typical tornado has been estimated by Thom to be 2.82 mi^2.[17] The 1-degree square at this latitude (32°45') has an area of ~ 4050 mi^2. A conservative estimate of the chance that a given point will be affected by a tornado in a given year, P_s, is therefore approximately:

$$P_s = \frac{\left[\frac{13}{13}\right] x\, 2.82}{4050} = 0.0007$$

Thus, a given point can be expected to be affected by a tornado about once in 1436 years, on the average.

G. Probability of High Windspeeds Due to Tornadoes

Probabilities of high windspeeds due to tornadoes have been estimated for this site, using a document issued by the NRC.[18] The probability of strong winds in a tornado striking a specific site is a function of two factors:

1. The frequency of tornadoes in the site region.

2. The intensity probability.

The frequency has been estimated above; however, there are few actual observations of winds associated with tornadoes. Data concerning tornado intensity was collected by NOAA climatologists during 1971 and 1972.[18] For the contiguous United States, 1612 tornadoes were graded on intensity as shown in table 2.3-3. The 1612 tornadoes are categorized into wind groups in table 2.3-4 and into cumulative probability of intensity in table 2.3-5. Tables 2.3-3, 2.3-4, and 2.3-5 are from reference 18.

The probabilities in table 2.3-5 have been plotted on log probability paper in figure 2.3-7 to show the probability of a tornado with a given windspeed.

In reference 18, it is suggested that the design basis tornado should have a probability of occurrence of about 10^{-7} per year. The tornado wind with a

probability of occurrence on this order has been estimated, using the following computational procedure from reference 18.

The intensity probability, P_i, can be calculated using the following relationships:

$$P_s P_i \leq 10^{-7}$$

$$\frac{1}{1436} P_i \leq 10^{-7}$$

$$P_i \leq 0.000144$$

$$P_i \leq 0.0144\%$$

From figure 2.3-7, the 0.0144% probable windspeed in a tornado is about 350 mph. This would be represented by a tornado having a rotational speed of about 300 mph moving horizontally at a speed of ~ 50 mph.

Figure 2.3-8 shows calculated tornado windspeeds by 5-degree squares for 10^{-7} probability.

H. Strong Winds

The frequency of strong winds (50 knots or greater), as estimated from damage reports, has been analyzed in the WBTM FCST 12[(4)] for the 13-year period 1955 through 1967. The results are shown in figures 2.3-9 and 2.3-10, giving frequencies for 1- and 2-degree squares, respectively. For the site, the number of occurrences in the 13-year period was ~ 11 per 1-degree square, and ~ 40 for the 2-degree square, or about 1 per year for the 1-degree square.

The occurrence of strong winds is usually in conjunction with strong cyclonic disturbances and with thunderstorms, mostly in the summer.

I. High Air Pollution Potential

The site region experiences a relatively high incidence of slow-moving anticyclones, resulting in high air pollution potential, especially in the autumn. Korshhover has reported on the climatology of stagnating anticyclones east of the Rocky Mountains between 1936 and 1970.[(7)] In his study, he found that in the region of the site there were ~ 8 stagnation days per year. These forecasts are based mainly on expected duration of conditions that cause accumulation of pollutants over a large area.

2.3.2 LOCAL METEOROLOGY

2.3.2.1 Data Sources

Climatological information for the site is provided by the first-order Weather Bureau observations at the Savannah and Macon, Georgia, airports. The climate of the site region will be somewhat different due to the slight modification of the climate around Savannah by the ocean and the slight modification of the Macon area by the higher ground.

In addition, the site meteorological measurement program described in subsection 2.3.3 has been in operation since May 1970. Several years of these data have been summarized and used where appropriate in the sections which follow.

2.3.2.2 Normal and Extreme Values of Meteorological Parameters

A. Wind

The mean windspeed for each month and the most frequent wind direction are listed in table 2.3-1 for Savannah and table 2.3-2 for Macon. Maximum winds occur in the winter and spring, with maximum speeds of 9.5 mph and 9.6 mph in March at Macon and Savannah, respectively. The slowest winds occur in the summer and early fall with a minimum mean speed of 6.5 and 7.0 mph in August at Macon and Savannah, respectively. The average windspeed for the fastest mile on record (25 years ending 1973) was 70 mph in August 1961 at Macon and 90 mph in August 1940 at Savannah (33 years ending 1973). At the site region, Thom[8] estimates that, at 30 ft above ground, speeds of 80 mph occur once in 50 years and speeds of 100 mph occur once in 100 years.

B. Temperatures

Table 2.3-1 lists monthly averages of the daily maximum, daily minimum, and daily mean (the arithmetic average of the maximum and minimum) temperatures for the climatological normal period 1941 through 1970, compiled from the Savannah Airport. Table 2.3-2 lists similar information for the Macon Airport. The normal daily maximum ranges from 61°F at Savannah and 59°F at Macon in January to 91°F and 92°F in July at Savannah and Macon, respectively. The average daily minimum ranges from 39°F in January and 71°F in July at Savannah, to 37°F in January and 71°F in July at Macon. At Savannah, the record maximum was 105°F in July 1879, and the record minimum was 8°F in February 1899. At Macon a record maximum temperature of 106°F was recorded in June 1954 and a record low of 3°F in January 1966.

For the 30-year period 1944 through 1973, the extreme maximum and minimum temperatures were calculated using the Lieblein Analysis[19] for return periods of 50 and 100 years. Using data from Savannah and Macon and the reduced variate

Lieblein Analysis, the extreme temperature values for the site region are as follows:

Savannah

Return Period	Maximum Temperature (°F)	Minimum Temperature (°F)
50 years	105.5	8.0
100 years	111.0	-1.4

Macon

Return Period	Maximum Temperature (°F)	Minimum Temperature (°F)
50 years	107.0	3.1
100 years	113.3	-7.5

Based on a 9-year period of record, there are about 70 days (80 days for Macon, 58 days for Savannah) in which the maximum temperature is 90°F or above in the site region (tables 2.3-1 and 2.3-2). Most of these days occur during the summer months.

The growing season in the site region averages about 260 days, from an average date of last freeze of March 5 to a first freeze in autumn of November 20.

C. Water Vapor

Normal relative humidities at 4 synoptic hours, based on 9 years of data, are given in tables 2.3-1 and 2.3-2. They illustrate a moderately humid climate with normal afternoon humidities around 50% in both winter and summer. The spring is the least humid time in terms of relative humidity.

D. Precipitation

Tables 2.3-1 and 2.3-2 list the normal monthly precipitation, the maximum, the minimum observed in a month, and the maximum in 24 h for the respective periods at Savannah and Macon. The maximum 24-h precipitation at Savannah was 11.44 in. in September 1928. At Macon, the maximum 24-h precipitation was 8.36 in. in August 1928.

E. Fog

Heavy fog, with visibility less than 1/4 mile, occurs annually 39 days at Savannah (table 2.3-1) and 25 days at Macon (table 2.3-2). The fog days at Macon are more representative of the site because both Macon and the site region are inland. At Macon, a maximum of about 4 heavy fog days would occur during each winter

month and a minimum of about 1 day a month during the months of April, May, June, and July.

2.3.2.3 Potential Influence of the Plant and Its Facilities on Local Meteorology

The HNP-1 cooling towers consist of four mechanical draft counter-flow cooling towers, while HNP-2 utilizes three mechanical draft cross-flow cooling towers and one counter-flow mechanical draft cooling tower to dissipate waste heat to the atmosphere. The HNP cooling towers utilize state-of-the-art drift eliminators that reduce the maximum drift loss for the cross-flow towers to 0.008% of the circulating water flow and for the counter-flow tower to 0.005% of the circulating water flow. Thus, the plant and its facilities are not expected to have any significant effect on local meteorological conditions.

Experience with cooling towers of the general type located at the plant has resulted in no significant adverse environmental effects. Sustained ground fog is not expected to occur from tower operation; however, during high winds, wisps of the visible plume may briefly intersect the ground near the towers. In the NRC's analysis[20] of the effects of the mechanical draft cooling towers at the proposed Barton Nuclear Plant, the NRC staff concluded that icing conditions were not expected to occur because of the buoyancy of the cooling tower plumes. Therefore, as a result of the small amount of drift (i.e., water droplets) emitted from the towers and the buoyancy of the cooling tower plumes, no ice deposition problem at HNP is anticipated. To protect the cooling towers from freezing during low temperature operation, the capability exists to bypass the cooling towers until the water temperature in the cooling tower flumes and basins has increased. Negligible increases in relative humidity in the site region would result from tower operation.

2.3.2.4 Topographical Description

A topographic map of the site region is shown in figure 2.3-11. Topographic cross-sections for each of the 16 direction sectors are included in figure 2.3-12. A site topographic map is shown in figure 2.3-13.

2.3.3 ONSITE METEOROLOGICAL MEASUREMENT PROGRAM

The onsite meteorological measurement program began in April 1970. The original 150-ft tower is located in a cleared area (figure 2.3-13) and now serves as a backup tower for the new primary tower. Pertinent meteorological parameter instrument elevations and descriptions are given in table 2.3-7. The meteorological tower, instrumentation, and recorders were installed and shared with Unit 1. Windspeed, direction, and vertical temperature differences are recorded in the main control room (MCR) for use by both units. Data are continuously recorded on recorders. Preventive and routine maintenance are performed by Southern Nuclear Operating Company personnel in accordance with the instrument manuals. These personnel also perform emergency repair work to minimize outages and to ensure maximum data recovery. Calibrations are performed semiannually.

Meteorological data are normally reduced to 15-min averages centered on the hour. These data are then converted to engineering units and summarized to provide averages representative of each hour of data. These hourly averages provide the information from which monthly, seasonal, and annual summaries can be prepared as required.

For this report, 4 years of records collected from the original site tower from June 1970 through September 1974 have been used. During each of the 1-year periods, the following approximate percentages of data recovery were achieved for each parameter used in this report:

		Percent Recovery			
Item	Parameter	6/70-5/71	9/71-8/72	9/72-8/73	9/73-8/74
1	75-ft windspeed	97.8	98.1	95.7	93.7
2	150-ft windspeed	98.8	80.3	97.9	96.4
3	75-ft wind direction	99.2	97.6	96.5	96.7
4	150-ft wind direction	99.1	99.6	97.5	95.2
5	$\Delta T_{150\text{-}33\text{ ft}}$	99.6	98.7	97.0	90.4
6	Combined 75-ft windspeed, 75-ft wind direction, $\Delta T_{150\text{-}33\text{ ft}}$	97.3	96.2	92.0	85.3
7	Combined 150-ft windspeed, 150-ft wind direction, $\Delta T_{150\text{-}33\text{ ft}}$	98.2	78.5	95.2	86.7

In September 1972, the lower temperature sensor was moved from 10 ft to 33 ft to avoid ground effects. For the first 2 years of data, a temperature difference correction factor was applied assuming a logarithmic relationship between temperature and elevation above grade. The correction factor for the ΔT between 150 ft and 10 ft to provide an effective ΔT between 150 ft and 33 ft was determined as follows:

$$\Delta T_{150\text{ ft - }33\text{ ft}} = f \times \Delta T_{150\text{ ft - }10\text{ ft}}$$

$$f = \left[\frac{\mathrm{Ln}\frac{150}{33}}{\mathrm{Ln}\frac{150}{10}} \right] = 0.56$$

Figure 2.3-14 is a wind rose from the 150-ft level for the 4-year period of record. Figure 2.3-15 shows wind roses for each month and season for the 150-ft data. Joint frequency of windspeed and direction by temperature difference group are shown in table 2.3-9 for the 75-ft level, and in table 2.3-10 for the 150-ft level for the period from June 1, 1970, to August 31, 1974.

As part of the station emergency response plans, an upgraded meteorological system has been installed on site in accordance with the meteorological guidance of the Proposed Revision 1 to Regulatory Guide 1.23, "Meteorological Programs in Support of Nuclear Power Plants," and Revision 1 of NUREG-0654, "Criteria for Preparation and Evaluation of Radiological Emergency Response Plans and Preparedness in Support of Nuclear Power Plants." The upgraded system is capable of making reliable meteorological measurements. The new meteorological measurement program is described in the following paragraphs.

The upgraded measurement system includes a 100-m meteorological tower, designated as the primary tower, and the existing 45-m (150-ft) tower, which has been reinstrumented to serve as a backup system to the primary tower.

The new 100-m primary tower has been erected in an open field, 0.75 miles south-southwest of the power blocks within the plant boundaries, as shown in figure 2.3-13. The meteorological tower is instrumented at three levels (10 m, 60 m, and 100 m) to characterize the conditions for diffusion estimates of radiological releases at different levels. The meteorological parameters measured on the tower, models of the sensors employed, and parameter accuracy are given in table 2.3-7. The tower is equipped with a boom elevator, which eliminates climbing the tower to perform sensor maintenance.

The backup meteorological tower is instrumented at the 10-m and 45-m levels. Parameters measured on the tower and the instruments' accuracies are listed in table 2.3-7.

Both the meteorological towers and their associated equipment buildings are designed for lightning protection and are also connected to a power system that includes redundant power sources. In addition, a heating, ventilating, and air-conditioning system for each equipment building is provided to maintain building temperatures within equipment tolerance limits.

Signals from the meteorological sensors are conditioned for transmission in the associated equipment building. The signals are then transmitted independently to the recorders and the digital data acquisition system in the MCR. The recorders, which serve as backup data recording equipment to the digital data acquisition system, are located in the MCR. The recorders and data acquisition system are shared by HNP-1 and HNP-2. The required 15-min averages of meteorological parameters for diffusion estimates are reduced from data recorded by the data acquisition system.

2.3.4 SHORT-TERM (ACCIDENT) DIFFUSION ESTIMATES

2.3.4.1 Objective

In this section, estimates of atmospheric dilution factors are made based on 4 years of HNP site meteorological data. Probability distributions are drawn and values are reported which have a 5% and 50% probability of occurrence for each time period used in the safety analysis in chapter 15. Estimates of atmospheric dilution factors are applicable to HNP-1 and HNP-2.

Methods used to estimate diffusion conditions for evaluating short-term accident releases (< 1 h) are discussed in paragraph 2.3.4.2.1, and methods for assessing the consequences of longer term accident releases (from 1 h to 30 days) are discussed in paragraph 2.3.4.2.2. Diffusion conditions for the main steam line break accident (MSLBA) are also discussed in paragraph 2.3.4.2.1. However, methods used to estimate diffusion conditions for evaluating design basis accident (DBA) releases for the MCR and technical support center (TSC) are discussed in subsection 2.3.6, instead of this subsection.

2.3.4.2 Calculations

Conservative values of accident diffusion estimates are given in table 2.3-11 for both stack and ground-level releases. Derivation of these values is as follows:

2.3.4.2.1 Short-Term Accident Diffusion Estimates

A. Releases From Vents or Leaks Which Are Trapped in the Wake of Plant Structures

To determine the atmospheric dispersion appropriate for short-term (1 h or less) releases in the wake of plant structures, a plot of cumulative centerline X/Q values as a function of probability of occurrence is made for each of the 4-year periods of site hourly data. Statistical distributions plotted for the 1-h cases are constructed by computing X/Q values for each hour of the period of onsite records and then counting all of the hours that had X/Q values equal to or greater than selected values. The number of hours so obtained is then divided by the number of hours in the total period of record to obtain the probability that the selected X/Q value would be equaled or exceeded. Values found for each separate year were averaged to obtain the 4-year estimate. The resulting probabilities are independent of wind direction.

Equations and methods used to compute X/Q values are discussed in paragraph 2.3.4.2.4. Pasquill diffusion categories used for each hour are based on vertical temperature difference measurements as described in paragraph 2.3.4.2.3. Building wake is accounted for as described in paragraph 2.3.4.2.4. Calms are assumed to have a windspeed of 1.0 mph and the measured diffusion condition.

For analyses of ground-level releases, measured values of speed at 75 ft were extrapolated to the 33-ft level, using the following general equation:[21]

$$\overline{u}_{33ft} = \overline{u}_{75ft}\left[\frac{h}{z}\right]^n$$

where:

$\overline{u}_{33ft}$ = extrapolated speed at 33 ft (mph).

$\overline{u}_{75ft}$ = measured speed at 75 ft (mph).

z = height at which measurement is made (75 ft).

n = exponent based on stability.

h = height to which extrapolation is made (33 ft).

Average values of n used for each diffusion group are assumed to be as given in the table below. These values are in accordance with reference 21 with the exception of group D, which was found to be between the reference 21 values on one tower studied.[22] Assuming an n value of 0.33 for group D results in lower speeds at the 33-ft level than would result using the 0.25 value for n. Therefore, use of 0.33 is conservative compared with the suggested value of 0.25 in reference 21.

Diffusion Group	n
A	0.25
B	0.25
C	0.25
D	0.33
E	0.5
F	0.5
G	0.5

B. Diffusion Estimates for Stack Releases

On the average, windspeed increases with height within the first several hundred meters above the ground. This increase has been predicted by investigators[23] using the exponential relationship given in paragraph 2.3.4.2.1.A. Windspeeds are known to increase more rapidly under stable conditions than for unstable conditions. Therefore, estimates of windspeed increases with height are based on measured values of ΔT from the tower.

Figure 2.3-16 shows examples of measured vertical average speed profiles from ORNL, Oyster Creek, Savannah River, Sterling, Douglas Point, Ginna, and a 400-ft tower in central Pennsylvania. The figure shows that the predicted average windspeed at 120 m (stack height) for Hatch is lower, and thus conservative, compared with measured values from other sites. Therefore, it is concluded that the method used to extrapolate 150-ft windspeed measurements to the 120-m level at the HNP site is appropriate and conservative.

For elevated releases at the stack height of 120 m, the windspeed extrapolation equation discussed in A, above, was used with h = 120 m and z = 150 ft (46 m) (the instrument height used for stack estimates). Probability plots were made using hourly values of X/Q for 4 years as in A, above. The elevated diffusion

equation given in paragraph 2.3.4.2.4 was used to estimate X/Q values. The 5% probable 1-h offsite X/Q values are given in table 2.3-11. These values are the peak computed at the site boundary (low population distance) of 1250 m.

C. Diffusion of MSLBA Releases

In a MSLBA, fission products in the steam will rise with the steam and not be entrapped in the building wake. Therefore, to estimate the appropriate diffusion conditions for such an event, the elevated diffusion model for a 30-m release was used as described in paragraph 2.3.4.2.4. The 5% probable X/Q was calculated based on the conditions given in Regulatory Guide 1.5-1971. The diffusion conditions assumed for control room operator doses due to a MSLBA are discussed in subsection 15.3.4.

2.3.4.2.2 Long-Term Accident Diffusion Estimates

For releases which occur over a longer period of time (> 1 h), it is appropriate to consider changes in wind direction, atmospheric stability, and windspeed which result in lower concentrations at any given offsite location. Using the available onsite data, a computer evaluation was made to estimate the probability that any particular average diffusion condition (or poorer one) would exist during a selected interval of time at any offsite location.

Starting with each hour of data for 1 year, the computed X/Q values are added in each of 16 direction sectors for the duration of the release time period being evaluated. The maximum integrated value of all 16 directions is stored, and a new integration period spaced 1 hour later is started. Again, the maximum value from this next integration period is stored regardless of the direction sector in which it occurred, and so on. After processing all hours of data, cumulative probability plots are made for each release time period considered. Table 2.3-11 gives these values for both the site boundary and low-population zones. Estimates in table 2.3-11 are the average of four separate 1-year runs at the given probability level. The diffusion models and assumptions are described in paragraphs 2.3.4.2.3 and 2.3.4.2.4.

2.3.4.2.3 Selection of Diffusion Condition

Table 2.3-12 gives the temperature difference categories (from Regulatory Guide 1.23-1972) used to classify the site data into Pasquill groups for use in computing σ_y and σ_z in the diffusion equations.

2.3.4.2.4 Methods for Dispersion Computations

Plume centerline values of X/Q for ground-level releases are estimated using the following model:

$$X/Q = \frac{1}{\bar{u}_{33}\left(\pi\sigma_y\sigma_z + cA\right)}$$

where:

X	=	concentration (μCi/m^3).
Q	=	release rate (μCi/s).
$\bar{u}_{33}$	=	average windspeed at 33 ft (m/s).
σ_y	=	horizontal diffusion coefficient based on temperature difference and Pasquill curves (m).[24][25]
σ_z	=	vertical diffusion coefficient based on vertical temperature difference and Pasquill curves (m).[24][25]
cA	=	building wake factor (800 m^2) (c = 0.5 and A = 1600 m^2).

Sector average X/Q values are determined using the general equation:

$$X/Q \text{ sector average} = \frac{2.03}{x\bar{u}\sigma_{z_{(eff)}}}$$

This is an integrated form of the Pasquill diffusion relationship[24][25] which uses an effective σ_z term to account for dilution in the vertical direction from the building wake. The symbols have the following meanings:

x	=	distance from source (m).
X	=	average concentration at ground level in the given 22 1/2-degree sector (ci/m^3).
Q	=	average release rate (ci/s).
u	=	windspeed (m/s) at the 33-ft level (extrapolated from 75-ft level).
σ_z	=	vertical diffusion coefficient (m).

Values of $\sigma_{z_{(eff)}}$ were determined for each stability group using the relationship:

$$\sigma_{z_{(eff)}} = \sqrt{(\sigma_z)^2 + \frac{cH^2}{\pi}}, \text{ with a limit of } \sqrt{3}\,\sigma_z$$

where:

H	=	height of plant structure (assumed 47 m).
c	=	0.5 as before.

The Pasquill diffusion condition (for determination of σ_z) is assumed to be a function of vertical temperature difference as derived in paragraph 2.3.4.2.3.

Plume centerline X/Q values are used for post-accident time periods less than 8 h, and sector average values are used for time periods greater than 8 h. The site boundary and the low population distances are assumed to be 1250 m.

For stack releases, the elevated equation for ground-level centerline concentration was used as follows:

$$X/Q = \frac{1.0}{u\pi\sigma_y\sigma_z} e^{-\left[\frac{h^2}{2(\sigma_z)^2}\right]}$$

where parameters are as above except h is the stack height in meters and u is the windspeed at stack height in meters per second. This equation is also used for MSLB calculations of atmospheric dispersion factors. The sector average version of the elevated equation is used for time periods beyond 8 h.

The sector average diffusion model for releases from the stack is as follows:

$$X/Q = \frac{2.03}{\bar{u}x\sigma_z} e^{-\left[\frac{h^2}{2(\sigma_z)^2}\right]}$$

where symbols are as before and h is the stack height. For average annual calculations, the local terrain height above plant grade (figure 2.3-12) was subtracted from the plume centerline height at each distance for which estimates were made. Only the peak offsite value is used in the averaging technique of paragraph 2.3.4.2.2 for simplicity and conservatism.

2.3.5 LONG-TERM (ROUTINE) DIFFUSION ESTIMATES

2.3.5.1 Objective

The objective of this section is to calculate annual average diffusion conditions for use in evaluating routine ground-level and elevated releases from the plant. Low-level releases into wakes of buildings are considered as ground-level releases. Annual average diffusion conditions are applicable to HNP-1 and HNP-2; and also, are used as input to dose calculations described in subsection 11.3.4.

2.3.5.2 Calculations

Data used in the analyses are presented in this section as joint frequency tables. For the HNP site, these tables were compiled for 2 levels over a 4-year period of record. Table 2.3-13 is a

joint frequency table of windspeed, wind direction, and stability group for the 150-ft level using ΔT between 150 ft and 35 ft.

These data are used for evaluations of stack effluents. An exponential speed adjustment is made to the 393-ft stack height. Table 2.3-14 is similar to Table 2.3-13 for the 75-ft level with a speed adjustment to the 33-ft level. Table 2.3-14 is used for evaluations of ground-level release effluents. A logarithmic adjustment to the ΔT to be representative of temperature difference between 150 ft and 10 ft is made for all data prior to September 1972, when the lower temperature sensor was moved to the 35-ft level for ground release calculations. Table 2.3-15 is a 150-ft level joint frequency table similar to table 2.3-13 for each of the 12 months, and table 2.3-16 is a 75-ft level joint frequency table similar to table 2.3-14 for each of the 12 months.

2.3.5.2.1 Airflow Trajectory and Terrain Influences

As indicated by the 4-year (1970-1974) 150-ft wind rose from the HNP meteorological tower, the general flow pattern in the plant site region is from the northwest to the southwest and from the east. (See figure 2.3-14.) During the fall and winter months, high-pressure systems generally passing to the north of the plant site dominate the eastern two-thirds of the United States. The clockwise circulation around these high-pressure centers produces NWW winds when to the west of the plant site and NEE winds when to the north and east of the plant site. During the spring and summer months and at various other times throughout the year, the southern U.S. comes under the influence of Gulf and South Atlantic high-pressure centers. These would produce predominately west and southwest winds when to the west of the site area and SSE winds when to the east of the site. The plant site region is influenced by a number of low-pressure centers; however, these centers generally move rapidly and affect the area only for short periods.

Topography is gently rolling in the site area and has little effect on wind trajectory. During periods of light winds, local terrain affects wind trajectory. The most pronounced terrain feature is the river depression; however, this is a relatively small, wide depression which has little influence. Since it is not considered practical at the present time to compute estimates using particle-in-cell or puff trajectory diffusion models, correction factors suggested in Regulatory Guide 1.111 for open terrain are used in this analysis. This is considered to result in diffusion estimates at distances near the plant which are very unlikely to be exceeded.

2.3.5.2.2 Description of Atmospheric Diffusion Models

Models described in this section follow those described in Regulatory Guide 1.111. The following paragraphs describe the models used in these evaluations with frequent references to Regulatory Guide 1.111, since most assumptions are identical to those in the guide. These models are used to determine routine (average) X/Q and D/Q values applicable to the site.

2.3.5.2.2.1 **Atmospheric Diffusion Model.** Average atmospheric dispersion evaluated using the straight line airflow model as follows:

$$\overline{\left(X/Q'\right)}_D = 2.032 \sum_{ij} n_{ij} \left[N x \bar{u}_i \Sigma_{zj}(x)\right]^{-1} \exp\left[-h_e^2 / 2\sigma_{zj}^2(x)\right] \qquad (1)$$

where:

h_e = the effective release height.

n_{ij} = the length of time (hours of valid data) weather conditions are observed to be at a given wind direction, windspeed class, i, and atmospheric stability class, j.

N = the total hours of valid data.

$\bar{u}_i$ = the geometrical mean of all speeds in the windspeed class, i, at a height representative of release; calms are one-half the threshold anemometer speed or less; extrapolation to higher levels, if necessary, is done by raising the ratio of the two heights to the n power, where n = 0.25, 0.33 and 0.5 for unstable, neutral, and stable conditions, respectively.

$\sigma_{zj}(x)$ = the vertical plume spread without volumetric correction at distance, x, for stability class, j (figure 1 of Regulatory Guide 1.111) based on vertical temperature difference (ΔT) and Regulatory Guide 1.23 categorization of Pasquill Groups by ΔT.

$\Sigma_{zj}(x)$ = the vertical plume spread with a volumetric correction for a release within the building wake cavity, at a distance, x, for stability class, j; otherwise $\Sigma_{zj}(x) = \sigma_{zj}(x)$.

$\overline{\left(X/Q'\right)}_D$ = the average effluent concentration, X, normalized by source strength, Q′, at distance, x, in a given downwind direction, D.

2.032 = $(2/\pi)^{1/2}$ divided by the width in radians of a 22.5-degree sector.

In some cases, hourly data were used and the summation over i and j in the above equation was deleted; this summation was accomplished for all hours at all distances for each direction. Dilution was decreased according to terrain correction factors in figure 2 of Regulatory Guide 1.111. These factors were multiplied by the results from equation 1 and varied in accordance with the direction and distance being evaluated.

This general Gaussian diffusion model has been used extensively for both nuclear reactor and air pollution diffusion analysis for at least 10 years; therefore, it is considered appropriate for use in this specific application. With regard to model accuracy, the greatest weakness results from determining stability using vertical temperature difference. A more appropriate representation of

turbulence and resulting diffusion could be obtained using bivane data or some other measurement of turbulence.

Actual model input assumptions and source-term configurations are discussed below.

2.3.5.2.2.2 Source Configuration Considerations. If a release point is elevated and there are no buildings which would obstruct the plume in its normal trajectory, equation 1 is used with the height of release defined as follows (from equation 4 of Regulatory Guide 1.111):

$$h_e = h_s + h_{pr} - h_t - c \qquad (2)$$

where:

c	=	correction for low relative exit velocity (equation 5 of Regulatory Guide 1.111).
h_e	=	effective release height.
h_{pr}	=	rise of the plume above the release point based on Briggs. (See further explanation below.)
h_s	=	is the physical height of the release point. (The elevation of the stack base should be assumed to be zero.)
h_t	=	maximum terrain height between the release point and the point for which the calculation is made.

Values of h_{pr} are computed as follows for a jet since nuclear plant vents have an insignificant amount of buoyancy resulting from heated discharges:

$$h_{pr} = 1.44D\left(\frac{W_O}{\bar{u}}\right)^{2/3}\left(\frac{x}{D}\right)^{1/3} \qquad (3)$$

up to the point where h_{pr} is the minimum of the following two equations:

$$h_{pr_{max}} = 3\left(\frac{W_O}{\bar{u}}\right)D,\ \text{or} \qquad (4a)$$

$$h_{pr_{max}} = 1.5\left(\frac{F_m}{\bar{u}}\right)^{1/3} s^{-1/6} \qquad (4b)$$

where symbols are as before, and

D = stack or vent effective inside diameter (m).

W_O = stack or vent exit velocity (m/s).

$\bar{u}$ = windspeed at discharge level (m/s).

F_m = momentum flux (m^4/s^2).

s = stability parameter (s^{-2}).

If the plume trajectory from a release point (vent) does not remain outside of building wake influences near large structures, all or portions of the plume are considered to be entrapped and brought to ground level in the turbulent wake of the building. The criteria for determining the portion of the plume treated as an elevated or ground release follows from equations 6, 7, and 8 of Regulatory Guide 1.111 and are repeated below for completeness:

If $W_O/\bar{u} > 5.0$, use h_e as calculated above.

If $W_O/\bar{u} \le 1.0$, use $h_e = 0$.

If $1 < W_O/\bar{u} \le 1.5,\ E_t = 2.58 - 1.58\left(\frac{W_O}{\bar{u}}\right)$

If $1.5 < W_O/\bar{u} \le 5.0,\ E_t = 0.30 - 0.06\left(\frac{W_O}{\bar{u}}\right)$

The appropriate diffusion estimate is then computed by assuming an elevated release 100 (1 - E_t) percent of the time and by assuming ground release 100 E_t percent of the time. Calculations using this mixed model are referred to as wake-split calculations in this report. A building wake correction is computed for all ground releases near structures in accordance with the following general equation:

$$\Sigma = \sqrt{\sigma_z^2 + \frac{cH^2}{\pi}} \le 1.73\sigma_z \qquad (5)$$

where:

Σ = effective dispersion coefficient for use in equation 1 (m).

c = building wake coefficient (c = 0.5).

H = height of the tallest structure in the nuclear plant power block (m).

2.3.5.2.2.3 <u>Removal Mechanisms</u>. As radioactive effluent in a plume travels downwind, it is subject to several removal mechanisms including radioactive decay, dry deposition, and wet deposition (during rain). Corrections for radioactive decay are not made in the estimates reported in this section.

Dry deposition which results in depletion of halogen and particulate isotopes from the plume is considered only to the extent suggested in Regulatory Guide 1.111, figures 3 through 6. Depletion factors in these curves are a function of height and distance; therefore, for sites where elevated releases occur, the terrain must be subtracted from the plume height before entering the curves at the appropriate distance. Each elevated or ground level, X/Q is multiplied by the depletion and the terrain correction factors before combining to give the final depleted X/Q value.

To determine relative deposition rate as a function of distance and stability, the curves given in figures 7 through 10 of Regulatory Guide 1.111 are used. Again, terrain heights are subtracted before the table lookup. Each D/Q value is multiplied by terrain correction factors, if any. Values from the curves are divided by the sector cross-width (arc) at the point of calculation.

Since seasonal rainfall is fairly uniform, dry deposition is believed to adequately represent overall deposition rates; therefore, wet deposition has not been considered.

2.3.5.2.3 Diffusion Model Inputs and Results

Computer runs have been made using site data in the diffusion models given in paragraph 2.3.5.2.2. A list of runs, input assumptions, and results are given in the following sections.

2.3.5.2.3.1 **List of Computer Runs.** Table 2.3-17 tabulates computer runs which used the diffusion models described in paragraph 2.3.5.2.2. Since the grazing season is assumed to exist all year, separate runs for the grazing season were not necessary.

2.3.5.2.3.2 **Summary of Plant Discharges.** A summary of plant vent information for each discharge point is given in tables 2.3-18 and 2.3-19. Only vents used during routine operation are considered in this evaluation. Inspection of tables 2.3-18 and 2.3-19 shows that two calculations are required to determine diffusion conditions applicable for each vent.

2.3.5.2.3.3 **Input Assumptions.** Table 2.3-20 tabulates all pertinent input information utilized in making the model calculations. Terrain elevations for all distances out to 10 miles are found in figure 2.3-12. Terrain height is conservatively not allowed to decrease with increasing distance or to decrease below plant grade in accordance with Regulatory Guide 1.111.

2.3.5.2.3.4 **Results.** Resulting X/Q and D/Q values are listed in tables 2.3-21 and 2.3-22 for each direction sector for 10 distances. These results are used as input for the dose calculations described in subsection 11.3.4. Tables 2.3-23 and 2.3-24 summarize the resulting diffusion factors for each of the receptor locations. Each table represents model results for one vent location. One set of calculations was made for the stack, and the second set of calculations was made for all other vents. Since the main plant vent has a top-hat, no vertical jet exists and

use of a wake-split model is not appropriate. Thus, all effluents were assumed to be entrapped in the building wake at ground level.

2.3.6 ACCIDENT DIFFUSION ESTIMATES FOR MCR AND TECHNICAL SUPPORT CENTER

For the design basis accidents, (loss-of-coolant accident (LOCA), main steam line break (MSLB), control rod drop accident (CRDA), and fuel handling accident (FHA), the MCR and the TSC X/Q values were determined using the computer code ARCON96. ARCON96 was developed by Pacific Northwest Laboratory for the NRC to determine X/Q values for onsite receptors near building structures.

Three years of meteorological data, from 1996 through 1998, were used in determining the X/Q values for the MCR and the TSC. X/Q values are calculated for averaging periods ranging in duration from 2 h to 30 days. These X/Q values are applicable to HNP-1 and HNP-2.

ARCON96 was used to determine X/Q values for both ground-level and elevated releases. Releases considered to be ground-level releases are those with release heights less than the estimated building wake cavity height. The basic diffusion model implemented in the ARCON96 code is a straight-line Gaussian model that assumes the release rate is constant for the entire period of release. A sector-average relative concentration model is used to estimate concentrations for periods after the initial 0- to 8-h period.

ARCON 96 includes corrections to account for enhanced dispersion under low windspeed conditions and in building wakes. The wake correction model included in ARCON96 treats diffusion under low windspeed conditions much better than previous models. Thus, the diffusion coefficients in ARCON96 account for both low windspeed meander and wake effects.

The calculated X/Q values for the MCR and the TSC for various release paths are provided in table 2.3-25.

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TABLE 2.3-1

METEROLOGICAL DATA FOR SAVANNAH, GEORGIA

Station: SAVANNAH, GEORGIA MUNICIPAL AIRPORT Standard time used: EASTERN Latitude: 32° 08' N Longitude: 81° 12' W Elevation (ground) : 46 feet Year: 1973

Month	Temperature – Averages – Daily maximum	Daily minimum	Monthly	Extremes – Highest	Date	Lowest	Date	Degree days (Base 65°) – Heating	Cooling	Precipitation – Total	Greatest in 24 hrs.	Date	Snow, Ice pellets – Total	Greatest in 24 hrs.	Date
JAN	60.2	39.4	49.8	72	22+	23	14	462	0	3.61	0.93	22	T	T	12
FEB	61.1	38.1	49.6	74	8	19	12	423	0	4.46	1.84	8-9	3.2	3.2	10
MAR	73.0	54.2	63.6	87	14	38	22	98	63	5.36	1.53	8-9	0.0	0.0	
APR	74.9	53.0	64.0	85	23	38	11	85	63	4.43	2.37	25-26	0.0	0.0	
MAY	84.5	62.5	73.5	95	23	48	5	4	275	1.23	0.43	29-30+	0.0	0.0	
JUN	87.9	70.6	79.3	93	17	64	3	0	434	9.19	3.31	10-11	0.0	0.0	
JUL	91.5	72.8	82.2	97	29	69	3+	0	540	2.89	1.03	11	0.0	0.0	
AUG	88.5	71.4	80.0	94	13	63	22	0	473	6.45	1.73	3	0.0	0.0	
SEP	87.4	70.6	79.0	93	9	66	23	0	429	3.65	1.02	13-14	0.0	0.0	
OCT	79.9	57.9	68.9	88	5+	40	30	47	177	0.19	0.10	28-29	0.0	0.0	
NOV	73.9	49.2	61.6	85	4	32	30+	148	52	0.68	0.55	9	0.0	0.0	
DEC	62.7	39.6	51.2	80	31	20	22	435	13	3.26	1.16	15-16	0.0	0.0	
YEAR	77.1	56.6	66.9	97	JUL. 29	19	FEB. 12	1702	2519	45.40	3.31	JUN. 10-11	3.2	3.2	FEB. 10

Month	Relative humidity – Hour 01 (Local time)	Hour 07	Hour 13	Hour 19	Wind & – Resultant – Direction	Speed	Average speed	Fastest mile – Speed	Direction #	Date	Percent of possible sunshine	Average sky cover sunrise to sunset	Number of days – Sunrise to sunset – Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog	Temperatures – Maximum – 90° and above ‡	32° and below	Minimum – 32° and below	0° and below	Average daily solar radiation - langleys
JAN	79	84	57	68	32	2.1	8.0	26	NW	29+	45	6.8	7	7	17	12	0	0	4	0	0	10	0	
FEB	75	80	50	58	32	2.6	9.3	32	W	3	57	6.0	10	5	13	8	1	1	3	0	0	6	0	
MAR	85	88	59	70	18	1.8	9.1	40	W	17	48	7.4	5	6	20	11	0	1	8	0	0	0	0	
APR	79	83	48	58	22	3.0	9.3	30	W	10	70	5.3	10	9	11	6	0	4	0	0	0	0	0	
MAY	81	82	47	60	23	3.8	7.4	39	W	29	68	5.8	10	6	15	7	0	6	1	5	0	0	0	
JUN	89	89	63	74	19	1.7	6.3	25	E	18	42	7.6	1	13	16	12	0	11	2	7	0	0	0	
JUL	89	89	57	73	16	0.8	6.1	34	NW	11	57	6.2	3	19	9	9	0	14	1	25	0	0	0	
AUG	92	92	65	78	12	0.4	6.2	26	E	21	45	6.8	5	11	15	14	0	13	3	12	0	0	0	
SEP	92	93	64	79	08	2.8	6.4	25	NW	14	47	5.8	9	11	10	12	0	6	3	3	0	0	0	
OCT	88	91	51	72	02	3.1	7.6	29	W	29	64	4.8	11	13	7	5	0	1	4	0	0	0	0	
NOV	84	87	51	73	28	1.1	7.2	25	NW	21	67	4.3	15	7	8	2	0	1	9	0	0	2	0	
DEC	81	83	57	71	26	2.2	8.7	29	W	21+	52	5.9	9	9	13	9	0	0	2	0	0	8	0	
YEAR	85	87	56	70	25	0.6	7.6	40	W	MAR. 17	55	6.1	95	116	154	107	1	58	40	52	0	26	0	

Normals, Means, and Extremes

Month	Temperature – Normal – Daily maximum	Daily minimum	Monthly	Extremes – Record highest	Year	Record lowest	Year	Normal heating degree days (Base 65°)	Precipitation – Normal total	Maximum monthly	Year	Minimum monthly	Year	Maximum in 24 hrs.	Year
(a)	(b)	(b)	(b)	9		9		(b)	(b)	23		23		23	
J	61.1	38.7	49.9	81	1972+	9	1966	483	2.92	7.18	1967	0.51	1957	2.80	1967
F	63.6	40.5	52.1	84	1966	16	1967	379	2.86	7.92	1964	1.16	1968+	3.46	1964
M	69.5	46.4	58.0	90	1967	26	1971+	256	4.41	9.57	1959	0.18	1955	4.65	1959
A	77.8	54.3	66.1	95	1967	33	1966	63	2.93	7.74	1961	0.71	1969	3.66	1961
M	84.8	61.8	73.3	98	1967	40	1971	0	4.20	10.08	1957	0.51	1953	4.23	1969
J	89.3	68.8	79.1	102	1971	53	1966	0	5.89	14.39	1963	0.84	1954	4.06	1963
J	90.8	71.3	81.1	99	1968	61	1972+	0	7.87	20.10	1964	1.35	1972	6.36	1957
A	90.3	70.9	80.6	100	1968	61	1966	0	6.47	14.94	1971	1.33	1954	7.04	1971
S	85.4	66.9	76.2	97	1972	43	1967	0	5.57	13.47	1953	0.36	1972	5.87	1956
O	78.2	55.9	67.1	93	1971	33	1965	60	2.81	8.54	1959	0.02	1963	3.57	1959
N	69.3	44.9	57.1	85	1973+	15	1970	253	1.94	4.91	1972	0.15	1966	5.02	1969
D	62.1	38.7	50.4	83	1971+	19	1968	458	3.28	5.50	1953	0.40	1955	3.47	1964
YR	76.8	54.9	65.9	102	JUN. 1971	9	JAN. 1966	1952	51.15	20.10	JUL. 1964	0.02	OCT. 1963	7.04	AUG. 1971

Month	Snow, Ice pellets – Mean total	Maximum monthly	Year	Maximum in 24 hrs.	Year	Relative humidity – Hour 01 (Local time)	Hour 07	Hour 13	Hour 19	Wind & – Mean speed	Prevailing direction	Fastest mile – Speed	Direction	Year	Pct. of possible sunshine	Mean sky cover sunrise to sunset	Mean number of days – Sunrise to sunset – Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog	Temperatures – Max. – 90° and above ‡	32° and below	Min. – 32° and below	0° and below	Average daily solar radiation - langleys
(a)	23	23		23		9	9	9	9	23	13	23	23		23	23	23	23	23	23	23	23	23	9	9	9	9	
J	T	T	1973+	T	1973+	81	84	55	69	8.7	WNW	46	NW	1955	54	6.1	9	7	15	9	0	1	5	0	*	12	0	
F	0.3	3.6	1968	3.6	1968	76	81	50	62	9.6	NE	44	WNW	1960	57	6.1	8	6	14	9	*	1	3	0	0	10	0	
M	T	T	1968+	T	1968+	78	83	47	59	9.6	WNW	40	W	1973	64	5.9	9	9	13	10	0	3	3	*	0	4	0	
A	0.0	0.0		0.0		81	85	47	60	9.3	SSE	40	N	1963	70	5.5	10	9	11	7	0	4	2	2	0	0	0	
M	0.0	0.0		0.0		84	85	51	64	8.2	SW	42	N	1965	69	5.7	9	11	11	9	0	8	3	4	0	0	0	
J	0.0	0.0		0.0		87	88	57	70	7.7	SW	66	E	1953	66	6.1	7	11	12	11	0	10	3	11	0	0	0	
J	0.0	0.0		0.0		89	89	59	74	7.4	SW	51	SE	1951	65	6.4	5	15	11	15	0	16	1	20	0	0	0	
A	0.0	0.0		0.0		90	91	63	78	7.0	SW	58	N	1954	64	6.2	6	14	11	12	0	12	2	16	0	0	0	
S	0.0	0.0		0.0		89	91	60	76	7.7	NE	56	NW	1959	60	6.2	6	12	12	10	0	6	4	5	0	0	0	
O	0.0	0.0		0.0		86	88	54	75	7.9	NNE	37	NE	1952	64	5.0	12	8	11	6	0	1	4	1	0	0	0	
N	0.0	0.0		0.0		84	86	49	72	7.9	NNE	34	W	1957	63	5.1	12	7	11	6	0	*	5	0	0	4	0	
D	T	T	1962+	T	1962+	82	85	54	71	8.3	NE	37	NW	1962	56	5.9	10	7	14	8	0	*	4	0	0	9	0	
YR	0.3	3.6	FEB. 1968	3.6	FEB. 1968	84	86	54	69	8.3	SW	66	E	JUN. 1953	63	5.8	103	116	146	111	*	63	39	58	*	39	0	

Means and extremes above are from existing and comparable exposures. Annual extremes have been exceeded at other sites in the locality as follows: Highest temperature 105 in July 1879; lowest temperature 8 in February 1899; maximum monthly precipitation 22.88 in September 1924; minimum monthly precipitation T in December 1889; maximum precipitation in 24 hours 11.44 in September 1928; fastest mile wind 90 in August 1940.

(a) Length of record, years, based on January data. Other months may be for more or fewer years if there have been breaks in the record.
(b) Climatological normals (1941-1970).
* Less than one half.
\+ Also on earlier dates, months, or years.
T Trace, an amount too small to measure.
\- Below zero temperatures are preceded by a minus sign.
‡ ≦ 70° at Alaskan stations.

The prevailing direction for wind in the Normals, Means, and Extremes table is from records through 1963.

Unless otherwise indicated, dimensional units used in this bulletin are: temperature in degrees F.; precipitation, including snowfall, in inches; wind movement in miles per hour; and relative humidity in percent. Heating degree day totals are the sums of negative departures of average daily temperatures from 65° F. Cooling degree day totals are the sums of positive departures of average daily temperatures from 65° F. Sleet was included in snowfall totals beginning with July 1948. The term "ice pellets" includes solid grains of ice (sleet) and particles consisting of snow pellets encased in a thin layer of ice. Heavy fog reduces visibility to 1/4 mile or less.

Sky cover is expressed in a range of 0 for no clouds or obscuring phenomena to 10 for complete sky cover. The number of clear days is based on average cloudiness 0-3, partly cloudy days 4-7, and cloudy days 8-10 tenths.

Solar radiation data are the averages of direct and diffuse radiation on a horizontal surface. The langley denotes one gram calorie per square centimeter.

& Figures instead of letters in a direction column indicate direction in tens of degrees from true North; i.e., 09 - East, 18 - South, 27 - West, 36 - North, and 00 - Calm. Resultant wind is the vector sum of wind directions and speeds divided by the number of observations. If figures appear in the direction column under "Fastest mile" the corresponding speeds are fastest observed 1-minute values.

\# To 8 compass points only.

TABLE 2.3-2

METEROLOGICAL DATA FOR MACON, GEORGIA

Station: MACON, GEORGIA   LEWIS B. WILSON AIRPORT   Standard time used: EASTERN   Latitude: 32° 42' N   Longitude: 83° 39' W   Elevation (ground): 354 feet   Year: 1973

Month	Temperature – Averages – Daily maximum	Daily minimum	Monthly	Extremes – Highest	Date	Lowest	Date	Degree days (Base 65°) – Heating	Cooling	Precipitation – Total	Greatest in 24 hrs.	Date	Snow, Ice pellets – Total	Greatest in 24 hrs.	Date
JAN	57.5	35.7	46.6	74	18	20	14	562	0	6.10	3.05	31-1	T	T	12 +
FEB	58.6	33.3	46.0	72	27+	9	11	529	0	5.71	2.36	1-2	16.5	16.5	9-10
MAR	72.2	51.8	62.0	85	15+	33	22	137	53	6.49	1.84	30-31	0.0	0.0	
APR	73.7	50.4	62.1	85	21+	32	11	128	47	6.33	2.56	7	0.0	0.0	
MAY	82.9	57.2	70.1	95	23	43	16	27	189	4.25	2.58	28-29	0.0	0.0	
JUN	89.7	69.0	79.4	95	28+	62	30	0	439	3.29	1.03	21-22	0.0	0.0	
JUL	93.6	72.1	82.9	99	30+	67	20	0	562	2.81	1.42	6	0.0	0.0	
AUG	91.4	70.4	80.9	96	1	65	22	0	500	7.09	1.82	17	0.0	0.0	
SEP	90.0	69.0	79.5	96	9+	56	19	0	446	0.98	0.50	10	0.0	0.0	
OCT	81.7	52.9	67.3	91	5	38	18	56	136	0.67	0.28	28	0.0	0.0	
NOV	73.4	45.7	59.6	85	26	29	12	189	30	1.98	0.75	20-21	0.0	0.0	
DEC	61.1	37.7	49.4	79	4	18	22	480	6	2.78	0.76	31	0.0	0.0	
YEAR	77.2	53.8	65.5	99	JUL. 30+	9	FEB. 11	2108	2408	48.48	3.05	JAN. 31-1	16.5	16.5	FEB. 9-10

Month	Relative humidity – Hour 01 (Local time)	Hour 07	Hour 13	Hour 19	Wind & – Resultant – Direction	Speed	Average speed	Fastest mile – Speed	Direction #	Date	Percent of possible sunshine	Average sky cover sunrise to sunset	Number of days – Sunrise to sunset – Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog	Temperatures – Maximum – ‡ 90° and above	32° and below	Minimum – 32° and below	0° and below	Average daily solar radiation - langleys
JAN	79	85	60	64	34	2.3	7.6	29	S	19	52	6.8	8	4	19	13	0	1	2	0	0	13	0	
FEB	74	83	50	58	32	2.7	8.6	26	SE	1	67	6.2	9	7	12	7	2	1	5	0	0	14	0	
MAR	81	88	62	66	16	2.5	9.3	42	SW	4	46	7.6	4	7	20	13	0	4	3	0	0	0	0	
APR	77	87	52	55	21	2.5	9.1	26	W	10	61	6.2	10	5	15	6	0	3	1	0	0	1	0	
MAY	82	86	52	55	25	3.3	8.6	33	NW	19	73	C.1	9	9	13	9	0	6	1	2	0	0	0	
JUN	91	90	61	71	07	0.4	6.4	27	W	6	68	7.0	3	11	16	13	0	10	4	18	0	0	0	
JUL	90	92	59	71	14	0.5	6.2	28	NW	16	70	6.7	2	16	13	7	0	9	2	29	0	0	0	
AUG	91	93	61	74	07	0.6	5.9	28	SE	31	76	5.6	8	13	10	9	0	10	5	25	0	0	0	
SEP	91	93	61	76	08	2.7	6.7	21	NE	26	71	5.8	6	12	12	8	0	2	2	19	0	0	0	
OCT	86	89	44	60	02	2.8	6.7	22	NW	29	80	4.0	17	7	7	3	0	1	1	3	0	0	0	
NOV	84	88	50	65	25	1.0	6.3	51	NW	21	67	4.5	14	6	10	8	0	1	4	0	0	3	0	
DEC	78	82	55	65	26	1.6	8.9	27	SW	13	54	5.5	12	5	14	10	0	4	2	0	0	11	0	
YEAR	84	88	56	65	29	0.3	7.5	51	NW	NOV. 21	66	6.0	102	102	161	106	2	52	32	96	0	42	0	

Normals, Means, and Extremes

Month	Temperature – Normal – Daily maximum	Daily minimum	Monthly	Extremes – Record highest	Year	Record lowest	Year	Normal heating degree days (Base 65°)	Precipitation – Normal total	Maximum monthly	Year	Minimum monthly	Year	Maximum in 24 hrs.	Year	Snow, Ice pellets – Mean total	Maximum monthly	Year	Maximum in 24 hrs.	Year
(a)	(b)	(b)	(b)	9		9		(b)	(b)	25		25		25		25	25		25	
J	58.7	36.9	47.8	78	1971+	3	1966	543	3.67	8.30	1964	0.69	1954	4.44	1962	0.2	3.7	1955	3.7	1955
F	61.8	39.0	50.4	80	1965	9	1973	423	4.38	9.12	1949	1.39	1968	2.95	1952	0.8	16.5	1973	16.5	1973
M	68.4	44.6	56.5	88	1967	22	1968	298	5.15	9.69	1962	1.26	1968	3.94	1970	T	T	1908+	T	1968+
A	78.3	53.2	65.8	94	1970	31	1971	66	3.55	8.42	1964	0.53	1972	3.65	1955	0.0	0.0		0.0	
M	86.0	60.9	73.5	99	1967	40	1971+	6	3.57	11.77	1957	0.32	1956	4.01	1957	0.0	0.0		0.0	
J	90.9	68.2	79.6	100	1970+	46	1972	0	3.86	9.06	1965	0.99	1954	4.97	1965	0.0	0.0		0.0	
J	92.1	70.7	81.4	104	1970	54	1967	0	4.52	9.04	1953	1.21	1952	2.69	1965	0.0	0.0		0.0	
A	91.8	70.0	80.9	104	1968	56	1969+	0	3.65	7.09	1973	1.39	1968	2.96	1959	0.0	0.0		0.0	
S	86.6	64.9	75.8	99	1972+	35	1967	0	3.10	8.82	1953	0.41	1970	4.60	1956	0.0	0.0		0.0	
O	78.2	53.2	65.7	94	1971	29	1965	82	2.31	9.39	1959	0.00	1963+	5.35	1970	0.0	0.0		0.0	
N	67.9	42.4	55.2	85	1973+	16	1970	304	2.39	5.89	1957	0.43	1956	1.63	1968	T	0.2	1950	0.2	1950
D	59.6	37.0	48.3	82	1972+	18	1973+	518	4.31	10.39	1972	0.58	1955	4.50	1972	T	0.5	1963	0.5	1963
YR	76.7	53.4	65.1	104	JUL. 1970+	3	JAN. 1966	2240	44.46	11.77	MAY 1957	0.00	OCT. 1963+	5.35	OCT. 1970	1.0	16.5	FEB. 1973	16.5	FEB. 1973

Month	Relative humidity – Hour 01 (Local time)	Hour 07	Hour 13	Hour 19	Wind & – Mean speed	Prevailing direction	Fastest mile – Speed	Direction #	Year	Pct. of possible sunshine	Mean sky cover sunrise to sunset	Mean number of days – Sunrise to sunset – Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog	Temperatures – Max. – ‡ 90° and above	32° and below	Min. – 32° and below	0° and below	Average daily solar radiation - langleys
(a)	9	9	9	9	25	15	25	25		25	25	25	25	25	25	25	25	25	9	9	9	9	
J	78	83	58	64	8.3	WNW	47	SW	1959+	54	6.4	8	7	16	10	*	1	4	0	*	15	0	
F	74	81	54	58	9.2	WNW	60	NE	1960	57	6.1	8	6	14	10	*	2	3	0	0	13	0	
M	74	83	50	53	9.6	WNW	60	S	1954	61	6.1	9	8	14	11	0	3	2	0	0	6	0	
A	78	86	48	52	9.2	WNW	57	SE	1957	67	5.5	11	7	12	8	0	4	1	2	0	*	0	
M	82	87	50	56	7.9	NW	65	SE	1952	70	5.6	10	10	11	9	0	6	1	7	0	0	0	
J	86	87	54	62	7.5	NW	56	NW	1954	68	5.9	8	11	11	10	0	9	1	16	0	0	0	
J	88	89	58	69	7.0	SW	59	SW	1954	66	6.5	5	12	14	14	0	14	1	22	0	0	0	
A	89	91	59	70	6.5	NE	70	S	1961	69	5.7	9	12	10	11	0	10	2	19	0	0	0	
S	89	92	57	71	7.2	NE	40	NW	1956	65	5.8	9	10	11	8	0	3	2	13	0	0	0	
O	85	88	52	68	7.1	NE	32	NW	1954+	68	4.6	14	7	10	6	0	1	2	2	0	1	0	
N	81	86	50	65	7.4	WNW	54	SW	1957	64	5.0	12	7	11	7	0	1	3	0	0	7	0	
D	80	83	56	67	7.9	NW	50	NW	1967	56	6.0	9	8	14	10	0	1	4	0	0	11	0	
YR	82	86	54	63	7.9	WNW	70	S	AUG. 1961	62	5.8	112	105	148	112	*	56	25	80	*	53	0	

Means and extremes above are from existing and comparable exposures. Annual extremes have been exceeded at other sites in the locality as follows:
Highest temperature 106 in June 1954;maximum monthly precipitation 20.52 in August 1928; maximum precipitation in 24 hours 8.36 in August 1928.

(a) Length of record, years, based on January data. Other months may be for more or fewer years if there have been breaks in the record.
(b) Climatological normals (1941-1970).
* Less than one half.
\+ Also on earlier dates, months, or years.
T Trace, an amount too small to measure.
— Below zero temperatures are preceded by a minus sign.
‡ ≥ 70° at Alaskan stations.

The prevailing direction for wind in the Normals, Means, and Extremes table is from records through 1963.

Unless otherwise indicated, dimensional units used in this bulletin are: temperature in degrees F.; precipitation, including snowfall, in inches; wind movement in miles per hour; and relative humidity in percent. Heating degree day totals are the sums of negative departures of average daily temperatures from 65° F. Cooling degree day totals are the sums of positive departures of average daily temperatures from 65° F. Sleet was included in snowfall totals beginning with July 1948. The term "Ice pellets" includes solid grains of ice (sleet) and particles consisting of snow pellets encased in a thin layer of ice. Heavy fog reduces visibility to 1/4 mile or less.

Sky cover is expressed in a range of 0 for no clouds or obscuring phenomena to 10 for complete sky cover. The number of clear days is based on average cloudiness 0-3, partly cloudy days 4-7, and cloudy days 8-10 tenths.

Solar radiation data are the averages of direct and diffuse radiation on a horizontal surface. The langley denotes one gram calorie per square centimeter.

& Figures instead of letters in a direction column indicate direction in tens of degrees from true North; i.e., 09 - East, 18 - South, 27 - West, 36 - North, and 00 - Calm. Resultant wind is the vector sum of wind directions and speeds divided by the number of observations. If figures appear in the direction column under "Fastest mile" the corresponding speeds are fastest observed 1-minute values.

\# To 8 compass points only.

TABLE 2.3-3 (SHEET 1 OF 2)

TABLE OF FUJITA-PEARSON TORNADO SCALE[a]

F-Scale Maximum Windspeed				P-Scale Path Length			P-Scale Path Width			
Scale	(mph)	(kts)	(m/s)	Scale	(mi)	(km)	Scale	(ft)	(yd)	(m)
F 0.0	40	35	18	P 0.0	0.3	0.5	P 0.0	17	6	5
0.1	43	37	19	0.1	0.4	0.6	0.1	19	6	6
0.2	46	40	21	0.2	0.4	0.6	0.2	21	7	6
0.3	49	43	22	0.3	0.5	0.7	0.3	24	8	7
0.4	52	46	23	0.4	0.5	0.8	0.4	26	9	8
0.5	56	48	25	0.5	0.6	0.9	0.5	30	10	9
0.6	59	51	26	0.6	0.6	1.0	0.6	33	11	10
0.7	63	54	28	0.7	0.7	1.1	0.7	37	13	11
0.8	66	57	30	0.8	0.8	1.3	0.8	42	14	13
0.9	70	60	31	0.9	0.9	1.4	0.9	47	16	14
F 1.0	73	64	33	P 1.0	1.0	1.6	P 1.0	53	18	16
1.1	77	67	34	1.1	1.1	1.8	1.1	59	20	18
1.2	81	70	36	1.2	1.3	2.0	1.2	66	22	20
1.3	84	73	38	1.3	1.4	2.3	1.3	74	25	23
1.4	88	77	40	1.4	1.6	2.6	1.4	84	28	26
1.5	92	80	41	1.5	1.8	2.9	1.5	94	31	29
1.6	96	84	43	1.6	2.0	3.2	1.6	105	35	32
1.7	100	87	45	1.7	2.2	3.6	1.7	118	39	36
1.8	104	91	47	1.8	2.5	4.0	1.8	133	44	40
1.9	109	94	49	1.9	2.8	4.5	1.9	149	50	45
F 2.0	133	98	50	P 2.0	3.2	5.1	P 2.0	167	56	51
2.1	117	102	52	2.1	3.5	5.7	2.1	187	62	57
2.2	121	105	54	2.2	4.0	6.4	2.2	210	70	64
2.3	126	109	56	2.3	4.5	7.2	2.3	235	78	72
2.4	130	113	58	2.4	5.0	8.1	2.4	265	88	81
2.5	135	117	60	2.5	5.6	9.0	2.5	297	99	90
2.6	139	121	62	2.6	6.3	10.2	2.6	333	111	102
2.7	144	125	64	2.7	7.1	11.4	2.7	374	125	114
2.8	148	129	66	2.8	7.9	12.8	2.8	419	140	128
2.9	153	132	68	2.9	8.9	14.3	2.9	470	157	143

TABLE 2.3-3 (SHEET 2 OF 2)

F-Scale Maximum Windspeed				P-Scale Path Length			P-Scale Path Width			
Scale	(mph)	(kts)	(m/s)	Scale	(mi)	(km)	Scale	(ft)	(yd)	(m)
F 3.0	158	137	70	P 3.0	10.0	16.1	P 3.0	528	176	161
3.1	162	141	73	3.1	11.2	18.0	3.1	591	197	180
3.2	167	145	75	3.2	12.6	20.3	3.2	665	222	203
3.3	172	149	77	3.3	14.1	22.7	3.3	744	248	227
3.4	177	154	79	3.4	15.9	25.6	3.4	837	279	256
3.5	182	158	81	3.5	17.8	28.6	3.5	940	313	286
3.6	187	162	83	3.6	20.0	32.2	3.6	1054	351	322
3.7	192	167	86	3.7	22.4	36.0	3.7	1183	394	360
3.8	197	171	88	3.8	25.1	40.4	3.8	1326	442	404
3.9	202	175	90	3.9	28.2	45.4	3.9	1489	496	454
F 4.0	207	180	93	P 4.0	31.6	50.9	P 4.0	1670	557	509
4.1	212	184	95	4.1	35.5	57.1	4.1	1874	625	571
4.2	218	189	97	4.2	39.8	64.1	4.2	2102	701	641
4.3	223	194	100	4.3	44.7	71.8	4.3	2354	785	718
4.4	228	198	102	4.4	50.1	80.6	4.4	2646	882	806
4.5	233	203	104	4.5	56.2	90.4	4.5	2967	989	904
4.6	238	207	107	4.6	63.1	102.0	4.6	3332	1111	1.0 km
4.7	244	212	109	4.7	70.8	114.0	4.7	3738	1246	1.1
4.8	250	217	112	4.8	79.4	128.0	4.8	4194	1398	1.3
4.9	255	222	114	4.9	89.1	143.0	4.9	4704	1568	1.4
F 5.0	261	227	117	P 5.0	100	161.0	P 5.0	1.0 mi	1760	1.6
5.1	267	232	119	5.1	112	181.0	5.1	1.1	1971	1.8
5.2	272	236	122	5.2	126	203.0	5.2	1.3	2218	2.0
5.3	278	241	124	5.3	141	227.0	5.3	1.4	2482	2.3
5.4	284	246	127	5.4	159	255.0	5.4	1.6	2798	2.6
5.5	289	251	129	5.5	178	286.0	5.5	1.8	3133	2.9
5.6	295	256	132	5.6	200	321.0	5.6	2.0	3520	3.2
5.7	301	261	135	5.7	224	360.0	5.7	2.2	3942	3.6
5.8	307	267	137	5.8	251	404.0	5.8	2.5	4418	4.0
5.9	313	272	140	5.9	282	454.0	5.9	2.8	4963	4.5

a. Characteristics of a tornado can be expressed as a combination of Fujita-scale windspeed and Pearson-scale path length and width. This scale permits us to classify tornadoes between two extreme FPP scales: 0,0,0 and 5,5,5.

TABLE 2.3-4

WINDSPEED DISTRIBUTION

Windspeed Classification	No. of Tornadoes	Percent of Total
F5 (windspeed > 260 mph)	2	0.12
F4 (207 to 260 mph)	34	2.1
F3 (158 to 206 mph)	115	7.2
F2 (113 to 157 mph)	430	26.6
F1 (73 to 112 mph)	710	44.0
F0 (40 to 72 mph)	321	19.9

TABLE 2.3-5

CUMULATIVE WINDSPEED DISTRIBUTION

Windspeed Classification	No. of Tornadoes	Percent of Total
F5 and above (windspeed > 260 mph)	2	0.12
F4 and above (> 206 mph)	36	2.2
F3 and above (> 157 mph)	151	9.3
F2 and above (> 112 mph)	581	36.0
F1 and above (> 74 mph)	1291	80.0
F0 and above (> 40 mph)	1612	100.0

TABLE 2.3-7 (SHEET 1 OF 3)

METEOROLOGICAL INSTRUMENTATION AT THE PLANT SITE (HNP-1 AND HNP-2)

Primary Tower

Height Above Tower Base (m)	Sensed Parameter	Instrument Characteristics
100	Wind speed and direction	Climatronics, F460 sensor (speed), ± 0.07 m/s accuracy, 0.26 m/s threshold, 1.52 m/s distance constant, model 100075 transmitter, F460 sensor (direction), ± 2 degrees accuracy, 0.26 m/s threshold, 1.13 m/s distance constant, model 100076 transmitter with vane
	Wind direction variability	Climatronics, 101035 sigma theta computer, sampling rate 3600/h, resolution 1 part in 256
	Vertical temperature difference (100-10 m)	Climatronics, 100950-1 platinum dual ΔT translator, accuracy < ± 0.1°C, 100826 precision platinum 100-ohm 4-wire sensor
60	Wind speed and direction	(same as 100-m level)
	Wind direction variability	(same as 100-m level)
10	Wind speed and direction	(same as 100-m level)
	Wind direction variability	(same as 100-m level)
	Ambient temperature	Climatronics, 100826 platinum 100-ohm 4-wire, RTD temperature sensor, 100950 platinum temperature translator, accuracy < ± 0.1°C

TABLE 2.3-7 (SHEET 2 OF 3)

Primary Tower

Height Above Tower Base (m)	Sensed Parameter	Instrument Characteristics
	Dew point	Climatronics, 100743, lithium chloride dew point sensor, ± 0.5°C accuracy, 100089 lithium chloride dew point translator
Near tower base	Precipitation	Climatronics, 100097-1, tipping bucket gage with heater, accuracy ± 1% up to 3 in./h, 1000 ws wind screen, 100747 precipitation integrator

TABLE 2.3-7 (SHEET 3 OF 3)

Backup Tower

Height Above Tower Base (m)	Sensed Parameter	Instrument Characteristics
45	Vertical temperature difference (45-10 m)	Climatronics, 100950-1 platinum dual ΔT translator, accuracy < ± 0.1°C, 100826 precision platinum 100-ohm 4-wire sensors
	Wind speed and direction	Climatronics, F460 sensor (speed), ± 0.07 m/s accuracy, 0.26 m/s threshold, distance constant 1.52 m/s, 100075 transmitter, F460 sensor (direction), ± 2 degrees accuracy, 0.26 m/s threshold, distance constant 1.13 m/s, 100076 transmitter with vane
	Wind direction variability	Climatronics, 10135 computed sampling rate 3600/h, resolution 1 part in 256
10	Ambient temperature	Climatronics, 100950 platinum temperature translator, accuracy < ± 0.1°C, 100826 platinum sensor

TABLE 2.3-9 (SHEET 1 OF 4)

JOINT FREQUENCY TABLES OF WINDSPEED AND DIRECTION FOR 75-ft LEVEL

FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT

WIND DIRECTION

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	0	0	0	0	1	0	0	1	0	0	0	0	1	0	1	2	5	.1
CALM+ 3.5	60	58	56	61	53	43	21	18	14	24	32	21	16	31	38	54	610	6.8
3.6- 7.5	221	176	227	335	346	184	109	81	94	141	191	185	227	281	319	216	3333	36.9
7.6- 12.5	108	121	220	263	290	231	152	99	114	186	227	254	322	398	340	148	3473	38.5
12.6- 18.5	17	21	43	45	48	57	64	43	42	105	108	101	158	200	172	46	1270	14.1
18.6- 24.5	7	8	13	11	4	7	25	21	12	24	8	14	39	59	16	7	275	3.0
24.6- 32.5	1	2	0	5	0	1	14	10	1	1	1	1	7	4	3	0	51	.6
32.6+	0	1	0	0	0	0	2	4	0	0	0	0	0	0	0	0	7	.1
TOTAL	414	387	569	720	742	523	387	277	277	481	567	576	770	973	889	473	9025	100.0
PERCENT	4.6	4.3	6.3	8.0	8.2	5.8	4.3	3.1	3.1	5.3	6.3	6.4	8.5	10.8	9.9	5.2	100.0	
AV SPD	6.6	7.2	7.8	7.6	7.6	8.4	10.7	10.8	9.4	9.9	9.1	9.4	10.3	10.3	9.2	7.6		

AVERAGE SPEED FOR THIS TABLE EQUALS 8.9
HOURS OF UNSTEADY WIND = 285

FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT

WIND DIRECTION

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
CALM+ 3.5	28	17	19	19	20	6	7	6	7	7	10	5	10	8	24	12	205	12.7
3.6- 7.5	33	40	46	61	76	43	28	25	27	33	53	45	50	38	40	22	660	41.0
7.6- 12.5	14	16	36	41	37	40	34	26	21	39	51	38	41	39	31	27	531	33.0
12.6- 18.5	1	4	10	6	6	14	11	10	12	11	15	18	18	13	15	7	171	10.6
18.6- 24.5	0	0	0	2	3	1	3	2	3	3	7	5	1	5	1	0	36	2.2
24.6- 32.5	0	0	2	0	0	1	0	0	0	0	0	0	0	0	1	0	4	.2
32.6+	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	1	.1
TOTAL	76	77	113	129	142	105	83	70	70	93	136	111	120	103	112	68	1608	100.0
PERCENT	4.7	4.8	7.0	8.0	8.8	6.5	5.2	4.4	4.4	5.8	8.5	6.9	7.5	6.4	7.0	4.2	100.0	
AV SPD	5.1	5.9	7.4	7.1	7.0	8.4	8.9	8.9	9.0	8.5	8.7	8.9	8.3	8.8	7.6	7.7		

AVERAGE SPEED FOR THIS TABLE EQUALS 7.9
HOURS OF UNSTEADY WIND = 53

TABLE 2.3-9 (SHEET 2 OF 4)

FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	.1
CALM+ 3.5	17	13	11	14	10	7	9	1	7	10	12	5	8	10	6	13	153	12.4
3.6- 7.5	22	24	26	44	41	29	22	13	23	28	35	30	26	28	27	29	447	36.3
7.6- 12.5	28	18	26	25	32	31	17	13	23	33	42	32	41	30	34	18	443	36.0
12.6- 18.5	2	1	10	7	13	9	9	4	8	14	16	17	13	13	11	5	152	12.3
18.6- 24.5	0	0	1	1	0	2	1	0	1	6	5	2	4	3	4	0	30	2.4
24.6- 32.5	0	0	0	0	0	1	0	0	0	0	1	0	0	3	0	0	5	.4
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
TOTAL	69	56	74	91	96	79	58	31	62	91	111	87	92	87	82	65	1231	0.0
PERCENT	5.6	4.5	6.0	7.4	7.8	6.4	4.7	2.5	5.0	7.4	9.0	7.1	7.5	7.1	6.7	5.3	100.0	
AV SPD	6.4	6.2	7.9	7.1	7.8	8.5	8.0	8.3	8.0	8.8	9.0	9.0	9.3	9.6	9.2	6.9		

AVERAGE SPEED FOR THIS TABLE EQUALS 8.2
HOURS OF UNSTEADY WIND = 26

FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	8	5	5	6	5	11	7	2	5	4	3	4	2	3	7	3	80	1.0
CALM+ 3.5	89	73	86	91	119	80	67	55	48	53	62	64	59	50	56	53	1095	13.6
3.6- 7.5	155	180	234	286	322	222	199	151	186	255	294	179	173	196	145	136	3283	40.8
7.6- 12.5	94	94	154	256	165	169	147	61	108	242	301	182	146	161	134	136	2550	31.7
12.6- 18.5	15	19	41	60	49	55	42	26	52	79	103	49	49	83	78	41	841	10.5
18.6- 24.5	1	2	4	4	3	4	3	6	13	16	18	10	9	41	13	4	151	1.9
24.6- 32.5	0	0	0	0	0	0	0	0	2	1	2	8	11	13	3	3	43	.5
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
TOTAL	353	373	494	703	662	541	465	301	414	650	783	496	449	547	436	376	8043	0.0
PERCENT	4.4	4.6	6.1	8.7	8.2	6.7	5.8	3.7	5.1	8.1	9.7	6.2	5.6	6.8	5.4	4.7	100.0	
AV SPD	6.0	6.3	7.1	7.4	6.6	7.3	7.1	6.7	7.9	8.4	8.6	8.1	8.4	9.6	8.6	7.8		

AVERAGE SPEED FOR THIS TABLE EQUALS 7.7
HOURS OF UNSTEADY WIND = 88

TABLE 2.3-9 (SHEET 3 OF 4)

FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	5	10	3	8	8	7	3	4	3	6	4	3	2	8	4	3	81	1.0
CALM+ 3.5	78	77	99	92	110	112	85	83	70	70	73	74	61	63	70	58	1265	15.8
3.6- 7.5	117	128	219	295	323	312	334	290	319	410	335	217	222	205	156	122	4004	50.2
7.6- 12.5	37	36	61	63	80	139	172	124	207	285	322	172	140	107	117	90	2152	27.0
12.6- 18.5	1	6	3	20	9	12	26	13	35	61	64	32	41	57	41	20	441	5.5
18.6- 24.5	0	0	1	2	1	0	0	1	1	2	4	3	3	8	13	0	39	.5
24.6- 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	.0
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
TOTAL	238	257	376	480	531	582	620	515	635	834	802	501	469	449	401	293	7983	0.0
PERCENT	3.0	3.2	4.7	6.0	6.7	7.3	7.8	6.5	8.0	10.4	10.0	6.3	5.9	5.6	5.0	3.7	100.0	
AV SPD	4.8	5.0	5.3	5.6	5.3	5.9	6.5	6.2	7.0	7.4	7.6	7.0	7.2	7.7	7.6	6.6		

AVERAGE SPEED FOR THIS TABLE EQUALS 6.6
HOURS OF UNSTEADY WIND = 66

FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	9	1	7	5	4	6	5	6	1	0	4	4	10	5	11	9	87	3.1
CALM+ 3.5	45	50	35	41	87	47	47	44	34	38	26	26	44	39	42	37	682	24.3
3.6- 7.5	52	51	69	80	106	130	123	103	116	142	144	109	116	94	72	50	1557	55.5
7.6- 12.5	4	4	11	9	8	18	24	23	55	67	83	52	36	32	21	5	452	16.1
12.6- 18.5	0	0	0	0	0	1	0	1	1	5	2	3	2	3	6	0	24	.9
18.6- 24.5	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.0
24.6- 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
TOTAL	111	106	122	135	205	202	199	177	207	252	259	194	208	173	152	101	2803	0.0
PERCENT	4.0	3.8	4.4	4.8	7.3	7.2	7.1	6.3	7.4	9.0	9.2	6.9	7.4	6.2	5.4	3.6	100.0	
AV SPD	4.0	4.0	4.3	4.3	4.0	4.8	5.1	4.9	5.9	6.3	6.3	6.1	5.2	5.3	5.0	4.0		

AVERAGE SPEED FOR THIS TABLE EQUALS 5.1
HOURS OF UNSTEADY WIND = 27

TABLE 2.3-9 (SHEET 4 OF 4)

FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT

WIND DIRECTION

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	5	2	10	4	6	2	4	1	7	6	3	5	3	6	8	7	79	3.3
CALM+ 3.5	56	46	40	30	42	42	34	30	27	43	46	43	60	49	72	53	713	29.5
3.6- 7.5	39	50	45	48	42	58	49	45	72	91	174	109	155	149	103	58	1287	53.3
7.6- 12.5	2	2	4	13	9	6	13	9	22	58	62	47	42	14	11	4	318	13.2
12.6- 18.5	0	0	2	0	0	0	0	1	2	1	1	0	0	5	5	1	18	.7
18.6- 24.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	.0
24.6- 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
TOTAL	102	100	101	95	99	108	100	86	130	200	286	204	260	223	199	123	2416	0.0
PERCENT	4.2	4.1	4.2	3.9	4.1	4.5	4.1	3.6	5.4	8.3	11.8	8.4	10.8	9.2	8.2	5.1	100.0	
AV SPD	3.3	3.9	3.8	4.5	3.7	4.3	4.5	4.7	5.2	5.9	5.8	5.4	5.1	4.9	4.4	3.8		

AVERAGE SPEED FOR THIS TABLE EQUALS 4.8
HOURS OF UNSTEADY WIND = 24

TABLE 2.3-10 (SHEET 1 OF 4)

JOINT FREQUENCY TABLES OF WINDSPEED AND DIRECTION FOR 150-ft LEVEL

FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT AND
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	1	0	2	0	0	C	0	0	0	0	0	0	0	0	2	0	5	.1
CALM+ - 3.5	59	48	49	40	38	32	27	11	12	14	11	15	24	20	30	40	470	5.2
3.6 - 7.5	174	169	206	228	282	157	97	61	57	105	138	148	162	206	260	187	2637	29.1
7.6 - 12.5	144	126	191	279	416	259	137	92	114	159	191	261	291	379	311	174	3524	38.9
12.6 - 18.5	40	23	58	59	115	8C	71	48	54	110	123	163	199	238	229	114	1724	19.0
18.6 - 24.5	12	11	7	3	25	29	26	6	15	32	23	31	74	88	74	48	504	5.6
24.6 - 32.5	8	7	2	1	6	30	13	1	2	4	6	4	23	32	19	6	164	1.8
32.6+	1	1	0	0	1	18	2	0	0	0	0	0	1	5	6	1	36	.4
TOTAL	439	385	515	610	883	605	373	219	254	424	492	622	774	968	931	570	9064	100.0
PERCENT	4.8	4.2	5.7	6.7	9.7	6.7	4.1	2.4	2.8	4.7	5.4	6.9	8.5	10.7	10.3	6.3	100.0	
AV SPD	8.2	8.0	8.0	8.2	9.3	11.3	10.8	9.9	10.4	10.9	10.4	10.7	11.8	11.8	11.1	10.1		

AVERAGE SPEED FOR THIS TABLE EQUALS 10.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 225

REQUEST NUMBER

FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT AND
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	.1
CALM+ - 3.5	15	16	13	9	16	13	10	7	5	7	8	7	8	8	18	17	177	10.7
3.6 - 7.5	32	32	40	35	48	46	28	16	21	31	27	36	33	30	41	21	517	31.2
7.6 - 12.5	30	18	24	46	60	45	38	26	25	32	48	44	43	32	28	24	563	34.0
12.6 - 18.5	7	5	12	15	22	12	11	20	11	20	26	32	30	24	28	15	290	17.5
18.6 - 24.5	4	2	2	3	4	3	11	3	9	5	6	5	8	11	7	5	88	5.3
24.6 - 32.5	1	1	1	0	1	1	0	1	1	0	0	2	0	4	2	0	15	.9
32.6+	0	0	0	1	0	1	0	0	0	0	0	0	0	4	0	0	6	.4
TOTAL	89	74	92	109	151	122	98	73	72	95	115	126	122	113	124	82	1657	100.0
PERCENT	5.4	4.5	5.6	6.6	9.1	7.4	5.9	4.4	4.3	5.7	6.9	7.6	7.4	6.8	7.5	4.9	100.0	
AV SPD	8.2	7.1	7.8	8.8	8.7	8.7	9.6	10.2	10.5	9.6	9.8	10.4	10.1	12.0	9.4	8.8		

AVERAGE SPEED FOR THIS TABLE EQUALS 9.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 45

TABLE 2.3-10 (SHEET 2 OF 4)

FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT AND
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	2	.2
CALM+ - 3.5	18	5	6	11	3	6	4	4	6	7	9	5	5	9	6	11	115	9.2
3.6 - 7.5	16	25	24	34	33	24	17	15	20	29	33	26	20	26	22	26	390	31.3
7.6 - 12.5	27	22	22	22	50	23	20	15	23	27	42	38	33	22	20	14	420	33.7
12.6 - 18.5	7	3	8	17	11	16	4	9	7	12	23	18	26	20	21	13	215	17.3
18.6 - 24.5	0	4	2	3	6	5	1	1	3	9	8	9	7	4	15	4	81	6.5
24.6 - 32.5	0	0	0	0	1	4	1	0	0	0	2	0	3	2	4	1	18	1.4
32.6+	0	0	0	0	1	0	0	0	0	0	0	0	1	1	1	0	4	.3
TOTAL	68	59	62	88	105	78	47	44	60	84	117	96	95	84	89	69	1245	100.0
PERCENT	5.5	4.7	5.0	7.1	8.4	6.3	3.8	3.5	4.8	6.7	9.4	7.7	7.6	6.7	7.1	5.5	100.0	
AV SPD	7.3	8.6	8.1	8.5	9.9	10.5	8.7	9.0	8.7	9.9	10.2	10.6	11.4	10.3	12.6	8.7		

AVERAGE SPEED FOR THIS TABLE EQUALS 9.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 25

FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT AND
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	4	0	1	1	1	1	0	0	1	1	2	1	1	1	3	0	18	.2
CALM+ - 3.5	43	48	57	64	77	51	57	40	38	36	43	29	36	40	43	48	750	9.3
3.6 - 7.5	132	162	173	231	217	178	166	129	147	132	171	133	138	118	113	107	2447	30.3
7.6 - 12.5	113	98	176	254	258	193	152	126	135	266	315	267	178	154	169	121	2975	36.9
12.6 - 18.5	28	35	65	111	97	80	63	33	64	111	167	113	67	112	121	75	1342	16.6
18.6 - 24.5	13	3	10	9	30	25	14	16	24	36	34	20	25	43	57	18	377	4.7
24.6 - 32.5	3	1	0	1	2	11	1	2	8	14	9	20	17	26	16	5	136	1.7
32.5+	0	0	0	0	0	0	0	0	2	4	2	0	2	6	7	0	23	.3
TOTAL	336	347	482	671	682	539	453	346	419	600	743	583	464	500	529	374	8068	100.0
PERCENT	4.2	4.3	6.0	8.3	8.5	6.7	5.6	4.3	5.2	7.4	9.2	7.2	5.8	6.2	6.6	4.6	100.0	
AV SPD	8.0	7.3	8.3	8.6	8.9	9.2	8.4	8.5	9.5	10.6	10.4	10.5	10.2	11.6	11.6	9.4		

AVERAGE SPEED FOR THIS TABLE EQUALS 9.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 103

TABLE 2.3-10 (SHEET 3 OF 4)

FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT AND
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	1	2	2	3	5	2	3	2	2	2	3	2	3	2	1	4	39	.5
CALM+ - 3.5	36	18	30	30	39	36	50	27	39	31	33	35	37	35	21	28	519	6.7
3.6 - 7.5	73	82	122	177	208	239	196	177	165	147	158	116	107	100	97	101	2265	29.3
7.6 - 12.5	66	91	157	197	220	264	266	234	325	348	378	251	252	207	170	111	3537	45.8
12.6 - 18.5	10	6	24	29	42	67	54	59	106	151	171	87	99	83	74	49	1111	14.4
18.6 - 24.5	3	1	1	6	17	18	7	13	19	21	25	13	15	30	20	5	214	2.8
24.6 - 32.5	0	0	0	0	4	2	0	0	2	5	0	7	4	8	4	0	36	.5
32.6+	0	0	0	0	0	0	0	0	1	0	0	0	0	1	0	0	2	.0
TOTAL	189	210	336	442	535	622	576	512	659	705	768	511	517	466	387	298	7723	100.0
PERCENT	2.4	2.6	4.4	5.7	6.9	8.1	7.5	6.6	8.5	9.1	9.9	6.6	6.7	6.0	5.0	3.9	100.0	
AV SPD	7.1	7.4	7.8	7.9	8.4	8.5	8.2	8.8	9.5	10.1	10.2	9.8	9.8	10.4	10.1	8.5		

AVERAGE SPEED FOR THIS TABLE EQUALS 9.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 40

FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT AND
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	0	2	4	3	1	2	2	2	2	1	3	0	1	1	1	0	25	.9
CALM+ - 3.5	12	12	20	12	16	14	28	11	22	16	12	17	9	16	21	7	245	8.9
3.6 - 7.5	38	32	50	75	75	69	114	75	82	58	59	62	60	48	64	45	1006	36.7
7.6 - 12.5	26	36	44	56	87	77	73	64	96	106	120	102	111	91	56	32	1177	42.9
12.6 - 18.5	4	1	3	6	9	15	14	13	23	44	35	31	15	24	11	5	253	9.2
18.6 - 24.5	0	0	0	1	1	5	2	1	4	1	1	4	2	2	2	1	27	1.0
24.6 - 32.5	1	0	0	0	1	1	0	0	0	0	2	1	1	0	1	0	8	.3
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
TOTAL	81	83	121	153	190	183	233	166	229	226	232	217	199	182	156	90	2741	0.0
PERCENT	3.0	3.0	4.4	5.6	6.9	6.7	8.5	6.1	8.4	8.2	8.5	7.9	7.3	6.6	5.7	3.3	100.0	
AV SPD	7.0	6.6	6.5	7.1	7.6	8.2	7.0	7.5	8.2	9.1	9.1	8.9	8.8	8.8	7.4	7.2		

AVERAGE SPEED FOR THIS TABLE EQUALS 8.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 12

TABLE 2.3-10 (SHEET 4 OF 4)

REQUEST NUMBER

FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
TEMPERATURE DIFFERENCE BETWEEN 150 AND 33
FROM 7006010101 TO 72083124 , TEMP DIFF WAS MESURED BETWEEN 150-10 FT AND
ADJUSTED TO BE EQUIVALENT TO 150-33 FT BY MULTIPLYING BY A FACTOR OF .56

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT
CALM	0	2	0	0	3	2	0	2	1	0	1	1	1	2	3	1	19	.8
CALM+ - 3.5	18	15	14	10	18	9	12	24	13	17	22	17	25	21	21	15	271	11.0
3.6 - 7.5	76	44	46	78	61	58	69	39	50	48	80	76	111	81	77	76	1070	43.3
7.6 - 12.5	19	20	28	30	45	36	30	36	58	62	98	124	134	112	57	45	934	37.8
12.6 - 18.5	5	2	1	4	2	1	1	3	5	22	32	28	13	13	9	9	150	6.1
18.6 - 24.5	0	3	0	0	0	0	1	0	2	2	1	3	2	4	1	3	22	.9
24.6 - 32.5	1	0	0	0	0	0	0	1	0	0	0	0	3	0	0	0	5	.2
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
TOTAL	119	86	89	122	129	106	113	105	129	151	234	249	289	233	168	149	2471	0.0
PERCENT	4.8	3.5	3.6	4.9	5.2	4.3	4.6	4.2	5.2	6.1	9.5	10.1	11.7	9.4	6.8	6.0	100.0	
AV SPD	6.3	6.3	6.2	6.5	6.3	6.4	6.3	6.5	7.6	8.5	8.3	8.5	8.0	7.8	6.9	7.1		

AVERAGE SPEED FOR THIS TABLE EQUALS 7.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

TABLE 2.3-11

ATMOSPHERIC DISPERSION FACTORS FOR ACCIDENT EVALUATION BASED ON 4 YEARS OF SITE DATA (6/70 - 9/74)

		5% Probable X/Q Values (s/m^3)			50% Probable X/Q Values (s/m^3)	
Time Period (h)	Location of Release	Exclusion Area (1250 m)	Low-Population Distance (1250 m)[(a)]	Worst Condition[(b)]	Exclusion Area Site Boundary[(a)]	Low-Population Distance
0-1	Elevated for SLB (30 m)	2.8(-4)[(c)]	2.8(-4)	NA	3.7(-5)	3.7(-5)
0-1	Stack release (120 m)	2.5(-6)	2.5(-6)	NA	3.5(-7)	4.5(-7)
0-2	Stack release (120 m)	1.7(-6)	1.7(-6)	NA	4.0(-7)	4.0(-7)
2-8[(d)]	Stack release (120 m)	NA	9.4(-7)	3.2(-6)	NA	3.4(-7)
8-24[(d)]	Stack release (120 m)	NA	3.9(-7)	1.2(-6)	NA	1.5(-7)
24-96[(d)]	Stack release (120 m)	NA	2.0(-7)	3.3(-7)	NA	1.0(-7)
96-720[(d)]	Stack release (120 m)	NA	8.0(-8)	9.9(-8)	NA	5.5(-8)
0-1	Ground level in bldg wake	4.1(-4)	4.1(-4)	NA	3.7(-5)	3.7(-5)
0-2	Ground level in bldg wake	3.1(-4)	3.1(-4)	NA	3.1(-5)	3.1(-5)
2-8	Ground level in bldg wake	NA	1.7(-4)	4.3(-4)	NA	2.8(-5)
8-24	Ground level in bldg wake	NA	2.3(-5)	4.9(-5)	NA	8.5(-6)
24-96	Ground level in bldg wake	NA	1.1(-5)	1.7(-5)	NA	5.5(-6)
96-720	Ground level in bldg wake	NA	4.5(-6)	5.3(-6)	NA	3.2(-7)

a. Site boundary.
b. Highest average value calculated at low-population distance.
c. 2.8(-4) is 2.8×10^{-4}.
d. For 2-8, 8-24, 24-96, and 96-720 time periods, averaging periods of 8, 16, 72, and 624, respectively, were used.

TABLE 2.3-12

TEMPERATURE DIFFERENCE GROUPS FOR DETERMINING PASQUILL STABILITY CATEGORIES

Pasquill Category	AEC ΔT Model[a] (°F/100 ft)
A	$\Delta T < -1.0$
B	$-1.0 \leq \Delta T < -0.9$
C	$-0.9 \leq \Delta T < -0.8$
D	$-0.8 \leq \Delta T < -0.3$
E	$-0.3 \leq \Delta T < 0.8$
F	$0.8 \leq \Delta T < 2.2$
G	$2.2 \leq \Delta T$

a. In conversion from °C/100 m (Regulatory Guide 1.23) to °F/100 ft, values were rounded to nearest tenth of a degree.

TABLE 2.3-13 (SHEET 1 OF 4)

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-23R2
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	2	0	0	0	0	0	0	0	0	0	0	0	2	0	5	.1	.30
CALM+ - 3.5	21	20	21	17	14	14	14	4	7	6	5	6	11	12	14	16	202	2.3	2.49
3.6 - 7.5	148	114	129	114	150	78	57	33	32	48	74	69	80	99	132	144	1521	17.4	5.47
7.6 - 12.5	136	148	202	308	381	229	119	73	77	134	165	190	222	266	276	155	3081	35.3	9.75
12.6 - 18.5	83	67	108	126	246	166	100	72	81	121	145	194	206	275	243	122	2359	27.0	14.91
18.6 - 24.5	23	12	31	23	46	33	41	23	31	66	63	92	127	141	148	80	980	11.2	20.92
24.6 - 32.5	10	7	3	3	24	33	25	7	10	30	16	24	64	74	53	41	434	5.0	27.46
32.6+	8	8	1	6	4	42	12	0	2	1	5	4	17	31	17	6	158	1.8	37.67
TOTAL	430	376	497	593	865	595	368	212	240	406	473	599	729	898	895	564	8740	100.0	9.53
PERCENT	4.9	4.3	5.7	6.8	9.9	6.8	4.2	2.4	2.7	4.6	5.4	6.9	8.3	10.3	10.2	6.5	100.0		
AV SPD	10.5	10.2	10.3	10.4	11.5	14.4	13.8	12.6	13.3	14.0	13.2	13.6	15.1	15.2	14.3	12.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 210

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-23R2
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	.1	.30
CALM+ - 3.5	7	7	4	6	6	3	7	3	2	2	2	4	4	4	8	8	77	4.9	2.75
3.6 - 7.5	24	27	32	20	32	29	16	11	11	19	21	20	18	23	29	18	350	22.2	5.15
7.6 - 12.5	34	26	25	46	63	48	32	22	27	33	37	33	36	19	34	21	538	34.1	9.61
12.6 - 18.5	15	8	21	23	27	21	24	19	13	21	36	32	36	30	21	19	366	23.2	15.12
18.6 - 24.5	3	2	4	3	12	7	7	10	9	14	10	20	14	17	19	7	158	10.0	20.89
24.6 - 32.5	4	3	3	3	4	3	7	3	7	3	3	5	5	8	5	5	71	4.5	26.90
32.6+	1	0	0	1	1	2	0	1	1	0	0	1	0	8	2	0	18	1.1	38.17
TOTAL	88	73	89	104	145	114	93	69	70	92	109	115	113	109	118	76	1579	100.0	8.35
PERCENT	5.6	4.6	5.6	6.6	9.2	7.2	5.9	4.4	4.4	5.8	6.9	7.3	7.2	6.9	7.5	4.9	100.0		
AV SPD	10.4	9.1	10.1	11.2	11.2	11.1	12.3	13.2	13.4	12.3	12.4	13.3	13.1	15.4	12.3	11.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 44

TABLE 2.3-13 (SHEET 2 OF 4)

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-23R2
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	2	.2	.30
CALM+ - 3.5	11	1	3	6	1	3	2	3	4	3	3	1	3	6	1	6	57	4.9	2.39
3.6 - 7.5	15	20	12	17	16	15	9	9	10	16	20	14	16	17	12	22	240	20.8	5.32
7.6 - 12.5	22	18	22	28	45	26	17	13	22	28	29	21	21	17	25	17	371	32.2	9.77
12.6 - 18.5	15	13	13	17	18	17	12	12	11	12	32	29	31	19	15	14	280	24.3	15.16
18.6 - 24.5	4	3	4	7	6	6	2	4	7	11	16	11	8	9	15	6	119	10.3	20.99
24.6 - 32.5	0	3	1	3	5	4	2	1	1	7	7	5	7	3	13	3	65	5.6	27.67
32.6+	0	0	0	0	2	4	0	0	0	0	1	0	3	3	4	1	18	1.6	37.16
TOTAL	67	58	55	79	93	75	44	42	56	77	108	81	89	74	85	69	1152	100.0	8.20
PERCENT	5.8	5.0	4.8	6.9	8.1	6.5	3.8	3.6	4.9	6.7	9.4	7.0	7.7	6.4	7.4	6.0	100.0		
AV SPD	9.4	11.0	10.8	11.0	12.6	13.1	11.3	11.6	11.2	13.1	13.4	13.6	14.6	13.1	16.4	11.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 25

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-23R2
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	4	0	1	1	1	0	0	0	0	0	2	1	1	1	3	0	15	.2	.30
CALM+ - 3.5	14	26	18	26	29	25	27	19	18	14	9	9	14	16	19	19	302	4.0	2.38
3.6 - 7.5	83	103	117	117	120	107	101	72	71	65	86	85	76	75	76	75	1434	18.9	5.42
7.6 - 12.5	108	165	144	225	222	165	129	105	125	143	191	129	119	111	114	96	2231	29.4	9.33
12.6 - 18.5	83	70	124	179	164	118	105	86	85	192	237	198	117	129	128	94	2110	27.8	14.94
18.6 - 24.5	17	26	49	74	63	49	46	22	46	76	107	68	42	70	89	53	897	11.8	21.00
24.6 - 32.5	14	4	10	13	27	25	15	20	27	36	38	23	29	47	58	22	408	5.4	27.70
32.6+	2	1	1	1	6	15	3	3	10	20	13	22	21	37	28	5	188	2.5	38.16
TOTAL	325	335	464	636	632	504	426	327	383	549	683	535	419	486	517	364	7585	100.0	8.71
PERCENT	4.3	4.4	6.1	8.4	8.3	6.6	5.6	4.3	5.0	7.2	9.0	7.1	5.5	6.4	6.8	4.8	100.0		
AV SPD	11.1	10.2	11.5	11.9	12.4	12.8	11.7	11.9	13.3	14.9	14.4	14.7	14.3	16.1	16.0	13.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 95

TABLE 2.3-13 (SHEET 3 OF 4)

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-23R2
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	2	1	3	4	2	3	2	2	2	3	2	3	2	1	4	37	.5	.30
CALM+ - 3.5	10	6	8	9	9	8	13	5	12	12	6	12	10	10	8	8	146	2.0	2.25
3.6 - 7.5	38	26	47	54	58	65	71	45	43	34	48	36	42	41	29	46	723	10.0	5.53
7.6 - 12.5	63	65	102	149	182	189	159	145	132	116	130	88	85	77	81	74	1837	25.3	9.97
12.6 - 18.5	54	60	134	169	191	217	188	175	246	260	261	193	183	158	130	88	2727	37.6	15.18
18.6 - 24.5	11	8	29	26	37	63	58	72	121	137	174	100	99	100	79	54	1168	16.1	20.91
24.6 - 32.5	3	3	6	18	20	35	26	18	34	59	73	24	41	30	38	13	441	6.1	27.44
32.6+	3	1	1	0	17	12	4	10	13	18	11	19	14	30	18	4	175	2.4	36.55
TOTAL	183	191	328	428	518	591	522	472	603	638	706	474	477	448	384	291	7254	100.0	9.56
PERCENT	2.5	2.6	4.5	5.9	7.1	8.1	7.2	6.5	8.3	8.8	9.7	6.5	6.6	6.2	5.3	4.0	100.0		
AV SPD	11.5	12.1	12.7	12.7	13.7	14.0	13.4	14.4	15.6	16.6	16.7	16.1	16.1	17.1	16.5	13.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 35

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-23R2
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	4	3	1	2	2	2	2	1	2	0	1	1	1	0	23	.9	.30
CALM+ - 3.5	4	5	5	4	4	4	7	4	8	8	6	5	3	5	3	2	77	2.9	2.20
3.6 - 7.5	18	12	22	26	27	19	39	18	32	18	16	25	18	18	29	14	351	13.2	5.51
7.6 - 12.5	32	25	43	59	66	61	100	64	68	49	52	47	52	41	56	36	851	32.0	9.93
12.6 - 18.5	17	33	36	49	74	65	58	53	65	76	90	88	78	76	47	29	934	35.2	14.33
18.6 - 24.5	6	2	7	8	9	12	14	13	36	55	47	31	37	28	11	4	320	12.0	20.85
24.6 - 32.5	2	1	0	2	5	7	5	3	6	11	3	10	3	11	4	2	75	2.8	26.95
32.6+	1	0	0	0	1	5	1	1	4	0	3	4	3	0	2	0	25	.9	36.89
TOTAL	80	79	117	151	187	175	226	158	221	218	219	210	195	180	153	87	2656	100.0	7.75
PERCENT	3.0	3.0	4.4	5.7	7.0	6.6	8.5	5.9	8.3	8.2	8.2	7.9	7.3	6.8	5.8	3.3	100.0		
AV SPD	11.5	11.0	10.7	11.5	12.4	13.3	11.4	12.1	13.3	14.8	14.7	14.5	14.4	14.3	12.1	11.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 12

TABLE 2.3-13 (SHEET 4 OF 4)

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74063124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

REQUEST NUMBER 604-2382

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	2	0	0	2	2	0	2	1	0	1	1	1	2	3	1	18	.7	.30
CALM+ - 3.5	7	5	6	4	6	4	4	5	6	5	8	3	5	9	7	2	86	3.5	2.32
3.6 - 7.5	29	23	28	19	27	17	25	30	19	21	36	34	43	31	33	30	445	18.1	5.56
7.6 - 12.5	60	37	27	64	49	46	56	30	41	39	58	61	93	65	62	65	853	34.7	9.78
12.6 - 18.5	16	11	24	29	41	30	26	31	49	52	80	102	116	104	49	38	798	32.5	14.85
18.6 - 24.5	3	5	4	5	3	4	1	3	9	19	47	40	24	13	6	7	193	7.9	20.70
24.6 - 32.5	3	3	0	0	0	0	0	1	2	12	3	4	2	6	7	3	45	1.9	27.61
32.5+	1	0	0	0	0	0	1	1	2	0	0	3	5	1	0	3	17	.7	37.71
TOTAL	119	86	89	121	128	103	113	103	129	148	233	248	289	231	167	149	2456	100.0	7.45
PERCENT	4.8	3.5	3.6	4.9	5.2	4.2	4.6	4.2	5.3	6.0	9.5	10.1	11.8	9.4	6.8	6.1	100.0		
AV SPD	10.1	10.2	10.1	10.6	10.3	10.5	10.2	10.4	12.3	13.7	13.4	13.8	12.9	12.7	11.2	11.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-14 (SHEET 1 OF 4)

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-24R2

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	1	0	0	1	0	0	0	0	1	0	1	2	6	.1	.30
CALM+ - 1.5	6	3	5	4	8	6	0	2	2	7	3	4	3	2	1	4	60	.7	1.12
1.6 - 2.5	33	40	39	36	28	19	12	9	8	11	15	14	7	11	21	24	327	3.7	2.07
2.6 - 3.5	75	61	54	70	56	46	19	16	20	23	39	20	33	52	54	64	704	7.9	2.99
3.6 - 7.5	226	186	309	431	457	250	149	114	123	193	270	259	329	401	444	239	4390	49.4	5.33
7.6 - 12.5	59	73	125	141	166	162	149	78	82	182	190	216	278	343	289	119	2652	29.8	9.34
12.6 - 18.5	10	16	24	23	10	24	35	35	29	56	37	43	92	126	65	19	644	7.2	14.50
18.6+	3	3	3	6	3	1	21	18	5	3	2	1	16	18	5	0	110	1.2	20.66
TOTAL	412	382	559	713	731	518	385	273	269	475	556	557	759	953	880	471	8893	100.0	5.47
PERCENT	4.6	4.3	6.3	8.0	8.2	5.8	4.3	3.1	3.0	5.3	6.3	6.3	8.5	10.7	9.9	5.3	100.0		
AV SPD	5.4	5.9	6.4	6.2	6.2	6.8	8.7	8.8	7.7	8.0	7.4	7.6	8.4	8.4	7.5	6.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 280

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-24R2

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	3	1	0	2	1	0	1	1	0	1	1	0	0	1	1	3	16	1.0	1.10
1.6 - 2.5	16	12	10	10	8	5	6	4	5	5	9	2	7	5	16	6	126	8.0	2.03
2.6 - 3.5	16	17	16	15	20	7	3	4	9	5	9	14	11	6	17	3	172	10.9	2.96
3.6 - 7.5	36	34	54	71	83	57	35	35	30	49	66	50	57	45	47	27	776	49.4	5.24
7.6 - 12.5	4	12	30	27	22	22	29	17	14	28	34	34	34	33	22	25	387	24.6	9.25
12.6 - 18.5	1	1	0	2	4	5	7	5	10	4	15	5	9	10	8	1	87	5.5	14.41
18.6+	0	0	2	1	0	1	0	1	1	0	0	1	0	0	1	0	8	.5	21.31
TOTAL	76	77	112	128	138	97	61	67	69	92	134	106	118	100	112	65	1572	100.0	4.75
PERCENT	4.8	4.9	7.1	8.1	8.8	6.2	5.2	4.3	4.4	5.9	8.5	6.7	7.5	6.4	7.1	4.1	100.0		
AV SPD	4.2	4.8	6.1	5.8	5.7	6.9	7.3	7.3	7.3	7.0	7.1	7.2	6.8	7.3	6.2	6.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 50

TABLE 2.3-14 (SHEET 2 OF 4)

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-24R2
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	.1	.30
CALM+ - 1.5	3	2	2	2	1	2	2	1	1	1	1	1	0	1	2	1	23	1,9	1,18
1,6 - 2,5	7	7	2	5	6	4	3	0	5	7	7	2	4	5	3	8	75	6,3	2,01
2,6 - 3,5	10	9	9	14	7	3	7	3	4	7	8	10	9	7	8	8	123	10.4	2,94
3,6 - 7,5	38	25	31	41	53	39	25	14	30	43	49	35	33	35	34	29	555	47,0	5,29
7,6 - 12,5	10	10	24	20	21	26	18	9	15	21	30	27	33	19	26	15	324	27,4	9,32
12,6 - 18,5	0	0	2	1	2	1	3	2	4	10	9	8	9	9	7	3	70	5.9	14,44
18,6+	0	0	0	0	0	1	0	0	0	0	3	0	1	5	1	0	11	.9	20,54
TOTAL	66	54	70	83	90	76	58	29	59	89	107	84	89	81	81	64	1182	100.0	4,78
PERCENT	5.8	4.6	5.9	7.0	7.6	6.4	4.9	2.5	5.0	7.5	9.1	7.1	7.5	6.9	6.9	5.4	100.0		
AV SPD	5.3	5.1	6.6	5.9	6.2	6.7	6.6	6.9	6.6	7.2	7.4	7.2	7.6	8.0	7.5	5.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 26

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-24R2
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	6	5	4	3	4	8	6	2	4	4	3	4	2	3	7	3	68	.9	.30
CALM+ - 1.5	21	25	12	24	26	21	17	12	8	15	15	10	16	7	13	12	254	3,3	1,04
1,6 - 2,5	48	36	55	56	74	52	37	37	25	30	37	38	34	34	32	34	659	8,4	1,99
2,6 - 3,5	56	56	69	73	98	36	72	42	46	58	52	50	44	48	49	31	682	11.3	2,94
3,6 - 7,5	164	188	246	364	323	285	228	149	200	334	396	240	214	231	174	194	3930	50,3	5,06
7,6 - 12,5	47	51	87	156	103	107	84	37	84	147	213	114	95	144	122	85	1676	21,4	9,15
12,6 - 18,5	4	5	11	10	12	13	10	9	21	35	38	13	22	59	31	9	302	3,9	14,39
18,6+	0	0	0	0	0	0	0	0	1	1	2	8	12	13	3	3	43	.6	20,12
TOTAL	346	360	484	686	640	524	454	288	389	624	756	477	439	539	431	371	7814	100.0	3,75
PERCENT	4.4	4.7	6.2	8.8	8.2	6.7	5.8	3.7	5.0	8.0	9.7	6.1	5.6	6.9	5.5	4.7	100.0		
AV SPD	4.6	4.8	5.4	5.7	5.1	5.6	5.4	5.2	6.0	6.4	6.6	6.3	6.4	7.4	6.6	6.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 79

TABLE 2.3-14 (SHEET 3 OF 4)

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-24R2
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	5	9	3	8	4	6	3	4	3	5	4	3	2	8	4	3	74	1.0	.30
CALM+ - 1.5	32	32	45	33	40	32	25	34	14	21	23	29	21	24	36	30	471	6.1	.97
1.6 - 2.5	51	53	46	69	98	94	58	60	66	59	60	52	47	49	37	30	931	12.0	2.01
2.6 - 3.5	45	55	93	123	149	130	128	114	113	118	112	78	68	60	54	49	1459	19.2	2.97
3.6 - 7.5	95	96	175	206	207	283	324	262	349	509	484	276	259	226	194	141	4087	52.6	4.50
7.6 - 12.5	4	9	5	26	15	20	45	26	58	95	100	51	60	71	63	31	683	8.5	8.57
12.6 - 18.5	0	0	1	2	0	0	0	1	1	1	1	2	2	8	11	0	30	.4	13.62
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	.0	18.84
TOTAL	232	254	372	467	513	565	584	501	606	808	784	491	459	447	399	284	7766	100.0	2.95
PERCENT	3.0	3.3	4.8	6.0	6.6	7.3	7.5	6.5	7.8	10.4	10.1	6.3	5.9	5.8	5.1	3.7	100.0		
AV SPD	3.2	3.3	3.6	3.7	3.5	3.9	4.3	4.1	4.7	4.9	5.1	4.7	4.8	5.1	5.0	4.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 64

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-24R2
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	8	1	7	5	3	6	4	6	1	0	4	4	10	5	10	9	83	3.0	.30
CALM+ - 1.5	18	21	17	21	33	9	17	21	14	15	8	6	20	17	15	9	261	9.5	.95
1.6 - 2.5	31	31	23	27	59	43	34	28	27	27	24	22	32	26	30	30	494	18.0	1.97
2.6 - 3.5	24	24	33	35	54	80	52	49	43	39	47	43	48	50	36	23	680	24.7	2.98
3.6 - 7.5	28	28	41	46	51	58	82	65	118	154	167	106	95	65	47	26	1179	42.9	4.60
7.6 - 12.5	0	0	0	0	0	2	1	4	3	8	4	9	2	8	9	2	52	1.9	8.45
12.6 - 18.5	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.0	13.33
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	110	105	121	134	200	198	190	173	206	243	254	192	207	171	147	99	2750	0.0	2.13
PERCENT	4.0	3.8	4.4	4.9	7.3	7.2	6.9	6.3	7.5	8.8	9.2	7.0	7.5	6.2	5.3	3.6	100.0		
AV SPD	2.7	2.6	2.9	2.9	2.7	3.1	3.4	3.3	3.9	4.2	4.2	4.1	3.5	3.6	3.4	2.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 27

TABLE 2.3-14 (SHEET 4 OF 4)

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-24R2

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	5	2	10	4	5	2	4	1	7	6	3	5	3	6	8	7	78	3.2	.30
CALM+ - 1.5	29	16	14	15	24	13	12	12	11	14	14	22	22	12	19	23	272	11.3	.99
1.6 - 2.5	29	35	32	18	24	34	25	21	20	33	37	27	46	54	62	36	533	22.1	1.98
2.6 - 3.5	22	26	24	27	23	22	21	19	32	36	58	44	85	78	51	27	595	24.7	2.95
3.6 - 7.5	17	21	19	30	22	36	36	32	58	105	171	105	102	67	50	28	899	37.3	4.88
7.6 - 12.5	0	0	2	1	0	0	0	1	2	4	3	1	2	5	7	2	30	1.2	8.61
12.6 - 18.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	.0	14.53
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	102	100	101	95	98	107	98	86	130	199	286	204	260	222	197	123	2408	0.0	1.93
PERCENT	4.2	4.2	4.2	3.9	4.1	4.4	4.1	3.6	5.4	8.3	11.9	8.5	10.8	9.2	8.2	5.1	100.0		
AV SPD	2.2	2.6	2.5	3.0	2.5	2.9	3.0	3.1	3.4	3.9	3.9	3.6	3.4	3.2	2.9	2.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 23

TABLE 2.3-15 (SHEET 1 OF 48)

MONTH OF JANUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	1	2	1	1	3	5	1	1	1	0	0	0	1	1	0	19	3.0	2.69
3.6 - 7.5	4	8	5	7	8	3	8	2	1	1	5	1	7	2	5	4	71	11.4	5.50
7.6 - 12.5	15	32	22	27	20	16	9	5	7	12	9	17	19	20	28	16	275	44.0	9.71
12.6 - 18.5	11	16	19	11	22	8	1	1	0	4	9	11	9	15	23	6	163	26.1	14.75
18.6 - 24.5	0	1	2	1	2	2	3	4	4	0	5	6	5	15	16	2	69	11.0	21.25
24.6 - 32.5	0	0	0	0	0	0	2	0	0	0	2	0	1	7	12	4	23	4.5	27.67
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	32	58	50	47	53	32	28	13	13	18	30	35	42	60	82	32	625	0.0	9.82
PERCENT	5.1	9.3	8.0	7.5	8.5	5.1	4.5	2.1	2.1	2.9	4.8	5.6	6.7	9.6	13.1	5.1	100.0		
AV SPD	10.8	10.5	11.3	10.3	11.8	10.5	9.8	13.4	13.0	10.3	14.2	13.4	12.3	15.8	16.0	13.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	1	1	1	2	0	1	1	0	0	0	1	0	0	1	0	10	7.9	2.65
3.6 - 7.5	3	4	5	1	4	2	1	0	3	0	0	0	3	1	2	2	31	24.4	4.96
7.6 - 12.5	5	4	3	1	5	2	4	0	2	1	2	5	2	5	5	2	49	38.6	9.69
12.6 - 18.5	0	1	0	0	0	1	0	0	1	4	4	3	5	1	3	2	26	20.5	15.00
18.6 - 24.5	0	0	1	0	0	0	0	1	0	0	3	2	1	1	2	0	11	8.7	19.29
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	9	10	10	3	12	5	6	2	6	5	9	11	12	8	13	6	127	0.0	7.32
PERCENT	7.1	7.9	7.9	2.4	9.4	3.9	4.7	1.6	4.7	3.9	7.1	8.7	9.4	6.3	10.2	4.7	100.0		
AV SPD	6.9	7.7	7.3	5.8	7.1	8.9	7.5	12.3	8.5	14.8	16.8	12.8	12.5	10.8	11.2	10.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 10.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-15 (SHEET 2 OF 48)

MONTH OF JANUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	1	1	0	1	1	0	0	2	0	0	0	0	1	0	8	8.8	2.32
3.6 - 7.5	1	6	2	2	2	2	0	2	0	1	1	0	0	1	0	2	22	24.2	5.34
7.6 - 12.5	1	3	2	1	5	2	0	2	1	0	3	2	1	2	2	4	31	34.1	9.49
12.6 - 18.5	1	2	0	1	1	1	1	1	0	2	3	2	1	1	2	2	21	23.1	15.17
18.6 - 24.5	0	0	1	0	0	0	0	0	0	1	2	0	0	1	0	0	5	5.5	20.29
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	1	0	0	1	1	0	3	3.3	26.58
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	1.1	33.84
TOTAL	4	11	6	5	8	6	2	5	1	6	10	4	2	6	7	8	91	100.0	7.22
PERCENT	4.4	12.1	6.6	5.5	8.8	6.6	2.2	5.5	1.1	6.6	11.0	4.4	2.2	6.6	7.7	8.8	100.0		
AV SPD	8.0	8.4	8.6	7.9	9.9	7.7	7.1	8.3	10.6	10.6	15.1	12.8	12.3	15.1	16.7	9.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 10.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	5	2	1	7	5	5	4	0	2	4	0	1	2	4	5	5	52	6.3	2.41
3.6 - 7.5	14	11	16	15	14	11	15	11	8	6	11	11	7	4	9	8	171	20.7	5.21
7.6 - 12.5	15	10	8	11	23	12	16	9	22	21	21	9	14	13	26	23	253	30.7	9.77
12.6 - 18.5	21	6	12	17	11	3	4	4	15	19	26	17	11	19	26	19	230	27.9	14.75
18.6 - 24.5	4	1	4	2	0	1	0	0	5	12	12	5	4	2	10	14	76	9.2	20.60
24.6 - 32.5	0	0	0	0	1	0	0	0	2	9	4	0	1	2	13	3	35	4.2	27.25
32.6+	0	0	0	0	0	0	0	0	0	1	2	0	0	3	2	0	8	1.0	34.54
TOTAL	59	30	41	52	54	32	39	24	54	72	76	43	39	47	91	72	825	100.0	8.16
PERCENT	7.2	3.6	5.0	6.3	6.5	3.9	4.7	2.9	6.5	8.7	9.2	5.2	4.7	5.7	11.0	8.7	100.0		
AV SPD	10.8	9.0	10.1	9.6	9.5	8.0	7.7	8.2	11.9	14.9	14.2	11.9	11.3	14.3	15.0	13.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 9

TABLE 2.3-15 (SHEET 3 OF 48)

MONTH OF JANUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	.1	.30
CALM+ - 3.5	1	0	2	2	2	1	2	0	0	1	3	2	1	1	0	2	17	2.3	2.43
3.6 - 7.5	2	3	5	3	5	8	6	2	4	1	5	5	5	5	2	5	67	9.1	5.56
7.6 - 12.5	4	7	2	11	14	15	4	12	8	10	20	9	5	10	7	7	145	19.8	10.02
12.6 - 18.5	3	6	5	10	12	15	12	11	20	33	35	18	20	26	22	13	261	35.6	15.46
18.6 - 24.5	1	0	0	2	1	5	0	5	16	39	24	9	5	8	18	7	140	19.1	21.31
24.6 - 32.5	0	0	0	0	0	0	0	0	8	19	34	4	3	3	11	2	84	11.4	27.13
32.6+	0	0	0	0	0	0	0	0	0	4	3	0	2	4	5	1	19	2.6	34.33
TOTAL	11	16	14	28	34	45	24	30	56	107	121	47	41	57	66	37	734	100.0	11.47
PERCENT	1.5	2.2	1.9	3.8	4.6	6.1	3.3	4.1	7.6	14.6	16.5	6.4	5.6	7.8	9.0	5.0	100.0		
AV SPD	11.0	11.2	9.6	12.0	11.3	12.1	11.0	13.2	17.7	19.8	19.3	14.7	16.0	16.6	19.4	14.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 16.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	0	1	0	0	0	0	0	2	0	1	0	0	0	0	4	1.5	2.92
3.6 - 7.5	2	1	1	0	5	4	3	4	2	2	0	4	2	2	5	0	38	14.2	5.74
7.6 - 12.5	4	3	4	6	2	6	16	7	9	5	8	3	7	5	8	5	98	33.6	10.14
12.6 - 18.5	0	2	2	3	3	7	3	5	11	7	15	9	5	10	9	0	93	35.0	15.02
18.6 - 24.5	0	0	0	3	0	0	0	0	2	8	2	4	1	0	0	0	24	9.0	21.41
24.6 - 32.5	0	0	0	0	0	0	0	0	0	4	0	2	0	0	0	0	6	2.2	26.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	6	6	7	13	10	17	22	16	24	28	25	23	18	25	23	5	268	0.0	10.29
PERCENT	2.2	2.2	2.6	4.9	3.7	6.3	8.2	6.0	9.0	10.4	9.3	8.6	6.7	9.3	8.6	1.9	100.0		
AV SPD	9.5	10.3	11.4	12.7	9.9	10.5	10.3	10.8	13.4	16.2	14.6	14.6	12.4	13.6	10.9	9.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

TABLE 2.3-15 (SHEET 4 OF 48)

MONTH OF JANUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	1	0	1	0	0	0	0	1	0	0	2	2	0	0	8	3.5	1.93
3.6 - 7.5	2	2	0	0	3	2	4	1	1	0	3	4	3	2	7	8	42	18.5	5.37
7.6 - 12.5	2	3	4	3	8	5	6	5	4	3	5	2	10	8	10	8	86	37.9	9.77
12.6 - 18.5	3	3	3	7	4	2	3	0	4	1	10	8	5	3	5	10	72	31.7	15.02
18.6 - 24.5	0	1	0	0	0	0	0	1	2	0	5	5	1	0	0	1	16	7.0	20.32
24.6 - 32.5	0	0	0	0	0	0	0	0	0	3	0	0	0	0	0	0	3	1.3	26.40
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	8	9	8	10	16	9	13	7	11	8	23	19	22	15	22	27	227	0.0	8.80
PERCENT	3.5	4.0	3.5	4.4	7.0	4.0	5.7	3.1	4.8	3.5	10.1	8.4	9.7	6.6	9.7	11.9	100.0		
AV SPD	10.3	11.8	10.1	13.5	10.1	10.0	10.2	11.2	13.8	15.5	14.4	14.4	10.7	9.2	9.9	10.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 5 OF 48)

MONTH OF FEBRUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	2	2	1	0	0	1	1	0	0	0	0	0	3	1	2	13	1.8	2.35
3.6 - 7.5	8	8	11	9	7	8	3	0	1	2	3	5	3	5	6	4	83	11.6	5.26
7.6 - 12.5	17	11	17	34	35	19	6	3	2	3	9	6	14	17	15	19	227	31.6	9.90
12.6 - 18.5	14	3	10	6	12	2	5	7	2	3	9	27	23	29	40	24	216	30.1	15.10
18.6 - 24.5	3	1	2	2	0	0	1	5	2	6	8	9	10	21	38	24	132	18.4	20.98
24.6 - 32.5	0	0	0	1	0	0	0	0	0	2	0	6	7	4	15	3	38	5.3	27.00
32.6+	0	0	0	0	0	0	0	0	0	0	1	1	1	2	4	0	9	1.3	37.83
TOTAL	42	25	42	53	54	29	16	16	7	16	30	54	58	81	119	76	718	100.0	13.80
PERCENT	5.8	3.5	5.8	7.4	7.5	4.0	2.2	2.2	1.0	2.2	4.2	7.5	8.1	11.3	16.6	10.6	100.0		
AV SPD	11.7	8.3	10.1	10.2	10.3	8.8	10.9	14.9	14.0	16.1	15.1	16.7	16.5	16.2	18.5	15.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 15

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	0	2	1	0	0	0	0	0	0	0	0	0	0	0	4	2.7	3.11
3.6 - 7.5	3	0	3	1	3	1	0	0	0	2	1	2	1	0	1	1	19	13.0	4.92
7.6 - 12.5	6	2	0	4	3	4	1	1	5	2	2	3	0	0	3	1	37	25.3	10.23
12.6 - 18.5	5	0	1	4	1	1	0	1	1	4	2	5	4	2	4	8	43	29.5	15.14
18.6 - 24.5	1	0	0	0	0	0	1	3	6	3	1	4	3	2	5	3	32	21.9	21.18
24.6 - 32.5	0	0	0	0	0	0	0	0	1	1	1	1	0	1	2	2	9	6.2	26.29
32.6+	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	0	2	1.4	35.15
TOTAL	16	2	4	11	8	6	2	5	13	12	7	16	8	5	16	15	146	100.0	13.82
PERCENT	11.0	1.4	2.7	7.5	5.5	4.1	1.4	3.4	8.9	8.2	4.8	11.0	5.5	3.4	11.0	10.3	100.0		
AV SPD	11.1	8.8	6.9	10.6	7.8	9.8	17.4	17.4	17.3	15.4	15.2	16.4	16.7	19.8	18.4	17.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 6

TABLE 2.3-15 (SHEET 6 OF 48)

MONTH OF FEBRUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEC MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	2	2.5	3.11
3.6 - 7.5	0	2	2	0	0	0	0	0	0	0	0	0	0	1	0	0	5	6.3	5.28
7.6 - 12.5	0	1	1	2	2	1	0	1	1	3	2	1	1	2	2	0	20	25.0	10.39
12.6 - 18.5	1	0	1	2	0	1	0	3	1	1	2	4	0	2	3	5	26	32.5	15.62
18.6 - 24.5	0	0	0	1	0	0	0	1	3	1	0	0	1	4	2	1	14	17.5	20.71
24.6 - 32.5	0	0	0	0	0	0	0	0	1	2	1	1	2	0	1	1	9	11.2	26.23
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	3	0	1	0	4	5.0	39.24
TOTAL	2	3	4	5	2	2	0	5	6	7	6	6	7	9	9	7	80	100.0	12.70
PERCENT	2.5	3.7	5.0	6.3	2.5	2.5	0.0	6.3	7.5	8.7	7.5	7.5	8.7	11.2	11.2	8.7	100.0		
AV SPD	8.0	7.9	9.7	15.2	10.5	12.8	0.0	15.8	19.3	17.4	14.4	15.2	29.5	15.0	20.2	17.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 15.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEC MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	2	1	2	2	1	1	1	0	1	0	0	0	1	1	1	14	2.0	2.60
3.6 - 7.5	6	7	6	11	12	4	7	1	4	2	2	3	3	4	3	4	79	11.1	5.45
7.6 - 12.5	9	4	19	26	15	13	10	11	8	10	12	18	8	5	9	7	185	26.1	10.20
12.6 - 18.5	5	12	15	37	16	7	5	4	10	21	29	22	10	16	21	11	241	34.0	14.92
18.6 - 24.5	0	5	5	12	5	0	1	0	8	9	12	3	5	16	17	1	99	14.0	21.20
24.6 - 32.5	0	0	0	0	0	0	0	1	7	8	3	3	4	20	10	1	57	8.0	27.51
32.6+	0	0	0	0	0	0	0	0	0	1	0	9	9	10	5	0	34	4.8	38.70
TOTAL	20	30	46	88	51	25	24	18	37	52	58	58	39	72	66	25	709	100.0	11.47
PERCENT	2.8	4.2	6.5	12.4	7.2	3.5	3.4	2.5	5.2	7.3	8.2	8.2	5.5	10.2	9.3	3.5	100.0		
AV SPD	9.3	12.2	12.1	13.0	11.6	10.5	9.9	11.4	16.8	16.8	15.8	18.4	21.8	22.0	19.5	12.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 15.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

TABLE 2.3-15 (SHEET 7 OF 48)

MONTH OF FEBRUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.2	.30
CALM+ - 3.5	0	1	1	0	0	0	1	0	0	2	0	1	1	0	0	0	7	1.2	2.32
3.6 - 7.5	2	2	3	5	4	3	3	2	3	1	3	2	0	1	0	3	37	6.6	5.43
7.6 - 12.5	5	5	4	6	13	13	8	9	2	4	8	5	3	4	5	7	101	18.0	10.11
12.6 - 18.5	6	11	18	8	11	17	8	14	11	17	25	18	14	19	30	5	232	41.4	15.37
18.6 - 24.5	1	2	4	2	1	2	6	11	13	7	24	9	9	15	11	5	122	21.8	21.04
24.6 - 32.5	0	0	0	0	0	0	1	2	8	4	2	5	5	9	6	0	42	7.5	27.55
32.6+	0	0	0	0	0	0	0	0	2	1	0	1	0	8	6	0	18	3.2	37.36
TOTAL	14	21	31	21	29	35	27	38	39	36	62	41	32	56	58	20	560	100.0	12.16
PERCENT	2.5	3.7	5.5	3.7	5.2	6.3	4.8	6.8	7.0	6.4	11.1	7.3	5.7	10.0	10.4	3.6	100.0		
AV SPD	12.8	13.0	13.3	12.2	11.6	12.6	14.6	16.0	19.9	16.9	17.2	17.8	18.3	22.1	19.6	12.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 16.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	1	0	0	0	1	0	0	0	0	1	1	1	0	0	5	2.7	2.21
3.6 - 7.5	0	1	0	0	0	0	3	1	1	3	0	0	2	4	2	1	18	9.6	4.94
7.6 - 12.5	4	3	2	3	5	4	10	5	4	5	1	2	2	2	5	1	59	31.4	9.39
12.6 - 18.5	0	0	1	1	4	10	8	5	4	8	5	9	9	10	9	3	86	45.7	14.81
18.6 - 24.5	0	0	0	0	0	1	2	1	1	1	7	3	1	1	1	0	19	10.1	20.48
24.6 - 32.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	.5	24.77
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	4	4	4	4	9	15	24	12	10	18	13	15	15	18	18	5	188	0.0	10.09
PERCENT	2.1	2.1	2.1	2.1	4.8	8.0	12.8	6.4	5.3	9.6	6.9	8.0	8.0	9.6	9.5	2.7	100.0		
AV SPD	9.8	8.8	9.4	9.8	12.5	13.4	11.4	12.3	12.7	12.8	18.2	15.6	13.1	11.9	12.4	11.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 8 OF 48)

MONTH OF FEBRUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0	2	.9	.30
CALM+ - 3.5	1	0	0	0	0	0	0	1	1	1	0	1	0	2	0	1	8	3.5	2.65
3.6 - 7.5	3	2	2	0	2	4	0	3	4	1	1	2	2	6	4	4	40	17.6	5.66
7.6 - 12.5	14	2	0	7	3	4	5	0	0	2	0	4	3	4	11	13	77	33.9	9.92
12.6 - 18.5	0	0	0	0	7	6	2	7	2	1	5	12	14	12	7	4	79	34.8	15.07
18.6 - 24.5	0	0	0	0	0	0	0	1	1	1	9	5	2	0	1	0	20	8.8	21.44
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	.4	25.74
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	18	4	2	7	12	14	7	12	8	6	16	24	26	26	23	22	227	0.0	7.39
PERCENT	7.9	1.8	.9	3.1	5.3	6.2	3.1	5.3	3.5	2.6	7.0	10.6	11.5	11.5	10.1	9.7	100.0		
AV SPD	8.8	8.0	5.9	9.1	12.3	11.2	11.7	11.9	10.2	10.3	19.3	15.0	13.7	10.3	11.3	9.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 9 OF 48)

MONTH OF MARCH

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	2	1	1	0	1	1	0	1	1	2	0	0	2	0	0	1	13	1.5	2.41
3.6 - 7.5	6	8	3	11	11	7	3	8	2	1	3	6	4	7	13	8	101	11.5	5.46
7.6 - 12.5	7	10	13	21	51	21	19	11	10	9	16	21	16	15	29	10	279	31.7	9.81
12.6 - 18.5	6	2	2	12	33	18	10	10	17	19	12	23	40	16	23	11	254	28.9	15.02
18.6 - 24.5	0	0	0	0	1	0	3	3	13	15	1	5	30	26	13	6	116	13.2	21.26
24.6 - 32.5	0	0	0	0	0	0	0	1	6	9	0	2	17	36	9	2	82	9.3	27.24
32.6+	0	0	0	0	0	0	0	0	0	0	0	1	12	21	1	0	35	4.0	35.46
TOTAL	21	21	19	44	97	47	35	34	49	55	32	58	121	121	88	38	880	100.0	11.19
PERCENT	2.4	2.4	2.2	5.0	11.0	5.3	4.0	3.9	5.6	6.3	3.6	6.6	13.7	13.7	10.0	4.3	100.0		
AV SPD	9.3	8.7	9.5	10.5	11.4	11.2	11.7	11.7	16.4	17.5	11.8	13.6	19.5	22.5	14.3	13.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 15.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 27

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	1	0	0	0	0	0	0	0	0	0	0	0	0	1	0	2	1.3	2.43
3.6 - 7.5	1	3	2	3	5	4	1	1	0	2	1	0	3	1	3	2	33	20.8	5.01
7.6 - 12.5	4	2	5	13	7	5	5	1	2	1	1	4	7	0	0	0	57	35.8	10.08
12.6 - 18.5	0	1	1	2	3	2	2	2	2	1	4	3	9	4	2	4	42	26.4	15.29
18.6 - 24.5	0	0	1	0	1	0	0	2	2	1	0	2	1	2	3	4	19	11.9	20.64
24.6 - 32.5	0	0	0	0	0	0	0	0	2	0	0	0	0	1	0	0	3	1.9	25.59
32.6+	0	0	0	0	0	0	0	0	1	0	0	0	0	1	1	0	3	1.9	33.32
TOTAL	5	7	9	18	17	11	8	6	9	5	6	9	20	9	10	10	159	100.0	9.40
PERCENT	3.1	4.4	5.7	11.3	10.7	6.9	5.0	3.8	5.7	3.1	3.8	5.7	12.6	5.7	6.3	6.3	100.0		
AV SPD	9.2	7.5	10.4	9.8	10.0	9.4	10.7	14.8	19.5	11.1	12.9	14.5	12.8	18.4	14.4	16.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

TABLE 2.3-15 (SHEET 10 OF 48)

MONTH OF MARCH

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
3.6 - 7.5	3	4	1	1	2	0	2	1	0	2	0	2	2	1	0	0	21	15.7	5.52
7.6 - 12.5	1	3	0	2	4	3	6	5	3	4	4	2	1	0	0	2	40	29.9	9.73
12.6 - 18.5	4	0	0	2	1	4	1	0	3	0	11	2	12	5	1	1	47	35.1	15.22
18.6 - 24.5	1	0	0	0	1	0	1	0	2	2	3	2	2	0	3	1	18	13.4	21.36
24.6 - 32.5	0	0	0	1	0	0	0	0	0	2	0	0	3	0	1	0	7	5.2	29.23
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	.7	37.15
TOTAL	9	7	1	6	8	7	10	6	8	10	18	8	20	7	5	4	134	100.0	11.05
PERCENT	6.7	5.2	.7	4.5	6.0	5.2	7.5	4.5	6.0	7.5	13.4	6.0	14.9	5.2	3.7	3.0	100.0		
AV SPD	11.8	7.3	6.2	13.4	10.8	13.0	10.6	9.0	15.3	15.7	14.8	13.6	17.3	16.8	22.3	14.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	2	0	0	3	2	2	1	0	1	1	2	2	2	0	0	19	2.8	2.59
3.6 - 7.5	6	6	3	8	5	12	5	7	7	8	4	4	8	2	6	4	96	14.0	5.53
7.6 - 12.5	14	6	8	11	16	15	19	12	8	10	15	10	8	9	6	7	174	25.4	9.92
12.6 - 18.5	15	7	8	10	6	5	12	14	15	19	19	21	21	10	14	11	207	30.3	14.93
18.6 - 24.5	4	2	8	5	4	5	7	2	6	13	15	18	5	8	8	11	121	17.7	21.05
24.6 - 32.5	0	0	1	0	0	4	0	0	2	2	6	2	7	10	3	2	39	5.7	27.62
32.6+	0	0	0	0	0	1	0	0	2	4	0	1	3	16	1	0	28	4.1	37.73
TOTAL	40	23	28	34	35	44	45	36	40	57	60	58	54	57	38	35	684	100.0	10.69
PERCENT	5.8	3.4	4.1	5.0	5.1	6.4	6.6	5.3	5.8	8.3	8.8	8.5	7.9	8.3	5.6	5.1	100.0		
AV SPD	11.8	10.9	14.9	11.7	10.8	12.5	12.0	11.5	15.1	16.0	15.7	16.1	16.6	23.8	16.2	15.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 15

TABLE 2.3-15 (SHEET 11 OF 48)

MONTH OF MARCH

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	0	3	1	0	0	0	1	2	0	0	0	0	0	0	7	1.1	2.21
3.6 - 7.5	2	0	4	2	2	4	2	3	2	4	1	1	3	1	3	2	36	5.7	5.50
7.6 - 12.5	1	2	2	5	7	11	7	8	10	4	13	7	4	4	7	5	97	15.5	10.26
12.6 - 18.5	8	2	C	4	17	12	11	19	41	24	24	28	27	14	15	13	259	41.3	15.36
18.6 - 24.5	1	0	1	0	0	5	3	10	13	24	28	14	15	19	13	15	161	25.7	20.92
24.6 - 32.5	0	0	0	2	0	0	0	3	4	4	5	2	10	9	10	4	53	8.5	27.12
32.6+	0	0	0	0	0	0	0	1	1	2	1	0	2	6	1	0	14	2.2	36.82
TOTAL	12	4	7	16	27	32	23	44	72	64	72	52	61	53	49	39	627	100.0	13.62
PERCENT	1.9	.6	1.1	2.6	4.3	5.1	3.7	7.0	11.5	10.2	11.5	8.3	9.7	8.5	7.8	6.2	100.0		
AV SPD	14.8	12.5	8.8	11.9	13.4	13.1	13.8	16.2	16.5	17.7	17.6	16.6	18.7	22.0	18.1	17.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 17.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	1	1	1	0	0	0	0	1	0	1	0	0	1	0	0	6	2.5	2.31
3.6 - 7.5	1	0	1	2	3	1	3	2	0	0	3	3	0	1	1	1	22	9.1	5.44
7.6 - 12.5	1	C	2	3	6	3	3	5	1	3	3	4	6	6	5	2	53	21.9	10.07
12.6 - 18.5	1	1	2	3	3	8	1	4	7	14	15	10	5	9	3	3	90	37.2	15.19
18.6 - 24.5	0	0	0	0	0	0	0	2	12	16	9	7	9	12	0	0	67	27.7	20.36
24.6 - 32.5	0	0	0	0	0	0	0	0	1	0	1	2	0	0	0	0	4	1.7	26.76
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	3	2	6	9	12	12	7	13	22	33	32	26	21	29	9	6	242	0.0	11.38
PERCENT	1.2	.8	2.5	3.7	5.0	5.0	2.9	5.4	9.1	13.6	13.2	10.7	8.7	12.0	3.7	2.5	100.0		
AV SPD	9.3	8.4	9.9	10.3	9.1	13.1	9.0	12.2	17.9	17.3	15.4	15.8	16.3	16.2	12.2	11.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-15 (SHEET 12 OF 48)

MONTH OF MARCH

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.4	.30
CALM+ - 3.5	0	0	1	0	0	1	1	2	1	1	1	0	0	0	2	0	10	4.4	2.58
3.6 - 7.5	2	0	1	0	4	1	6	4	3	3	5	3	6	1	3	0	42	18.6	5.44
7.6 - 12.5	2	2	1	4	3	5	6	4	1	1	2	6	15	7	3	9	71	31.4	9.96
12.6 - 18.5	0	0	0	2	3	2	1	3	8	4	10	6	9	14	3	5	70	31.0	15.00
18.6 - 24.5	0	0	0	0	1	1	0	1	1	5	11	2	5	4	0	0	31	13.7	20.29
24.6 - 32.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	.4	25.57
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	4	3	3	6	11	10	14	14	14	15	29	17	35	26	11	14	226	0.0	7.99
PERCENT	1.8	1.3	1.3	2.7	4.9	4.4	6.2	6.2	6.2	6.6	12.8	7.5	15.5	11.5	4.9	6.2	100.0		
AV SPD	8.1	8.4	5.7	10.5	10.4	10.0	7.9	9.5	11.9	14.7	15.1	12.0	12.4	15.0	9.0	11.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-15 (SHEET 13 OF 48)

MONTH OF APRIL

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	2	2.0	.00
CALM+ - 3.5	0	1	0	2	0	0	0	1	1	0	0	0	0	0	0	0	5	5.1	2.15
3.6 - 7.5	0	2	0	3	1	1	0	0	1	1	1	0	0	1	1	2	14	14.1	4.87
7.6 - 12.5	1	1	0	4	5	3	0	1	0	3	1	1	2	3	2	0	27	27.3	9.75
12.6 - 18.5	0	0	0	0	0	0	2	1	3	2	5	6	9	1	0	0	30	30.3	15.13
18.6 - 24.5	0	0	0	0	1	1	0	0	0	0	4	5	3	1	2	0	17	17.2	21.12
24.6 - 32.5	0	0	0	0	0	0	0	0	0	1	1	1	1	0	0	0	4	4.0	28.08
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	1	4	0	10	7	5	3	3	6	7	12	13	15	6	5	2	99	0.0	5.64
PERCENT	1.0	4.0	0.0	10.1	7.1	5.1	3.0	3.0	6.1	7.1	12.1	13.1	15.2	6.1	5.1	2.0	100.0		
AV SPD	7.9	5.3	0.0	5.6	11.0	11.2	15.0	9.3	8.1	13.7	16.5	18.9	16.9	12.4	13.9	4.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	1	0	0	0	0	0	0	0	0	1	0	0	0	2	.4	.00
CALM+ - 3.5	3	1	1	1	0	1	1	2	1	0	0	0	1	0	0	1	13	2.3	2.22
3.6 - 7.5	1	4	5	3	5	4	6	3	4	9	7	5	3	3	3	2	67	12.0	5.63
7.6 - 12.5	10	7	6	7	17	15	7	9	9	19	9	13	10	9	5	5	157	28.0	10.20
12.6 - 18.5	10	5	2	8	11	13	12	10	6	19	21	35	17	18	10	3	200	35.7	15.22
18.6 - 24.5	1	3	2	6	3	7	0	1	5	6	16	14	7	12	8	1	92	16.4	20.90
24.6 - 32.5	0	0	0	4	3	0	0	0	0	4	6	1	1	1	2	0	22	3.9	28.12
32.6+	0	0	0	1	0	0	0	0	1	0	3	1	1	0	0	0	7	1.2	35.50
TOTAL	25	20	16	31	39	40	26	25	26	57	62	69	41	43	28	12	560	100.0	9.50
PERCENT	4.5	3.6	2.9	5.5	7.0	7.1	4.6	4.5	4.6	10.2	11.1	12.3	7.3	7.7	5.0	2.1	100.0		
AV SPD	11.1	11.8	10.2	15.7	13.1	13.2	11.3	11.4	12.9	13.9	17.6	15.8	14.7	15.7	16.0	10.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

TABLE 2.3-15 (SHEET 14 OF 48)

MONTH OF APRIL

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	1	0	0	1	0	1	0	0	0	3	.6	.33
CALM+ - 3.5	0	0	1	1	0	0	1	1	0	1	0	0	1	1	2	0	9	1.7	1.73
3.6 - 7.5	2	0	3	0	3	4	4	1	1	0	1	1	2	1	0	3	26	4.9	5.58
7.6 - 12.5	3	3	3	9	14	2	7	12	8	6	7	7	9	6	3	6	105	19.8	10.37
12.6 - 18.5	5	1	5	14	12	13	20	17	19	20	38	21	19	9	9	8	230	43.3	15.14
18.6 - 24.5	0	0	0	2	0	0	6	2	14	14	35	25	10	9	4	1	122	23.0	20.95
24.6 - 32.5	0	0	0	0	0	0	1	0	2	5	12	6	5	0	0	1	33	6.2	26.42
32.6+	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0	0	3	.6	35.66
TOTAL	10	4	12	26	29	19	39	34	44	46	94	63	48	26	18	19	531	100.0	10.13
PERCENT	1.9	.8	2.3	4.9	5.5	3.6	7.3	6.4	8.3	8.7	17.7	11.9	9.0	4.9	3.4	3.6	100.0		
AV SPD	12.0	11.6	10.6	12.6	12.0	12.3	13.9	12.8	16.7	17.4	18.5	19.1	16.3	15.3	14.6	13.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 15.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	3	1	0	1	0	0	0	0	0	0	0	0	0	0	5	2.3	.33
CALM+ - 3.5	0	0	0	0	0	0	1	0	1	1	1	1	1	0	0	0	6	2.8	1.93
3.6 - 7.5	0	0	0	2	2	0	1	0	4	1	1	4	0	0	1	1	17	7.9	5.57
7.6 - 12.5	3	1	1	4	5	4	8	11	6	4	3	4	3	4	2	2	66	30.6	10.06
12.6 - 18.5	2	2	1	3	3	3	7	7	2	7	11	7	8	11	2	1	77	35.6	15.03
18.6 - 24.5	1	0	1	1	0	0	0	3	6	8	5	3	9	4	0	0	41	19.0	20.75
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	1	2	0	1	0	0	4	1.9	26.32
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	6	3	6	11	11	8	17	21	19	21	22	21	21	20	5	4	216	0.0	5.89
PERCENT	2.8	1.4	2.8	5.1	5.1	3.7	7.9	9.7	8.8	9.7	10.2	9.7	9.7	9.3	2.3	1.9	100.0		
AV SPD	13.4	14.6	7.5	10.8	11.2	13.8	11.6	13.1	12.8	15.8	15.6	13.3	16.4	15.1	10.4	9.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-15 (SHEET 15 OF 48)

MONTH OF APRIL

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	0	0	0	0	0	0	0	0	3	0	0	0	1	1	5	2.6	2.47
3.6 - 7.5	1	3	4	0	0	1	2	1	1	1	3	2	5	2	2	3	31	16.4	5.46
7.6 - 12.5	2	2	1	2	0	1	2	4	5	4	2	4	7	5	2	1	44	23.3	9.53
12.6 - 18.5	0	0	3	0	3	3	4	4	3	5	7	15	25	9	1	1	83	43.9	15.21
18.6 - 24.5	0	0	0	0	0	0	0	0	2	3	7	10	3	0	0	0	25	13.2	20.42
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	.5	24.77
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	3	5	8	2	3	5	8	9	11	13	22	32	40	16	6	6	189	0.0	9.92
PERCENT	1.6	2.6	4.2	1.1	1.6	2.6	4.2	4.8	5.8	6.9	11.6	16.9	21.2	8.5	3.2	3.2	100.0		
AV SPD	7.8	8.4	10.2	9.3	15.5	12.7	10.6	11.9	12.4	14.3	13.5	15.2	13.5	11.6	9.2	8.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 16 OF 48)

MONTH OF MAY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	2	.2	.30
CALM+ - 3.5	0	1	2	0	0	0	1	0	2	0	0	0	0	0	0	0	6	.7	2.39
3.6 - 7.5	10	8	7	5	11	3	3	8	3	3	9	7	11	19	18	13	138	15.0	5.75
7.6 - 12.5	20	16	19	12	15	17	10	13	13	10	20	26	31	41	54	21	338	36.6	9.83
12.6 - 18.5	8	8	3	3	5	11	12	15	20	31	29	36	33	47	39	9	310	33.6	14.32
18.6 - 24.5	0	0	3	0	3	3	4	8	7	17	10	19	14	7	9	4	108	11.7	20.59
24.6 - 32.5	0	0	0	0	0	0	0	0	2	10	2	0	3	2	1	0	20	2.2	26.57
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1	.1	33.93
TOTAL	39	33	35	20	35	34	30	44	47	71	70	88	93	116	121	47	923	100.0	9.81
PERCENT	4.2	3.6	3.8	2.2	3.8	3.7	3.3	4.8	5.1	7.7	7.6	9.5	10.1	12.6	13.1	5.1	100.0		
AV SPD	9.8	9.2	9.4	9.5	10.8	12.6	13.0	12.5	14.2	17.1	13.4	14.4	14.0	12.3	11.9	10.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 31

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	0	0	0	2	2	1	1	0	1	0	0	0	0	0	8	5.6	2.32
3.6 - 7.5	5	3	0	0	1	2	3	2	2	1	4	3	2	3	5	1	37	25.7	5.32
7.6 - 12.5	2	2	1	1	2	1	2	5	6	4	7	2	7	2	4	2	50	34.7	10.01
12.6 - 18.5	1	0	3	0	1	1	2	3	4	5	9	3	1	0	2	1	36	25.0	14.71
18.6 - 24.5	0	0	0	0	1	0	0	1	1	0	0	5	2	0	1	0	11	7.6	20.22
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	2	1.4	31.23
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	9	5	4	1	5	6	9	12	14	10	21	15	12	5	12	4	144	0.0	7.39
PERCENT	6.3	3.5	2.8	.7	3.5	4.2	6.3	8.3	9.7	6.9	14.6	10.4	8.3	3.5	8.3	2.8	100.0		
AV SPD	7.0	7.4	12.4	11.8	12.8	6.5	7.6	11.3	10.8	12.7	11.5	16.4	11.4	6.6	9.4	9.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 10.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

TABLE 2.3-15 (SHEET 17 OF 48)

MONTH OF MAY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	0	1	0	0	0	1	1	0	1	0	0	0	0	2	7	6.6	2.74
3.6 - 7.5	1	0	0	0	1	2	0	1	0	2	4	2	1	1	2	2	19	17.9	5.44
7.6 - 12.5	2	0	4	1	5	2	3	0	3	2	3	2	0	4	6	4	41	38.7	9.93
12.6 - 18.5	2	1	1	1	3	1	1	0	0	5	4	4	2	2	0	0	27	25.5	14.43
18.6 - 24.5	0	0	1	0	0	0	0	2	0	1	1	0	0	0	0	1	6	5.7	20.39
24.6 - 32.5	0	0	0	0	0	0	0	0	0	1	0	3	0	0	1	0	5	4.7	27.33
32.6+	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	.9	32.57
TOTAL	6	1	6	3	9	5	4	4	4	11	14	11	3	7	9	9	106	100.0	8.44
PERCENT	5.7	.9	5.7	2.8	8.5	4.7	3.8	3.8	3.8	10.4	13.2	10.4	2.8	6.6	8.5	8.5	100.0		
AV SPD	8.3	14.8	12.8	6.8	12.0	8.9	11.8	12.7	8.0	14.3	11.6	15.9	11.2	10.6	10.9	8.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	2	1	0	4	1	1	2	1	0	1	1	1	1	1	1	18	2.9	2.37
3.6 - 7.5	12	9	6	5	7	12	5	8	4	1	6	8	7	7	10	8	115	18.8	5.21
7.6 - 12.5	15	13	11	11	18	23	4	4	14	7	14	12	10	18	7	8	189	30.9	9.99
12.6 - 18.5	6	9	8	14	15	11	14	14	5	15	19	21	13	10	6	11	191	31.3	15.02
18.6 - 24.5	2	7	1	4	3	5	10	5	5	5	5	6	5	3	3	2	71	11.6	20.76
24.6 - 32.5	0	2	5	2	0	0	1	1	4	2	1	2	1	0	2	0	23	3.8	27.39
32.6+	0	0	1	0	0	0	0	0	0	1	2	0	0	0	0	0	4	.7	35.45
TOTAL	35	42	33	36	47	52	35	34	33	31	48	50	37	39	29	30	611	100.0	9.30
PERCENT	5.7	6.9	5.4	5.9	7.7	8.5	5.7	5.6	5.4	5.1	7.9	8.2	6.1	6.4	4.7	4.9	100.0		
AV SPD	9.9	12.6	14.0	14.1	11.3	10.9	14.5	12.8	13.9	16.1	14.0	13.5	12.9	11.3	11.4	10.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.7
HOURS IN ABOVE TABLE WITH VARIAPLE DIRECTION = 14

TABLE 2.3-15 (SHEET 18 OF 48)

MONTH OF MAY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	.2	.30
CALM+ - 3.5	0	0	0	0	0	1	0	1	1	1	0	0	1	1	1	0	7	1.1	2.66
3.6 - 7.5	4	1	5	2	5	9	2	2	4	0	2	7	0	8	1	6	59	9.7	5.23
7.6 - 12.5	9	3	7	4	9	13	11	17	12	9	18	6	12	12	6	13	166	27.2	9.96
12.6 - 18.5	9	10	0	5	6	22	13	19	36	40	30	16	15	5	13	9	248	40.6	15.11
18.6 - 24.5	1	3	0	1	1	5	12	16	26	5	15	8	3	4	7	4	111	18.2	20.70
24.6 - 32.5	0	2	0	0	0	0	1	1	3	4	1	0	1	0	1	0	14	2.3	27.94
32.6+	0	1	1	0	0	0	0	0	0	1	0	1	0	1	0	0	5	.8	34.01
TOTAL	23	20	13	12	22	50	39	56	82	60	66	38	32	31	29	38	611	100.0	10.87
PERCENT	3.8	3.3	2.1	2.0	3.6	8.2	6.4	9.2	13.4	9.8	10.8	6.2	5.2	5.1	4.7	6.2	100.0		
AV SPD	11.5	17.3	10.2	11.7	10.6	12.7	15.1	15.2	16.3	16.0	15.5	14.0	13.8	11.4	14.9	11.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	2	1.0	.30
CALM+ - 3.5	2	0	0	0	1	1	0	0	2	1	1	0	1	0	0	0	9	4.6	2.02
3.6 - 7.5	3	1	4	0	1	0	4	1	0	4	3	3	2	2	3	1	32	16.2	5.85
7.6 - 12.5	4	1	1	0	2	4	5	4	8	5	4	4	1	4	8	7	62	31.5	9.80
12.6 - 18.5	5	3	4	2	3	1	1	6	4	6	7	8	6	5	4	5	70	35.5	14.70
18.6 - 24.5	1	1	0	0	0	0	0	1	5	3	7	0	0	1	1	0	20	10.2	20.72
24.6 - 32.5	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	2	1.0	24.92
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	15	6	9	3	7	6	10	12	20	20	23	15	10	12	15	13	197	0.0	6.66
PERCENT	7.6	3.0	4.6	1.5	3.5	3.0	5.1	6.1	10.2	10.2	11.7	7.6	5.1	6.1	8.1	6.6	100.0		
AV SPD	10.0	13.8	9.9	12.3	9.5	9.9	9.2	12.3	12.4	12.6	15.3	11.6	11.9	12.3	10.8	11.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-15 (SHEET 19 OF 48)

MONTH OF MAY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN [illegible]
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

REQUEST NUMBER 604-43

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1	0	2	1.2	.33
CALM+ - 3.5	1	1	1	1	0	1	0	0	0	0	1	1	1	0	2	0	13	7.6	2.34
3.6 - 7.5	1	2	0	1	1	0	0	6	2	2	3	1	3	7	2	1	32	18.7	5.56
7.6 - 12.5	5	4	1	0	2	2	4	1	4	2	5	4	6	7	9	5	64	37.4	10.03
12.6 - 18.5	2	1	0	1	0	3	1	1	6	3	7	4	8	2	2	2	43	25.1	14.92
18.6 - 24.5	0	0	0	0	0	1	0	0	0	0	9	2	0	0	0	0	12	7.0	20.57
24.6 - 32.5	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	2	1.2	24.92
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	12	8	2	3	3	7	5	8	12	8	27	12	18	22	16	8	171	0.0	6.32
PERCENT	7.0	4.7	1.2	1.8	1.8	4.1	2.9	4.7	7.0	4.7	15.8	7.0	10.5	12.9	9.4	4.7	100.0		
AV SPD	9.4	8.8	6.1	7.2	8.5	13.0	10.7	7.5	12.0	12.5	14.7	12.4	11.2	11.0	8.7	10.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-15 (SHEET 20 OF 48)

MONTH OF JUNE

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	2	1	4	1	1	1	1	0	0	1	1	0	1	1	1	0	16	2.0	2.53
3.6 - 7.5	25	21	24	24	17	15	11	2	3	4	5	9	12	11	22	20	225	27.8	5.39
7.6 - 12.5	19	19	18	23	42	26	19	10	9	11	15	21	31	36	39	26	364	45.0	9.59
12.6 - 18.5	1	4	9	17	13	25	9	8	5	1	5	19	10	19	11	6	168	20.8	14.69
18.6 - 24.5	0	0	1	0	3	5	3	1	1	0	3	4	4	2	1	0	28	3.5	20.37
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	1	3	3	1	0	8	1.0	27.37
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	47	45	55	65	82	72	43	21	18	17	29	54	61	72	75	52	809	0.0	8.28
PERCENT	5.8	5.6	6.9	8.0	10.1	8.9	5.3	2.6	2.2	2.1	3.6	6.7	7.5	8.9	9.3	6.4	100.0		
AV SPD	7.3	8.0	8.4	9.3	10.7	11.7	10.1	11.6	11.2	8.7	11.1	12.4	11.4	11.7	9.4	8.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 10.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 17

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	1	1	1	1	0	1	0	0	0	0	0	2	1	0	1	10	6.6	2.64
3.6 - 7.5	2	2	4	2	5	4	4	1	2	1	3	3	2	2	2	5	44	28.9	5.35
7.6 - 12.5	2	4	1	2	10	5	4	1	1	2	3	4	4	2	2	2	49	32.2	9.45
12.6 - 18.5	2	0	0	0	2	2	4	6	0	1	0	7	4	3	0	0	31	20.4	14.73
18.6 - 24.5	0	0	0	0	2	0	0	0	0	0	2	3	3	3	1	0	14	9.2	21.74
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	1	2	1	0	4	2.6	25.17
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	7	7	6	5	20	11	13	8	3	4	8	17	16	13	6	8	152	0.0	7.39
PERCENT	4.6	4.6	3.9	3.3	13.2	7.2	8.6	5.3	2.0	2.6	5.3	11.2	10.5	8.6	3.9	5.3	100.0		
AV SPD	9.5	7.0	5.2	6.6	9.7	9.1	9.7	13.2	6.6	8.8	11.4	12.9	13.3	15.1	12.6	6.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 10.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-15 (SHEET 21 OF 48)

MONTH OF JUNE

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	2	0	0	0	0	0	0	0	0	0	0	0	1	2	0	0	5	4.7	2.35
3.6 - 7.5	2	1	3	1	2	4	3	0	0	1	3	3	0	2	3	5	33	31.1	5.23
7.6 - 12.5	1	1	1	5	2	1	3	0	4	3	3	3	4	1	2	1	35	33.0	8.70
12.6 - 18.5	0	0	0	0	0	3	2	2	2	0	2	3	1	1	1	0	17	16.0	14.32
18.6 - 24.5	0	0	0	0	1	1	1	1	0	0	1	2	0	0	1	0	8	7.5	19.93
24.6 - 32.5	0	0	0	0	0	0	0	1	0	0	0	0	0	2	4	0	7	6.6	28.79
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	.9	36.77
TOTAL	5	2	4	6	5	9	9	4	6	4	9	11	6	9	11	6	106	100.0	7.85
PERCENT	4.7	1.9	3.8	5.7	4.7	8.5	8.5	3.8	5.7	3.8	8.5	10.4	5.7	8.5	10.4	5.7	100.0		
AV SPD	5.2	7.8	5.3	8.9	10.5	11.2	10.6	20.7	10.4	9.6	10.5	11.5	10.9	15.5	16.7	6.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	0	0	1	0	0	0	0	0	0	0	0	1	0	0	3	.5	.33
CALM+ - 3.5	0	3	4	5	4	3	3	1	4	2	3	1	3	0	1	2	39	6.3	2.39
3.6 - 7.5	5	14	9	9	13	15	15	11	12	6	13	13	5	5	4	5	154	25.0	5.42
7.6 - 12.5	10	3	16	16	23	15	16	17	14	13	22	19	13	12	9	4	222	36.0	9.63
12.6 - 18.5	0	1	10	5	7	10	25	4	3	11	19	16	5	9	6	2	133	21.6	14.32
18.6 - 24.5	1	0	0	1	4	3	8	2	2	3	9	3	1	6	1	0	44	7.1	21.07
24.6 - 32.5	0	0	0	0	0	2	1	2	2	0	3	2	1	1	6	0	20	3.2	27.24
32.6+	0	0	0	0	0	1	0	0	0	0	0	0	0	0	1	0	2	.3	33.05
TOTAL	17	21	39	36	52	49	68	37	37	35	69	54	28	34	28	13	617	100.0	6.39
PERCENT	2.8	3.4	6.3	5.8	8.4	7.9	11.0	6.0	6.0	5.7	11.2	8.8	4.5	5.5	4.5	2.1	100.0		
AV SPD	8.6	6.3	9.2	8.9	9.7	11.2	12.2	10.3	9.6	11.3	12.2	11.9	10.2	12.9	15.1	8.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 10.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-15 (SHEET 22 OF 48)

MONTH OF JUNE

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
REQUEST NUMBER 634-43
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	1	1	1	0	1	0	1	0	1	0	0	6	.8	.30
CALM+ - 3.5	1	0	0	1	1	0	2	0	0	0	1	0	0	3	0	2	11	1.5	2.49
3.5 - 7.5	3	1	5	7	9	6	9	7	5	5	3	3	5	6	3	8	85	11.7	5.57
7.6 - 12.5	1	6	7	13	24	18	24	21	17	18	13	9	4	6	6	7	194	26.6	9.91
12.6 - 18.5	4	2	8	15	25	23	41	19	27	23	22	22	15	16	9	8	279	38.3	15.03
18.6 - 24.5	1	0	0	3	3	5	13	5	4	6	13	12	10	4	3	5	87	12.0	20.74
24.6 - 32.5	0	0	0	1	8	5	4	1	4	2	4	0	5	2	0	0	36	4.9	28.72
32.6+	0	0	0	0	3	0	1	4	1	1	0	0	4	8	2	0	30	4.1	36.63
TOTAL	10	9	20	40	79	58	95	58	58	56	56	47	43	46	23	30	728	100.0	8.67
PERCENT	1.4	1.2	2.7	5.5	10.9	8.0	13.0	8.0	8.0	7.7	7.7	6.5	5.9	6.3	3.2	4.1	100.0		
AV SPD	11.9	9.7	10.8	12.4	16.5	13.9	14.2	14.3	14.6	13.8	15.7	15.3	18.7	17.1	15.3	11.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
REQUEST NUMBER 634-43
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	1	0	0	1	1	1	0	0	0	1	0	5	2.1	.30
CALM+ - 3.5	1	1	0	1	0	1	2	0	2	1	1	0	0	1	1	1	13	5.4	2.29
3.5 - 7.5	4	2	4	3	7	2	7	1	8	2	1	4	3	5	1	3	57	23.7	5.53
7.6 - 12.5	2	2	4	6	5	8	10	7	8	5	2	3	6	3	5	8	84	34.9	9.73
12.6 - 18.5	0	1	2	3	3	3	8	9	4	3	8	6	4	6	3	1	64	26.6	14.99
18.6 - 24.5	0	0	1	0	0	0	3	0	0	1	2	2	4	1	2	0	16	6.6	20.32
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1	.4	24.93
32.6+	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.4	43.54
TOTAL	8	6	11	13	15	15	30	17	23	13	15	15	18	16	13	13	241	100.0	5.15
PERCENT	3.3	2.5	4.6	5.4	6.2	6.2	12.4	7.1	9.5	5.4	6.2	6.2	7.5	6.6	5.4	5.4	100.0		
AV SPD	10.5	7.7	10.7	9.9	8.5	9.3	10.9	12.4	8.2	9.8	13.5	12.5	13.5	10.9	11.0	9.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 10.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-15 (SHEET 23 OF 48)

MONTH OF JUNE

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

REQUEST NUMBER 604-43

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	2	1	0	0	2	1	1	1	0	0	1	0	0	0	0	0	9	8.6	2.42
3.6 - 7.5	5	2	4	2	1	1	2	2	1	2	1	0	2	3	4	3	35	33.3	5.17
7.6 - 12.5	1	0	0	1	5	1	1	1	3	1	5	4	5	4	4	0	35	33.3	9.73
12.6 - 18.5	0	0	0	0	1	1	1	0	2	5	1	2	1	5	3	1	23	21.9	13.92
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0	1	0	1	0	0	1	3	2.9	20.07
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	8	3	4	3	7	4	5	4	6	8	9	6	10	12	11	5	105	0.0	6.62
PERCENT	7.6	2.9	3.8	2.9	6.7	3.8	4.8	3.8	5.7	7.6	8.6	5.7	9.5	11.4	10.5	4.8	100.0		
AV SPD	5.3	3.6	6.0	7.9	7.6	7.3	7.4	6.3	10.5	11.4	9.5	11.4	10.7	10.5	9.1	10.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 24 OF 48)

MONTH OF JULY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	5	3	2	3	3	2	0	1	0	0	1	2	4	4	7	3	40	4.2	2.33
3.6 - 7.5	32	7	16	25	23	10	9	5	3	13	21	25	15	18	25	36	285	29.8	5.47
7.6 - 12.5	7	9	29	47	35	21	11	10	4	30	33	39	39	46	37	16	413	43.2	9.62
12.6 - 18.5	5	1	3	7	20	12	6	5	2	16	24	21	22	22	16	5	187	19.5	14.64
18.6 - 24.5	1	0	0	0	4	1	0	0	0	3	2	5	3	2	2	1	24	2.5	20.19
24.6 - 32.5	0	0	0	0	1	1	0	0	1	1	0	1	0	0	0	2	7	.7	26.33
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	50	20	52	82	87	47	26	21	10	63	81	93	82	92	87	63	956	0.0	7.59
PERCENT	5.2	2.1	5.4	8.6	9.1	4.9	2.7	2.2	1.0	6.6	8.5	9.7	8.6	9.6	9.1	6.6	100.0		
AV SPD	7.0	6.8	9.4	8.5	10.2	10.4	9.4	9.4	11.8	11.1	10.5	10.7	10.5	10.2	9.1	8.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	0	0	0	1	0	1	0	0	0	0	1	0	0	2	5	3.9	2.33
3.6 - 7.5	3	4	2	1	1	3	3	2	1	4	2	4	1	5	5	3	45	35.2	5.13
7.6 - 12.5	1	2	3	6	3	2	4	3	3	6	3	4	5	1	2	3	51	39.8	9.71
12.6 - 18.5	0	1	1	1	0	3	3	1	1	1	5	0	1	4	0	0	22	17.2	14.21
18.6 - 24.5	0	0	0	0	0	0	0	1	0	2	1	0	0	0	0	0	4	3.1	20.27
24.6 - 32.5	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1	.8	27.61
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	4	7	6	8	4	9	11	8	5	13	11	8	8	10	8	8	128	0.0	7.25
PERCENT	3.1	5.5	4.7	6.3	3.1	7.0	8.6	6.3	3.9	10.2	8.6	6.3	6.3	7.8	6.3	6.3	100.0		
AV SPD	6.3	7.4	9.0	10.1	8.7	8.9	11.6	10.8	9.2	10.4	12.4	7.8	8.7	9.9	6.3	5.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-15 (SHEET 25 OF 48)

MONTH OF JULY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-42
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	2	0	0	0	0	1	1	0	1	0	1	0	0	0	0	1	7	9.2	2.56
3.6 - 7.5	0	2	0	2	0	1	1	0	2	1	4	1	5	1	1	1	22	28.9	5.28
7.6 - 12.5	0	2	0	0	3	2	0	3	3	2	4	2	5	2	4	1	33	43.4	9.32
12.6 - 18.5	0	0	1	0	1	0	0	2	0	2	2	2	0	0	0	1	11	14.5	14.57
18.6 - 24.5	0	0	0	1	0	0	0	0	0	0	0	1	1	0	0	0	3	3.9	18.72
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	2	4	1	3	4	4	2	5	6	5	11	6	11	3	5	4	76	0.0	6.80
PERCENT	2.6	5.3	1.3	3.9	5.3	5.3	2.6	6.6	7.9	6.6	14.5	7.9	14.5	3.9	6.6	5.3	100.0		
AV SPD	2.5	7.1	13.9	11.0	9.6	6.9	3.2	12.8	7.4	12.8	8.4	12.5	8.6	7.9	9.2	8.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	1	0	0	0	0	0	0	0	1	1	0	0	3	0	7	1.5	.32
CALM+ - 3.5	0	2	1	0	2	1	4	5	2	1	0	1	1	1	1	1	23	4.9	2.53
3.6 - 7.5	5	5	18	9	7	4	7	4	5	10	9	8	11	13	11	6	132	28.3	5.41
7.6 - 12.5	1	6	8	6	7	13	12	14	9	24	27	15	15	10	6	2	175	37.5	9.68
12.6 - 18.5	1	2	1	2	4	5	3	7	5	28	19	18	3	2	0	1	101	21.6	14.51
18.6 - 24.5	0	0	2	0	0	1	3	2	3	3	2	2	0	2	1	0	21	4.5	20.32
24.6 - 32.5	1	0	0	0	1	0	0	2	0	0	0	0	0	1	0	0	5	1.1	23.58
32.6+	0	0	0	0	0	0	0	0	1	0	0	0	0	1	1	0	3	.6	36.25
TOTAL	9	15	31	17	21	24	29	34	25	66	58	45	30	30	23	10	467	100.0	5.62
PERCENT	1.9	3.2	6.6	3.6	4.5	5.1	6.2	7.3	5.4	14.1	12.4	9.6	6.4	6.4	4.9	2.1	100.0		
AV SPD	8.9	8.2	7.5	7.6	9.5	10.2	3.5	11.3	11.9	11.9	11.3	11.4	8.5	10.4	7.5	6.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 10.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-15 (SHEET 26 OF 48)

MONTH OF JULY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	1	0	2	3	1	0	0	0	0	1	0	1	1	0	3	14	2.5	.30
CALM+ - 3.5	5	1	2	0	0	1	0	0	5	1	1	1	4	2	3	2	28	4.9	2.16
3.6 - 7.5	8	2	6	3	5	2	7	8	6	4	8	4	7	7	8	7	92	16.2	5.46
7.6 - 12.5	6	5	14	4	7	25	26	14	15	16	19	14	21	15	7	6	214	37.7	9.77
12.6 - 18.5	1	8	11	5	5	15	13	13	19	28	23	20	9	6	2	2	181	31.9	15.02
18.6 - 24.5	0	1	0	0	2	0	5	2	5	5	2	6	2	4	1	0	35	6.2	20.52
24.6 - 32.5	1	0	0	0	0	1	0	0	0	1	0	0	0	0	1	0	4	.7	28.33
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	22	18	33	14	23	45	51	37	50	55	54	45	44	35	22	20	568	0.0	5.06
PERCENT	3.9	3.2	5.8	2.5	4.0	7.9	9.0	6.5	8.8	9.7	9.5	7.9	7.7	5.2	3.9	3.5	100.0		
AV SPD	7.0	11.5	10.1	9.5	9.9	11.2	12.0	11.7	12.1	13.4	12.0	13.5	9.7	10.2	9.2	6.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	1	0	0	0	1	0	0	0	0	0	2	1.4	.33
CALM+ - 3.5	0	2	1	0	0	0	1	0	0	0	1	1	0	2	1	0	9	6.3	2.06
3.6 - 7.5	0	1	3	3	1	5	2	1	4	2	1	0	0	0	3	1	27	18.7	5.49
7.6 - 12.5	0	0	0	1	2	3	4	7	7	6	2	5	9	1	1	0	48	33.3	9.67
12.6 - 18.5	0	0	1	3	3	2	4	1	6	2	5	10	9	3	0	0	49	34.0	14.67
18.6 - 24.5	0	0	0	0	0	0	1	1	2	1	2	1	0	0	1	0	9	6.3	21.89
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	0	3	5	7	6	10	13	10	19	11	12	17	18	6	6	1	144	0.0	5.84
PERCENT	0.0	2.1	3.5	4.9	4.2	6.9	9.0	6.9	13.2	7.6	8.3	11.8	12.5	4.2	4.2	.7	100.0		
AV SPD	0.0	3.6	6.7	10.0	12.3	8.9	9.9	11.2	12.1	10.8	12.2	13.0	12.5	9.5	8.5	7.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 10.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-15 (SHEET 27 OF 48)

MONTH OF JULY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 634-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	0	0	0	0	0	0	1	0	0	1	0	0	0	0	3	5.0	.35
CALM+ - 3.5	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	2	3.3	3.23
3.6 - 7.5	1	0	0	1	0	1	1	4	2	2	1	1	2	0	2	0	18	30.0	5.09
7.6 - 12.5	0	0	1	1	2	0	0	1	3	1	6	0	6	1	0	0	22	36.7	10.21
12.6 - 18.5	0	0	0	0	0	0	0	1	0	2	0	8	3	0	0	0	14	23.3	14.46
18.6 - 24.5	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	1.7	19.42
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	1	1	1	2	2	1	1	6	7	6	7	11	11	1	2	0	60	0.0	3.46
PERCENT	1.7	1.7	1.7	3.3	3.3	1.7	1.7	10.0	11.7	10.0	11.7	18.3	18.3	1.7	3.3	0.0	100.0		
AV SPD	7.4	.3	9.4	7.8	11.0	3.6	3.6	6.9	6.0	8.9	10.0	13.2	11.3	10.7	4.5	0.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-15 (SHEET 28 OF 48)

MONTH OF AUGUST

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	3	0	3	2	2	2	0	0	0	0	0	1	0	0	0	1	14	3.4	2.74
3.6 - 7.5	9	8	8	11	12	5	4	1	1	7	5	7	3	17	9	10	115	28.2	5.42
7.6 - 12.5	7	15	30	31	28	6	4	2	5	16	17	11	8	8	14	3	197	48.3	9.5[illegible]
12.6 - 18.5	3	2	5	2	10	5	3	1	0	6	5	10	12	9	1	0	74	18.1	14.43
18.6 - 24.5	0	0	0	0	1	0	0	0	1	1	2	1	1	1	0	0	8	2.0	19.42
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	21	25	46	46	45	18	11	4	7	30	29	30	24	35	23	14	408	0.0	7.76
PERCENT	5.1	6.1	11.3	11.3	11.0	4.4	2.7	1.0	1.7	7.4	7.1	7.4	5.9	8.6	5.6	3.4	100.0		
AV SPD	7.4	8.8	9.1	8.4	9.7	8.8	9.4	10.5	10.6	9.8	10.7	10.6	12.4	9.6	8.4	6.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	3	0	2	0	0	0	0	0	0	0	0	0	2	0	1	8	9.5	2.31
3.6 - 7.5	3	6	7	2	1	0	0	0	0	3	2	4	1	2	1	1	33	39.3	5.55
7.6 - 12.5	1	2	1	1	4	2	1	0	3	3	5	4	3	1	3	2	36	42.9	9.22
12.6 - 18.5	1	0	0	1	0	0	0	1	0	0	1	0	1	0	0	0	5	6.0	14.49
18.6 - 24.5	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	2	2.4	18.89
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	5	12	8	6	5	2	2	1	3	6	8	8	5	5	4	4	84	0.0	6.40
PERCENT	6.0	14.3	9.5	7.1	6.0	2.4	2.4	1.2	3.6	7.1	9.5	9.5	6.0	6.0	4.8	4.8	100.0		
AV SPD	7.9	5.8	6.3	7.2	9.4	3.2	14.0	14.8	8.7	7.8	8.6	7.3	10.4	5.4	9.[illegible]	6.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-15 (SHEET 29 OF 48)

MONTH OF AUGUST

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	1	1	0	0	0	0	1	0	0	0	0	2	0	1	7	10.9	2.11
3.6 - 7.5	3	0	2	1	2	2	0	0	0	3	3	2	3	2	1	2	26	40.6	5.35
7.6 - 12.5	0	1	0	1	2	4	2	0	1	4	2	3	3	0	0	1	24	37.5	9.97
12.6 - 18.5	0	0	0	0	0	0	1	0	0	0	1	0	2	0	0	0	4	6.3	14.15
18.6 - 24.5	1	0	0	0	0	0	0	0	0	0	1	0	1	0	0	0	3	4.7	21.37
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	5	1	3	3	4	6	3	0	2	7	7	5	9	4	1	4	64	0.0	5.66
PERCENT	7.8	1.6	4.7	4.7	6.3	9.4	4.7	0.0	3.1	10.9	10.9	7.8	14.1	6.3	1.6	6.3	100.0		
AV SPD	7.3	7.5	3.6	6.3	7.3	8.5	13.3	0.0	5.9	8.7	11.5	8.1	11.3	4.2	4.7	5.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 8.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	2	.4	.33
CALM+ - 3.5	2	4	3	2	3	2	5	3	2	2	3	0	0	1	1	1	34	6.9	2.52
3.6 - 7.5	4	11	14	11	12	11	16	8	16	12	19	8	9	9	4	6	170	34.7	5.54
7.6 - 12.5	8	9	10	18	13	7	10	8	24	23	24	7	5	2	5	5	178	36.3	9.75
12.6 - 18.5	0	0	0	0	7	5	3	3	4	21	21	5	9	3	1	3	85	17.3	14.35
18.6 - 24.5	0	0	0	0	0	3	0	0	0	2	4	1	0	0	0	0	10	2.0	20.53
24.6 - 32.5	0	0	0	0	1	1	3	1	0	1	2	0	0	0	0	0	9	1.8	23.23
32.6+	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	0	2	.4	35.93
TOTAL	15	24	27	31	36	31	37	23	46	61	74	21	23	15	11	15	490	100.0	6.46
PERCENT	3.1	4.9	5.5	6.3	7.3	6.3	7.6	4.7	9.4	12.4	15.1	4.3	4.7	3.1	2.2	3.1	100.0		
AV SPD	6.7	7.0	6.7	7.7	9.2	12.0	9.1	9.2	8.2	11.3	11.0	10.4	10.0	7.7	8.5	8.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-15 (SHEET 30 OF 48)

MONTH OF AUGUST

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 60443
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	0	1	1	0	1	0	1	1	1	1	1	0	0	0	9	2.2	.30
CALM+ - 3.5	1	1	0	0	1	2	2	0	0	0	1	3	1	1	2	0	15	3.7	2.32
3.6 - 7.5	2	3	3	6	4	8	13	4	7	4	8	6	4	0	1	1	74	18.4	5.44
7.6 - 12.5	5	1	9	6	14	16	15	11	23	23	7	6	5	0	3	4	149	37.1	9.53
12.6 - 18.5	0	0	6	6	5	10	8	15	28	17	18	13	5	0	0	0	131	32.6	14.96
18.6 - 24.5	0	0	0	0	5	1	0	1	2	5	1	4	0	0	0	0	19	4.7	20.72
24.6 - 32.5	0	0	0	0	0	1	2	2	0	0	0	0	0	0	0	0	5	1.2	27.02
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	8	6	18	19	30	38	41	33	61	50	36	33	17	1	6	5	402	0.0	5.25
PERCENT	2.0	1.5	4.5	4.7	7.5	9.5	10.2	8.2	15.2	12.4	9.0	8.2	4.2	.2	1.5	1.2	100.0		
AV SPD	7.5	4.9	10.1	9.7	11.7	10.8	9.6	12.8	12.0	12.4	11.4	11.5	9.5	3.4	6.2	8.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 60443
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0	2	1.9	.30
CALM+ - 3.5	0	1	1	0	1	2	0	1	0	1	1	0	0	0	0	0	8	7.7	2.16
3.6 - 7.5	0	3	5	4	0	0	4	1	4	1	1	1	2	1	3	1	31	29.8	5.13
7.6 - 12.5	1	3	1	1	1	2	5	2	5	3	5	4	2	0	2	1	38	36.5	9.41
12.6 - 18.5	0	0	0	1	0	2	2	2	2	1	2	4	1	0	0	1	18	17.3	14.90
18.6 - 24.5	0	0	0	0	0	1	0	3	0	1	1	1	0	0	0	0	7	6.7	20.54
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	1	7	7	6	2	7	11	11	11	7	10	10	5	1	5	3	104	0.0	4.73
PERCENT	1.0	6.7	6.7	5.8	1.9	6.7	10.6	10.6	10.6	6.7	9.6	9.6	4.8	1.0	4.8	2.9	100.0		
AV SPD	9.4	6.3	5.3	8.1	5.9	11.6	9.1	10.6	9.1	10.5	10.9	12.2	6.8	6.2	6.6	9.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.3
HOURS IN ABOVE TAPLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 31 OF 48)

MONTH OF AUGUST

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION

REQUEST NUMBER 634-43

FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2

SITE HATCH

PERIOD OF RECORD FROM 70060101 TO 74083124

SPEED AND DIRECTION FROM 150 LEVEL

TEMPERATURE DIFFERENCE BETWEEN 150 AND 35

SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	SEC MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	2	12.5	2.50
3.6 - 7.5	0	0	1	0	0	0	0	0	0	0	0	1	0	0	1	0	3	18.7	6.03
7.6 - 12.5	0	0	0	0	1	1	1	0	1	0	0	2	0	0	0	2	8	50.0	10.55
12.6 - 18.5	0	0	0	0	0	0	0	1	0	0	2	0	0	0	0	0	3	18.7	16.25
18.5 - 24.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
24.6 - 32.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	0	0	1	1	1	1	1	1	2	0	2	3	0	0	1	2	16	0.0	7.24
PERCENT	0.0	0.0	6.3	6.3	6.3	6.3	6.3	6.3	12.5	0.0	12.5	18.7	0.0	0.0	6.3	12.5	100.0		
AV SPD	0.0	0.0	7.0	3.4	11.8	11.7	11.7	15.7	7.0	0.0	16.6	9.1	0.0	0.0	5.0	9.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.9

HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 32 OF 48)

MONTH OF SEPTEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
REQUEST NUMBER 604-43
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	1	0	1	2	1	0	0	1	0	1	0	1	0	1	1	10	1.4	2.35
3.6 - 7.5	13	11	15	10	25	9	2	4	6	1	1	9	4	2	13	10	136	19.0	5.39
7.6 - 12.5	8	9	16	51	74	32	3	7	7	6	7	9	11	4	9	4	257	35.9	9.85
12.6 - 18.5	8	17	18	31	39	17	7	5	7	1	3	6	6	15	10	7	196	27.4	15.05
18.6 - 24.5	4	4	7	9	9	6	10	2	1	0	1	3	3	1	6	7	73	10.2	21.01
24.6 - 32.5	4	4	1	0	4	2	7	2	0	0	1	1	3	1	2	5	37	5.2	27.52
32.6+	2	1	0	0	0	0	1	0	0	0	0	0	0	0	2	0	6	.8	39.94
TOTAL	39	47	58	102	152	67	30	20	22	8	14	28	23	23	43	34	715	100.0	9.73
PERCENT	5.5	6.6	8.1	14.3	21.3	9.4	4.2	2.8	3.1	1.1	2.0	3.9	3.9	3.2	6.0	4.8	100.0		
AV SPD	14.0	14.1	11.9	12.1	11.5	11.8	20.6	13.2	10.6	8.9	12.2	11.6	14.0	14.6	14.3	14.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 33

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
REQUEST NUMBER 604-43
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	0	0	0	0	1	0	0	0	0	0	0	0	1	0	3	2.6	2.79
3.6 - 7.5	1	1	4	1	0	1	2	0	0	0	0	0	0	0	1	0	11	9.4	5.00
7.6 - 12.5	3	3	4	4	3	6	1	2	0	3	0	1	2	1	5	4	42	35.9	9.55
12.6 - 18.5	2	2	3	5	9	2	4	0	1	0	2	1	2	1	3	1	41	35.0	13.57
18.6 - 24.5	0	0	0	1	3	1	2	0	0	0	0	0	0	0	0	0	7	6.0	20.33
24.6 - 32.5	1	2	0	0	1	0	3	1	3	0	0	1	0	0	0	0	12	10.3	26.49
32.6+	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.9	40.84
TOTAL	9	8	11	14	15	10	13	3	4	3	2	3	4	2	10	5	117	100.0	10.53
PERCENT	7.7	6.8	9.4	12.0	13.7	8.5	11.1	2.6	3.4	2.6	1.7	2.6	3.4	1.7	8.5	4.3	100.0		
AV SPD	14.5	14.6	9.6	14.1	16.0	11.6	15.5	15.3	24.1	9.3	14.1	16.9	13.7	12.9	10.8	10.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION =)

TABLE 2.3-15 (SHEET 33 OF 48)

MONTH OF SEPTEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	2	2.5	2.92
3.5 - 7.5	1	1	1	1	0	2	2	2	1	1	2	0	2	1	1	1	19	23.7	5.33
7.5 - 12.5	2	1	5	4	7	2	0	0	0	1	3	1	2	2	3	0	33	41.2	9.36
12.5 - 18.5	2	1	1	3	1	4	1	1	0	0	0	1	0	2	1	0	18	22.5	15.35
18.6 - 24.5	0	0	1	3	0	1	0	0	0	0	0	1	0	0	0	0	6	7.5	20.22
24.5 - 32.5	0	0	0	0	1	0	0	0	0	0	0	0	1	0	0	0	2	2.5	28.21
32.5+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	6	3	8	11	9	10	3	3	1	2	5	3	5	5	5	1	80	0.0	8.60
PERCENT	7.5	3.7	10.0	13.7	11.2	12.5	3.7	3.7	1.2	2.5	6.3	3.7	6.3	6.3	6.3	1.2	100.0		
AV SPD	10.1	10.0	11.0	14.0	12.4	11.6	8.8	9.4	5.3	6.7	7.2	15.0	12.5	11.1	10.5	3.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	2	1	1	1	2	4	2	0	0	0	0	2	1	1	17	3.3	2.49
3.5 - 7.5	3	8	12	14	15	9	7	5	4	2	4	2	5	4	5	5	104	20.1	5.62
7.5 - 12.5	8	9	21	42	30	13	13	4	6	4	8	7	10	7	8	3	193	37.3	9.73
12.5 - 18.5	4	7	13	23	21	17	5	3	4	11	11	2	5	6	4	2	138	26.6	14.71
18.6 - 24.5	1	1	2	5	8	4	3	4	2	0	0	0	2	0	1	2	35	6.8	21.08
24.5 - 32.5	0	0	1	0	1	4	0	8	5	0	3	3	0	1	0	0	26	5.0	28.42
32.5+	0	0	0	0	1	0	0	0	0	0	0	1	3	0	0	0	5	1.0	35.15
TOTAL	16	25	51	85	79	47	30	28	23	17	26	15	25	20	19	13	518	100.0	9.06
PERCENT	3.1	4.8	9.8	16.4	15.1	9.1	5.8	5.4	4.4	3.3	5.0	2.9	4.8	3.9	3.7	2.5	100.0		
AV SPD	11.3	10.7	10.6	10.7	12.3	13.9	10.3	15.5	14.4	12.6	14.4	15.7	14.2	10.4	10.2	10.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

TABLE 2.3-15 (SHEET 34 OF 48)

MONTH OF SEPTEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	1	2	1	1	1	1	1	0	2	2	1	1	0	0	0	14	1.7	2.39
3.6 - 7.5	3	7	7	14	9	8	15	10	5	8	8	2	3	5	1	3	108	12.9	5.95
7.6 - 12.5	9	15	28	59	45	37	30	24	17	8	10	10	4	5	1	2	304	36.3	10.05
12.6 - 18.5	4	10	28	38	41	35	32	19	8	7	6	14	12	12	2	3	271	32.4	14.96
18.6 - 24.5	2	1	3	3	5	15	6	6	3	5	4	1	4	8	6	0	73	8.7	20.50
24.6 - 32.5	1	0	1	3	1	7	4	4	2	5	7	3	3	3	0	0	44	5.3	27.75
32.5+	0	0	0	0	0	0	2	2	4	2	3	5	5	0	0	0	23	2.7	35.30
TOTAL	19	34	69	118	103	103	90	66	39	37	40	36	32	33	10	8	837	100.0	10.67
PERCENT	2.3	4.1	8.2	14.1	12.3	12.3	10.8	7.9	4.7	4.4	4.8	4.3	3.8	3.9	1.2	1.0	100.0		
AV SPD	12.1	10.8	12.7	12.0	12.5	14.0	13.1	13.8	14.9	15.2	15.6	17.3	18.7	16.6	17.7	10.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1	.3	.30
CALM+ - 3.5	0	0	0	1	1	0	1	1	1	2	0	0	0	0	1	0	8	2.6	2.44
3.6 - 7.5	2	1	3	6	5	4	6	3	2	0	0	3	2	1	3	2	43	14.2	5.55
7.6 - 12.5	4	5	11	14	12	10	12	4	5	3	5	9	5	4	0	3	106	35.1	9.75
12.6 - 18.5	2	4	3	8	5	10	6	2	10	4	3	6	5	2	2	5	78	25.8	14.63
18.6 - 24.5	3	1	2	2	1	5	5	1	0	2	1	4	3	1	1	0	32	10.6	21.63
24.6 - 32.5	1	0	0	0	1	6	2	3	2	1	0	0	1	3	1	0	21	7.0	27.53
32.5+	0	0	0	0	0	0	1	1	2	0	2	3	3	0	1	0	13	4.3	35.53
TOTAL	12	11	19	31	26	35	34	15	22	12	11	25	19	11	9	10	302	100.0	9.12
PERCENT	4.0	3.6	6.3	10.3	8.6	11.6	11.3	5.0	7.3	4.0	3.6	8.3	6.3	3.6	3.0	3.3	100.0		
AV SPD	15.2	12.1	11.1	11.2	11.2	15.6	13.1	15.1	15.7	13.8	17.4	15.5	17.8	15.3	14.2	11.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-15 (SHEET 35 OF 48)

MONTH OF SEPTEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 634-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	1	1.1	.30
CALM+ - 3.5	0	1	1	1	0	0	0	1	0	0	0	0	1	0	0	0	5	5.6	2.34
3.6 - 7.5	1	1	1	2	0	1	1	1	0	1	0	0	1	1	2	3	16	17.8	5.15
7.6 - 12.5	1	0	2	2	1	0	0	3	3	1	0	4	1	1	3	0	22	24.4	9.39
12.6 - 18.5	1	0	1	0	0	0	0	0	5	2	0	1	2	6	3	2	23	25.6	15.24
18.6 - 24.5	1	0	0	0	0	0	0	0	0	1	0	2	4	2	1	0	11	12.2	21.35
24.6 - 32.5	1	0	0	0	0	0	0	0	0	0	1	1	1	1	1	0	6	6.7	28.48
32.6+	0	0	0	0	0	0	0	0	0	0	0	2	3	1	0	0	6	6.7	37.34
TOTAL	5	2	5	5	1	1	1	6	8	5	1	10	13	12	10	5	90	100.0	6.82
PERCENT	5.6	2.2	5.6	5.6	1.1	1.1	1.1	5.7	8.9	5.6	1.1	11.1	14.4	13.3	11.1	5.6	100.0		
AV SPD	16.1	3.4	9.5	5.7	8.4	7.4	3.6	7.1	13.2	13.1	30.6	20.0	21.8	18.5	12.9	10.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 36 OF 48)

MONTH OF OCTOBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
REQUEST NUMBER 634-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	3	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
3.6 - 7.5	13	13	16	6	14	4	3	0	2	2	1	3	2	2	7	9	97	12.4	5.43
7.6 - 12.5	13	13	15	27	46	25	12	2	4	3	4	4	8	10	6	11	203	25.9	9.78
12.6 - 18.5	9	7	21	21	44	21	7	3	4	4	2	1	7	16	15	9	192	24.5	14.95
18.5 - 24.5	5	3	11	7	15	9	12	0	0	3	2	3	4	8	14	2	99	12.6	20.84
24.5 - 32.5	5	3	2	2	18	29	14	4	0	0	1	1	0	3	8	8	99	12.6	28.28
32.6+	6	7	1	0	4	42	11	0	0	0	2	2	0	0	6	4	85	10.8	39.25
TOTAL	53	49	66	63	143	131	59	9	10	12	12	14	21	39	58	46	785	100.0	11.93
PERCENT	6.8	6.2	8.4	8.0	18.2	16.7	7.5	1.1	1.3	1.5	1.5	1.8	2.7	5.0	7.4	5.9	100.0		
AV SPD	15.9	14.8	13.2	13.1	15.6	25.3	22.4	20.5	10.7	13.9	18.3	16.4	13.6	15.5	18.9	15.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 17.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 13

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
REQUEST NUMBER 634-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	2	1.8	3.16
3.6 - 7.5	6	0	0	4	5	4	0	0	1	2	2	0	1	2	2	0	23	21.1	5.59
7.6 - 12.5	2	1	4	9	7	2	0	3	2	3	2	1	0	0	4	2	42	38.5	10.21
12.6 - 18.5	1	2	5	1	4	1	2	0	0	0	1	0	0	0	0	1	18	16.5	15.39
18.6 - 24.5	0	1	1	0	2	2	1	0	0	0	0	0	0	0	3	0	10	9.2	20.37
24.6 - 32.5	1	1	1	1	0	1	3	1	0	0	0	0	1	0	0	1	11	10.1	27.43
32.6+	0	0	0	1	1	1	0	0	0	0	0	0	0	0	0	0	3	2.8	42.87
TOTAL	5	5	12	16	19	11	6	4	3	5	5	1	2	2	9	4	109	100.0	9.89
PERCENT	4.6	4.6	11.0	14.7	17.4	10.1	5.5	3.7	2.8	4.6	4.6	.9	1.8	1.8	8.3	3.7	100.0		
AV SPD	12.4	19.9	14.7	13.0	12.7	16.7	23.6	13.6	8.8	7.5	9.1	8.9	16.4	4.9	12.9	15.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

TABLE 2.3-15 (SHEET 37 OF 48)

MONTH OF OCTOBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
3.6 - 7.5	3	1	0	3	2	1	1	0	3	4	0	1	1	3	1	3	27	26.2	5.47
7.6 - 12.5	3	3	5	1	4	0	2	0	5	1	1	0	1	0	2	0	28	27.2	9.51
12.6 - 18.5	0	1	5	5	7	2	1	0	1	0	0	0	0	2	1	1	26	25.2	15.55
18.6 - 24.5	2	0	0	2	2	3	0	0	0	0	0	0	0	0	1	2	12	11.7	20.61
24.6 - 32.5	0	2	0	0	0	2	2	0	0	0	1	0	0	0	1	0	8	7.8	28.18
32.6+	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	2	1.9	37.73
TOTAL	8	7	10	11	16	9	6	0	9	5	2	1	2	5	6	6	103	100.0	9.94
PERCENT	7.8	6.8	9.7	10.7	15.5	8.7	5.8	0.0	8.7	4.9	1.9	1.0	1.9	4.9	5.8	5.8	100.0		
AV SPD	10.4	15.4	13.0	13.4	15.1	21.1	16.7	0.0	9.1	7.1	20.1	6.5	7.8	10.2	14.6	12.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	3	4	1	5	3	1	1	0	2	1	1	1	2	0	4	3	32	4.6	2.15
3.6 - 7.5	14	12	8	10	6	14	4	2	3	1	2	6	9	9	7	6	113	16.1	5.73
7.6 - 12.5	7	16	25	46	29	17	9	2	4	2	3	2	6	6	6	6	186	26.5	9.33
12.6 - 18.5	3	9	42	41	37	15	3	3	1	5	2	5	3	7	5	5	186	26.5	15.09
18.6 - 24.5	2	0	12	24	15	12	8	3	1	2	4	0	1	9	5	6	106	15.1	21.13
24.6 - 32.5	8	1	2	1	9	10	6	3	0	2	3	2	3	1	3	7	61	8.7	27.60
32.6+	2	0	0	0	3	7	2	0	1	0	2	2	0	0	0	0	19	2.7	36.53
TOTAL	39	42	90	127	103	76	33	13	12	13	17	18	24	32	31	33	703	100.0	9.59
PERCENT	5.5	6.0	12.8	18.1	14.7	10.8	4.7	1.8	1.7	1.8	2.4	2.6	3.4	4.6	4.4	4.7	100.0		
AV SPD	14.3	9.3	13.9	13.3	15.9	17.0	17.5	17.3	11.4	15.8	19.7	14.7	10.7	13.7	12.7	14.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 9

TABLE 2.3-15 (SHEET 38 OF 48)

MONTH OF OCTOBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	1	0	1	1	0	1	1	0	1	0	2	0	1	0	1	10	1.5	2.50
3.6 - 7.5	5	2	2	3	3	5	5	3	3	2	0	0	1	1	5	1	41	6.1	5.63
7.6 - 12.5	11	8	8	17	22	18	15	10	6	5	4	5	2	4	15	3	154	23.1	9.94
12.6 - 18.5	2	23	41	47	37	33	15	5	6	8	10	4	7	22	9	9	278	41.7	15.30
18.6 - 24.5	4	1	16	8	11	14	2	3	7	5	4	3	9	4	4	5	103	15.4	21.07
24.6 - 32.5	1	1	4	3	9	13	7	5	1	4	1	2	0	1	2	3	57	8.5	27.90
32.6+	3	0	0	0	2	12	0	2	0	1	2	0	0	0	0	2	24	3.6	35.98
TOTAL	26	36	71	79	85	95	45	29	23	26	21	16	19	33	36	27	667	100.0	12.65
PERCENT	3.9	5.4	10.6	11.8	12.7	14.2	6.7	4.3	3.4	3.9	3.1	2.4	2.8	4.9	5.4	4.0	100.0		
AV SPD	14.9	13.6	16.9	14.9	16.3	19.2	14.1	16.2	14.9	16.7	19.0	14.7	17.0	15.7	12.7	19.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 15.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	2	.8	1.77
3.6 - 7.5	2	0	0	2	2	1	1	1	2	1	0	1	1	1	3	0	18	7.2	5.77
7.6 - 12.5	2	4	12	10	8	4	12	4	7	4	6	2	3	1	6	3	88	35.2	10.25
12.6 - 18.5	1	12	13	13	12	8	6	4	6	4	5	6	7	3	3	2	105	42.0	14.91
18.6 - 24.5	0	0	0	0	1	2	0	1	1	2	4	1	2	2	0	2	18	7.2	21.64
24.6 - 32.5	1	1	0	0	1	0	2	0	1	0	0	1	1	5	0	1	14	5.6	27.56
32.6+	0	0	0	0	0	5	0	0	0	0	0	0	0	0	0	0	5	2.0	36.76
TOTAL	7	17	25	25	24	20	21	10	18	11	15	11	14	12	12	8	250	100.0	11.71
PERCENT	2.8	5.8	10.0	10.0	9.6	8.0	8.4	4.0	7.2	4.4	6.0	4.4	5.6	4.8	4.8	3.2	100.0		
AV SPD	11.3	14.2	12.4	12.7	13.5	19.5	13.1	12.9	12.4	14.2	15.3	15.0	16.5	20.9	9.6	17.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 39 OF 48)

MONTH OF OCTOBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 634-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	1	2	0	0	0	2	0	0	0	0	0	1	0	0	0	6	2.3	1.84
3.6 - 7.5	1	3	4	6	4	0	3	2	1	2	1	1	0	0	0	1	29	10.9	5.07
7.6 - 12.5	9	9	4	22	6	4	4	1	3	3	9	6	3	0	1	9	93	35.1	9.68
12.6 - 18.5	5	3	8	14	7	3	3	1	2	8	5	1	3	9	8	5	91	34.3	15.05
18.6 - 24.5	1	2	2	1	1	0	0	0	0	1	3	0	3	4	1	3	22	8.3	20.93
24.6 - 32.5	2	3	0	0	0	0	0	1	0	1	0	0	1	4	2	2	16	6.0	28.02
32.6+	1	0	0	0	0	0	1	1	0	0	0	1	2	0	0	2	8	3.0	33.70
TOTAL	19	21	20	43	18	7	13	6	6	15	18	9	13	17	12	22	265	100.0	10.39
PERCENT	7.2	7.9	7.5	16.2	6.8	2.6	4.9	2.3	2.3	5.7	6.8	3.4	7.2	6.4	4.5	8.3	100.0		
AV SPD	15.2	13.6	11.5	11.4	11.3	12.1	11.2	17.2	12.1	14.8	12.6	13.0	18.5	20.1	16.9	16.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-15 (SHEET 40 OF 48)

MONTH OF NOVEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

REQUEST NUMBER 604-43

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	4	6	0	2	0	1	1	0	0	1	0	1	0	0	0	5	21	4.6	2.57
3.5 - 7.5	7	5	8	3	5	1	1	0	2	1	4	2	4	2	4	9	59	13.1	5.24
7.5 - 12.5	4	4	7	12	9	13	4	6	6	8	10	5	11	15	12	6	132	29.2	9.75
12.6 - 18.5	8	2	10	9	11	8	5	4	3	11	12	5	11	15	12	14	140	31.0	15.02
18.5 - 24.5	7	1	3	2	3	0	0	0	0	3	6	1	5	9	9	8	57	12.6	20.84
24.6 - 32.5	0	0	0	0	0	1	0	0	1	4	1	1	5	6	3	7	29	6.4	27.53
32.5+	0	0	0	0	0	0	0	0	2	0	0	0	1	5	4	2	14	3.1	40.93
TOTAL	30	18	28	28	29	24	11	10	14	28	33	15	37	52	44	51	452	100.0	9.74
PERCENT	6.6	4.0	6.2	6.2	6.4	5.3	2.4	2.2	3.1	6.2	7.3	3.3	8.2	11.5	9.7	11.3	100.0		
AV SPD	11.8	7.1	11.8	11.8	12.2	12.2	11.3	12.3	15.7	15.7	13.8	12.8	15.8	18.6	16.9	15.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 10

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

REQUEST NUMBER 604-43

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	1	0	1	0	1	0	2	0	0	0	0	2	0	0	3	2	12	9.2	2.95
3.5 - 7.5	1	3	3	1	2	3	1	0	0	0	2	0	1	4	5	0	26	19.8	5.41
7.6 - 12.5	2	0	3	4	8	3	1	3	0	2	2	0	0	2	1	0	31	23.7	9.96
12.6 - 18.5	1	0	4	4	3	1	3	2	1	1	0	4	2	4	1	1	32	24.4	15.15
18.6 - 24.5	0	0	1	1	2	2	1	0	0	1	1	0	0	4	1	0	14	10.7	21.39
24.6 - 32.5	2	0	2	2	0	0	0	0	1	1	0	0	0	0	2	1	11	8.4	27.93
32.5+	0	0	0	0	0	0	0	1	0	0	0	0	0	4	0	0	5	3.8	43.27
TOTAL	7	3	14	12	16	9	8	6	2	5	5	6	3	18	13	4	131	100.0	8.53
PERCENT	5.3	2.3	10.7	9.2	12.2	6.9	6.1	4.6	1.5	3.8	3.8	4.6	2.3	13.7	9.9	3.1	100.0		
AV SPD	14.1	5.7	13.1	15.6	11.4	11.5	11.2	16.7	24.1	17.7	10.8	10.8	12.2	21.0	10.5	13.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

TABLE 2.3-15 (SHEET 41 OF 48)

MONTH OF NOVEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	2	0	1	1	1	0	0	0	0	1	0	0	1	1	0	2	10	9.1	2.15
3.6 - 7.5	1	0	1	2	4	0	0	0	1	0	2	2	1	2	1	3	20	18.2	5.51
7.6 - 12.5	3	0	2	4	4	5	0	1	1	2	1	1	0	0	1	1	26	23.6	9.59
12.6 - 18.5	1	5	2	3	1	0	1	0	1	0	0	3	1	0	3	2	23	20.9	15.23
18.6 - 24.5	0	3	1	0	0	0	0	0	2	0	0	0	0	3	0	1	10	9.1	22.73
24.6 - 32.5	0	1	1	2	2	2	0	0	0	0	1	0	0	0	2	2	13	11.8	27.53
32.6+	0	0	0	0	1	3	0	0	0	0	0	0	0	1	2	1	8	7.3	37.23
TOTAL	7	9	8	12	13	10	1	1	5	3	4	6	3	7	9	12	110	100.0	8.00
PERCENT	6.4	8.2	7.3	10.9	11.8	9.1	.9	.9	4.5	2.7	3.6	5.5	2.7	6.4	8.2	10.9	100.0		
AV SPD	8.5	19.1	13.0	12.8	13.7	23.9	15.4	11.5	16.0	6.5	11.7	10.7	8.6	17.6	21.7	14.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	3	2	1	1	7	3	0	0	2	0	1	1	2	3	2	28	4.0	2.28
3.6 - 7.5	10	10	14	15	14	7	12	6	1	6	4	6	7	13	12	17	154	22.0	5.17
7.6 - 12.5	8	10	6	17	12	14	9	8	4	5	14	6	6	7	7	17	150	21.4	9.80
12.6 - 18.5	11	9	4	3	15	15	11	3	1	4	6	19	11	13	13	15	153	21.8	15.22
18.6 - 24.5	2	3	0	3	12	5	3	2	1	5	10	5	8	5	17	15	96	13.7	21.15
24.6 - 32.5	4	0	0	2	7	4	1	1	1	2	1	4	3	7	15	7	59	8.4	28.14
32.6+	0	0	0	0	1	3	0	3	5	12	3	7	3	7	12	5	61	8.7	40.63
TOTAL	35	35	26	41	62	55	39	23	13	36	38	48	39	54	79	78	701	100.0	9.23
PERCENT	5.0	5.0	3.7	5.8	8.8	7.8	5.6	3.3	1.9	5.1	5.4	6.8	5.6	7.7	11.3	11.1	100.0		
AV SPD	12.7	10.1	7.7	10.0	14.3	14.2	13.8	14.3	25.8	22.6	16.0	18.0	17.3	17.7	21.1	15.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 15.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 13

TABLE 2.3-15 (SHEET 42 OF 48)

MONTH OF NOVEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
REQUEST NUMBER 604-43
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	1	.2	.30
CALM+ - 3.5	1	0	0	0	2	0	1	1	4	1	0	0	0	0	0	1	11	2.0	2.12
3.6 - 7.5	5	5	4	7	6	6	0	1	1	5	5	3	6	6	3	5	68	12.7	5.53
7.6 - 12.5	6	7	15	13	11	14	6	5	4	5	8	5	12	7	16	6	140	26.1	9.93
12.6 - 18.5	6	0	4	17	10	13	12	8	13	16	14	5	27	14	4	13	176	32.8	14.93
18.6 - 24.5	0	0	2	3	6	9	2	5	4	2	8	4	15	8	2	6	76	14.2	21.11
24.6 - 32.5	0	0	0	4	2	8	5	0	2	5	1	0	1	0	6	3	37	6.9	27.64
32.6+	0	0	0	0	6	0	1	0	3	6	0	8	1	1	1	1	28	5.2	38.59
TOTAL	18	12	25	44	43	49	27	20	32	40	36	25	63	36	32	35	537	100.0	10.24
PERCENT	3.4	2.2	4.7	8.2	8.0	9.1	5.0	3.7	6.0	7.4	6.7	4.7	11.7	6.7	6.0	6.5	100.0		
AV SPD	9.7	9.5	11.1	13.5	16.5	15.4	17.2	14.2	15.9	19.0	14.6	22.6	15.4	14.8	14.7	15.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 15.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
REQUEST NUMBER 604-43
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	1	1	1	0	0	0	0	0	0	0	1	1	0	0	6	2.0	.30
CALM+ - 3.5	0	0	1	0	1	0	0	1	0	0	0	0	0	0	0	0	3	1.0	3.03
3.6 - 7.5	3	2	0	4	1	2	3	3	1	1	1	0	2	1	1	2	27	9.2	5.74
7.6 - 12.5	4	1	5	7	15	13	12	4	3	3	6	5	4	9	11	2	104	35.4	9.99
12.6 - 18.5	2	6	4	7	27	4	6	3	3	11	3	10	6	3	7	6	108	36.7	15.06
18.6 - 24.5	1	0	0	0	5	2	1	0	2	3	4	3	1	0	1	2	25	8.5	21.53
24.6 - 32.5	0	0	0	2	3	1	0	0	1	2	0	3	0	0	2	1	15	5.1	25.92
32.6+	0	0	0	0	1	0	0	0	2	0	1	1	0	0	1	0	6	2.0	38.34
TOTAL	10	10	11	21	54	22	22	11	12	20	15	22	14	14	23	13	294	100.0	6.51
PERCENT	3.4	3.4	3.7	7.1	18.4	7.5	7.5	3.7	4.1	6.8	5.1	7.5	4.8	4.8	7.8	4.4	100.0		
AV SPD	11.5	10.9	10.8	12.6	15.0	12.1	11.5	8.6	18.7	16.3	15.2	17.9	12.2	9.8	15.3	14.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 13.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 43 OF 48)

MONTH OF NOVEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND #35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	2	2	0	1	0	0	0	0	1	0	2	0	8	1.7	.30
CALM+ - 3.5	1	1	0	1	2	1	0	0	2	1	1	0	0	3	0	0	13	2.7	2.43
3.6 - 7.5	7	6	10	6	8	5	5	4	3	2	8	6	10	3	4	3	90	19.0	5.67
7.6 - 12.5	14	12	8	17	13	13	16	9	10	7	9	10	11	18	9	8	186	39.3	9.53
12.6 - 18.5	2	2	7	2	11	3	3	9	5	6	10	26	27	18	10	4	145	30.7	14.49
18.6 - 24.5	0	0	2	1	1	1	0	0	1	4	0	10	3	0	0	1	24	5.1	20.79
24.6 - 32.5	0	0	0	0	0	0	0	0	1	0	0	2	0	0	0	1	4	.8	27.94
32.6+	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	1	3	.6	35.98
TOTAL	24	21	27	27	37	25	26	23	24	20	28	54	52	42	25	18	473	100.0	6.01
PERCENT	5.1	4.4	5.7	5.7	7.8	5.3	5.5	4.9	5.1	4.2	5.9	11.4	11.0	8.9	5.3	3.8	100.0		
AV SPD	8.5	9.8	10.9	9.3	9.5	8.5	9.8	10.5	13.1	12.9	10.5	14.6	11.3	11.0	10.6	12.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 11.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 44 OF 48)

MONTH OF DECEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	0	0	0	0	0	1	0	0	0	1	1	1	0	0	4	3.9	2.51
3.6 - 7.5	0	1	0	1	0	0	0	3	2	0	0	1	1	1	1	1	12	11.7	5.63
7.6 - 12.5	8	2	2	3	2	1	1	0	0	3	2	3	1	1	1	3	33	32.0	10.30
12.6 - 18.5	4	3	2	0	3	1	0	2	0	0	2	2	3	3	3	2	30	29.1	15.55
18.6 - 24.5	6	0	0	0	1	0	0	0	0	6	4	0	0	0	6	0	17	16.5	21.37
24.6 - 32.5	0	0	0	0	2	0	0	0	0	1	2	0	0	0	2	0	7	6.8	27.09
32.5+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	12	6	4	4	8	2	1	6	2	10	10	7	6	6	13	6	103	0.0	10.38
PERCENT	11.7	5.8	3.9	3.9	7.8	1.9	1.0	5.8	1.9	9.7	9.7	6.8	5.8	5.8	12.6	5.8	100.0		
AV SPD	12.1	12.7	12.6	8.5	18.9	14.1	11.3	8.2	6.1	18.7	19.2	10.2	11.3	11.0	18.7	11.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.1	.33
CALM+ - 3.5	0	1	1	2	1	0	0	0	2	0	0	1	1	2	1	1	13	1.9	2.32
3.6 - 7.5	3	6	6	7	8	5	2	6	3	5	5	11	2	2	4	4	79	11.3	5.54
7.6 - 12.5	3	12	6	14	18	8	4	7	3	5	22	11	14	13	20	9	169	24.1	10.01
12.6 - 18.5	7	3	9	19	14	12	8	17	17	19	45	17	9	16	22	11	245	34.9	15.39
18.6 - 24.5	0	4	13	12	8	3	3	1	8	16	18	11	4	7	17	1	126	17.9	21.16
24.6 - 32.5	1	1	1	4	4	0	3	1	4	6	6	4	8	3	4	2	52	7.4	27.33
32.6+	0	1	0	0	1	1	1	0	0	1	1	1	2	0	6	0	15	2.1	37.06
TOTAL	15	28	36	58	54	29	21	32	37	52	97	56	40	43	74	28	703	100.0	10.84
PERCENT	2.1	4.0	5.1	8.3	7.7	4.1	3.0	4.6	5.3	7.4	13.8	8.0	5.7	6.1	10.5	4.0	100.0		
AV SPD	12.2	12.1	15.4	14.5	14.1	12.7	17.1	12.5	16.3	17.6	15.9	14.9	16.8	14.8	17.6	12.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 15.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 9

TABLE 2.3-15 (SHEET 45 OF 48)

MONTH OF DECEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.2	.30
CALM+ - 3.5	2	1	5	5	3	2	4	0	0	0	1	0	3	2	1	0	29	5.1	2.30
3.6 - 7.5	7	9	6	2	11	9	6	1	5	7	12	9	6	9	6	6	111	19.4	5.26
7.5 - 12.5	6	4	8	6	13	14	13	0	4	6	6	16	20	35	16	8	175	30.6	10.04
12.6 - 18.5	4	4	6	4	1	1	5	2	7	8	13	5	10	29	21	23	143	25.0	14.79
13.6 - 24.5	2	0	0	0	1	1	1	0	1	7	4	5	6	13	15	21	78	13.7	20.33
24.6 - 32.5	0	0	0	0	0	0	0	0	0	1	5	1	2	5	9	10	33	5.3	27.41
32.6+	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	.2	33.59
TOTAL	21	18	26	17	29	27	29	3	17	29	42	36	47	93	69	68	571	100.0	8.27
PERCENT	3.7	3.2	4.6	3.0	5.1	4.7	5.1	.5	3.0	5.1	7.4	6.3	8.2	16.3	12.1	11.9	100.0		
AV SPD	9.7	7.8	8.0	8.3	8.1	8.8	9.5	11.4	11.6	13.6	13.8	11.5	12.7	13.7	15.8	17.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 11

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	1	0	0	1	0	0	0	1	0	1	1	1	1	2	2	11	7.3	2.92
3.6 - 7.5	2	1	1	3	2	1	0	4	1	3	1	3	1	2	0	2	27	18.0	4.91
7.6 - 12.5	4	3	0	1	6	12	3	3	2	4	5	4	4	3	2	1	57	38.0	9.78
12.6 - 18.5	1	1	2	1	2	1	0	0	0	2	2	1	2	3	4	1	23	15.3	15.19
18.6 - 24.5	0	0	0	1	0	1	1	0	0	5	1	1	2	0	2	0	14	9.3	21.09
24.6 - 32.5	0	0	0	0	3	2	0	1	0	0	1	1	1	4	0	1	14	9.3	26.73
32.6+	0	0	0	0	0	1	0	0	0	0	0	0	0	3	0	0	4	2.7	34.42
TOTAL	7	6	3	6	14	18	4	8	4	14	11	11	11	16	10	7	150	100.0	8.37
PERCENT	4.7	4.0	2.0	4.0	9.3	12.0	2.7	5.3	2.7	9.3	7.3	7.3	7.3	10.7	6.7	4.7	100.0		
AV SPD	9.3	8.1	12.8	9.7	13.7	13.8	12.9	9.4	6.6	13.7	12.1	10.9	13.8	13.0	13.9	8.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

TABLE 2.3-15 (SHEET 46 OF 48)

MONTH OF DECEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .6
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1	.2	.30
CALM+ - 3.5	1	1	0	0	0	2	2	0	1	0	1	2	0	0	0	0	10	2.2	2.75
3.6 - 7.5	0	0	0	2	2	2	5	2	2	0	4	2	5	0	2	2	30	6.6	5.32
7.6 - 12.5	3	3	3	2	2	6	6	2	10	8	3	5	4	4	4	3	68	15.0	10.39
12.6 - 18.5	6	7	9	0	9	9	3	16	18	27	16	14	13	15	15	5	181	40.0	15.34
18.6 - 24.5	0	0	3	2	1	3	3	6	14	20	16	5	16	17	10	3	119	26.3	21.21
24.6 - 32.5	0	0	1	5	0	0	1	0	0	6	6	2	7	3	1	0	32	7.1	26.68
32.6+	0	0	0	0	0	0	0	1	2	0	2	1	0	2	3	0	11	2.4	38.06
TOTAL	10	11	15	11	14	22	21	27	47	61	48	31	45	41	35	13	452	100.0	11.85
PERCENT	2.2	2.4	3.3	2.4	3.1	4.9	4.6	5.0	10.4	13.5	10.6	6.9	10.0	9.1	7.7	2.9	100.0		
AV SPD	11.8	12.3	16.9	18.5	13.2	12.7	10.8	16.0	16.8	19.1	18.2	15.8	17.6	19.4	18.2	14.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 16.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .6 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFFRENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 3.5	0	0	0	0	0	0	1	1	0	0	0	1	0	0	0	1	4	1.9	2.19
3.6 - 7.5	1	0	1	0	0	0	2	0	4	1	5	2	2	0	2	1	21	10.0	5.42
7.6 - 12.5	3	2	0	4	2	0	3	4	5	3	7	2	4	2	2	2	45	21.4	9.63
12.6 - 18.5	4	2	3	2	7	7	6	5	6	9	11	3	11	8	5	2	91	43.3	15.00
18.6 - 24.5	0	0	3	2	2	1	2	0	5	9	3	2	5	4	4	0	42	20.0	21.02
24.6 - 32.5	0	0	0	0	0	0	1	0	1	2	0	0	0	2	1	0	7	3.3	25.22
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	8	4	7	8	11	8	15	10	21	24	26	10	22	16	14	6	210	0.0	11.22
PERCENT	3.8	1.9	3.3	3.8	5.2	3.8	7.1	4.8	10.0	11.4	12.4	4.8	10.5	7.6	6.7	2.9	100.0		
AV SPD	11.4	10.8	15.8	13.7	15.0	15.3	13.0	11.7	14.0	17.5	12.7	13.0	14.7	17.4	16.0	10.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 14.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 47 OF 48)

MONTH OF DECEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION

REQUEST NUMBER 604-43

FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2

SITE HATCH

PERIOD OF RECORD FROM 70060101 TO 74083124

SPEED AND DIRECTION FROM 150 LEVEL

TEMPERATURE DIFFERENCE BETWEEN 150 AND 35

SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	.2	.33
CALM+ - 3.5	1	0	0	0	1	0	0	0	0	0	1	1	0	2	2	0	8	2.0	2.13
3.6 - 7.5	5	2	1	1	4	1	1	2	1	5	10	13	9	6	2	4	67	16.5	5.69
7.6 - 12.5	7	3	5	5	7	10	9	1	4	14	15	15	20	10	10	10	145	35.6	9.33
12.6 - 18.5	3	2	2	3	5	7	8	4	12	15	23	19	12	20	7	4	146	35.9	14.89
18.6 - 24.5	1	2	0	3	0	1	1	0	2	4	2	3	2	3	3	1	28	6.9	20.63
24.6 - 32.5	0	0	0	0	0	0	0	0	1	6	0	0	0	1	4	0	12	2.9	27.96
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	17	9	8	12	17	19	19	7	20	44	51	51	43	42	28	20	407	0.0	9.00
PERCENT	4.2	2.2	2.0	2.9	4.2	4.7	4.7	1.7	4.9	10.8	12.5	12.5	10.6	10.3	6.9	4.9	100.0		
AV SPD	9.6	12.5	10.4	13.7	10.1	12.1	12.4	12.3	14.9	14.8	11.9	11.4	11.0	12.8	13.8	10.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.3

HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-15 (SHEET 48 OF 48)

MONTH OF DECEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-43
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 150 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 150FT ADJUSTED TO 393 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	.2	.33
CALM+ - 3.5	1	0	0	0	1	0	0	0	0	0	1	1	0	2	2	0	8	2.0	2.13
3.6 - 7.5	5	2	1	1	4	1	1	2	1	5	10	13	9	6	2	4	67	16.5	5.69
7.6 - 12.5	7	3	5	5	7	10	9	1	4	14	15	15	20	10	10	10	145	35.6	9.33
12.6 - 18.5	3	2	2	3	5	7	8	4	12	15	23	19	12	20	7	4	146	35.9	14.89
18.6 - 24.5	1	2	0	3	0	1	1	0	2	4	2	3	2	3	3	1	28	6.9	20.69
24.6 - 32.5	0	0	0	0	0	0	0	0	1	6	0	0	0	1	4	0	12	2.9	27.96
32.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	17	9	8	12	17	19	19	7	20	44	51	51	43	42	28	20	407	0.0	9.00
PERCENT	4.2	2.2	2.0	2.9	4.2	4.7	4.7	1.7	4.9	10.8	12.5	12.5	10.5	10.3	6.9	4.9	100.0		
AV SPD	9.6	12.5	10.4	13.7	10.1	12.1	12.4	12.3	14.9	14.8	11.9	11.4	11.0	12.8	13.8	10.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 12.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-16 (SHEET 1 OF 48)

MONTH OF JANUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	[illegible]
CALM+ - 1.5	0	0	0	1	1	2	0	0	0	0	0	0	0	0	0	1	5	.9	1.14
1.6 - 2.5	1	1	3	2	3	3	1	0	0	1	0	2	0	1	0	1	19	3.4	[illegible]
2.6 - 3.5	5	3	7	2	4	3	0	1	0	0	0	[illegible]	2	2	2	-	27	4.8	[illegible]
3.6 - 7.5	39	26	38	18	21	8	10	2	6	19	18	16	26	23	25	19	317	56.0	[illegible]
7.6 - 12.5	18	12	15	22	8	3	1	1	4	8	9	10	10	23	8	8	160	28.3	6.19
12.6 - 18.5	0	1	0	0	0	0	0	0	0	0	1	0	8	13	10	7	36	6.4	14.51
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	[illegible]	2	.4	18.77
TOTAL	63	42	59	45	37	19	12	4	10	27	28	28	46	67	47	71	566	100.0	[illegible]
PERCENT	11.1	7.6	10.4	8.0	6.5	3.4	2.1	.7	1.8	4.8	4.9	4.9	8.1	11.8	8.3	5.5	100.0		
AV SPD	6.3	6.5	6.2	7.2	5.3	4.4	5.9	6.1	6.9	6.7	7.3	7.0	7.9	8.5	8.9	[illegible]			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	2	0	0	0	0	0	1	1	0	0	0	0	0	1	0	0	5	4.1	.95
1.6 - 2.5	0	1	1	3	1	0	1	0	0	2	0	1	0	0	1	[illegible]	11	8.9	[illegible]
2.6 - 3.5	3	5	1	2	2	0	0	1	0	0	0	0	1	2	0	0	17	13.8	[illegible]
3.6 - 7.5	3	5	3	3	6	6	1	1	0	3	4	4	9	5	7	4	64	52.0	5.41
7.6 - 12.5	1	0	2	1	0	1	0	0	1	4	4	4	1	2	1	1	23	18.7	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	1	0	1	0	1	[illegible]	3	2.4	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	9	11	7	9	9	7	3	3	1	9	9	9	12	10	10	5	123	0.0	3.94
PERCENT	7.3	8.9	5.7	7.3	7.3	5.7	2.4	2.4	.8	7.3	7.3	7.3	9.8	8.1	8.1	4.1	100.0		
AV SPD	4.1	3.7	5.5	4.0	4.6	5.6	3.3	3.7	9.6	6.4	8.7	7.0	6.8	4.9	6.2	5.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-16 (SHEET 2 OF 48)

MONTH OF JANUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	1	0	0	0	1	0	0	0	0	0	0	2	2.2	1.47
1.6 - 2.5	2	1	1	2	1	0	1	0	0	3	0	0	0	0	0	1	12	13.5	1.95
2.6 - 3.5	1	0	2	1	1	0	0	0	0	0	1	0	0	0	0	1	7	7.9	2.53
3.6 - 7.5	5	5	3	1	5	4	2	1	1	1	2	2	0	5	2	4	43	48.3	5.07
7.6 - 12.5	0	1	1	1	1	2	0	0	0	4	5	0	2	0	2	1	20	22.5	9.13
12.6 - 18.5	0	0	0	0	0	0	0	0	0	1	1	0	0	1	1	0	4	4.5	14.06
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	1.1	18.91
TOTAL	8	7	7	5	8	7	3	1	1	10	9	2	2	6	6	7	89	100.0	4.20
PERCENT	9.0	7.9	7.9	5.6	9.0	7.9	3.4	1.1	1.1	11.2	10.1	2.2	2.2	6.7	6.7	7.9	100.0		
AV SPD	4.4	4.8	5.2	4.0	5.2	5.6	3.8	5.9	5.5	6.4	8.6	5.1	9.4	6.7	10.5	4.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	2	3	2	0	0	1	0	1	1	0	0	1	1	2	1	16	2.0	.35
CALM+ - 1.5	4	1	3	4	3	0	3	1	1	1	1	2	1	1	2	1	29	3.7	1.14
1.6 - 2.5	6	6	7	13	12	2	2	7	2	2	3	6	6	3	5	5	87	11.0	1.94
2.6 - 3.5	8	8	9	4	10	2	7	7	6	4	4	4	5	4	5	4	91	11.5	2.93
3.6 - 7.5	26	10	15	29	35	17	20	14	24	40	40	23	21	27	30	35	406	51.5	5.07
7.6 - 12.5	14	2	5	8	5	2	2	0	7	14	12	6	3	8	14	19	121	15.3	9.23
12.6 - 18.5	1	0	0	0	1	0	0	0	1	8	8	0	0	3	11	5	38	4.8	14.32
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	.1	19.69
TOTAL	60	29	42	60	66	23	35	29	42	70	68	41	37	48	69	70	789	100.0	3.14
PERCENT	7.6	3.7	5.3	7.6	8.4	2.9	4.4	3.7	5.3	8.9	8.6	5.2	4.7	6.1	8.7	8.9	100.0		
AV SPD	5.2	3.8	4.0	4.5	4.4	5.0	4.1	3.7	5.2	6.8	6.9	5.2	4.6	6.1	7.1	6.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 10

TABLE 2.3-16 (SHEET 3 OF 48)

MONTH OF JANUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	2	1	2	0	2	0	0	0	0	3	0	1	2	1	1	15	2.1	.75
CALM+ - 1.5	1	2	4	3	1	3	1	2	0	1	3	5	0	3	1	6	36	5.1	.99
1.6 - 2.5	5	2	2	4	9	6	2	4	6	4	5	2	5	4	5	0	65	9.2	1.93
2.6 - 3.5	3	0	2	8	6	11	10	6	6	8	11	6	4	7	7	2	97	13.8	2.92
3.6 - 7.5	5	9	3	14	8	25	8	17	30	67	57	22	29	16	21	18	349	49.6	4.94
7.6 - 12.5	1	0	0	0	0	1	0	1	7	36	35	12	3	11	18	12	137	19.5	9.19
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	1	0	0	0	3	0	4	.6	13.64
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	15	15	12	31	24	48	21	30	49	116	115	47	42	43	56	29	703	0.0	2.95
PERCENT	2.1	2.1	1.7	4.4	3.4	6.8	3.0	4.3	7.0	16.5	16.4	6.7	6.0	6.1	8.0	5.5	100.0		
AV SPD	3.3	3.1	2.3	3.1	3.1	3.9	3.4	4.0	5.2	6.2	6.1	5.5	4.5	5.2	6.4	5.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	3	0	0	1	0	0	1	1	0	0	0	0	3	1	0	0	10	4.5	.30
CALM+ - 1.5	0	1	1	2	0	1	1	3	0	0	1	0	2	2	1	1	16	7.1	1.02
1.6 - 2.5	1	2	3	2	4	0	4	4	4	3	4	1	3	2	3	1	41	18.3	2.04
2.6 - 3.5	1	3	0	2	9	7	8	4	3	5	5	3	7	2	6	3	68	30.4	2.97
3.6 - 7.5	0	0	4	4	0	3	3	8	11	20	13	6	7	6	0	0	85	37.9	4.49
7.6 - 12.5	0	0	0	0	0	0	0	0	0	2	0	2	0	0	0	0	4	1.8	8.43
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	5	6	8	11	13	11	17	20	18	30	23	12	22	13	10	5	224	0.0	2.31
PERCENT	2.2	2.7	3.6	4.9	5.8	4.9	7.6	8.9	8.0	13.4	10.3	5.4	9.8	5.8	4.5	2.2	100.0		
AV SPD	1.2	2.1	3.2	3.0	2.6	3.1	2.7	2.9	3.7	4.7	3.7	4.7	2.9	2.8	2.6	2.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 6

TABLE 2.3-16 (SHEET 4 OF 48)

MONTH OF JANUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70063101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 6C4-4C

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	1	0	0	0	0	0	0	0	0	2	0	0	0	1	4	2.3	.10
CALM+ - 1.5	0	2	1	2	1	0	1	2	1	1	1	2	0	0	1	3	18	10.4	.59
1.6 - 2.5	2	1	4	1	1	5	3	1	4	2	0	4	3	2	5	5	43	24.9	1.95
2.6 - 3.5	3	2	4	1	1	2	2	2	5	3	1	3	8	7	3	2	49	28.3	[illegible]
3.6 - 7.5	1	0	1	3	3	1	1	2	5	7	15	6	2	3	4	3	57	32.9	[illegible]
7.6 - 12.5	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	2	1.2	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	6	5	11	7	6	8	7	7	15	15	17	17	13	12	13	14	173	0.0	2.10
PERCENT	3.5	2.9	6.4	4.0	3.5	4.6	4.0	4.0	8.7	8.7	9.8	9.8	7.5	6.9	7.5	8.1	100.0		
AV SPD	2.9	2.1	2.4	2.9	3.3	2.4	2.2	3.0	3.1	4.4	4.6	3.0	3.0	3.2	2.8	2.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-16 (SHEET 5 OF 48)

MONTH OF FEBRUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	3	.4	[illegible]
CALM+ - 1.5	2	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	3	.4	1.33
1.6 - 2.5	0	6	1	5	0	0	1	0	1	0	1	0	0	1	2	2	20	2.9	2.15
2.6 - 3.5	5	2	5	2	2	2	1	0	1	0	0	0	3	4	1	4	32	4.6	2.93
3.6 - 7.5	20	14	31	44	33	13	4	2	2	8	14	13	16	31	22	15	282	40.8	5.42
7.6 - 12.5	3	7	7	9	5	2	13	2	3	10	20	40	21	56	58	22	288	41.7	9.59
12.6 - 18.5	0	0	5	2	0	0	0	1	1	4	4	14	6	14	5	2	57	8.2	14.13
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	1	2	3	0	6	.9	22.28
TOTAL	30	29	49	60	40	17	19	5	8	22	39	67	47	109	92	58	691	100.0	5.92
PERCENT	4.3	4.2	7.1	8.7	5.8	2.5	2.7	.7	1.2	3.2	5.6	9.7	6.8	15.8	13.3	8.4	100.0		
AV SPD	5.7	5.6	6.7	5.8	6.0	5.9	8.3	9.1	7.0	9.7	8.5	9.8	8.7	9.3	9.5	7.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 8.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 20

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
1.6 - 2.5	1	0	0	3	1	0	0	0	0	0	0	0	0	0	1	1	7	5.6	2.17
2.6 - 3.5	0	1	1	1	0	0	0	0	1	1	1	2	0	0	1	0	9	7.2	2.97
3.6 - 7.5	1	2	1	6	6	1	0	2	2	6	2	4	1	2	3	2	40	32.0	5.62
7.6 - 12.5	0	0	1	4	0	0	4	4	5	4	3	6	7	3	8	6	55	44.0	9.27
12.6 - 18.5	0	0	0	0	0	0	1	1	2	0	3	1	0	1	3	0	12	9.6	13.77
18.6+	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	0	2	1.6	19.74
TOTAL	2	3	3	14	7	1	5	7	10	10	9	14	8	6	17	9	125	100.0	6.74
PERCENT	1.6	2.4	2.4	11.2	5.6	.8	4.0	5.6	8.0	8.0	7.2	11.2	6.4	4.8	13.6	7.2	100.0		
AV SPD	3.3	4.6	5.3	5.8	5.3	6.8	10.2	9.7	9.3	7.1	9.5	9.0	9.5	8.9	9.8	7.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 8.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 8

TABLE 2.3-16 (SHEET 6 OF 48)

MONTH OF FEBRUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
1.6 - 2.5	0	0	0	1	0	0	0	0	0	0	0	0	1	0	0	0	2	2.6	2.02
2.6 - 3.5	0	1	1	1	0	0	0	0	0	0	0	0	0	0	0	0	3	3.9	2.97
3.6 - 7.5	0	0	2	4	1	0	0	1	2	2	4	1	2	1	3	1	24	31.6	5.73
7.6 - 12.5	0	0	1	0	3	1	3	2	4	1	2	2	2	6	5	4	36	47.4	9.64
12.6 - 18.5	0	0	0	0	0	0	0	0	2	1	1	0	2	0	0	1	7	9.2	13.99
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	4	0	0	4	5.3	22.15
TOTAL	0	1	4	6	4	1	3	3	8	4	7	3	7	11	8	6	76	100.0	7.13
PERCENT	0.0	1.3	5.3	7.9	5.3	1.3	3.9	3.9	10.5	5.3	9.2	3.9	9.2	14.5	10.5	7.9	100.0		
AV SPD	0.0	3.0	6.2	4.8	9.9	9.0	9.5	9.0	9.9	9.4	8.3	7.3	8.7	13.9	8.3	9.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.7
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	2	.3	.30
CALM+ - 1.5	0	1	2	0	0	1	0	0	1	3	0	0	0	1	0	1	10	1.5	1.12
1.6 - 2.5	2	2	6	3	3	4	3	0	0	0	1	2	1	2	2	0	31	4.6	2.04
2.6 - 3.5	2	3	5	3	7	0	5	1	2	2	3	2	0	0	3	4	42	6.2	2.95
3.6 - 7.5	9	12	29	48	27	22	13	13	20	29	40	14	15	18	19	12	340	50.0	5.30
7.6 - 12.5	0	9	8	16	17	7	1	4	13	16	28	10	13	22	24	5	193	28.4	9.32
12.6 - 18.5	0	0	0	0	2	0	1	1	6	2	1	5	4	18	3	0	42	6.2	14.54
18.6+	0	0	0	0	0	0	0	0	0	1	0	8	7	3	1	0	20	2.9	15.91
TOTAL	13	27	50	70	56	34	22	19	42	54	73	41	40	64	52	23	680	100.0	5.02
PERCENT	1.9	4.0	7.4	10.3	8.2	5.0	3.2	2.8	6.2	7.9	10.7	6.0	5.9	9.4	7.6	3.4	100.0		
AV SPD	4.5	5.9	5.4	6.2	6.1	5.5	4.4	6.6	7.6	7.1	6.8	10.1	10.1	10.4	8.4	5.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 13

TABLE 2.3-16 (SHEET 7 OF 48)

MONTH OF FEBRUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	1	0	0	0	0	0	0	0	0	0	0	1	0	1	0	4	.8	[illegible]
CALM+ - 1.5	3	1	1	1	1	1	1	1	0	2	0	2	2	0	2	0	18	3.4	[illegible]
1.6 - 2.5	1	1	2	4	3	3	2	0	2	3	3	0	1	2	1	1	29	5.5	2.05
2.6 - 3.5	2	4	8	3	9	7	4	7	4	1	5	6	4	3	1	4	67	12.8	[illegible]
3.6 - 7.5	11	10	18	14	12	18	25	21	18	29	37	25	28	27	23	8	324	61.7	4.96
7.6 - 12.5	0	0	0	0	0	1	3	4	15	6	9	6	5	12	13	0	74	14.1	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	1	1	0	1	0	3	3	0	9	1.7	17.64
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	18	17	29	22	25	25	35	33	40	42	54	40	41	47	44	13	525	0.0	[illegible]
PERCENT	3.4	3.2	5.5	4.2	4.8	4.8	6.7	6.3	7.6	8.0	10.3	7.6	7.8	9.0	8.4	2.5	100.0		
AV SPD	3.7	3.9	4.0	3.9	3.6	4.2	4.8	5.4	6.8	5.5	5.5	5.4	5.0	6.7	6.6	4.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	1	3	1.8	[illegible]
CALM+ - 1.5	0	1	0	0	1	0	2	1	2	1	0	0	4	0	0	0	12	7.0	[illegible]
1.6 - 2.5	1	1	1	1	2	1	2	0	2	1	1	0	2	0	3	3	21	12.3	1.98
2.6 - 3.5	1	2	0	2	5	15	1	4	4	3	1	5	4	6	0	0	53	31.0	[illegible]
3.6 - 7.5	2	1	0	0	1	5	9	3	7	9	13	12	9	5	4	1	81	47.4	[illegible]
7.6 - 12.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	.6	7.76
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	4	5	1	3	9	21	14	8	15	15	15	17	20	11	8	5	171	0.0	[illegible]
PERCENT	2.3	2.9	.6	1.8	5.3	12.3	8.2	4.7	8.8	8.8	8.8	9.9	11.7	6.4	4.7	2.9	100.0		
AV SPD	3.4	2.9	1.8	2.5	2.7	3.4	3.7	3.4	3.2	4.5	4.6	4.5	3.0	3.6	2.8	2.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-16 (SHEET 8 OF 48)

MONTH OF FEBRUARY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	2	0	1	1	0	1	0	0	1	1	7	3.3	[illegible]
CALM+ - 1.5	4	1	0	1	0	0	0	1	0	1	1	1	4	1	2	0	17	7.9	.95
1.6 - 2.5	3	5	4	1	2	1	2	1	1	2	4	2	5	6	7	1	47	22.0	1.93
2.6 - 3.5	4	0	1	3	6	1	4	0	1	1	4	5	12	12	8	2	64	29.9	2.55
3.6 - 7.5	1	0	0	1	2	4	2	3	2	8	23	12	11	7	3	0	79	36.9	4.58
7.6 - 12.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	12	6	5	6	10	6	10	5	5	13	32	21	32	26	21	4	214	0.0	2.05
PERCENT	5.6	2.8	2.3	2.8	4.7	2.8	4.7	2.3	2.3	6.1	15.0	9.8	15.0	12.1	9.8	1.9	100.0		
AV SPD	2.1	1.9	2.3	2.8	3.0	3.7	2.6	3.6	3.0	3.6	4.3	3.6	3.2	3.1	2.4	2.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-16 (SHEET 9 OF 48)

MONTH OF MARCH

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	1	0	0	0	0	0	0	0	1	0	0	2	.2	1.11
1.6 - 2.5	4	0	2	0	1	1	0	1	1	1	0	1	2	1	0	3	18	2.1	2.10
2.6 - 3.5	3	1	1	9	5	2	3	1	0	0	3	3	1	3	5	1	41	4.8	2.97
3.6 - 7.5	14	8	17	31	52	24	23	15	7	16	22	20	25	26	32	18	350	40.6	5.47
7.6 - 12.5	4	3	12	8	15	21	17	9	13	37	23	25	48	27	28	12	302	35.0	5.45
12.6 - 18.5	0	0	0	0	0	0	0	1	5	17	3	3	32	54	11	3	129	14.9	14.53
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	10	11	0	0	21	2.4	15.77
TOTAL	25	12	32	48	73	49	43	27	26	71	51	52	118	123	76	37	863	100.0	6.64
PERCENT	2.9	1.4	3.7	5.6	8.5	5.7	5.0	3.1	3.0	8.2	5.9	6.0	13.7	14.3	8.8	4.3	100.0		
AV SPD	4.9	6.9	6.7	5.4	6.1	7.1	7.3	7.1	9.1	10.1	7.8	8.0	11.0	12.0	8.2	7.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 8.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 30

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	1	1	0	2	0	0	0	0	0	0	0	0	0	0	0	0	4	2.7	1.35
1.6 - 2.5	1	0	0	2	0	0	0	0	1	0	0	0	1	0	1	0	6	4.0	1.97
2.6 - 3.5	2	0	0	1	2	0	0	1	0	0	0	1	1	1	1	1	11	7.4	2.83
3.6 - 7.5	4	3	8	16	9	5	4	3	1	2	3	7	5	2	1	1	74	49.7	5.26
7.6 - 12.5	0	0	5	1	1	2	2	1	2	4	2	4	5	5	4	7	45	30.2	9.52
12.6 - 18.5	0	0	0	0	0	0	0	1	3	0	0	1	0	2	1	1	9	6.0	14.55
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	8	4	13	22	12	7	6	6	7	6	5	13	12	10	8	10	149	0.0	4.94
PERCENT	5.4	2.7	8.7	14.8	8.1	4.7	4.0	4.0	4.7	4.0	3.4	8.7	8.1	6.7	5.4	6.7	100.0		
AV SPD	3.8	3.9	7.4	4.6	4.9	7.2	6.6	6.9	9.9	9.0	7.8	7.3	6.5	10.0	8.4	9.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 8

TABLE 2.3-16 (SHEET 10 OF 48)

MONTH OF MARCH

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74033124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	.00
CALM+ - 1.5	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	2	1.6	1.42
1.6 - 2.5	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	1.6	2.20
2.6 - 3.5	0	1	0	1	1	0	1	1	0	1	0	2	0	0	0	0	8	6.2	3.17
3.6 - 7.5	2	2	2	2	5	5	4	4	1	8	3	8	6	3	0	3	58	45.0	5.44
7.6 - 12.5	2	0	1	1	3	3	0	0	4	4	7	4	8	1	3	3	44	34.1	9.12
12.6 - 18.5	0	0	1	0	0	0	0	0	1	3	2	0	3	0	1	2	13	10.1	14.43
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	2	1.6	19.36
TOTAL	6	4	4	4	9	8	6	5	6	16	12	14	18	5	4	8	129	100.0	6.09
PERCENT	4.7	3.1	3.1	3.1	7.0	6.2	4.7	3.9	4.7	12.4	9.3	10.9	14.0	3.9	3.1	6.2	100.0		
AV SPD	6.3	4.6	8.3	5.1	6.5	6.4	4.8	4.7	9.5	8.5	8.9	6.6	9.8	9.9	10.2	8.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.7
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74033124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	2	0	0	0	0	0	1	0	0	0	0	3	.5	.30
CALM+ - 1.5	0	0	0	3	3	2	1	0	0	2	0	0	1	0	3	1	16	2.5	1.12
1.6 - 2.5	2	1	2	2	3	4	1	2	2	5	1	0	3	1	3	0	32	4.9	1.96
2.6 - 3.5	6	2	0	7	6	5	3	2	3	2	3	3	2	2	1	2	49	7.5	2.92
3.6 - 7.5	23	11	14	10	24	24	20	13	18	38	33	11	27	13	19	15	313	48.1	5.18
7.6 - 12.5	2	3	12	5	6	9	9	1	12	20	32	10	13	13	14	16	177	27.2	9.36
12.6 - 18.5	0	0	0	0	1	4	0	0	4	5	9	2	8	16	3	1	53	8.1	14.80
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	3	5	0	0	8	1.2	21.45
TOTAL	33	17	28	27	43	50	34	18	39	72	78	27	57	50	43	35	651	100.0	6.62
PERCENT	5.1	2.6	4.3	4.1	6.6	7.7	5.2	2.8	6.0	11.1	12.0	4.1	8.8	7.7	6.6	5.4	100.0		
AV SPD	5.1	5.8	6.9	4.7	4.8	6.0	5.9	4.6	7.4	7.0	7.9	7.4	8.5	11.1	6.9	7.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 14

TABLE 2.3-16 (SHEET 11 OF 48)

MONTH OF MARCH

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
REQUEST NUMBER 604-44
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	1	0	0	0	0	1	0	0	2	.3	.30
CALM+ - 1.5	1	1	3	1	1	1	0	0	1	1	0	1	2	1	2	1	17	2.7	.92
1.6 - 2.5	2	0	1	2	3	3	3	6	1	1	2	2	1	2	1	1	31	5.0	1.99
2.6 - 3.5	1	4	1	6	7	5	3	5	7	3	14	4	4	2	3	7	72	11.6	2.92
3.6 - 7.5	12	0	2	7	12	20	22	31	39	55	56	35	37	17	31	28	404	65.2	5.02
7.6 - 12.5	1	0	1	2	0	0	4	2	8	12	10	6	11	13	10	8	88	14.2	8.91
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	2	3	0	0	5	.8	13.29
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	.2	18.84
TOTAL	17	5	8	18	23	29	32	44	57	72	82	48	57	40	47	41	620	100.0	3.93
PERCENT	2.7	.8	1.3	2.9	3.7	4.7	5.2	7.1	9.2	11.6	13.2	7.7	9.2	6.5	7.6	6.6	100.0		
AV SPD	4.9	2.7	2.9	4.0	3.7	4.2	4.9	4.5	5.2	5.8	5.2	5.3	5.9	7.3	5.9	5.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 6

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	1	2	2	1	0	0	0	1	0	0	1	2	1	11	4.7	.35
CALM+ - 1.5	0	0	2	2	1	1	0	1	0	1	0	0	1	0	1	1	11	4.7	.97
1.6 - 2.5	0	0	1	0	3	3	1	0	1	0	0	1	2	4	1	0	17	7.3	1.99
2.6 - 3.5	0	2	2	3	6	6	2	2	1	2	5	5	6	1	4	1	48	20.5	2.95
3.6 - 7.5	0	2	1	4	2	1	3	5	29	24	30	15	20	6	1	2	145	62.0	4.35
7.6 - 12.5	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	2	.9	8.15
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	0	4	6	10	14	13	7	8	31	27	37	22	29	12	9	5	234	0.0	2.26
PERCENT	0.0	1.7	2.6	4.3	6.0	5.6	3.0	3.4	13.2	11.5	15.8	9.4	12.4	5.1	3.8	2.1	100.0		
AV SPD	0.0	3.4	2.3	2.6	2.2	2.3	3.1	4.0	5.1	4.9	4.8	4.4	4.5	3.7	2.2	2.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-16 (SHEET 12 OF 48)

MONTH OF MARCH

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	1	0	1	0	1	0	0	1	1	0	0	0	0	0	5	2.4	.00
CALM+ - 1.5	3	2	2	3	4	1	2	0	1	0	1	2	2	0	1	0	24	11.5	.92
1.6 - 2.5	1	2	3	1	0	1	2	1	2	3	5	0	8	5	1	4	39	18.7	2.02
2.6 - 3.5	1	1	1	1	1	3	2	2	4	5	11	7	4	7	5	2	57	27.3	3.01
3.6 - 7.5	2	0	0	1	1	6	4	4	4	22	13	17	10	2	0	3	84	40.2	4.63
7.6 - 12.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	7	5	7	6	7	11	11	7	11	31	31	21	24	14	7	9	209	0.0	2.11
PERCENT	3.3	2.4	3.3	2.9	3.3	5.3	5.3	3.3	5.3	14.8	14.8	10.0	11.5	6.7	3.3	4.3	100.0		
AV SPD	2.3	1.7	1.7	1.8	1.5	3.4	3.0	3.9	3.4	4.3	3.4	3.7	3.5	3.1	2.6	2.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

TABLE 2.3-16 (SHEET 13 OF 48)

MONTH OF APRIL

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO [illegible] FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	VRBL	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	2	1	0	0	0	0		0	3	.3	1.16
1.6 - 2.5	0	1	1	1	1	0	0	0	1	0	1	1	1	2	2		2	14	1.6	[illegible]
2.6 - 3.5	4	1	1	2	5	1	0	0	1	1	3	0	4	2	2		[illegible]	35	3.9	[illegible]
3.6 - 7.5	17	12	11	23	34	19	13	10	9	27	28	24	25	34	33		[illegible]	340	37.8	[illegible]
7.6 - 12.5	4	4	4	5	19	34	39	16	13	32	28	40	42	69	54		12	414	46.0	9.61
12.6 - 18.5	0	2	2	0	4	3	1	1	1	6	7	19	21	17	6		1	91	10.1	14.34
18.6+	0	0	0	0	0	0	0	0	0	1	2	0	0	0	0		0	3	.3	20.50
TOTAL	25	20	19	31	63	57	52	27	24	69	70	84	93	124	97		45	900	100.0	6.76
PERCENT	2.8	2.2	2.1	3.4	7.0	6.3	5.8	3.0	2.7	7.7	7.8	9.3	10.3	13.8	10.8		5.0	100.0		
AV SPD	5.5	6.5	6.5	6.2	7.2	8.5	8.8	8.1	7.6	8.3	8.5	9.6	9.7	9.1	8.4		6.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 8.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 41

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	VRBL	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		0	0	0.0	0.00
1.6 - 2.5	0	1	1	0	0	0	2	0	1	0	1	0	0	0	0		0	6	4.5	2.77
2.6 - 3.5	1	0	0	1	1	0	0	0	0	1	1	0	1	0	0		0	6	4.5	[illegible]
3.6 - 7.5	3	0	1	4	10	6	0	4	2	5	7	2	5	9	5		[illegible]	72	53.7	5.46
7.6 - 12.5	1	0	1	0	1	2	5	2	1	5	6	4	3	7	1		1	40	29.9	5.79
12.6 - 18.5	0	1	0	0	0	2	1	0	0	1	2	0	3	0	0		0	10	7.5	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0		0	0	0.0	0.00
TOTAL	5	2	3	5	12	10	17	6	4	12	17	6	12	16	6		1	134	0.0	[illegible]
PERCENT	3.7	1.5	2.2	3.7	9.0	7.5	12.7	4.5	3.0	9.0	12.7	4.5	9.0	11.9	4.5		.7	100.0		
AV SPD	5.2	7.6	5.7	4.1	6.6	8.2	7.1	7.0	5.1	8.4	7.8	9.0	8.0	7.4	6.7		12.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-16 (SHEET 14 OF 48)

MONTH OF APRIL

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	1	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	3	2.8	1.25
1.6 - 2.5	0	0	0	1	0	0	1	0	1	0	1	0	0	0	0	2	6	5.5	1.95
2.6 - 3.5	1	1	0	2	1	0	0	0	0	0	0	0	1	0	0	0	6	5.5	3.07
3.6 - 7.5	1	2	0	5	6	4	2	1	4	7	2	3	2	5	2	1	47	43.1	5.55
7.6 - 12.5	0	0	0	0	2	7	1	0	1	2	6	8	8	2	1	0	38	34.9	9.33
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	3	3	2	0	1	0	9	8.3	14.57
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	3	3	0	10	9	11	4	1	6	9	12	14	13	7	4	3	109	0.0	6.31
PERCENT	2.8	2.8	0.0	9.2	8.3	10.1	3.7	.9	5.5	8.3	11.0	12.8	11.9	6.4	3.7	2.8	100.0		
AV SPD	2.9	3.7	0.0	4.0	6.1	8.6	5.2	5.1	5.8	6.6	10.0	10.3	8.5	7.3	8.5	3.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	.2	.73
CALM+ - 1.5	0	2	1	1	1	0	0	2	1	0	0	1	1	0	2	0	12	2.0	1.12
1.6 - 2.5	2	0	0	1	1	1	0	3	1	3	3	3	0	1	3	0	22	3.7	1.95
2.6 - 3.5	3	7	4	4	9	2	5	3	3	5	2	2	1	0	2	2	54	9.1	3.00
3.6 - 7.5	15	10	9	18	24	24	18	11	22	34	29	26	26	24	7	13	310	52.4	5.23
7.6 - 12.5	7	3	3	8	9	18	7	2	4	19	29	22	10	21	8	1	171	28.9	9.17
12.6 - 18.5	0	0	0	0	4	0	0	0	0	6	7	1	1	1	0	0	20	3.4	14.65
18.6+	0	0	0	0	0	0	0	0	0	0	1	0	1	0	0	0	2	.3	19.16
TOTAL	27	22	17	32	48	46	30	21	31	67	71	55	40	47	22	16	592	100.0	6.23
PERCENT	4.6	3.7	2.9	5.4	8.1	7.8	5.1	3.5	5.2	11.3	12.0	9.3	6.8	7.9	3.7	2.7	100.0		
AV SPD	6.1	4.7	5.5	6.0	6.1	6.9	5.8	4.6	5.5	7.2	8.1	7.0	6.7	7.1	5.8	5.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

TABLE 2.3-16 (SHEET 15 OF 48)

MONTH OF APRIL

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	2	.3	.30
CALM+ - 1.5	2	0	1	1	1	2	1	2	0	2	0	1	2	0	0	0	15	2.6	1.37
1.6 - 2.5	1	1	3	6	9	2	3	1	3	3	2	2	2	2	0	2	42	7.3	2.02
2.6 - 3.5	1	2	8	10	14	6	11	9	7	7	9	9	3	1	2	2	101	17.6	2.59
3.6 - 7.5	6	0	7	16	14	24	31	25	27	53	60	36	21	25	13	8	366	63.8	4.58
7.6 - 12.5	0	0	0	0	0	0	3	2	3	5	19	12	3	0	0	1	48	8.4	8.71
12.6 - 18.5	0	0	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	11	3	19	34	38	34	49	39	40	73	90	60	31	28	15	13	574	0.0	3.79
PERCENT	1.9	.5	3.3	5.9	6.6	5.9	8.5	6.8	7.0	12.7	15.7	10.5	5.4	4.9	2.6	2.3	100.0		
AV SPD	2.9	2.5	3.5	3.4	3.3	4.2	4.5	4.3	4.9	5.2	6.1	5.6	5.0	4.7	5.0	4.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	2	0	0	1	0	1	0	0	1	2	0	0	1	0	8	3.5	.30
CALM+ - 1.5	0	2	2	3	1	1	1	2	1	0	0	1	1	2	1	0	18	7.8	1.29
1.6 - 2.5	2	2	0	3	5	2	5	1	3	0	2	3	2	2	2	3	37	16.0	2.35
2.6 - 3.5	0	1	1	2	5	8	7	4	3	2	5	4	3	3	1	2	51	22.1	3.23
3.6 - 7.5	3	2	4	2	2	4	5	6	8	19	23	6	19	3	5	3	114	49.4	4.76
7.6 - 12.5	0	0	0	0	0	0	0	1	0	0	0	1	1	0	0	0	3	1.3	8.21
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	5	7	9	10	13	16	18	15	15	21	31	17	26	10	10	8	231	0.0	2.25
PERCENT	2.2	3.0	3.9	4.3	5.6	6.9	7.8	6.5	6.5	9.1	13.4	7.4	11.3	4.3	4.3	3.5	100.0		
AV SPD	3.6	2.8	2.6	2.4	2.7	2.9	3.1	4.0	4.0	5.0	4.2	3.5	4.6	3.2	3.2	3.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-16 (SHEET 16 OF 48)

MONTH OF APRIL

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	2	0	1	0	0	0	0	0	1	0	0	1	0	0	0	0	5	2.4	[illegible]
CALM+ - 1.5	7	2	0	2	3	1	0	2	1	1	2	7	1	0	1	0	25	12.2	.75
1.6 - 2.5	2	1	5	0	0	3	3	1	3	4	6	0	2	2	0	1	33	16.1	[illegible]
2.6 - 3.5	2	1	1	0	3	1	2	4	3	7	2	6	6	6	1	0	39	19.0	[illegible]
3.6 - 7.5	0	1	2	1	3	3	4	1	8	15	30	16	12	4	2	0	102	49.8	4.64
7.6 - 12.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	.5	8.55
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	13	5	9	3	9	7	9	8	16	23	40	25	21	12	4	1	205	0.0	[illegible]
PERCENT	6.3	2.4	4.4	1.5	4.4	3.4	4.4	3.9	7.8	11.2	19.5	12.2	10.2	5.9	2.0	.5	100.0		
AV SPD	1.3	2.0	2.6	1.9	2.8	2.8	3.0	2.3	3.3	4.3	4.2	3.9	4.0	3.2	3.2	1.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1.

TABLE 2.3-16 (SHEET 17 OF 48)

MONTH OF MAY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.1	1.39
1.6 - 2.5	0	0	3	1	1	1	0	0	1	1	1	1	0	0	1	1	12	1.2	2.13
2.6 - 3.5	7	4	4	2	0	2	4	1	3	[illegible]	2	1	4	5	10	5	57	5.9	3.03
3.6 - 7.5	30	23	24	21	29	11	11	19	23	29	30	43	41	73	83	74	523	54.5	5.49
7.6 - 12.5	11	2	[illegible]	1	7	19	14	18	18	38	32	37	41	51	29	6	326	34.0	9.20
12.6 - 18.5	0	0	0	0	0	2	0	0	2	16	6	1	4	6	3	1	41	4.3	14.34
19.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	48	29	35	25	37	34	29	38	47	86	71	83	90	135	126	47	960	0.0	6.34
PERCENT	5.0	3.0	3.6	2.6	3.9	3.5	3.0	4.0	4.9	9.0	7.4	8.5	9.4	14.1	13.1	4.9	100.0		
AV SPD	5.9	5.3	5.2	5.4	6.3	7.6	7.2	7.7	7.2	9.0	9.1	7.6	7.8	7.3	6.4	5.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 32

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	.7	1.14
1.6 - 2.5	1	2	0	2	0	2	1	0	1	1	1	0	0	0	2	0	13	8.5	2.03
2.6 - 3.5	2	2	0	0	0	1	1	0	0	1	2	[illegible]	2	0	4	1	18	11.8	2.99
3.6 - 7.5	3	4	1	[illegible]	5	3	5	6	9	8	11	5	8	4	7	[illegible]	84	54.9	5.14
7.6 - 12.5	1	1	1	0	2	1	3	2	1	4	10	4	3	0	2	0	35	22.9	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	2	1.3	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	7	8	2	4	7	7	10	8	11	15	24	13	13	4	15	4	153	0.0	4.54
PERCENT	4.6	5.2	1.3	2.6	4.6	4.6	6.5	5.2	7.2	9.8	15.7	8.5	8.5	2.6	9.8	2.6	100.0		
AV SPD	4.4	[illegible]	7.2	3.7	6.1	4.8	6.3	6.3	5.9	5.9	6.5	5.5	5.7	4.9	4.8	4.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 6

TABLE 2.3-16 (SHEET 18 OF 48)

MONTH OF MAY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	1	0	0	0	0	0	0	1	0	0	0	0	0	0	0	1	3	2.8	1.19
1.6 - 2.5	0	1	0	0	1	1	0	0	0	1	2	0	0	0	1	1	8	7.4	2.56
2.6 - 3.5	0	0	0	0	0	1	0	0	0	1	2	0	2	2	1	3	12	11.1	3.30
3.6 - 7.5	3	3	3	1	7	3	1	0	4	4	10	0	1	4	7	5	56	51.9	5.76
7.6 - 12.5	1	0	2	1	2	3	2	1	0	3	2	3	0	1	2	0	23	21.3	9.33
12.6 - 18.5	0	0	0	0	0	0	0	0	0	2	0	3	0	0	0	0	5	4.6	14.90
18.6+	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	.9	21.42
TOTAL	5	4	5	2	10	8	3	2	4	11	17	6	3	7	11	10	108	100.0	4.54
PERCENT	4.6	3.7	4.6	1.9	9.3	7.4	2.8	1.9	3.7	10.2	15.7	5.5	2.8	6.5	10.2	9.3	100.0		
AV SPD	5.3	4.4	8.1	6.3	5.4	5.7	8.6	6.5	4.9	8.0	6.3	11.7	4.5	5.3	6.1	3.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	1	0	0	1	0	0	0	0	0	0	0	2	.3	.10
CALM+ - 1.5	2	0	0	2	1	1	0	3	0	0	2	0	0	0	0	0	11	1.7	.58
1.6 - 2.5	8	4	1	4	6	6	5	2	1	3	3	5	4	4	2	3	61	9.3	2.29
2.6 - 3.5	13	5	6	2	9	4	5	4	2	1	3	4	8	1	4	2	73	11.1	2.93
3.6 - 7.5	14	11	12	20	31	33	17	11	24	21	29	28	26	26	15	25	343	52.2	5.57
7.6 - 12.5	8	9	9	12	3	14	15	8	11	7	14	11	7	10	5	4	147	22.4	9.14
12.6 - 18.5	0	1	7	2	0	0	1	1	1	2	2	1	0	0	1	0	19	2.9	14.19
18.6+	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	.2	20.67
TOTAL	45	30	35	42	50	59	43	29	40	34	54	49	45	41	27	34	657	100.0	4.15
PERCENT	6.8	4.6	5.3	6.4	7.6	9.0	6.5	4.4	6.1	5.2	8.2	7.5	6.8	6.2	4.1	5.2	100.0		
AV SPD	4.1	5.7	7.5	5.9	4.7	5.7	6.6	5.8	5.9	6.7	6.8	6.1	5.3	5.7	6.1	5.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

TABLE 2.3-16 (SHEET 19 OF 48)

MONTH OF MAY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION   REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	1	0	0	1	0	1	0	0	0	0	0	0	0	0	3	.5	.30
CALM+ - 1.5	3	3	2	2	5	1	1	2	0	0	3	0	2	4	3	1	32	4.9	1.15
1.6 - 2.5	3	1	5	6	5	11	4	5	5	2	3	6	6	8	4	5	79	12.2	1.96
2.6 - 3.5	6	3	2	4	9	10	13	7	13	6	11	9	5	4	7	12	122	18.8	2.95
3.6 - 7.5	12	9	5	10	13	25	23	35	55	43	55	17	14	15	19	18	373	57.6	4.69
7.6 - 12.5	0	4	0	0	0	3	5	3	6	6	3	1	1	2	2	1	38	5.9	8.60
12.6 - 18.5	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.2	14.20
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	24	20	16	22	32	51	52	53	79	57	75	33	29	33	35	37	648	0.0	3.15
PERCENT	3.7	3.1	2.5	3.4	4.9	7.9	8.0	8.2	12.2	8.8	11.6	5.1	4.5	5.1	5.4	5.7	100.0		
AV SPD	3.3	5.2	3.3	3.1	3.1	3.9	4.6	4.5	5.0	4.9	4.7	3.9	3.5	3.7	4.0	3.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION   REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	1	0	0	1	0	3	0	2	1	8	3.8	.32
CALM+ - 1.5	4	1	0	0	3	1	4	1	0	1	2	0	3	3	2	2	27	12.7	.56
1.6 - 2.5	0	2	1	3	3	4	2	3	3	1	2	2	4	2	6	2	40	18.8	1.96
2.6 - 3.5	2	0	0	2	1	5	7	5	3	5	3	7	4	9	3	0	56	26.3	3.05
3.6 - 7.5	4	3	2	2	3	2	1	3	11	12	12	5	8	3	5	6	82	38.5	4.51
7.6 - 12.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	10	6	3	7	10	12	14	13	17	19	20	14	22	17	18	11	213	0.0	1.93
PERCENT	4.7	2.8	1.4	3.3	4.7	5.6	6.6	6.1	8.0	8.9	9.4	6.6	10.3	8.0	8.5	5.2	100.0		
AV SPD	2.8	2.9	3.4	2.9	2.4	2.8	2.5	2.6	4.2	4.1	4.0	3.4	2.7	2.8	2.6	3.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-16 (SHEET 20 OF 48)

MONTH OF MAY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	1	0	0	0	1	0	1	1	1	1	2	1	2	1	12	6.7	.33
CALM+ - 1.5	3	0	0	1	1	0	1	1	1	0	0	0	1	1	1	2	13	7.3	1.[illegible]
1.6 - 2.5	0	1	0	0	1	2	6	1	1	2	5	2	7	6	8	3	45	25.3	2.01
2.6 - 3.5	1	2	1	1	1	2	1	3	2	5	6	5	5	7	7	2	51	28.7	2.92
3.6 - 7.5	0	0	0	0	0	2	2	2	3	8	16	2	9	4	6	3	57	32.0	4.43
7.6 - 12.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	4	3	2	2	3	6	11	7	8	16	28	10	24	19	24	11	178	0.0	1.69
PERCENT	2.2	1.7	1.1	1.1	1.7	3.4	6.2	3.9	4.5	9.0	15.7	5.6	13.5	10.7	13.5	6.2	100.0		
AV SPD	1.6	2.7	1.4	1.7	1.9	2.8	2.3	2.9	2.7	4.0	3.9	2.8	2.9	2.7	2.6	2.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 2.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-16 (SHEET 21 OF 48)

MONTH OF JUNE

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	1	2	0	1	0	0	0	0	1	0	0	0	5	.6	.98
1.6 - 2.5	1	5	3	2	2	5	2	0	2	3	3	0	0	2	1	1	32	3.9	2.06
2.6 - 3.5	12	17	15	5	4	9	1	2	0	1	1	2	5	7	5	9	90	11.0	2.96
3.6 - 7.5	28	30	36	49	49	43	19	11	16	15	19	25	38	29	66	40	513	62.9	5.20
7.6 - 12.5	0	3	11	9	21	15	15	3	4	6	9	16	26	9	12	4	163	20.0	9.07
12.6 - 18.5	0	0	0	0	0	1	0	0	0	0	2	3	5	1	0	0	12	1.5	13.97
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	41	50	65	65	77	75	37	17	22	25	34	45	75	48	84	54	815	0.0	4.93
PERCENT	5.0	6.1	8.0	8.0	9.4	9.2	4.5	2.1	2.7	3.1	4.2	5.6	9.2	5.9	10.3	6.6	100.0		
AV SPD	4.0	4.6	5.2	5.5	6.3	5.8	6.8	5.4	5.9	5.8	7.1	7.3	7.2	5.9	5.7	5.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 25

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	2	1.3	1.27
1.6 - 2.5	3	1	3	0	2	1	1	1	1	0	0	0	0	0	2	1	16	10.3	1.57
2.6 - 3.5	3	0	3	3	4	1	0	1	1	1	0	1	0	1	1	0	20	12.8	2.98
3.6 - 7.5	8	4	2	4	7	9	1	4	0	5	6	12	6	4	3	4	79	50.6	5.07
7.6 - 12.5	1	0	0	1	1	2	5	4	1	0	1	6	7	2	1	0	32	20.5	9.40
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	3	0	2	2	0	0	7	4.5	13.44
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	15	5	8	8	15	13	7	10	3	6	10	19	15	9	7	6	156	0.0	4.32
PERCENT	9.6	3.2	5.1	5.1	9.6	8.3	4.5	6.4	1.9	3.8	6.4	12.2	9.6	5.8	4.5	3.8	100.0		
AV SPD	4.1	4.7	3.0	5.1	4.3	6.0	7.1	6.5	4.4	4.9	7.7	6.9	8.7	7.4	4.3	3.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-16 (SHEET 22 OF 48)

MONTH OF JUNE

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	0	2	1.9	1.42
1.6 - 2.5	0	0	0	1	2	0	0	0	0	0	0	0	2	0	1	1	7	6.7	2.01
2.6 - 3.5	2	1	2	1	1	0	1	0	1	1	1	1	2	2	0	0	16	15.2	2.97
3.6 - 7.5	4	0	2	5	3	3	5	0	5	6	5	3	5	3	2	4	55	52.4	5.16
7.6 - 12.5	0	0	0	0	1	3	2	1	2	1	0	3	2	1	0	0	16	15.2	9.55
12.6 - 18.5	0	0	0	0	0	0	1	1	0	0	0	0	2	5	0	0	9	8.6	14.06
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	6	1	4	7	7	6	9	2	9	8	6	7	13	11	4	5	105	0.0	4.51
PERCENT	5.7	1.0	3.8	6.7	6.7	5.7	8.6	1.9	8.6	7.6	5.7	6.7	12.4	10.5	3.8	4.8	100.0		
AV SPD	4.1	2.8	3.9	4.8	5.0	7.2	7.1	11.3	5.6	6.2	5.6	7.2	6.6	9.4	3.6	3.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	2	0	0	0	1	0	0	0	0	1	0	4	.6	[illegible]
CALM+ - 1.5	2	5	0	4	2	3	1	2	0	2	2	1	0	1	1	2	28	4.4	[illegible]
1.6 - 2.5	6	4	8	4	6	11	9	5	3	4	6	4	3	2	3	2	80	12.6	[illegible]
2.6 - 3.5	4	6	5	8	12	2	9	8	5	6	5	12	7	6	3	1	99	15.5	2.94
3.6 - 7.5	12	10	14	27	17	25	21	27	13	19	49	31	18	19	7	7	316	49.6	[illegible]
7.6 - 12.5	2	1	1	0	9	11	14	5	3	7	17	11	4	15	2	1	103	16.2	9.15
12.6 - 18.5	0	1	0	0	0	2	0	0	0	1	1	0	1	1	0	0	7	1.1	13.72
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	26	27	28	43	46	56	54	47	24	40	80	59	33	44	17	13	637	0.0	3.21
PERCENT	4.1	4.2	4.4	6.8	7.2	8.8	8.5	7.4	3.8	6.3	12.6	9.3	5.2	6.9	2.7	2.0	100.0		
AV SPD	3.9	3.7	4.0	3.8	4.8	5.0	5.4	4.7	4.5	5.1	5.7	5.3	5.1	6.3	4.2	4.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-16 (SHEET 23 OF 48)

MONTH OF JUNE

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	0	1	0	0	0	1	1	0	0	0	0	0	0	1	5	.7	.30
CALM+ - 1.5	2	2	4	4	4	5	2	2	4	1	4	2	3	1	5	2	47	6.6	.85
1.6 - 2.5	1	3	5	4	6	6	7	3	10	4	3	4	2	5	4	6	73	10.1	2.05
2.6 - 3.5	4	1	4	13	21	13	20	15	12	16	2	3	3	3	8	5	143	19.8	2.93
3.6 - 7.5	4	3	12	21	30	21	57	23	25	41	49	34	19	14	19	7	379	52.5	4.76
7.6 - 12.5	0	0	0	11	9	3	10	4	4	4	6	3	11	4	1	1	71	9.8	9.17
12.6 - 18.5	0	0	0	2	0	0	0	0	0	0	0	0	0	1	0	0	3	.4	13.04
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	11	11	25	56	70	48	95	48	56	66	64	46	38	28	37	22	722	0.0	2.96
PERCENT	1.5	1.5	3.5	7.8	9.7	6.6	13.3	6.6	7.8	9.1	8.9	6.4	5.3	3.9	5.1	3.0	100.0		
AV SPD	3.1	2.1	3.0	5.1	4.4	4.2	4.7	4.2	3.9	4.6	4.9	5.0	5.6	4.9	3.7	3.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	2	0	0	1	1	0	0	0	0	0	0	0	0	0	0	1	5	2.1	.35
CALM+ - 1.5	2	5	7	2	8	2	2	2	3	7	0	0	3	0	2	3	37	15.2	.91
1.6 - 2.5	1	2	2	1	12	9	3	3	5	2	4	1	5	5	4	5	65	26.7	1.56
2.6 - 3.5	2	2	4	2	4	5	4	1	4	3	3	2	5	6	5	2	56	23.0	2.95
3.6 - 7.5	0	1	2	4	5	9	9	5	6	5	11	12	3	5	3	1	79	32.5	4.93
7.6 - 12.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
12.6 - 18.5	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.4	13.37
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	8	11	11	10	30	25	17	11	19	13	18	15	16	16	14	10	243	0.0	1.92
PERCENT	3.3	4.5	4.5	4.1	12.3	10.3	7.0	4.5	7.4	5.3	7.4	6.2	6.6	6.6	5.8	4.1	100.0		
AV SPD	3.0	2.0	2.6	2.7	2.3	3.0	3.5	3.1	3.0	3.0	4.2	4.4	2.5	3.0	2.6	2.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-16 (SHEET 24 OF 48)

MONTH OF JUNE

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	3	1	0	0	0	1	0	0	0	0	0	0	2	1	9	8.6	.30
CALM+ - 1.5	1	1	1	1	1	0	2	1	0	0	0	0	5	3	3	2	21	20.0	.94
1.6 - 2.5	2	1	0	1	1	2	2	1	0	4	2	1	5	3	4	3	32	30.5	1.99
2.6 - 3.5	0	0	1	0	1	0	1	0	2	2	2	2	5	4	1	0	21	20.0	3.05
3.6 - 7.5	0	0	0	0	0	2	0	0	2	2	3	6	1	4	1	1	22	21.0	4.39
7.6 - 12.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	4	2	5	3	3	4	5	3	4	8	7	9	16	14	11	7	105	0.0	1.26
PERCENT	3.8	1.9	4.8	2.9	2.9	3.8	4.8	2.9	3.8	7.6	6.7	8.6	15.2	13.3	10.5	6.7	100.0		
AV SPD	1.5	1.7	.9	1.2	2.1	3.5	1.7	1.1	3.3	2.9	3.4	4.0	2.2	2.8	1.6	1.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 2.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-16 (SHEET 25 OF 48)

MONTH OF JULY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION   REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	1	0	0	1	0	0	0	0	1	0	0	0	3	.3	.30
CALM+ - 1.5	1	2	1	2	1	0	0	0	0	1	1	1	0	0	1	1	12	1.3	1.16
1.6 - 2.5	9	6	8	9	9	2	4	2	0	0	1	3	2	0	7	3	64	6.7	2.12
2.6 - 3.5	8	9	2	15	12	10	3	3	7	11	16	7	5	11	9	11	139	14.6	3.31
3.6 - 7.5	19	9	30	43	57	40	25	21	18	30	58	42	58	48	57	[illegible]	584	61.2	5.10
7.6 - 12.5	1	2	3	10	6	12	5	2	5	10	26	13	27	11	9	2	144	15.1	9.05
12.6 - 18.5	0	1	0	0	0	3	1	0	0	0	0	1	2	1	0	0	9	.9	14.38
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	37	29	44	79	85	67	38	29	30	52	102	67	95	71	83	47	955	-0.0	4.19
PERCENT	3.9	3.0	4.6	8.3	8.9	7.0	4.0	3.0	3.1	5.4	10.7	7.0	9.9	7.4	8.7	4.9	100.0		
AV SPD	3.7	4.2	4.9	4.8	4.9	6.1	5.7	4.7	5.4	5.6	6.1	5.7	6.4	5.6	5.3	4.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 15

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION   REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	2	1.5	1.28
1.6 - 2.5	2	1	1	0	0	2	0	0	0	1	1	0	1	1	3	1	14	10.4	2.17
2.6 - 3.5	1	1	3	1	0	3	0	1	4	0	2	5	2	0	4	1	28	20.7	3.04
3.6 - 7.5	0	3	4	4	6	5	7	5	4	3	11	7	7	6	2	3	77	57.0	5.17
7.6 - 12.5	0	0	1	1	0	0	1	1	0	3	2	2	0	0	1	0	12	8.9	9.11
12.6 - 18.5	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	2	1.5	14.37
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	3	5	9	6	6	10	9	8	8	7	17	14	10	7	10	6	135	0.0	4.02
PERCENT	2.2	3.7	6.7	4.4	4.4	7.4	6.7	5.9	5.9	5.2	12.6	10.4	7.4	5.2	7.4	4.4	100.0		
AV SPD	2.6	4.7	4.2	5.3	4.5	4.3	7.3	7.3	4.0	7.5	5.2	5.0	4.4	4.6	4.2	3.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-16 (SHEET 26 OF 48)

MONTH OF JULY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1	1.3	.90
1.6 - 2.5	1	1	0	0	0	0	0	0	2	2	1	0	0	1	0	1	9	11.5	2.03
2.6 - 3.5	1	0	0	1	1	1	0	0	1	1	1	3	2	1	1	1	15	19.2	2.87
3.6 - 7.5	1	1	2	1	2	3	1	2	3	3	2	8	6	3	5	0	43	55.1	5.36
7.6 - 12.5	0	1	0	1	0	0	0	1	1	1	2	1	1	0	1	0	10	12.8	9.16
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	3	3	2	3	3	4	2	3	7	7	6	12	9	5	7	2	78	0.0	3.93
PERCENT	3.8	3.8	2.6	3.8	3.8	5.1	2.6	3.8	9.0	9.0	7.7	15.4	11.5	6.4	9.0	2.6	100.0		
AV SPD	2.8	4.8	4.5	5.6	5.0	4.7	2.9	7.1	4.7	4.9	5.8	5.1	4.9	4.1	5.9	2.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	2	1	0	0	1	0	1	0	1	0	0	0	1	0	0	0	7	1.4	.30
CALM+ - 1.5	2	1	2	4	4	2	4	0	1	1	2	1	2	2	0	3	31	6.2	.95
1.6 - 2.5	4	7	8	6	7	2	5	7	2	3	6	5	3	1	3	7	55	13.1	1.93
2.6 - 3.5	1	3	10	9	9	3	3	4	5	9	4	8	6	6	3	4	76	17.3	2.66
3.6 - 7.5	2	6	9	8	10	14	20	10	17	42	57	29	14	16	5	4	229	52.[illegible]	4.75
7.6 - 12.5	0	3	1	0	1	1	2	3	7	9	5	4	2	4	1	0	43	9.7	8.73
12.6 - 18.5	0	0	0	0	0	0	0	2	0	0	0	0	0	1	0	2	5	1.1	14.81
18.6+	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	1	.2	19.22
TOTAL	11	17	31	27	31	22	35	22	30	64	74	47	29	30	12	16	437	100.0	2.32
PERCENT	2.2	3.4	6.2	5.4	6.2	4.4	7.0	4.4	6.9	17.9	14.9	9.5	6.6	6.0	2.4	3.7	100.0		
AV SPD	2.1	4.1	3.3	2.9	3.4	4.1	3.9	5.7	5.7	5.1	4.9	4.6	4.0	5.1	3.7	4.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

TABLE 2.3-16 (SHEET 27 OF 48)

MONTH OF JULY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	2	1	0	0	1	2	1	0	0	1	0	1	0	1	0	1	11	1.7	.30
CALM+ - 1.5	6	6	6	3	4	2	3	3	3	3	4	6	2	2	8	6	64	9.7	.65
1.6 - 2.5	11	12	3	6	7	14	5	13	13	9	10	11	7	8	4	1	131	19.8	2.01
2.6 - 3.5	4	3	10	13	8	12	20	13	8	32	21	14	15	6	4	4	187	28.2	2.97
3.6 - 7.5	7	6	15	4	8	17	24	11	34	49	35	23	11	10	2	4	260	39.2	4.53
7.6 - 12.5	1	0	0	0	1	0	2	1	1	2	1	0	0	0	1	0	10	1.5	8.66
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	31	28	34	26	29	47	55	41	53	96	71	55	35	27	19	16	663	0.0	2.39
PERCENT	4.7	4.2	5.1	3.9	4.4	7.1	8.3	6.2	8.0	14.5	10.7	8.3	5.3	4.1	2.9	2.4	100.0		
AV SPD	2.7	2.4	3.1	2.7	3.2	2.9	3.7	3.1	4.1	3.9	3.7	3.4	3.2	3.1	2.5	2.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	3	1	0	1	0	3	0	0	1	0	1	2	1	0	14	8.0	.30
CALM+ - 1.5	0	3	0	2	4	1	2	1	3	1	2	1	1	2	1	1	25	14.4	.97
1.6 - 2.5	2	2	2	0	5	3	5	4	1	7	1	0	1	1	1	2	37	21.3	2.00
2.6 - 3.5	0	0	0	1	2	4	4	6	9	5	8	4	4	2	0	0	49	28.2	2.95
3.6 - 7.5	0	2	2	1	1	1	7	1	4	6	13	4	2	3	0	0	47	27.0	4.50
7.6 - 12.5	0	0	0	0	0	0	0	1	0	0	0	0	0	1	0	0	2	1.1	7.89
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	3	7	7	5	12	10	18	16	17	19	25	9	9	11	3	3	174	0.0	1.97
PERCENT	1.7	4.0	4.0	2.9	6.9	5.7	10.3	9.2	9.8	10.9	14.4	5.2	5.2	6.3	1.7	1.7	100.0		
AV SPD	1.5	2.2	1.9	1.8	2.2	2.3	3.2	2.5	3.2	3.0	3.7	3.9	2.7	2.7	.9	1.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 2.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-16 (SHEET 28 OF 48)

MONTH OF JULY

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 6[illegible]-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	2	0	0	0	0	0	0	2	0	0	0	0	0	1	5	8.6	.[illegible]
CALM+ - 1.5	0	0	0	0	0	0	1	2	3	2	0	2	1	0	0	1	12	20.7	1.0[illegible]
1.6 - 2.5	1	1	0	1	0	1	0	0	1	2	4	0	0	1	1	0	13	22.4	1.99
2.6 - 3.5	1	0	0	0	0	0	1	3	2	2	2	0	1	0	0	0	12	20.7	3.1[illegible]
3.6 - 7.5	0	0	1	0	0	1	1	0	2	2	7	1	1	0	0	0	16	27.6	4.1[illegible]
7.6 - 12.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-	0	0.0	0.00
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	2	1	3	1	0	2	3	5	9	10	13	3	3	1	1	2	59	0.0	1.35
PERCENT	3.4	1.7	5.2	1.7	0.0	3.4	5.2	8.6	13.8	17.2	22.4	5.2	5.2	1.7	1.7	3.4	100.0		
AV SPD	2.6	1.9	1.5	1.6	0.0	2.7	2.2	2.3	2.7	2.1	3.4	2.4	3.3	2.1	1.9	.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 2.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2.

TABLE 2.3-16 (SHEET 29 OF 48)

MONTH OF AUGUST

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION   REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	1	0	1	0	1	0	0	1	0	2	1	0	0	0	0	0	7	1.1	1.31
1.6 - 2.5	4	3	4	6	6	3	0	3	1	3	4	2	1	1	0	2	43	6.8	2.09
2.6 - 3.5	8	11	12	11	11	6	2	2	1	3	7	3	1	11	7	8	104	16.4	2.97
3.6 - 7.5	11	19	39	46	36	23	14	6	10	14	37	32	26	29	35	23	400	62.9	5.34
7.6 - 12.5	0	5	4	1	6	8	0	3	0	7	7	9	13	12	6	0	81	12.7	8.75
12.6 - 18.5	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	1	.2	14.09
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	24	38	60	64	60	40	17	15	12	29	56	46	41	53	48	33	636	0.0	4.24
PERCENT	3.8	6.0	9.4	10.1	9.4	6.3	2.7	2.4	1.9	4.6	8.8	7.2	6.4	8.3	7.5	5.2	100.0		
AV SPD	3.6	4.7	5.0	4.5	4.9	5.5	5.4	4.8	5.4	5.1	5.4	5.6	6.5	5.4	5.2	4.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 13

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION   REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.9 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
1.6 - 2.5	1	0	2	0	2	0	0	0	0	0	0	1	1	2	0	1	10	10.5	2.22
2.6 - 3.5	3	5	5	2	1	0	0	0	0	0	1	2	1	1	1	0	22	23.2	3.09
3.6 - 7.5	1	6	7	2	0	4	1	1	3	5	9	4	5	5	4	0	57	60.0	4.69
7.6 - 12.5	0	1	0	0	1	1	0	1	0	0	0	0	0	1	0	1	6	6.3	8.94
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	5	12	14	4	4	5	1	2	3	5	10	7	7	9	5	2	95	0.0	3.95
PERCENT	5.3	12.6	14.7	4.2	4.2	5.3	1.1	2.1	3.2	5.3	10.5	7.4	7.4	9.5	5.3	2.1	100.0		
AV SPD	2.8	4.4	3.4	3.6	4.6	6.2	6.5	7.9	5.8	5.3	4.9	4.5	3.9	5.1	5.1	4.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-16 (SHEET 30 OF 48)

MONTH OF AUGUST

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0	3	3.7	[illegible]
1.6 - 2.5	1	0	0	0	0	1	0	0	0	0	1	1	0	3	0	0	7	8.5	[illegible]
2.6 - 3.5	1	4	1	3	1	0	2	0	0	1	3	1	1	1	2	0	21	25.6	[illegible]
3.6 - 7.5	1	1	1	2	3	4	4	0	1	3	8	4	5	2	2	3	44	53.7	[illegible]
7.6 - 12.5	0	0	0	0	1	0	0	0	0	1	0	0	3	0	0	1	6	7.3	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	1.2	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	3	6	3	5	5	6	6	0	1	5	13	6	9	6	4	4	82	0.0	[illegible]
PERCENT	3.7	7.3	3.7	6.1	6.1	7.3	7.3	0.0	1.2	6.1	15.9	7.3	11.0	7.3	4.9	4.9	100.0		
AV SPD	3.1	3.4	2.5	3.3	5.7	4.5	4.8	0.0	5.7	5.8	5.5	5.0	7.1	2.9	3.7	5.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.7
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	1	0	0	0	0	0	0	0	0	1	1	0	0	0	0	4	.7	[illegible]
CALM+ - 1.5	0	4	2	0	3	5	4	2	3	2	6	3	1	1	2	[illegible]	[illegible]	6.5	[illegible]
1.6 - 2.5	5	5	9	9	5	9	2	5	9	9	8	5	3	8	1	4	[illegible]	16.7	2.?7
2.6 - 3.5	6	6	6	12	13	4	14	7	5	16	14	4	6	5	6	2	[illegible]	21.9	[illegible]
3.6 - 7.5	9	11	10	17	21	28	11	15	15	43	51	18	10	5	7	[illegible]	[illegible]	43.4	4.74
7.6 - 12.5	0	1	0	0	1	1	1	0	0	2	15	6	4	0	0	2	[illegible]	5.7	8.77
12.6 - 18.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	[illegible]	[illegible]	.2	14.??
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]	[illegible]	0.0	0.00
TOTAL	21	28	27	38	44	47	32	29	33	73	95	37	24	19	16	[illegible]	[illegible]	0.0	2.82
PERCENT	3.6	4.8	4.6	6.5	7.6	8.1	5.5	5.0	5.7	12.6	16.4	6.4	4.1	3.3	[illegible]	[illegible]	100.0		
AV SPD	3.4	3.2	3.2	3.3	3.4	3.9	3.5	3.7	3.4	4.7	5.0	4.9	4.7	3.3	3.4	[illegible]			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 6

TABLE 2.3-16 (SHEET 31 OF 48)

MONTH OF AUGUST

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION   REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	1	0	1	0	1	0	0	1	1	0	0	0	2	0	0	9	1.2	.30
CALM+ - 1.5	3	6	10	5	8	7	6	8	3	6	3	5	4	4	7	5	89	12.9	.99
1.6 - 2.5	9	6	5	11	18	16	9	12	19	18	19	12	10	6	4	3	177	25.7	2.30
2.6 - 3.5	3	5	11	13	14	13	9	24	14	23	17	12	6	14	8	5	191	27.8	2.97
3.6 - 7.5	3	0	4	6	11	8	17	14	27	55	36	18	11	4	2	2	218	31.7	4.59
7.6 - 12.5	0	0	0	0	1	0	1	1	0	1	0	1	0	0	0	0	5	.7	9.33
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	19	17	30	36	52	45	42	59	64	104	75	48	31	30	21	15	688	0.0	2.17
PERCENT	2.8	2.5	4.4	5.2	7.6	6.5	6.1	8.6	9.3	15.1	10.9	7.0	4.5	4.4	3.1	2.2	100.0		
AV SPD	2.2	1.7	2.5	2.5	2.8	2.5	3.4	3.0	3.3	3.7	3.7	3.5	3.1	2.6	2.2	2.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 13

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION   REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	2	0	0	0	0	0	2	0	1	0	0	1	0	0	1	0	7	4.0	.30
CALM+ - 1.5	4	1	2	4	3	0	2	3	2	1	0	0	1	2	2	4	31	17.7	.97
1.6 - 2.5	3	8	0	2	3	4	2	2	4	4	3	7	5	2	0	3	52	29.7	1.99
2.6 - 3.5	0	1	0	0	0	2	4	8	5	1	4	3	4	4	3	2	41	23.4	2.99
3.6 - 7.5	1	0	0	0	0	3	5	4	7	5	11	6	1	0	0	0	43	24.5	4.55
7.6 - 12.5	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	.6	7.96
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	10	10	2	6	6	10	15	17	19	11	18	17	11	8	6	9	175	0.0	1.65
PERCENT	5.7	5.7	1.1	3.4	3.4	5.7	8.6	9.7	10.9	6.3	10.3	9.7	6.3	4.6	3.4	5.1	100.0		
AV SPD	1.4	2.0	.9	1.2	1.3	3.4	2.9	3.0	3.1	3.1	4.0	3.1	2.6	2.2	2.0	1.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 2.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-16 (SHEET 32 OF 48)

MONTH OF AUGUST

JOINT FREQUENCY TABLES OF WIND SPEED AND [illegible]
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) [illegible] THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 15
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 6C4-44

WIND DIRECTION

SPEED (MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD (MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	5.3	.33
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
1.6 - 2.5	0	0	1	0	0	0	1	0	1	0	1	1	0	0	1	0	6	31.6	1.95
2.6 - 3.5	0	0	0	0	0	1	0	0	0	0	1	0	0	0	0	0	2	10.5	[illegible]
3.6 - 7.5	0	0	0	0	1	2	0	1	1	0	4	0	1	0	0	0	10	52.6	4.14
7.6 - 12.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	0	0	1	0	1	3	1	1	2	0	6	1	1	0	2	0	19	100.0	1.98
PERCENT	0.0	0.0	5.3	0.0	5.3	15.8	5.3	5.3	10.5	0.0	31.6	5.3	5.3	0.0	10.5	0.0	100.0		
AV SPD	0.0	0.0	1.5	0.0	3.7	3.6	2.4	4.7	2.7	0.0	3.7	2.5	4.4	0.0	.9	0.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-16 (SHEET 33 OF 48)

MONTH OF SEPTEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	1	1	0	0	0	0	0	0	0	1	0	0	1	4	.5	1.27
1.6 - 2.5	0	4	2	4	4	1	1	1	0	0	0	1	1	0	2	2	23	3.3	2.06
2.6 - 3.5	8	2	5	8	9	5	2	2	2	1	3	1	3	2	4	4	61	8.9	2.97
3.6 - 7.5	13	12	23	69	57	37	11	8	7	5	14	13	19	12	21	9	330	48.0	5.33
7.6 - 12.5	4	10	26	27	30	19	15	11	4	4	4	7	5	11	11	6	194	28.2	9.35
12.6 - 18.5	3	5	7	10	4	6	5	6	8	4	1	1	0	3	4	2	69	10.0	14.43
18.6+	0	2	2	0	0	0	0	0	2	1	0	0	0	0	0	0	7	1.0	20.94
TOTAL	28	35	65	119	105	68	34	28	23	15	22	23	29	28	42	24	688	100.0	5.57
PERCENT	4.1	5.1	9.4	17.3	15.3	9.9	4.9	4.1	3.3	2.2	3.2	3.3	4.2	4.1	6.1	3.5	100.0		
AV SPD	6.1	8.4	8.2	6.9	6.6	7.3	8.4	8.8	10.4	9.5	6.0	7.1	5.8	7.9	7.1	6.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 38

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	.9	1.14
1.6 - 2.5	0	2	0	0	0	0	0	0	0	0	0	0	0	1	1	0	4	3.4	2.34
2.6 - 3.5	0	1	3	0	2	0	1	0	0	0	1	0	0	0	0	0	8	6.8	2.81
3.6 - 7.5	2	0	8	5	9	3	0	3	1	3	1	0	3	2	4	4	48	41.0	5.72
7.6 - 12.5	0	5	7	7	3	7	4	0	2	0	2	1	0	1	0	3	42	35.9	9.01
12.6 - 18.5	0	0	0	1	1	1	1	1	3	1	3	0	0	1	0	0	13	11.1	15.01
18.6+	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	.9	24.51
TOTAL	2	8	19	13	15	11	6	4	6	4	7	1	3	5	5	8	117	100.0	5.77
PERCENT	1.7	6.8	16.2	11.1	12.8	9.4	5.1	3.4	5.1	3.4	6.0	.9	2.6	4.3	4.3	6.8	100.0		
AV SPD	7.2	6.5	7.3	8.3	6.1	8.9	9.2	7.8	11.9	7.9	11.1	8.5	6.3	6.8	5.1	6.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-16 (SHEET 34 OF 48)

MONTH OF SEPTEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70063101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1.2	1.47
1.6 - 2.5	0	0	0	0	0	2	1	0	0	0	1	0	0	1	0	1	6	7.5	2.17
2.6 - 3.5	0	0	1	0	0	1	1	0	0	1	0	1	0	1	1	0	7	8.7	2.92
3.6 - 7.5	2	3	6	5	8	5	2	0	0	2	5	1	4	2	1	0	46	57.5	4.91
7.6 - 12.5	1	2	5	1	0	2	2	2	1	0	0	0	0	2	0	0	18	22.5	9.23
12.6 - 18.5	0	0	0	0	0	0	1	0	0	0	0	0	0	0	1	0	2	2.5	16.22
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	4	5	12	6	8	10	7	2	1	3	6	2	4	6	3	1	80	0.0	4.61
PERCENT	5.0	6.3	15.0	7.5	10.0	12.5	8.7	2.5	1.2	3.7	7.5	2.5	5.0	7.5	3.7	1.2	100.0		
AV SPD	5.5	7.0	7.3	6.0	5.2	5.3	6.8	9.4	9.1	4.2	4.1	3.6	4.2	6.1	8.9	2.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.7
SITE HATCH
PERIOD OF RECORD FROM 70063101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	3	1	1	4	1	3	2	1	1	1	2	0	2	0	1	0	23	4.6	1.11
1.6 - 2.5	1	2	5	6	12	2	3	3	1	0	1	1	1	4	1	4	47	9.3	1.59
2.6 - 3.5	2	6	8	9	8	5	9	1	3	1	5	3	3	7	2	2	73	14.5	2.95
3.6 - 7.5	13	17	24	49	37	25	14	9	17	15	7	13	7	12	9	9	277	55.1	[illegible]
7.6 - 12.5	1	4	6	5	3	9	6	1	3	8	7	1	2	1	2	2	62	12.3	9.35
12.6 - 18.5	0	0	0	0	1	1	4	0	1	4	3	0	0	3	3	0	20	4.0	13.64
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	.2	[illegible]
TOTAL	20	30	44	73	62	45	37	15	26	30	25	18	15	27	19	17	503	100.0	3.75
PERCENT	4.0	6.0	8.7	14.5	12.3	8.9	7.4	3.0	5.2	6.0	5.0	3.6	3.0	5.4	3.8	3.4	100.0		
AV SPD	4.5	4.6	4.9	4.6	4.4	5.6	6.1	4.4	5.6	7.3	6.6	4.3	4.4	5.2	7.4	4.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 5.2
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-16 (SHEET 35 OF 48)

MONTH OF SEPTEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	1	1	1	0	0	0	0	1	0	1	0	0	0	0	6	.8	.32
CALM+ - 1.5	5	4	7	6	8	3	6	7	3	1	3	3	1	3	2	4	66	8.3	.99
1.6 - 2.5	7	18	10	14	18	15	18	7	3	8	4	5	6	4	1	4	142	17.9	2.01
2.6 - 3.5	10	16	23	33	26	29	15	11	12	5	6	8	6	0	1	4	205	25.8	2.98
3.6 - 7.5	11	16	25	35	29	43	44	26	13	8	18	22	18	16	10	8	342	43.0	4.42
7.6 - 12.5	1	0	0	1	1	0	3	1	3	4	7	1	2	3	7	0	34	4.3	9.44
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	.1	13.73
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	34	55	66	93	83	90	86	52	34	27	38	40	33	26	22	20	796	0.0	2.59
PERCENT	4.3	6.9	8.3	11.7	10.4	11.3	10.8	6.5	4.3	3.4	4.8	5.0	4.1	3.3	2.8	2.5	100.0		
AV SPD	3.1	2.9	3.1	3.2	3.1	3.7	3.9	3.5	3.9	3.9	4.8	3.9	4.1	4.4	6.2	3.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 6

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	1	0	1	0	0	0	0	0	1	1	1	2	2	9	3.1	.70
CALM+ - 1.5	4	5	3	5	6	1	1	2	1	3	2	0	2	1	2	0	38	13.0	.98
1.6 - 2.5	5	6	5	5	9	5	2	2	0	3	2	3	3	2	4	3	58	19.8	1.97
2.6 - 3.5	3	4	5	6	7	7	4	3	6	5	5	4	0	3	2	4	68	23.2	2.90
3.6 - 7.5	9	2	5	8	9	10	8	6	10	8	5	3	4	6	5	6	105	35.8	4.33
7.6 - 12.5	0	0	0	0	0	0	0	0	1	3	2	0	0	4	4	1	15	5.1	8.82
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	21	17	18	25	31	24	15	13	18	22	16	11	10	17	19	16	293	0.0	1.99
PERCENT	7.2	5.8	6.1	8.5	10.6	8.2	5.1	4.4	6.1	7.5	5.5	3.8	3.4	5.3	6.5	5.5	100.0		
AV SPD	3.1	2.7	2.9	2.7	2.7	3.1	3.7	3.1	4.1	4.0	4.1	2.8	2.9	4.7	3.9	3.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.3
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

TABLE 2.3-16 (SHEET 36 OF 48)

MONTH OF SEPTEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN [illegible]
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 674-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	UNK	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	1	0	2	0	0	0	0	0	0	0	1	0	0	4	4.4	.30
CALM+ - 1.5	1	0	0	0	0	0	1	0	0	0	0	1	0	0	4	3	10	11.1	1.[illegible]
1.6 - 2.5	0	2	2	2	3	0	0	1	1	1	0	0	0	3	1	0	16	17.8	[illegible]
2.6 - 3.5	3	0	2	2	0	1	0	0	3	1	1	0	2	3	2	2	22	24.4	2.57
3.6 - 7.5	3	0	2	0	0	0	0	2	2	4	1	1	6	5	2	1	29	32.2	4.[illegible]
7.6 - 12.5	0	0	0	1	0	0	0	0	0	0	0	0	1	2	3	2	9	10.0	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	7	2	6	6	3	3	1	3	6	6	2	2	9	14	12	8	90	0.0	1.95
PERCENT	7.8	2.2	6.7	6.7	3.3	3.3	1.1	3.3	6.7	6.7	2.2	2.2	10.0	15.6	13.3	8.9	100.0		
AV SPD	2.9	2.0	3.2	3.1	2.0	1.2	.9	3.7	3.4	4.2	3.8	4.5	4.2	4.9	4.2	3.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-16 (SHEET 37 OF 48)

MONTH OF OCTOBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	1	0	0	0	0	1	0	0	0	0	0	0	1	0	0	0	3	.4	1.04
1.6 - 2.5	7	2	4	0	0	0	0	0	0	0	0	1	0	0	1	5	20	2.6	2.37
2.6 - 3.5	4	8	3	8	3	3	0	0	2	0	1	0	0	2	3	7	44	5.6	3.04
3.6 - 7.5	12	19	32	47	56	14	11	8	10	8	6	6	10	16	19	12	286	36.5	5.37
7.6 - 12.5	8	18	31	34	37	17	25	8	8	6	4	3	10	25	11	8	254	32.4	9.45
12.6 - 18.5	6	7	9	11	2	8	26	24	9	3	1	0	4	2	4	1	117	14.9	15.29
18.6+	3	1	1	9	3	1	21	17	3	0	0	0	1	1	0	0	60	7.7	21.51
TOTAL	41	55	80	108	101	44	84	57	32	17	12	10	26	46	38	33	784	100.0	6.67
PERCENT	5.2	7.0	10.2	13.8	12.9	5.6	10.7	7.3	4.1	2.2	1.5	1.3	3.3	5.9	4.8	4.2	100.0		
AV SPD	7.9	7.7	8.1	9.6	7.6	9.2	14.2	16.0	11.0	9.6	8.1	6.3	9.5	8.6	7.3	5.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 9.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 34

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
1.6 - 2.5	1	0	1	0	1	0	0	0	0	0	1	0	0	1	0	0	5	4.6	2.31
2.6 - 3.5	0	0	0	2	3	1	0	0	1	0	1	0	0	0	1	0	9	8.3	2.96
3.6 - 7.5	2	3	6	10	9	4	1	3	2	3	3	1	2	0	5	4	58	53.2	5.12
7.6 - 12.5	0	2	5	2	4	3	1	1	0	1	1	0	0	1	1	1	23	21.1	9.73
12.6 - 18.5	1	0	0	0	3	2	2	0	1	0	1	0	0	0	1	0	11	10.1	14.66
18.6+	0	0	0	0	0	1	0	1	1	0	0	0	0	0	0	0	3	2.8	23.31
TOTAL	4	5	12	14	20	11	4	5	5	4	7	1	2	2	8	5	109	100.0	5.49
PERCENT	3.7	4.6	11.0	12.8	18.3	10.1	3.7	4.6	4.6	3.7	6.4	.9	1.8	1.8	7.3	4.6	100.0		
AV SPD	6.3	7.4	7.2	5.7	7.3	9.6	11.0	11.6	9.9	7.3	6.7	5.5	3.9	5.9	6.2	5.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

TABLE 2.3-16 (SHEET 38 OF 48)

MONTH OF OCTOBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
1.6 - 2.5	0	2	0	0	0	0	0	0	0	0	0	0	1	0	0	1	4	3.9	1.84
2.6 - 3.5	2	1	1	1	0	0	0	2	1	1	0	0	1	0	1	3	14	13.6	2.91
3.6 - 7.5	1	5	5	6	3	2	2	2	6	2	4	0	1	1	3	1	44	42.7	4.99
7.6 - 12.5	1	2	5	5	7	2	6	0	0	0	0	0	1	1	2	1	33	32.0	9.51
12.6 - 18.5	0	0	1	0	2	1	1	1	1	0	0	0	0	0	0	0	7	6.8	14.55
18.6+	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1	1.0	21.75
TOTAL	4	10	12	12	12	6	9	5	8	3	4	0	4	2	6	6	103	100.0	5.20
PERCENT	3.9	9.7	11.7	11.7	11.7	5.8	8.7	4.9	7.8	2.9	3.9	0.0	3.9	1.9	5.8	5.8	100.0		
AV SPD	5.9	5.3	7.4	7.2	9.2	11.5	9.3	5.9	6.7	4.0	5.5	0.0	5.3	5.9	5.9	3.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.3
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	0	1	0	2	0	2	0	1	1	2	1	0	1	4	1	17	2.4	.33
CALM+ - 1.5	3	4	0	0	4	1	0	0	0	0	0	0	4	0	0	2	18	2.6	1.07
1.6 - 2.5	4	2	4	2	6	1	1	2	1	0	0	4	5	3	6	7	48	6.8	1.91
2.6 - 3.5	7	3	9	8	5	4	2	0	4	1	3	1	2	5	6	1	61	8.7	[illegible]
3.6 - 7.5	9	44	71	57	43	22	26	7	6	6	6	6	9	11	9	13	345	49.1	5.16
7.6 - 12.5	3	5	19	47	24	18	22	5	6	5	1	3	6	7	7	5	183	26.0	[illegible]
12.6 - 18.5	0	0	4	5	2	3	3	3	3	1	1	0	1	3	1	0	30	4.3	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	.1	20.36
TOTAL	27	58	108	119	86	49	56	17	21	14	13	15	27	31	33	29	703	100.0	3.36
PERCENT	3.8	8.3	15.4	16.9	12.2	7.0	8.0	2.4	3.0	2.0	1.8	2.1	3.8	4.4	4.7	4.1	100.0		
AV SPD	3.8	4.9	6.1	6.9	6.1	6.9	7.5	7.9	7.0	7.3	5.0	5.1	5.0	6.9	4.5	4.5			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 4

TABLE 2.3-16 (SHEET 39 OF 48)

MONTH OF OCTOBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 624-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	1	1	0	1	1	1	0	0	0	0	1	[illegible]	[illegible]	7	1.0	[illegible]
CALM+ - 1.5	1	3	2	3	0	3	0	1	1	2	0	1	0	2	[illegible]	1	22	3.3	[illegible]
1.6 - 2.5	6	2	3	5	5	4	4	5	4	2	1	0	1	6	4	3	55	8.2	[illegible]
2.6 - 3.5	2	9	18	15	19	12	13	6	5	3	5	7	2	9	9	6	134	20.1	[illegible]
3.6 - 7.5	12	30	47	48	42	47	27	18	19	17	16	9	15	22	17	10	396	59.4	[illegible]
7.6 - 12.5	0	4	7	4	1	7	9	5	2	6	0	1	2	2	1	2	53	7.9	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	[illegible]
TOTAL	21	48	77	76	68	73	54	36	31	30	22	13	20	42	34	22	667	0.0	[illegible]
PERCENT	3.1	7.2	11.5	11.4	10.2	10.9	8.1	5.4	4.6	4.5	3.3	1.9	3.0	6.3	5.1	3.3	100.0		
AV SPD	3.8	4.5	4.6	4.3	4.2	4.9	4.9	4.6	4.7	5.3	4.7	4.5	4.6	3.9	3.6	4.0			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 9

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 624-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	[illegible]	1	.4	[illegible]
CALM+ - 1.5	2	0	1	1	2	0	1	0	1	0	0	1	1	3	1	[illegible]	14	5.6	[illegible]
1.6 - 2.5	8	2	1	4	6	2	1	4	4	3	0	1	2	1	1	3	43	17.2	[illegible]
2.6 - 3.5	11	2	10	4	7	9	5	3	2	2	2	3	5	6	2	2	75	30.0	[illegible]
3.6 - 7.5	6	12	11	6	6	5	3	9	5	10	9	8	3	4	6	[illegible]	112	44.8	[illegible]
7.6 - 12.5	0	0	0	0	0	1	0	2	0	0	0	1	0	1	0	0	5	2.0	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	[illegible]
TOTAL	27	16	23	15	21	17	15	18	12	15	11	14	11	15	10	[illegible]	250	0.0	[illegible]
PERCENT	10.8	6.4	9.2	6.0	8.4	6.8	6.0	7.2	4.8	6.0	4.4	5.6	4.4	6.0	4.0	[illegible]	100.0		
AV SPD	3.0	4.0	3.6	3.3	2.9	3.7	3.5	4.2	3.5	3.9	4.8	4.0	2.8	3.1	4.4	3.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-16 (SHEET 40 OF 48)

MONTH OF OCTOBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN [illegible]
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	1	4	1.6	[illegible]
CALM+ - 1.5	3	1	4	1	2	2	0	1	1	2	2	1	1	0	1	4	26	10.3	1.03
1.6 - 2.5	5	8	4	3	2	5	0	2	1	1	4	2	0	4	6	3	50	19.8	1.99
2.6 - 3.5	3	10	7	4	3	2	0	2	1	3	1	0	10	6	9	4	65	25.7	2.97
3.6 - 7.5	5	15	9	7	3	3	2	1	4	9	7	1	9	7	6	9	97	38.3	4.50
7.6 - 12.5	0	0	2	0	0	0	0	0	1	0	0	0	0	3	4	0	10	4.0	8.84
12.6 - 18.5	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	1	.4	14.53
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	17	35	26	16	10	12	2	6	8	16	14	4	20	20	26	21	253	0.0	2.34
PERCENT	6.7	13.8	10.3	6.3	4.0	4.7	.8	2.4	3.2	6.3	5.5	1.6	7.9	7.9	10.3	8.3	100.0		
AV SPD	2.7	3.2	3.6	3.5	3.0	2.5	4.7	2.4	4.7	4.4	3.2	2.6	3.8	4.2	3.9	3.[illegible]			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

TABLE 2.3-16 (SHEET 41 OF 48)

MONTH OF NOVEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 634-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	1	0	1	0	2	0	0	0	1	2	0	1	0	0	0	1	9	1.9	1.15
1.6 - 2.5	1	0	1	0	1	0	1	2	0	1	1	0	0	1	3	2	23	4.9	1.93
2.6 - 3.5	8	5	1	3	0	2	1	2	2	1	0	0	0	1	3	4	33	7.0	2.95
3.6 - 7.5	12	8	17	20	17	7	5	5	10	9	5	8	15	24	24	10	196	41.4	5.55
7.6 - 12.5	3	4	9	10	9	5	3	5	3	13	14	4	19	26	19	17	153	32.3	9.25
12.6 - 18.5	1	0	1	2	0	0	1	2	2	2	3	0	8	11	5	1	39	8.2	14.62
18.6+	0	0	0	0	0	0	0	1	0	1	0	1	4	4	0	0	11	2.3	19.74
TOTAL	26	26	30	35	29	14	11	17	18	29	23	14	46	67	54	35	474	100.0	5.46
PERCENT	5.5	5.5	6.3	7.4	6.1	3.0	2.3	3.6	3.8	6.1	4.9	3.0	9.7	14.1	11.4	7.4	100.0		
AV SPD	5.2	4.4	7.0	6.8	5.9	6.1	7.6	7.6	6.6	8.1	9.0	7.6	10.2	9.6	7.7	7.3			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.6
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 21

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 634-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
1.6 - 2.5	2	2	1	0	1	0	1	0	0	0	1	0	2	0	3	2	15	11.1	2.15
2.6 - 3.5	0	1	0	0	2	1	0	0	0	0	0	0	2	0	3	0	10	7.4	2.99
3.6 - 7.5	3	2	10	10	7	3	2	1	2	2	6	0	2	1	2	0	57	42.2	5.36
7.6 - 12.5	0	2	5	4	5	0	3	1	0	1	0	1	6	5	1	4	33	23.9	8.97
12.6 - 18.5	0	0	0	1	0	0	1	1	1	1	1	0	2	3	1	0	12	8.9	15.20
18.6+	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	2	1.5	19.44
TOTAL	5	8	17	15	15	4	7	3	3	4	8	1	14	14	10	6	135	100.0	5.35
PERCENT	3.7	5.9	12.6	11.9	11.1	3.0	5.2	2.2	2.2	3.0	5.9	.7	10.4	10.4	7.4	4.4	100.0		
AV SPD	7.5	5.1	7.5	8.0	6.2	5.0	7.6	10.8	9.3	8.7	6.7	10.3	7.4	8.8	5.0	6.7			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

TABLE 2.3-16 (SHEET 42 OF 48)

MONTH OF NOVEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1	.9	.30
CALM+ - 1.5	0	0	1	0	0	0	0	0	0	0	1	0	0	1	1	0	4	3.4	1.25
1.6 - 2.5	1	2	1	0	2	0	0	0	1	0	0	0	0	0	0	0	7	6.0	2.06
2.6 - 3.5	1	0	1	3	1	0	0	0	0	0	0	2	0	0	2	0	10	8.6	3.05
3.6 - 7.5	7	3	4	2	6	6	0	2	3	1	2	1	0	5	4	3	49	42.2	5.77
7.6 - 12.5	3	2	7	8	1	0	1	2	0	0	0	3	4	2	2	3	38	32.9	9.25
12.6 - 18.5	0	0	0	1	0	0	0	0	0	1	0	0	0	2	2	0	6	5.2	15.73
18.6+	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	.9	19.47
TOTAL	12	7	14	14	10	6	1	4	4	2	4	7	4	10	11	6	115	100.0	4.33
PERCENT	10.3	6.0	12.1	12.1	8.6	5.2	.9	3.4	3.4	1.7	3.4	6.0	3.4	8.6	9.5	5.2	100.0		
AV SPD	6.4	5.9	6.6	8.3	5.5	5.3	12.3	7.1	4.3	11.5	7.5	5.7	10.6	8.0	9.1	7.2			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.1
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.7
REQUEST NUMBER 604-44
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	0	1	1	2	2	1	0	0	0	0	0	1	0	0	9	1.2	.33
CALM+ - 1.5	5	4	0	1	2	1	2	0	0	2	0	1	2	1	2	2	25	3.5	1.17
1.6 - 2.5	6	6	4	4	5	7	3	2	1	0	4	1	1	1	0	6	51	7.1	1.99
2.6 - 3.5	3	5	4	3	4	4	7	0	2	4	1	4	2	7	8	5	63	8.7	2.91
3.6 - 7.5	21	15	18	39	32	24	16	2	8	14	25	17	21	26	27	35	340	47.1	5.35
7.6 - 12.5	1	1	8	16	8	8	2	8	9	11	18	12	24	23	21	22	192	26.6	9.74
12.6 - 18.5	1	0	0	2	0	1	2	2	0	2	1	4	3	8	8	1	35	4.8	14.33
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	1	3	0	3	7	1.0	19.73
TOTAL	37	32	34	66	52	47	34	15	20	33	49	39	54	70	66	74	722	100.0	3.86
PERCENT	5.1	4.4	4.7	9.1	7.2	6.5	4.7	2.1	2.8	4.6	6.8	5.4	7.5	9.7	9.1	10.2	100.0		
AV SPD	4.5	3.7	5.8	5.5	5.2	5.2	4.7	9.0	6.8	6.7	6.6	7.3	7.5	8.1	7.3	6.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

TABLE 2.3-16 (SHEET 43 OF 48)

MONTH OF NOVEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	0	0	0	0	0	0	0	0	1	0	0	0	0	0	2	.4	.30
CALM+ - 1.5	1	3	5	2	3	1	2	2	2	1	3	0	2	2	4	2	35	6.2	.97
1.6 - 2.5	3	4	5	7	10	11	0	0	1	2	5	5	1	0	7	1	62	10.9	2.21
2.6 - 3.5	2	4	4	2	10	11	4	3	8	8	6	2	3	5	2	2	76	13.4	3.[illegible]
3.6 - 7.5	8	7	28	22	17	15	21	12	29	29	27	15	28	23	24	[illegible]	322	56.6	4.[illegible]
7.6 - 12.5	0	0	0	0	1	4	5	2	6	4	1	4	13	19	8	3	70	12.3	9.[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	2	.4	13.3[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	14	19	42	33	41	45	31	19	46	43	43	26	47	49	47	24	569	0.0	3.29
PERCENT	2.5	3.3	7.4	5.8	7.2	7.9	5.4	3.3	8.1	7.6	7.6	4.6	8.3	8.6	8.3	4.2	100.0		
AV SPD	3.6	3.1	3.8	3.7	3.6	4.1	5.4	4.6	4.9	4.6	4.3	5.3	5.9	6.8	5.3	5.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.8
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	.3	.30
CALM+ - 1.5	0	1	2	[illegible]	3	0	1	2	1	1	0	1	1	0	1	[illegible]	14	4.7	.88
1.6 - 2.5	4	4	4	5	4	4	3	3	0	1	1	1	0	3	2	2	41	13.8	1.99
2.6 - 3.5	3	2	9	15	6	9	4	3	1	3	2	1	1	5	5	4	68	22.8	2.[illegible]
3.6 - 7.5	2	1	8	12	13	11	17	4	13	10	15	16	10	14	12	2	160	53.7	4.6[illegible]
7.6 - 12.5	0	0	0	0	0	0	1	0	1	0	1	4	1	1	4	1	14	4.7	8.42
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	9	8	23	27	26	24	26	12	16	15	19	23	13	23	24	10	298	0.0	3.03
PERCENT	3.0	2.7	7.7	9.1	8.7	8.1	8.7	4.0	5.4	5.0	6.4	7.7	4.4	7.7	8.1	3.4	100.0		
AV SPD	2.6	2.4	3.0	3.4	3.2	3.7	4.6	3.2	4.5	3.9	4.6	5.3	5.4	4.5	5.0	3.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 0

TABLE 2.3-16 (SHEET 44 OF 48)

MONTH OF NOVEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70110101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 624-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	STD MEAN SPD(MPH)
CALM	0	0	1	1	0	0	0	0	2	0	1	0	1	1	1	0	8	1.7	.10
CALM+ - 1.5	6	4	3	3	5	3	3	0	0	1	2	1	3	4	3	5	46	9.6	.99
1.6 - 2.5	10	10	6	5	7	5	5	6	1	0	3	3	7	13	14	8	103	21.5	2.11
2.6 - 3.5	4	8	5	7	6	4	5	7	2	5	9	5	17	15	8	10	112	23.4	2.99
3.6 - 7.5	2	1	3	12	2	9	17	8	9	10	28	36	28	19	13	6	203	42.5	4.69
7.6 - 12.5	0	0	0	0	0	0	0	1	1	0	2	1	1	5	0	0	8	1.7	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	[illegible]
TOTAL	22	23	18	28	20	21	30	17	15	16	45	46	57	52	39	29	479	0.0	2.31
PERCENT	4.6	4.8	3.8	5.9	4.2	4.4	6.3	3.6	3.1	3.3	9.4	9.6	11.9	10.9	8.2	6.1	100.0		
AV SPD	2.2	2.3	2.3	3.1	2.4	3.6	3.9	3.9	4.4	4.4	4.0	4.6	3.8	3.1	2.9	2.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.4
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 1

TABLE 2.3-16 (SHEET 45 OF 48)

MONTH OF DECEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) LESS THAN OR EQUAL TO -1.0
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	1	1	0	1	0	0	0	1	0	0	2	0	0	0	0	6	1.1	.57
1.6 - 2.5	6	2	7	6	1	3	2	0	1	1	3	2	0	2	2	0	39	7.0	2.01
2.6 - 3.5	3	3	2	3	3	1	2	2	1	2	3	3	5	2	3	2	41	7.3	2.89
3.6 - 7.5	12	8	11	20	16	21	3	7	6	15	19	17	30	51	27	9	269	48.0	5.34
7.6 - 12.5	3	3	0	5	3	8	2	0	7	11	14	13	16	23	44	12	163	29.1	9.42
12.6 - 18.5	0	0	0	0	0	1	0	0	1	4	9	1	2	4	17	4	43	7.7	14.22
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	24	16	21	34	24	34	9	9	17	33	48	37	53	82	93	27	561	0.0	5.37
PERCENT	4.3	2.9	3.7	6.1	4.3	6.1	1.6	1.6	3.0	5.9	8.6	6.6	9.4	14.6	16.6	4.8	100.0		
AV SPD	4.5	4.9	4.2	5.3	5.1	6.4	4.3	5.0	5.6	7.3	8.1	6.2	7.1	7.1	9.2	8.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.9
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -1.0 BUT LESS THAN OR EQUAL TO -.9
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74093124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
CALM+ - 1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	.7	1.39
1.6 - 2.5	4	2	0	0	0	0	0	3	1	1	4	0	2	0	2	0	19	13.5	1.91
2.6 - 3.5	1	1	0	2	3	0	1	0	2	1	0	1	1	0	1	0	14	9.9	3.08
3.6 - 7.5	6	2	3	5	9	8	4	2	4	5	3	4	4	1	4	2	66	46.8	5.29
7.6 - 12.5	0	0	2	6	4	3	1	0	1	2	3	2	2	6	2	1	35	24.8	8.54
12.6 - 18.5	0	0	0	0	0	0	0	0	0	1	1	1	1	1	1	0	6	4.3	14.13
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	11	5	5	13	16	11	6	5	8	10	11	8	10	8	11	3	141	0.0	4.43
PERCENT	7.8	3.5	3.5	9.2	11.3	7.8	4.3	3.5	5.7	7.1	7.8	5.7	7.1	5.7	7.8	2.1	100.0		
AV SPD	4.0	3.6	7.2	6.9	5.8	7.4	5.2	3.3	4.2	6.4	5.9	7.5	5.9	9.3	5.7	6.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 7

TABLE 2.3-16 (SHEET 46 OF 48)

MONTH OF DECEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.9 BUT LESS THAN OR EQUAL TO -.8
SITE HATCH
PERIOD OF RECORD FROM 73060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	CFQ MEAN SPD(MPH)
CALM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	[illegible]
CALM+ - 1.5	0	0	0	0	1	0	0	0	0	0	0	1	0	0	0	0	2	1.9	[illegible]
1.6 - 2.5	0	0	0	0	0	0	0	0	1	1	1	1	0	0	1	0	5	4.7	[illegible]
2.6 - 3.5	1	0	0	0	0	0	2	0	1	0	0	0	0	0	0	0	4	3.7	[illegible]
3.6 - 7.5	11	1	1	7	4	0	2	1	0	4	2	4	1	1	3	4	46	43.0	[illegible]
7.6 - 12.5	2	2	2	2	0	3	1	0	2	4	6	3	2	3	8	2	42	39.3	[illegible]
12.6 - 18.5	0	0	0	0	0	0	0	0	0	2	1	2	0	1	1	0	7	6.5	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	.9	[illegible]
TOTAL	14	3	3	9	5	3	5	1	4	11	11	11	3	5	13	6	107	100.0	[illegible]
PERCENT	13.1	2.8	2.8	8.4	4.7	2.8	4.7	.9	3.7	10.3	10.3	10.3	2.8	4.7	12.1	5.6	100.0		
AV SPD	6.2	7.5	7.5	6.6	5.0	9.9	4.5	7.1	6.3	9.0	9.7	7.3	6.9	9.1	8.5	6.9			

AVERAGE SPEED FOR THIS TABLE EQUALS 7.5
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 2

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.8 BUT LESS THAN OR EQUAL TO -.7
SITE HATCH
PERIOD OF RECORD FROM 73060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	CFQ MEAN SPD(MPH)
CALM	1	0	0	0	0	0	0	1	0	0	0	1	0	0	0	[illegible]	3	.4	[illegible]
CALM+ - 1.5	0	2	1	1	2	2	0	1	0	1	0	1	2	0	0	[illegible]	13	1.6	[illegible]
1.6 - 2.5	2	1	0	2	7	3	3	3	2	1	1	2	4	4	3	[illegible]	[illegible]	4.7	[illegible]
2.6 - 3.5	1	2	3	4	7	3	4	5	5	7	5	3	2	5	6	[illegible]	[illegible]	[illegible]	[illegible]
3.6 - 7.5	11	31	21	42	22	27	32	17	20	32	30	24	20	34	20	17	[illegible]	[illegible]	[illegible]
7.6 - 12.5	9	10	15	39	17	9	3	0	9	23	35	13	7	20	24	7	[illegible]	31.3	[illegible]
12.6 - 18.5	2	2	0	1	1	2	0	0	5	3	5	0	4	5	1	[illegible]	[illegible]	[illegible]	[illegible]
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	[illegible]	1	.1	[illegible]
TOTAL	26	49	40	89	56	46	42	27	41	72	76	49	39	63	55	[illegible]	[illegible]	100.0	[illegible]
PERCENT	3.2	6.1	5.0	11.1	7.0	5.7	5.2	3.4	5.1	9.1	9.5	6.1	4.9	8.0	6.9	7.2	100.0		
AV SPD	6.2	6.4	6.8	7.4	5.9	5.0	5.4	4.2	6.9	7.2	7.8	6.6	6.4	7.0	7.0	6.4			

AVERAGE SPEED FOR THIS TABLE EQUALS 6.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

TABLE 2.3-16 (SHEET 47 OF 48)

MONTH OF DECEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN -.3 BUT LESS THAN OR EQUAL TO .8
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	0	1	1	0	1	1	0	2	0	1	0	1	0	0	9	1.5	[illegible]
CALM+ - 1.5	4	1	0	2	4	3	2	4	0	1	0	3	1	2	0	2	29	4.9	1.16
1.6 - 2.5	2	3	2	0	5	3	1	4	4	3	3	3	5	2	2	3	45	7.6	2.37
2.6 - 3.5	7	4	2	3	6	6	6	8	17	6	5	3	12	6	3	0	94	15.9	2.99
3.6 - 7.5	4	6	10	9	11	17	21	29	33	64	38	20	28	37	13	14	354	59.9	4.99
7.6 - 12.5	0	1	0	8	1	1	0	0	3	9	9	4	9	5	2	3	55	9.3	8.51
12.6 - 18.5	0	0	0	0	0	0	0	1	0	0	0	1	0	1	2	0	5	.8	14.33
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	17	16	14	23	28	30	31	47	57	85	55	35	55	54	22	22	591	0.0	3.13
PERCENT	2.9	2.7	2.4	3.9	4.7	5.1	5.2	8.0	9.6	14.4	9.3	5.9	9.3	9.1	3.7	3.7	100.0		
AV SPD	2.8	3.5	4.4	5.5	3.2	3.7	3.9	4.3	4.6	5.2	5.7	4.8	5.0	5.4	5.8	4.8			

AVERAGE SPEED FOR THIS TABLE EQUALS 4.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 10

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION — REQUEST NUMBER 604-44
FOR TEMPERATURE DIFFERENCE(DEG F/100FT) GREATER THAN .8 BUT LESS THAN OR EQUAL TO 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	GEO MEAN SPD(MPH)
CALM	0	1	2	0	0	1	0	0	0	0	0	0	1	0	0	1	6	2.5	.30
CALM+ - 1.5	2	1	1	0	1	1	0	3	0	3	1	2	0	2	1	0	18	7.4	.81
1.6 - 2.5	4	1	3	1	3	6	4	2	0	2	4	2	3	2	3	2	42	17.2	1.9[illegible]
2.6 - 3.5	1	4	2	1	2	2	2	6	2	3	4	2	5	3	5	3	47	19.3	2.3[illegible]
3.6 - 7.5	1	1	2	3	9	5	8	11	7	26	12	15	9	10	6	1	126	51.6	4.77
7.6 - 12.5	0	0	0	0	0	0	0	0	1	2	0	0	0	1	1	0	5	2.0	7.93
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	8	8	10	5	15	15	14	22	10	36	21	21	18	18	16	7	244	0.0	2.2[illegible]
PERCENT	3.3	3.3	4.1	2.0	6.1	6.1	5.7	9.0	4.1	14.8	8.6	8.6	7.4	7.4	6.6	2.9	100.0		
AV SPD	2.3	2.6	2.4	4.3	3.6	2.9	3.7	3.5	4.9	4.3	4.0	3.9	3.5	4.6	4.1	2.6			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.7
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 3

TABLE 2.3-16 (SHEET 48 OF 48)

MONTH OF DECEMBER

JOINT FREQUENCY TABLES OF WIND SPEED AND DIRECTION
FOR TEMPERATURE DIFFERENCE (DEG F/100FT) GREATER THAN 2.2
SITE HATCH
PERIOD OF RECORD FROM 70060101 TO 74083124
SPEED AND DIRECTION FROM 75 LEVEL
TEMPERATURE DIFFERENCE BETWEEN 150 AND 35
SPEED MEASURED AT 75FT ADJUSTED TO 33 FT

REQUEST NUMBER 604-44

WIND DIRECTION

SPEED(MPH)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	TOTAL	PERCENT	FREQ MEAN SPD(MPH)
CALM	1	1	0	0	4	0	0	0	2	1	0	0	0	3	1	1	14	3.3	.35
CALM+ - 1.5	1	2	3	1	7	6	1	2	3	6	5	10	4	3	2	3	60	14.1	1.94
1.6 - 2.5	3	2	3	3	7	9	1	6	4	17	3	12	9	9	14	8	106	24.9	1.96
2.6 - 3.5	0	2	1	8	1	6	3	1	7	7	18	11	15	11	7	3	101	23.7	2.99
3.6 - 7.5	3	4	1	5	7	3	3	8	16	18	24	12	12	12	13	2	143	33.6	4.50
7.6 - 12.5	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	2	.5	8.35
12.6 - 18.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
18.6+	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0.00
TOTAL	8	17	8	17	26	24	9	17	32	45	51	45	40	38	37	17	426	0.0	1.99
PERCENT	1.9	3.1	1.9	4.0	6.1	5.6	1.9	4.0	7.5	10.6	12.0	10.6	9.4	8.9	8.7	4.0	100.0		
AV SPD	2.5	2.5	1.9	3.3	2.2	2.3	3.0	2.9	3.5	3.4	3.6	2.8	3.0	2.8	2.9	2.1			

AVERAGE SPEED FOR THIS TABLE EQUALS 3.0
HOURS IN ABOVE TABLE WITH VARIABLE DIRECTION = 5

TABLE 2.3-17

LIST OF COMPUTER RUNS

Run Number	Vent Identification	Data Used	Type of Run	Hourly or Joint Frequency Data Used	Grazing Season or Annual Data	To Be Used for Evaluating Releases from the Following Vent[(a)]	Location of Results in the Report
HX-1	Stack	4 years Hatch	Elevated release	Joint frequency	Annual	Stack	Tables 2.3-21 and 2.3-23
HX-2	Reactor building vent	4 years Hatch	Ground release in building wake	Joint frequency	Annual	Vent	Tables 2.3-22 and 2.3-24

a. See table 2.3-18.

TABLE 2.3-18

GASEOUS DISCHARGE POINTS (HNP-1 AND HNP-2)

System	Vent	Mode ID Number	Months Operating
Reactor building exhaust	Reactor building	1	All
Refueling floor exhaust	Reactor building	1	All
Turbine building exhaust	Reactor building	1	All
Radwaste building exhaust	Reactor building	1	All
Control building exhaust	Reactor building	1	All
Waste gas treatment	Stack	1	All
Off-gas	Stack	1	All
Gland-seal	Stack	1	All
Mechanical vacuum pump	Stack	1 (startup only)	All

TABLE 2.3-19

VENT DESIGN INFORMATION (HNP-1 AND HNP-2)

Vent	Location	Discharge Elevation Above Grade (m)	Height of Discharge Above Maximum Building Elevation (m)	Effective Vent Diameter (m)	Velocity at Point of Discharge (m/s)	Operating Mode Identification Number
Reactor bldg vent, Unit 1	Reactor bldg roof	49.7	3.1	4.6	5.9[a]	1
Reactor bldg vent, Unit 2	Reactor bldg	49.7	3.1	4.6	4.0[a]	1
Main stack	550-ft east of reactor bldg	120	73	0.81	4.1	1

a. Top-hat on vent prevents vertical discharge.

TABLE 2.3-20

TABULATION OF INPUT ASSUMPTIONS FOR CALCULATIONS

Parameter	Assumed Value or Characteristic
Height of meteorological instruments for stack runs	150-ft speed and direction, ΔT 150-35, speed adjusted to represent 393 ft
Height of meteorological instruments for ground-level releases	75-ft speed and direction, ΔT 150-35, 75-ft speed adjusted to 33 ft
Height of meteorological instruments for hourly wake-split runs	Not applicable to HNP
Height of meteorological instruments for wake-split runs using joint frequency tables	Not applicable to HNP
Method for determining stability and diffusion coefficients	Temperature difference using Regulatory Guide 1.23 and Pasquill curves
Calms treatment	Assumed 0.3 mph. Assumed to have same direction as measured
Upper limit for σ(m)	1000
Height of tallest structure for computation of Σ_{eff}(m)	46.6
Vent exit conditions	From table 2.3-19
Delta-temperature correction factor	0.56 prior to 9/72
Terrain height	See figure 2.3-12
Terrain correction factors	Figure 2 of Regulatory Guide 1.111

TABLE 2.3-21

ATMOSPHERIC DISPERSION FACTOR FOR HNP

VENT STACK SEASONAL-ANNUALREG-MX-1 JFT

RUN TYPE- X/Q SEC/M3 DISTANCE (METERS)

IN DIRECTION SECTOR	1250	2413	4022	5631	7240	12067	24135	40225	56315	72405
N	4.48E-08	3.30E-08	3.72E-08	3.42E-08	2.87E-08	1.35E-08	4.78E-09	2.70E-09	1.83E-09	1.36E-09
NNE	5.60E-08	4.65E-08	4.07E-08	3.44E-08	2.74E-08	1.34E-08	4.72E-09	2.62E-09	1.77E-09	1.30E-09
NE	6.85E-08	5.06E-08	3.85E-08	3.95E-08	3.08E-08	1.45E-08	5.12E-09	2.89E-09	1.97E-09	1.46E-09
ENE	6.79E-08	4.40E-08	2.92E-08	2.19E-08	2.18E-08	1.11E-08	4.00E-09	2.29E-09	1.57E-09	1.17E-09
E	7.31E-08	4.38E-08	2.80E-08	2.09E-08	1.68E-08	8.72E-09	3.32E-09	1.93E-09	1.33E-09	9.98E-10
ESE	8.67E-08	6.53E-08	4.32E-08	2.94E-08	2.26E-08	1.13E-08	4.06E-09	2.36E-09	1.63E-09	1.23E-09
SE	1.08E-07	7.91E-08	4.86E-08	3.63E-08	2.55E-08	1.18E-08	4.15E-09	2.40E-09	1.65E-09	1.24E-09
SSE	7.80E-08	6.37E-08	4.26E-08	2.93E-08	2.34E-08	1.06E-08	3.62E-09	2.04E-09	1.39E-09	1.03E-09
S	7.93E-08	6.67E-08	4.06E-08	2.68E-08	1.98E-08	8.95E-09	3.02E-09	1.69E-09	1.15E-09	8.49E-10
SSW	6.73E-08	5.28E-08	3.41E-08	2.48E-08	1.90E-08	8.89E-09	3.06E-09	1.73E-09	1.19E-09	8.79E-10
SW	9.41E-08	8.06E-08	4.82E-08	3.67E-08	2.58E-08	1.23E-08	4.30E-09	2.42E-09	1.64E-09	1.22E-09
WSW	9.60E-08	9.34E-08	7.09E-08	4.62E-08	3.46E-08	1.63E-08	5.53E-09	3.04E-09	2.04E-09	1.59E-09
W	1.11E-07	9.86E-08	7.88E-08	5.09E-08	3.75E-08	1.69E-08	5.78E-09	3.21E-09	2.17E-09	1.50E-09
WNW	7.94E-08	6.85E-08	3.27E-08	2.68E-08	2.00E-08	1.36E-08	4.78E-09	2.67E-09	1.81E-09	1.34E-09
NW	5.25E-08	3.71E-08	2.76E-08	2.77E-08	2.76E-08	1.47E-08	5.20E-09	2.91E-09	1.97E-09	1.45E-09
NNW	3.63E-08	3.57E-08	3.05E-08	2.84E-08	2.45E-08	1.15E-08	4.04E-09	2.30E-09	1.57E-09	1.16E-09

RUN TYPE- DEPLETED X/Q SEC/M3 DISTANCE (METERS)

IN DIRECTION SECTOR	1250	2413	4022	5631	7240	12067	24135	40225	56315	72405
N	4.31E-08	3.25E-08	3.54E-08	3.41E-08	2.70E-08	1.26E-08	4.42E-09	2.47E-09	1.66E-09	1.22E-09
NNE	5.39E-08	4.46E-08	4.42E-08	3.42E-08	2.56E-08	1.24E-08	4.27E-09	2.34E-09	1.55E-09	1.13E-09
NE	6.59E-08	4.86E-08	3.47E-08	3.71E-08	2.87E-08	1.33E-08	4.58E-09	2.54E-09	1.70E-09	1.25E-09
ENE	6.52E-08	4.21E-08	2.78E-08	2.06E-08	2.02E-08	1.01E-08	3.53E-09	1.97E-09	1.33E-09	9.76E-10
E	7.02E-08	4.17E-08	2.65E-08	1.96E-08	1.56E-08	7.99E-09	2.93E-09	1.65E-09	1.11E-09	8.17E-10
ESE	8.29E-08	6.19E-08	4.06E-08	2.73E-08	2.06E-08	1.02E-08	3.56E-09	2.02E-09	1.37E-09	1.01E-09
SE	1.03E-07	7.50E-08	4.52E-08	3.16E-08	2.32E-08	1.05E-08	3.56E-09	1.99E-09	1.34E-09	9.85E-10
SSE	7.45E-08	6.04E-08	4.00E-08	2.72E-08	2.15E-08	9.59E-09	3.18E-09	1.75E-09	1.17E-09	8.49E-10
S	7.60E-08	6.33E-08	3.79E-08	2.47E-08	1.79E-08	7.90E-09	2.54E-09	1.37E-09	8.99E-10	6.49E-10
SSW	6.44E-08	5.03E-08	3.20E-08	2.28E-08	1.73E-08	7.91E-09	2.63E-09	1.45E-09	9.66E-10	7.05E-10
SW	9.18E-08	7.67E-08	4.52E-08	3.21E-08	2.36E-08	1.10E-08	3.76E-09	2.06E-09	1.37E-09	9.94E-10
WSW	9.20E-08	8.95E-08	6.64E-08	4.27E-08	3.17E-08	1.47E-08	4.94E-09	2.60E-09	1.71E-09	1.23E-09
W	1.06E-07	9.37E-08	7.40E-08	4.72E-08	3.45E-08	1.53E-08	5.06E-09	2.73E-09	1.80E-09	1.30E-09
WNW	7.64E-08	6.65E-08	3.11E-08	2.36E-08	1.87E-08	1.25E-08	4.26E-09	2.33E-09	1.55E-09	1.12E-09
NW	5.05E-08	3.57E-08	2.65E-08	2.62E-08	2.59E-08	1.37E-08	4.75E-09	2.62E-09	1.75E-09	1.29E-09
NNW	3.49E-08	3.43E-08	3.77E-08	2.69E-08	2.30E-08	1.07E-08	3.72E-09	2.09E-09	1.41E-09	1.04E-09

RUN TYPE- DEPOSITION D/Q M-2 DISTANCE (METERS)

IN DIRECTION SECTOR	1250	2413	4022	5631	7240	12067	24135	40225	56315	72405
N	4.54E-09	1.84E-09	6.81E-10	3.38E-10	2.07E-10	7.12E-11	1.63E-11	7.04E-12	4.04E-12	2.63E-12
NNE	7.09E-09	2.90E-09	1.05E-09	5.06E-10	3.10E-10	1.07E-10	2.46E-11	1.07E-11	6.10E-12	3.95E-12
NE	8.53E-09	3.43E-09	1.22E-09	6.13E-10	3.77E-10	1.32E-10	3.03E-11	1.31E-11	7.49E-12	4.84E-12
ENE	9.51E-09	3.57E-09	1.22E-09	5.99E-10	3.66E-10	1.28E-10	2.99E-11	1.32E-11	7.64E-12	4.96E-12
E	1.09E-08	3.68E-09	1.29E-09	6.20E-10	3.77E-10	1.33E-10	3.16E-11	1.43E-11	8.47E-12	5.59E-12
ESE	1.34E-08	4.69E-09	1.50E-09	7.12E-10	4.34E-10	1.51E-10	3.57E-11	1.61E-11	9.37E-12	6.10E-12
SE	1.43E-08	4.88E-09	1.55E-09	7.31E-10	4.46E-10	1.55E-10	3.66E-11	1.66E-11	9.55E-12	6.20E-12
SSE	8.90E-09	3.22E-09	1.03E-09	4.86E-10	2.96E-10	1.03E-10	2.41E-11	1.09E-11	6.24E-12	4.05E-12
S	7.17E-09	2.69E-09	8.66E-10	4.11E-10	2.51E-10	8.77E-11	2.06E-11	9.16E-12	5.31E-12	3.45E-12
SSW	6.37E-09	2.39E-09	7.90E-10	3.79E-10	2.32E-10	8.06E-11	1.88E-11	8.29E-12	4.77E-12	3.09E-12
SW	8.93E-09	3.22E-09	1.04E-09	4.98E-10	3.05E-10	1.06E-10	2.45E-11	1.08E-11	6.19E-12	4.09E-12
WSW	1.00E-08	3.97E-09	1.34E-09	6.35E-10	3.90E-10	1.36E-10	3.10E-11	1.35E-11	7.75E-12	5.01E-12
W	1.39E-08	5.11E-09	1.67E-09	7.83E-10	4.78E-10	1.66E-10	3.89E-11	1.73E-11	9.98E-12	6.47E-12
WNW	9.33E-09	3.48E-09	1.18E-09	5.81E-10	3.55E-10	1.24E-10	2.89E-11	1.28E-11	7.41E-12	4.80E-12
NW	6.13E-09	2.39E-09	8.31E-10	4.15E-10	2.56E-10	8.89E-11	2.05E-11	8.94E-12	5.13E-12	3.33E-12
NNW	3.99E-09	1.68E-09	6.06E-10	2.91E-10	1.80E-10	6.18E-11	1.41E-11	6.13E-12	3.51E-12	2.28E-12

TABLE 2.3-22

ATMOSPHERIC DISPERSION FACTORS FOR HNP

VENT-REACTOR BLDG (GROUND) SEASON-ANNUAL REQ-HX-2 4 YRS JFT

RUN TYPE- X/Q SEC/M3 DISTANCE (METERS)

IN DIRECTION SECTOR	1250	2413	4022	5631	7240	12067	24135	40225	56315	72405
N	6.87E-06	2.16E-06	7.07E-07	3.46E-07	2.38E-07	[illegible]	2.59E-08	1.30E-08	8.53E-09	6.21E-09
NNE	9.71E-06	2.73E-06	8.88E-07	4.59E-07	2.98E-07	1.11E-07	3.23E-08	1.62E-08	1.06E-08	7.72E-09
NE	9.38E-06	2.93E-06	9.58E-07	4.96E-07	3.23E-07	1.21E-07	3.53E-08	1.78E-08	1.16E-08	8.49E-09
ENE	7.42E-06	2.32E-06	7.65E-07	3.98E-07	2.60E-07	[illegible]	2.88E-08	1.46E-08	9.58E-09	7.01E-09
E	8.14E-06	2.59E-06	8.59E-07	4.50E-07	2.95E-07	1.12E-07	3.33E-08	1.69E-08	1.12E-08	8.23E-09
ESE	7.77E-06	2.44E-06	8.11E-07	4.23E-07	2.77E-07	1.05E-07	3.11E-08	1.59E-08	1.04E-08	7.64E-09
SE	7.88E-06	2.48E-06	8.25E-07	4.32E-07	2.83E-07	1.07E-07	3.20E-08	1.63E-08	1.07E-08	7.89E-09
SSE	6.13E-06	1.93E-06	6.40E-07	3.34E-07	2.19E-07	8.27E-08	2.46E-08	1.25E-08	8.24E-09	6.05E-09
S	6.42E-06	2.01E-06	6.61E-07	3.46E-07	2.25E-07	8.43E-08	2.49E-08	1.26E-08	8.27E-09	6.06E-09
SSW	5.80E-06	1.80E-06	5.86E-07	3.03E-07	1.97E-07	7.31E-08	2.13E-08	1.07E-08	7.00E-09	5.09E-09
SW	7.15E-06	2.24E-06	7.33E-07	3.80E-07	2.48E-07	9.24E-08	2.72E-08	1.37E-08	9.01E-09	6.58E-09
WSW	7.58E-06	2.36E-06	7.58E-07	3.89E-07	2.53E-07	9.29E-08	2.68E-08	1.34E-08	8.73E-09	6.33E-09
W	8.79E-06	2.75E-06	8.92E-07	4.59E-07	2.99E-07	1.10E-07	3.19E-08	1.60E-08	1.05E-08	7.61E-09
WNW	7.84E-06	2.48E-06	7.99E-07	4.10E-07	2.66E-07	9.79E-08	2.81E-08	1.41E-08	9.16E-09	6.64E-09
NW	7.16E-06	2.25E-06	7.28E-07	3.74E-07	2.43E-07	8.97E-08	2.58E-08	1.29E-08	8.43E-09	6.12E-09
NNW	6.19E-06	1.97E-06	6.49E-07	3.29E-07	2.14E-07	7.89E-08	2.27E-08	1.14E-08	7.42E-09	5.39E-09

RUN TYPE- DEPLETED X/Q SEC/M3 DISTANCE (METERS)

IN DIRECTION SECTOR	1250	2413	4022	5631	7240	12067	24135	40225	56315	72405
N	5.63E-06	1.64E-06	5.01E-07	2.44E-07	1.53E-07	5.16E-08	1.25E-08	5.29E-09	3.02E-09	1.97E-09
NNE	7.14E-06	2.07E-06	6.29E-07	3.04E-07	1.91E-07	6.45E-08	1.56E-08	6.59E-09	3.76E-09	2.45E-09
NE	7.49E-06	2.22E-06	6.78E-07	3.31E-07	2.07E-07	7.01E-08	1.71E-08	7.22E-09	4.12E-09	2.69E-09
ENE	6.09E-06	1.76E-06	5.42E-07	2.65E-07	1.65E-07	5.69E-08	1.39E-08	5.90E-09	3.39E-09	2.22E-09
E	6.68E-06	1.96E-06	6.08E-07	3.00E-07	1.89E-07	6.50E-08	1.61E-08	6.87E-09	3.96E-09	2.61E-09
ESE	6.38E-06	1.86E-06	5.74E-07	2.82E-07	1.77E-07	6.09E-08	1.50E-08	6.41E-09	3.69E-09	2.42E-09
SE	6.47E-06	1.88E-06	5.84E-07	2.88E-07	1.81E-07	6.23E-08	1.55E-08	6.60E-09	3.80E-09	2.51E-09
SSE	5.03E-06	1.46E-06	4.53E-07	2.23E-07	1.40E-07	4.81E-08	1.19E-08	5.07E-09	2.92E-09	1.92E-09
S	5.26E-06	1.53E-06	4.68E-07	2.30E-07	1.44E-07	4.90E-08	1.20E-08	5.11E-09	2.93E-09	1.92E-09
SSW	4.75E-06	1.37E-06	4.15E-07	2.02E-07	1.26E-07	4.25E-08	1.03E-08	4.35E-09	2.48E-09	1.62E-09
SW	5.87E-06	1.70E-06	5.19E-07	2.53E-07	1.59E-07	5.39E-08	1.32E-08	5.57E-09	3.19E-09	2.09E-09
WSW	6.21E-06	1.79E-06	5.37E-07	2.60E-07	1.62E-07	5.41E-08	1.29E-08	5.44E-09	3.09E-09	2.01E-09
W	7.21E-06	2.09E-06	6.31E-07	3.06E-07	1.91E-07	6.42E-08	1.54E-08	6.51E-09	3.71E-09	2.41E-09
WNW	6.47E-06	1.88E-06	5.65E-07	2.74E-07	1.70E-07	5.69E-08	1.36E-08	5.71E-09	3.24E-09	2.11E-09
NW	5.87E-06	1.71E-06	5.14E-07	2.50E-07	1.56E-07	5.22E-08	1.25E-08	5.25E-09	2.99E-09	1.94E-09
NNW	5.08E-06	1.50E-06	4.53E-07	2.20E-07	1.37E-07	4.59E-08	1.10E-08	4.62E-09	2.63E-09	1.71E-09

RUN TYPE- DEPOSITION D/Q M-2 DISTANCE (METERS)

IN DIRECTION SECTOR	1250	2413	4022	5631	7240	12067	24135	40225	56315	72405
N	3.14E-08	7.73E-09	2.04E-09	9.27E-10	5.34E-10	1.69E-10	3.48E-11	1.28E-11	6.48E-12	4.02E-12
NNE	4.59E-08	1.13E-08	2.98E-09	1.36E-09	7.83E-10	2.48E-10	5.09E-11	1.87E-11	9.49E-12	5.89E-12
NE	5.23E-08	1.29E-08	3.39E-09	1.54E-09	8.90E-10	2.82E-10	5.79E-11	2.13E-11	1.08E-11	6.70E-12
ENE	3.83E-08	9.45E-09	2.49E-09	1.13E-09	6.53E-10	2.07E-10	4.25E-11	1.56E-11	7.92E-12	4.92E-12
E	4.23E-08	1.04E-08	2.75E-09	1.25E-09	7.21E-10	2.28E-10	4.69E-11	1.72E-11	8.74E-12	5.43E-12
ESE	4.56E-08	1.12E-08	2.96E-09	1.35E-09	7.77E-10	2.46E-10	5.06E-11	1.86E-11	9.43E-12	5.85E-12
SE	4.08E-08	1.01E-08	2.65E-09	1.21E-09	6.95E-10	2.20E-10	4.52E-11	1.66E-11	8.43E-12	5.23E-12
SSE	2.68E-08	6.61E-09	1.74E-09	7.92E-10	4.57E-10	1.45E-10	2.97E-11	1.09E-11	5.54E-12	3.44E-12
S	2.44E-08	6.02E-09	1.59E-09	7.22E-10	4.16E-10	1.32E-10	2.71E-11	9.96E-12	5.05E-12	3.13E-12
SSW	2.43E-08	5.99E-09	1.58E-09	7.19E-10	4.14E-10	1.31E-10	2.69E-11	9.90E-12	5.02E-12	3.12E-12
SW	3.30E-08	8.14E-09	2.14E-09	9.76E-10	5.63E-10	1.78E-10	3.66E-11	1.35E-11	6.82E-12	4.24E-12
WSW	4.19E-08	1.03E-08	2.72E-09	1.24E-09	7.13E-10	2.26E-10	4.64E-11	1.71E-11	8.65E-12	5.37E-12
W	4.38E-08	1.08E-08	2.84E-09	1.29E-09	7.45E-10	2.36E-10	4.85E-11	1.78E-11	9.04E-12	5.61E-12
WNW	3.79E-08	9.33E-09	2.46E-09	1.12E-09	6.45E-10	2.04E-10	4.20E-11	1.54E-11	7.82E-12	4.86E-12
NW	3.36E-08	8.28E-09	2.18E-09	9.92E-10	5.72E-10	1.81E-10	3.72E-11	1.37E-11	6.94E-12	4.31E-12
NNW	2.57E-08	6.34E-09	1.67E-09	7.60E-10	4.38E-10	1.39E-10	2.85E-11	1.05E-11	5.31E-12	3.30E-12

TABLE 2.3-23 (SHEET 1 OF 2)

DIFFUSION AND DEPOSITION ESTIMATES FOR ALL RECEPTOR LOCATIONS

Release Point: Stack | Season: Annual | Computer Run ID: HX-1

Direction	Distance to Nearest Milk Cow (m)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})	Distance to Nearest Meat Animal (m)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})	Distance to Nearest Milk Goat (m^3)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})
N	-[(a)]	2.5E-08	N/A[(b)]	1.7E-10	2574-4020	3.7E-08	N/A	1.6E-09	-	2.5E-09	N/A	1.7E-10
NNE	4827	4.2E-08		6.9E-10	4660	4.4E-08		7.3E-10	-	2.4E-08		2.5E-10
NE	-	2.7E-08		3.1E-10	5310	4.0E-08		6.8E-10	-	2.7E-08		3.1E-10
ENE	-	2.0E-08		3.0E-10	6760	2.2E-08		4.2E-10	-	2.0E-08		3.0E-10
E	-	1.5E-08		3.1E-10	-	1.5E-08		3.1E-10	-	1.5E-08		3.1E-10
ESE	-	2.0E-08		3.6E-10	5960	2.8E-08		6.4E-10	-	2.0E-08		3.6E-10
SE	6918	2.6E-08		4.7E-10	3700-4180	5.0E-08		1.8E-09	-	2.2E-08		3.6E-10
SSE	7080	2.4E-08		3.1E-10	3380	5.3E-08		1.5E-09	-	2.1E-08		2.4E-10
S	7400	1.8E-08		2.3E-10	2570-4340	6.3E-09		2.3E-09	-	1.7E-08		2.1E-10
SSW	-	1.6E-08		1.9E-10	3050	4.4E-08		1.4E-09	-	1.6E-08		1.9E-10
SW	6918	2.6E-08		3.2E-10	2090-4670	8.6E-08		4.2E-09	-	2.3E-08		2.5E-10
WSW	6275	4.1E-08		5.2E-10	1930	9.4E-08		5.8E-09	-	3.0E-08		3.2E-10
W	-	3.2E-08		3.9E-10	2730-4500	1.1E-07		4.0E-09	-	3.2E-08		3.9E-10
WNW	-	1.8E-08		2.9E-10	-	1.8E-08		3.0E-10	-	1.8E-08		2.9E-10
NW	-	2.4E-08		2.1E-10	7080	2.8E-08		2.6E-10	-	2.4E-08		2.1E-10
NNW	8000	2.1E-08		1.4E-10	4340-4670	3.7E-08		5.0E-10	-	2.1E-08		1.4E-10

a. (-) indicates receptor distance is greater than 8000 m; diffusion values given are for 8000 m.
b. N/A indicates that diffusion information for this run was not used in dose calculation for receptors in this column.

TABLE 2.3-23 (SHEET 2 OF 2)

Direction	Distance to Residences (m)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})	Distance to Vegetable Garden (m)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})	Nearest Site Boundary (m^3)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})
N	2574-4800	4.0E-08	3.7E-08	1.7E-09	2574-4800	4.0E-08	N/A	1.7E-09	1638	4.0E-08	3.8E-08	3.5E-09
NNE	4827	4.1E-08	4.0E-08	6.9E-10	4700-4820	4.1E-08		7.6E-10	1950	4.4E-08	4.1E-08	3.9E-09
NE	4505	3.9E-08	3.7E-08	9.1E-10	5470	4.0E-08		6.6E-10	1882	5.5E-08	5.3E-08	5.1E-09
ENE	7884	2.0E-08	1.8E-08	3.1E-10	8000	2.0E-08		3.1E-10	1547	5.8E-08	5.6E-08	7.4E-09
E	8000	1.5E-08	1.4E-08	3.1E-10	8000	1.5E-08		3.1E-10	1402	6.7E-08	6.4E-08	9.4E-09
ESE	4660	3.7E-08	3.4E-08	1.1E-09	4660	3.7E-08		1.1E-09	1753	7.9E-08	7.4E-08	8.5E-09
SE	3200-4800	5.7E-08	5.4E-08	2.5E-09	2090-4800	8.8E-08		7.0E-09	1814	9.5E-08	9.0E-08	8.5E-09
SSE	2090[3]-4800	6.5E-08	6.2E-08	4.5E-09	2413-4800	6.3E-08		3.2E-09	1530	7.2E-08	6.9E-08	7.1E-09
S	1760-4800	7.2E-08	6.8E-08	4.9E-09	1760-4800	7.2E-08		4.9E-09	1554	7.3E-08	7.0E-08	5.6E-09
SSW	3050-4800	4.4E-08	4.2E-08	1.5E-09	3540-4800	3.7E-08		1.4E-09	1585	6.2E-08	5.9E-08	5.6E-09
SW	1600-4800	9.7E-08	9.2E-08	6.7E-09	1930-4800	9.1E-08		4.6E-09	1410	9.8E-08	9.4E-08	7.9E-09
WSW	1760-4800	9.4E-08	9.0E-08	6.5E-09	1760-4800	9.4E-08		6.5E-09	1516	9.3E-08	8.9E-08	8.3E-09
W	2090-4800	1.0E-07	9.1E-08	6.5E-09	2090-4800	1.0E-07		6.5E-09	1501	1.0E-07	9.8E-08	1.1E-08
WNW	7884	1.8E-08	1.7E-08	2.9E-10	7884	1.8E-08		2.9E-10	1524	6.8E-08	6.5E-08	7.5E-09
NW	6275	3.0E-08	2.8E-08	3.3E-10	6275	3.0E-08		3.3E-10	1570	4.3E-08	4.3E-08	4.8E-09
NNW	3050-4800	4.1E-08	3.8E-09	1.1E-09	3378-4800	4.1E-08		8.5E-10	1646	3.1E-08	3.0E-08	2.9E-09

TABLE 2.3-24 (SHEET 1 OF 2)

DIFFUSION AND DEPOSITION ESTIMATES FOR ALL RECEPTOR LOCATIONS

Release Point: Assumed ground in building wake | Season: Annual | Computer Run ID: HX-1 604-29

Direction	Distance to Nearest Milk Cow (m)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})	Distance to Nearest Meat Animal (m)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})	Distance to Nearest Milk Goat (m^3)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})
N	-[a]	2.0E-07	N/A[b]	4.3E-10	2574	1.9E-06	N/A	6.5E-09	-	2.0E-07	N/A	4.3E-10
NNE	4827	6.1E-07		1.9E-09	4660	6.4E-07		2.0E-09	-	2.5E-07		6.3E-10
NE	-	2.7E-07		7.2E-10	5310	5.4E-07		1.7E-09	-	2.7E-07		7.2E-10
ENE	-	2.2E-07		5.3E-10	6760	2.9E-07		7.6E-10	-	2.2E-07		5.3E-10
E	-	2.5E-07		5.8E-10	-	2.5E-07		5.8E-10	-	2.5E-07		5.8E-10
ESE	-	2.3E-07		6.3E-10	5960	3.9E-07		1.2E-09	-	2.3E-07		6.3E-10
SE	6918	3.0E-07		7.4E-10	3700	9.6E-07		3.2E-09	-	2.4E-07		5.6E-10
SSE	7080	2.3E-07		4.9E-10	3380	8.9E-07		2.6E-09	-	1.8E-07		3.7E-10
S	7400	2.1E-07		3.9E-10	2570	1.7E-06		5.2E-09	-	1.9E-07		3.4E-10
SSW	-	1.6E-07		3.4E-10	3050	1.0E-06		3.1E-09	-	1.6E-07		3.4E-10
SW	6918	2.6E-07		6.0E-10	2090	3.0E-06		1.1E-08	-	2.1E-07		4.6E-10
WSW	6275	3.21E-07		1.0E-09	1930	3.6E-06		1.7E-08	-	2.1E-07		5.8E-10
W	-	2.5E-07		6.0E-10	2730	2.1E-06		7.6E-09	-	2.5E-07		6.0E-10
WNW	-	2.2E-07		5.2E-10	-	2.2E-07		5.2E-10	-	1.2E-07		5.2E-10
NW	-	2.0E-07		4.6E-10	7080	2.5E-07		6.0E-10	-	2.0E-07		4.6E-10
NNW	8000	1.8E-07		3.6E-10	4340	5.6E-07		1.4E-09	-	1.8E-07		3.6E-10

a. (-) indicates receptor distance is greater than 8000 m; diffusion values given are for 8000 m.
b. N/A indicates that diffusion information for this run was not used in dose calculation for receptors in this column.

TABLE 2.3-24 (SHEET 2 OF 2)

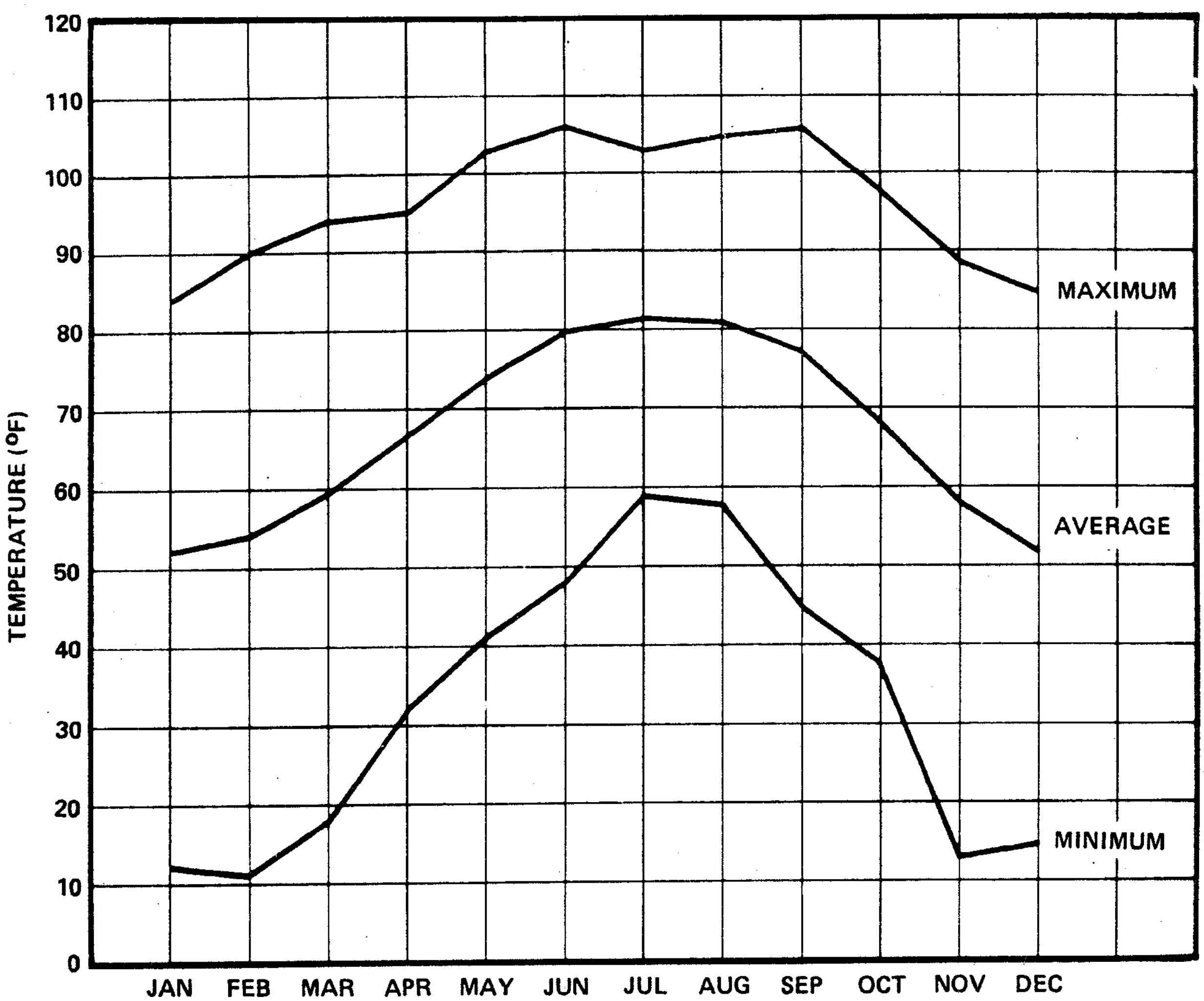
Direction	Distance to Nearest Milk Cow (m)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})	Distance to Vegetable Garden (m)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})	Nearest Site Boundary (m^3)	X/Q (s/m^3)	Depleted X/Q (s/m^3)	D/Q (m^{-2})
N	2574	1.8E-06	1.4E-06	5.7E-09	2574	1.9E-06	N/A	5.7E-09	1638	4.7E-06	3.7E-06	2.0E-06
NNE	4827	6.1E-07	4.2E-07	1.9E-09	4700	6.4E-07		2.0E-09	1950	4.2E-06	3.2E-06	1.9E-08
NE	4505	7.5E-07	5.2E-07	2.7E-09	5470	5.3E-07		1.6E-09	1882	4.8E-06	3.7E-06	2.3E-08
ENE	7884	2.2E-07	1.4E-07	5.6E-10	8000	2.2E-07		5.3E-10	1547	5.4E-06	4.3E-06	2.7E-08
E	8000	2.5E-07	1.5E-07	5.8E-10	8000	2.5E-07		5.8E-10	1402	6.9E-06	5.6E-06	3.5E-08
ESE	4660	6.0E-07	4.2E-07	2.1E-09	4660	6.1E-07		2.1E-09	1753	4.6E-06	3.7E-06	2.5E-08
SE	3200	1.3E-06	9.4E-07	4.5E-09	2090	3.3E-06		1.4E-09	1814	4.4E-06	3.5E-06	2.0E-08
SSE	2090	2.8E-06	2.0E-06	9.3E-09	2413	1.9E-06		6.7E-09	1530	4.6E-06	3.7E-06	1.9E-08
S	1760	3.8E-06	3.0E-06	1.3E-08	1760	3.8E-06		1.3E-06	1554	4.7E-06	3.8E-06	1.7E-08
SSW	3050	1.0E-06	7.7E-07	3.0E-09	3540	1.1E-06		2.1E-09	1585	4.0E-06	3.2E-06	1.6E-08
SW	1600	5.0E-06	4.0E-06	2.25-08	1930	3.6E-06		1.4E-08	1410	6.0E-06	4.9E-06	2.7E-08
WSW	1760	4.4E-06	3.5E-06	2.2E-08	1760	4.5E-06		2.2E-06	1516	5.8E-06	4.7E-06	3.1E-08
W	2090	3.7E-06	2.9E-06	1.4E-08	2090	3.7E-06		1.4E-08	1501	6.8E-06	5.4E-06	3.2E-08
WNW	7884	2.3E-07	1.5E-07	5.2E-10	7884	2.3E-07		5.2E-10	1524	6.0E-06	4.8E-06	2.7E-08
NW	6275	3.2E-07	2.1E-07	7.8E-10	6275	3.2E-07		7.8E-10	1570	5.1E-06	4.1E-06	2.3E-08
NNW	3050	1.2E-06	8.4E-07	3.4E-09	3378	9.5E-07		2.5E-09	1646	4.2E-06	3.3E-06	1.6E-08

TABLE 2.3-25

X/Q VALUES AT MCR AIR INTAKE AND TSC INTAKE[a] (s/m^3)

	Release Point	
Averaging Time	Reactor Building Vent	Main Stack
0 - 2 h	1.41E-03	3.76E-06
2 - 8 h	1.08E-03	2.88E-06
8 - 24 h	4.70E-04	7.50E-07
1 - 4 days	3.54E-04	7.67E-07
4 - 30 days	2.67E-04	5.04E-07

a. Values for MCR air intake bound those for the TSC air intake.



GLENNVILLE-24 MILES EAST OF SITE

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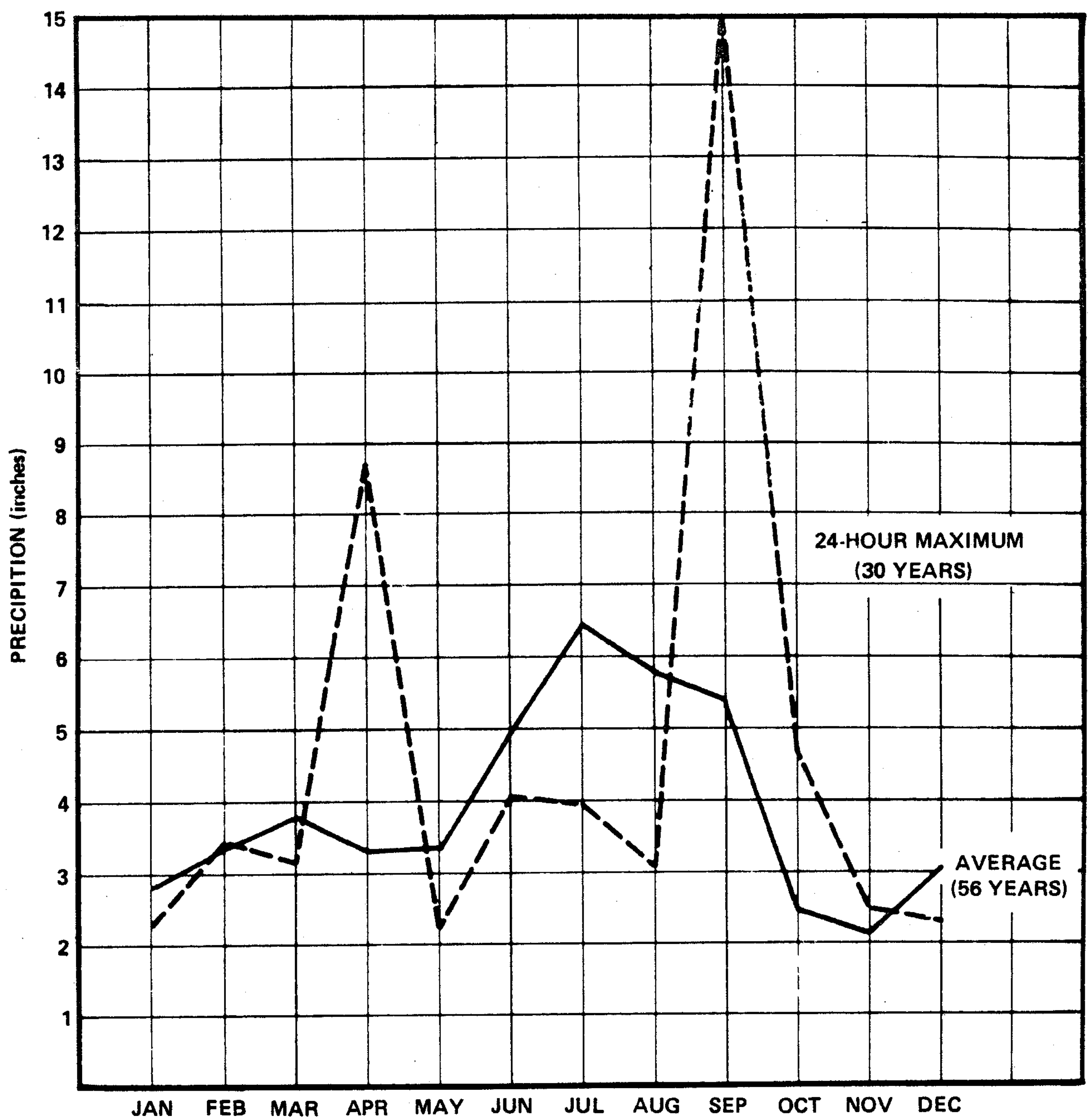
MONTHLY MAX., AVG., AND MIN.
TEMPERATURE FOR GLENNVILLE, GA.

FIGURE 2.3-1



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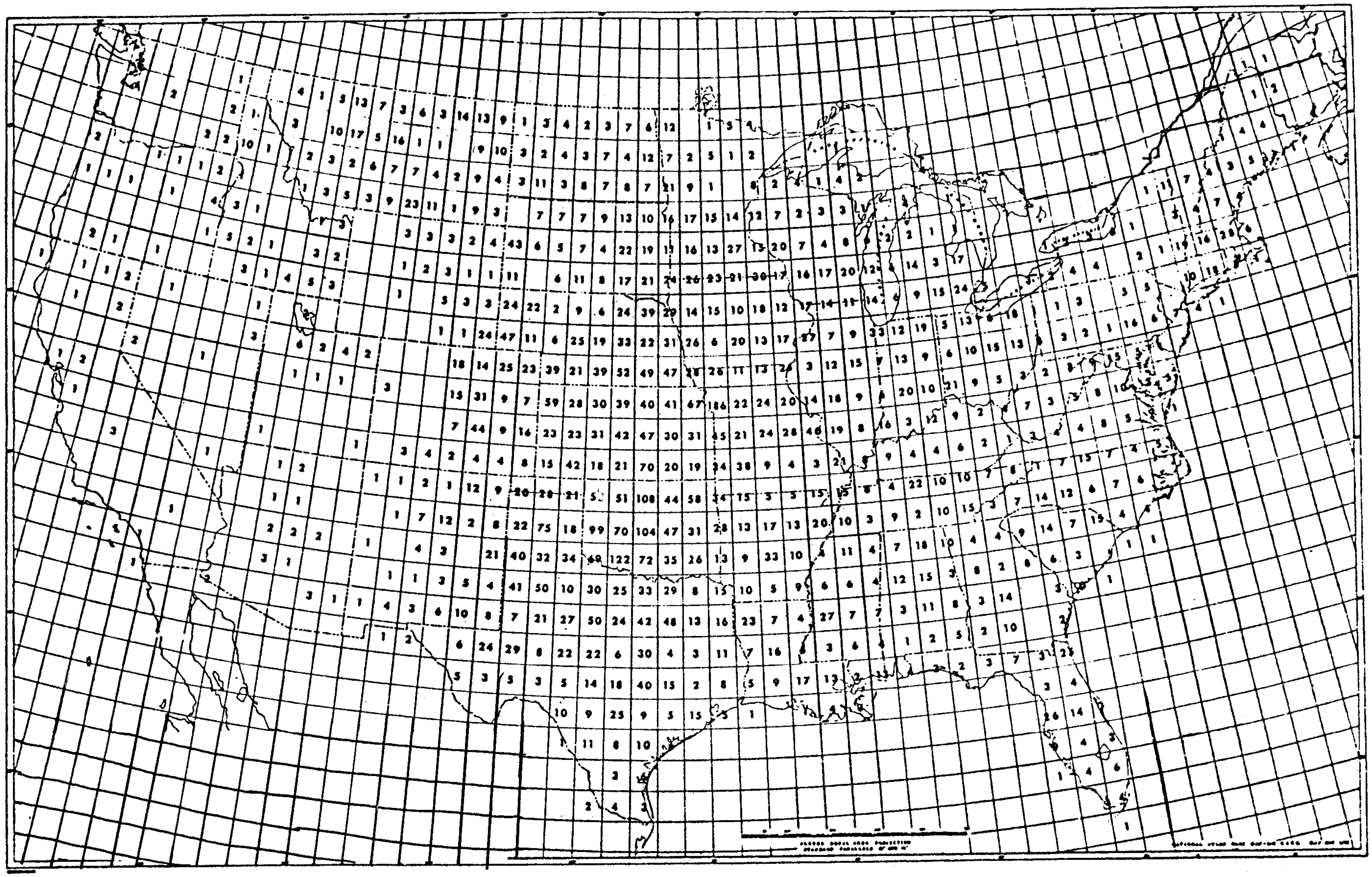
MONTHLY AVG. AND 24-h
PRECIPITATION FOR GLENNVILLE, GA.

FIGURE 2.3-2



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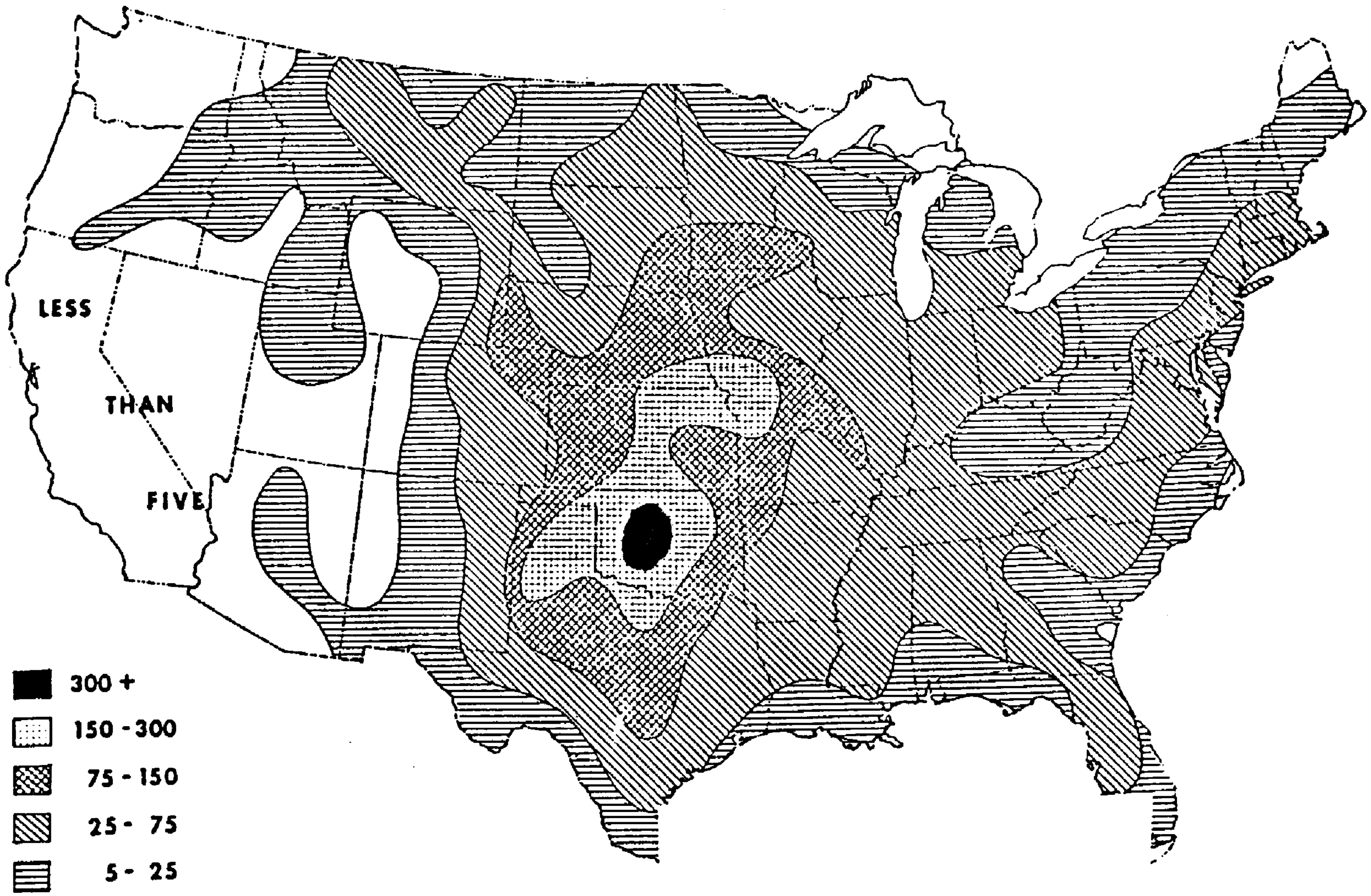
TOTAL NUMBER OF HAIL REPORTS ¾ in. AND GREATER, 1955-1967, BY 1 DEGREE SQUARES (BASED ON SELS LOG)

FIGURE 2.3-3



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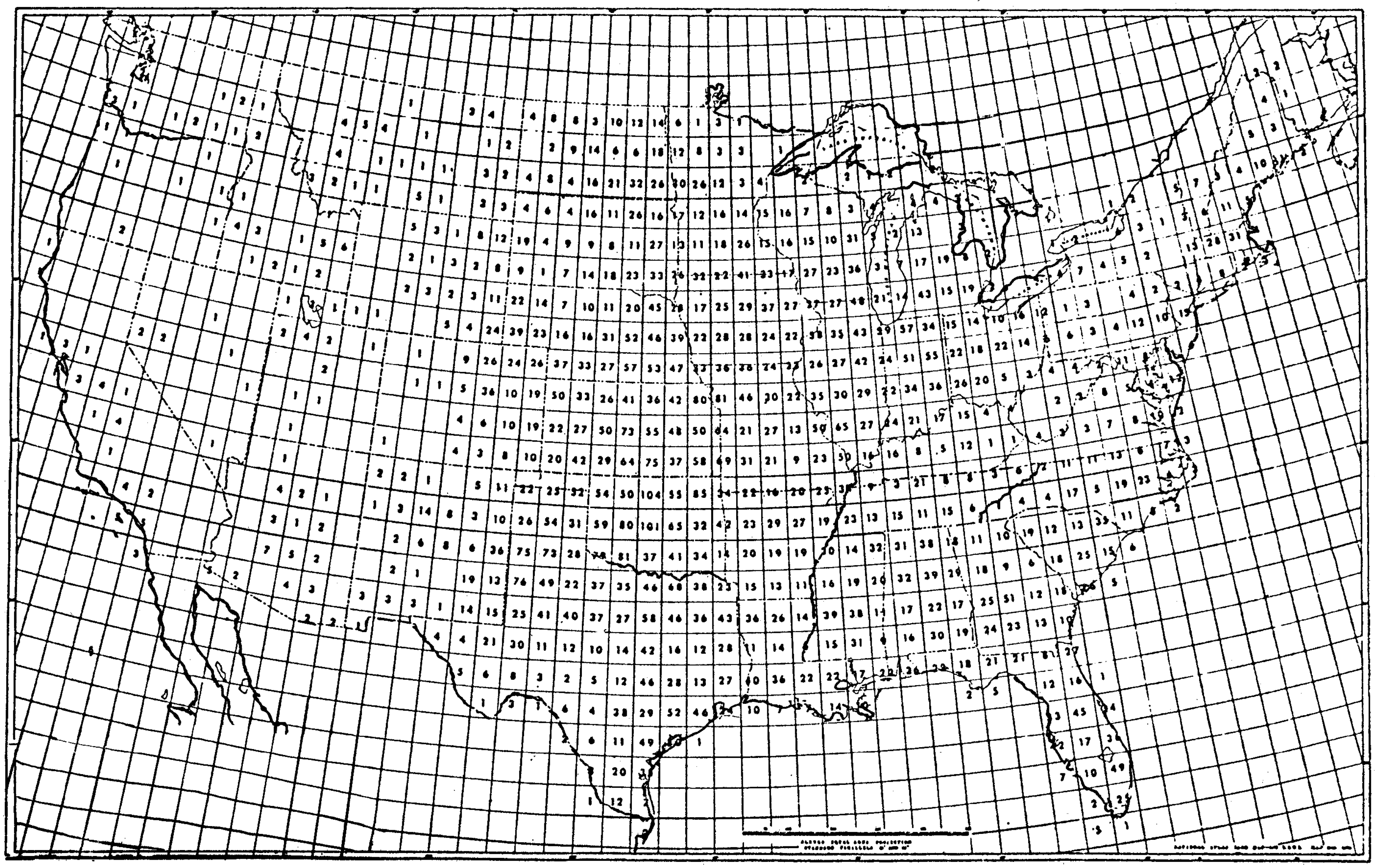
TOTAL NUMBER OF HAIL REPORTS ¾ in. AND GREATER, 1955-1967, BY 2 DEGREE SQUARES (BASED ON SELS LOGS)

FIGURE 2.3-4



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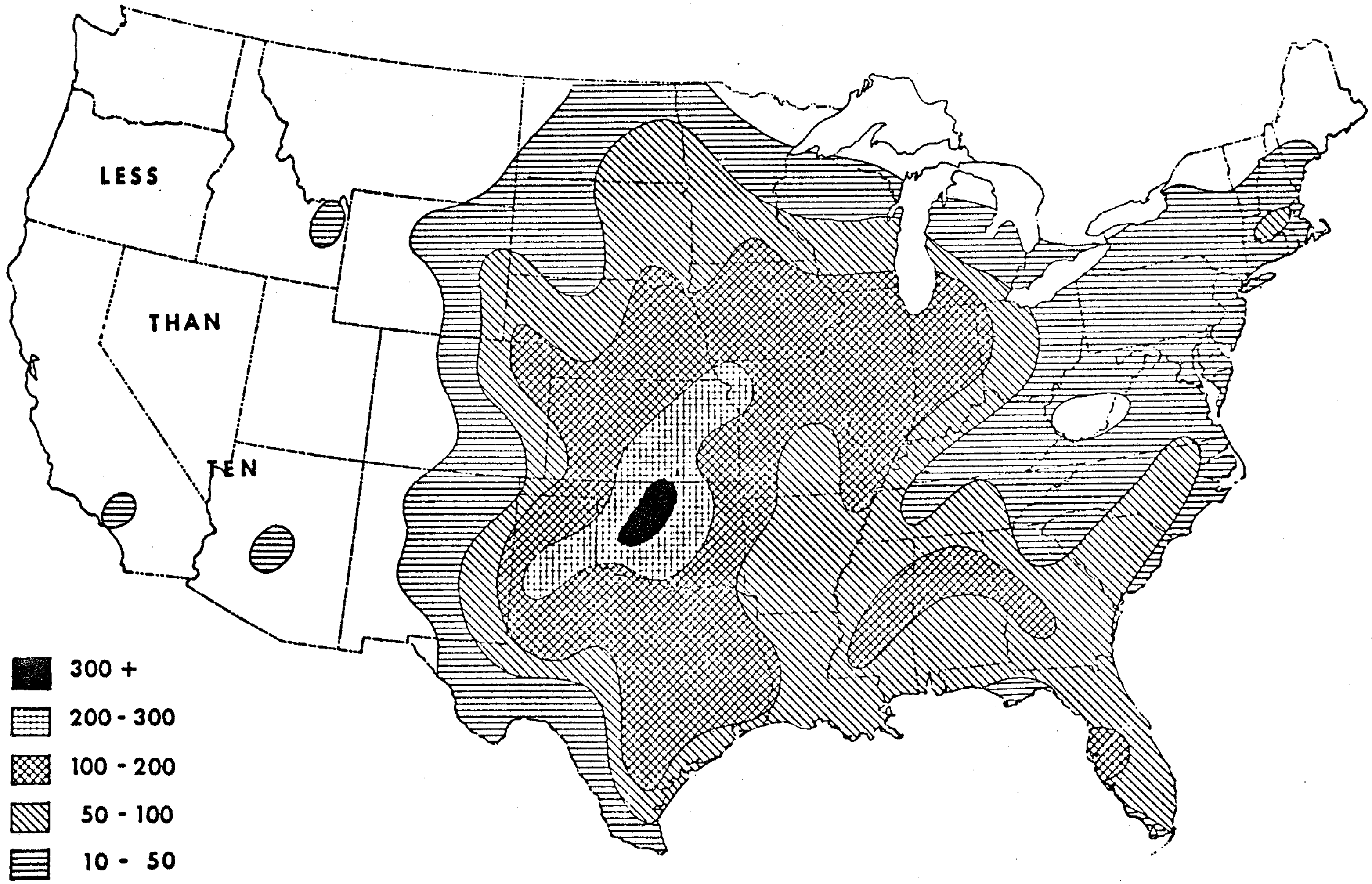
TOTAL TORNADOES, 1955-1967, BY 1 DEGREE SQUARES
(BASED ON SELS LOG)

FIGURE 2.3-5



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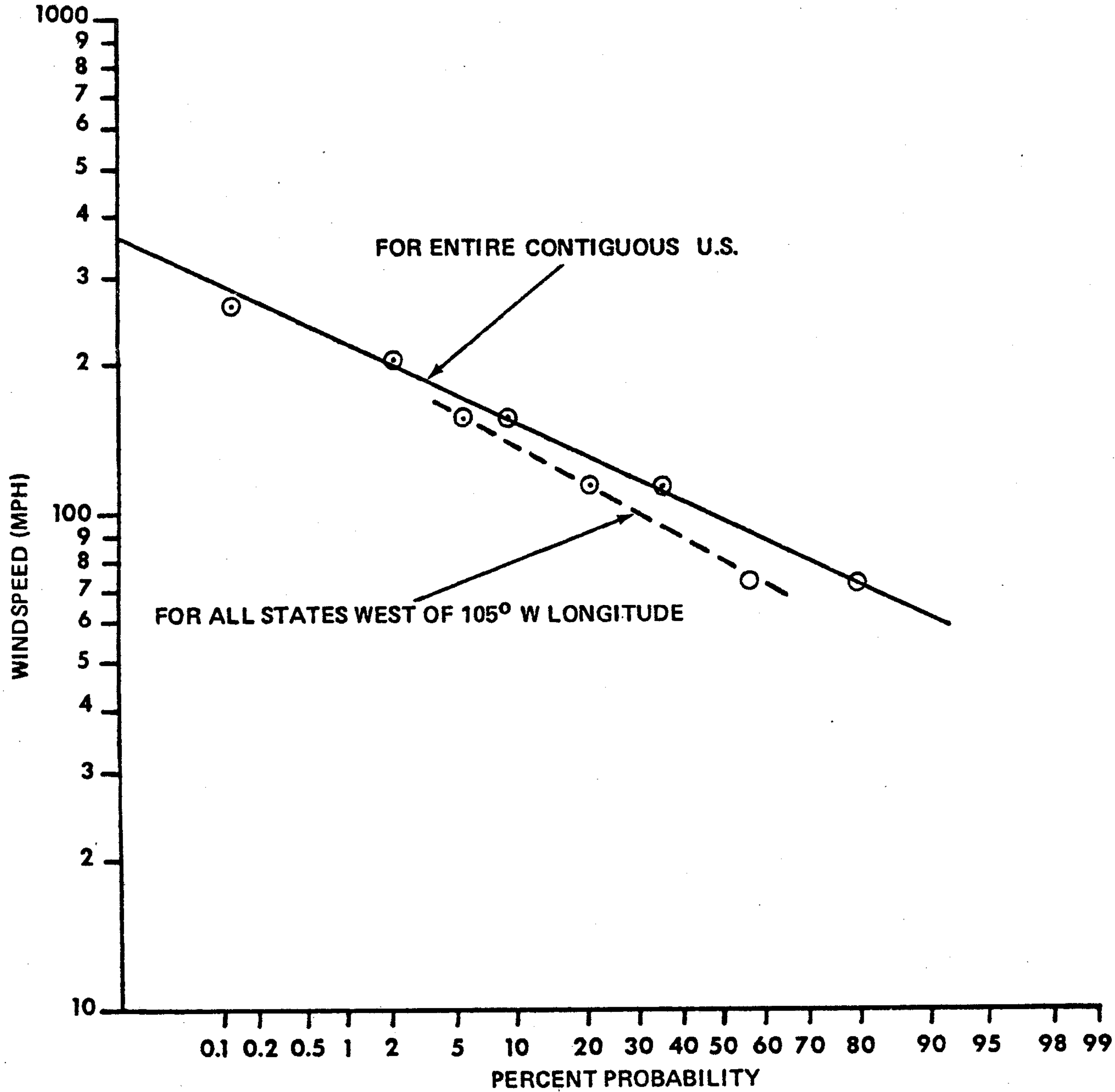
TOTAL TORNADOES, 1955-1967, BY 2 DEGREE SQUARES
(BASED ON SELS LOG)

FIGURE 2.3-6



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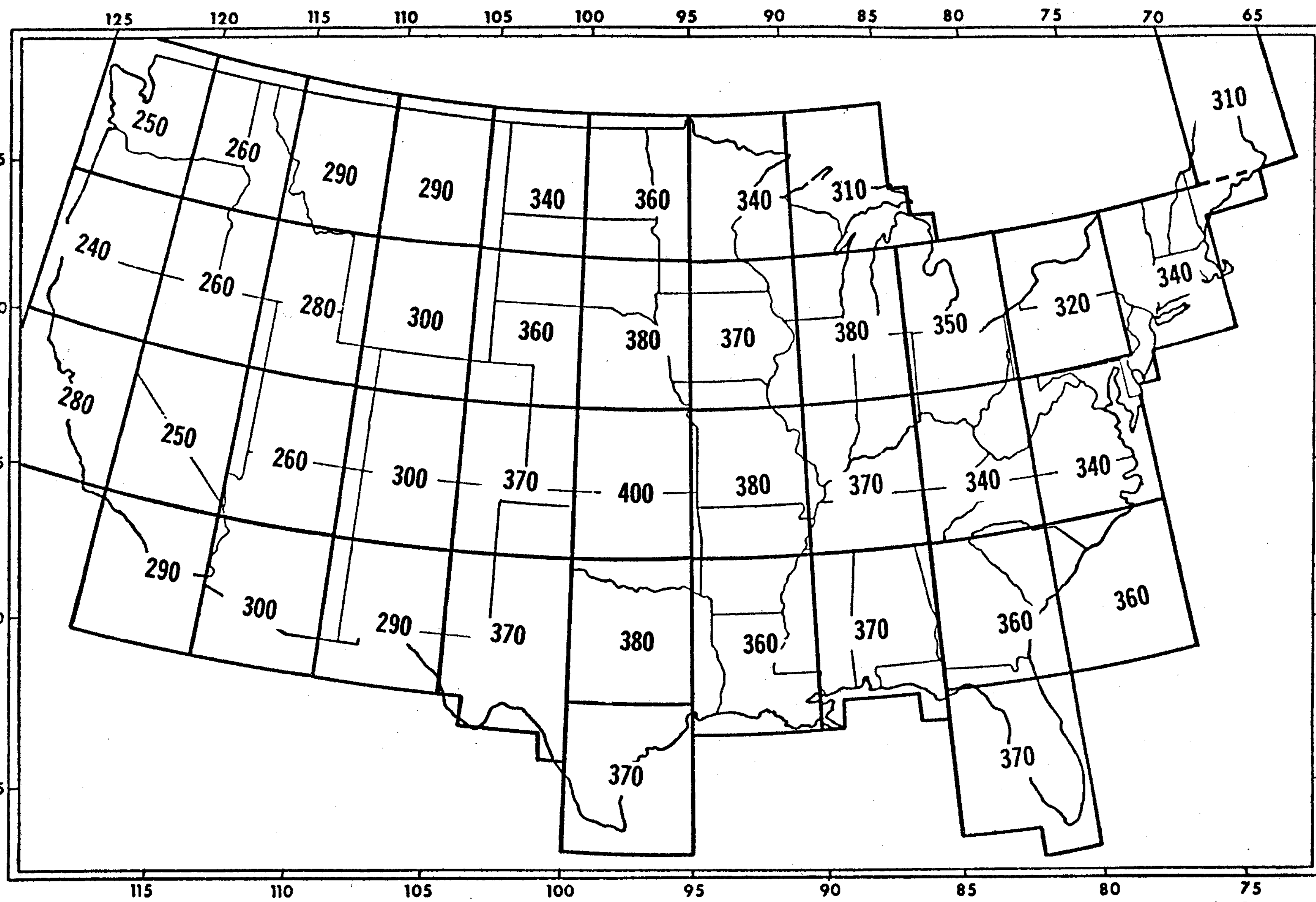
PERCENT OF PROBABILITY OF EXCEEDING ORDINATE VALUE OF THE WINDSPEED

FIGURE 2.3-7



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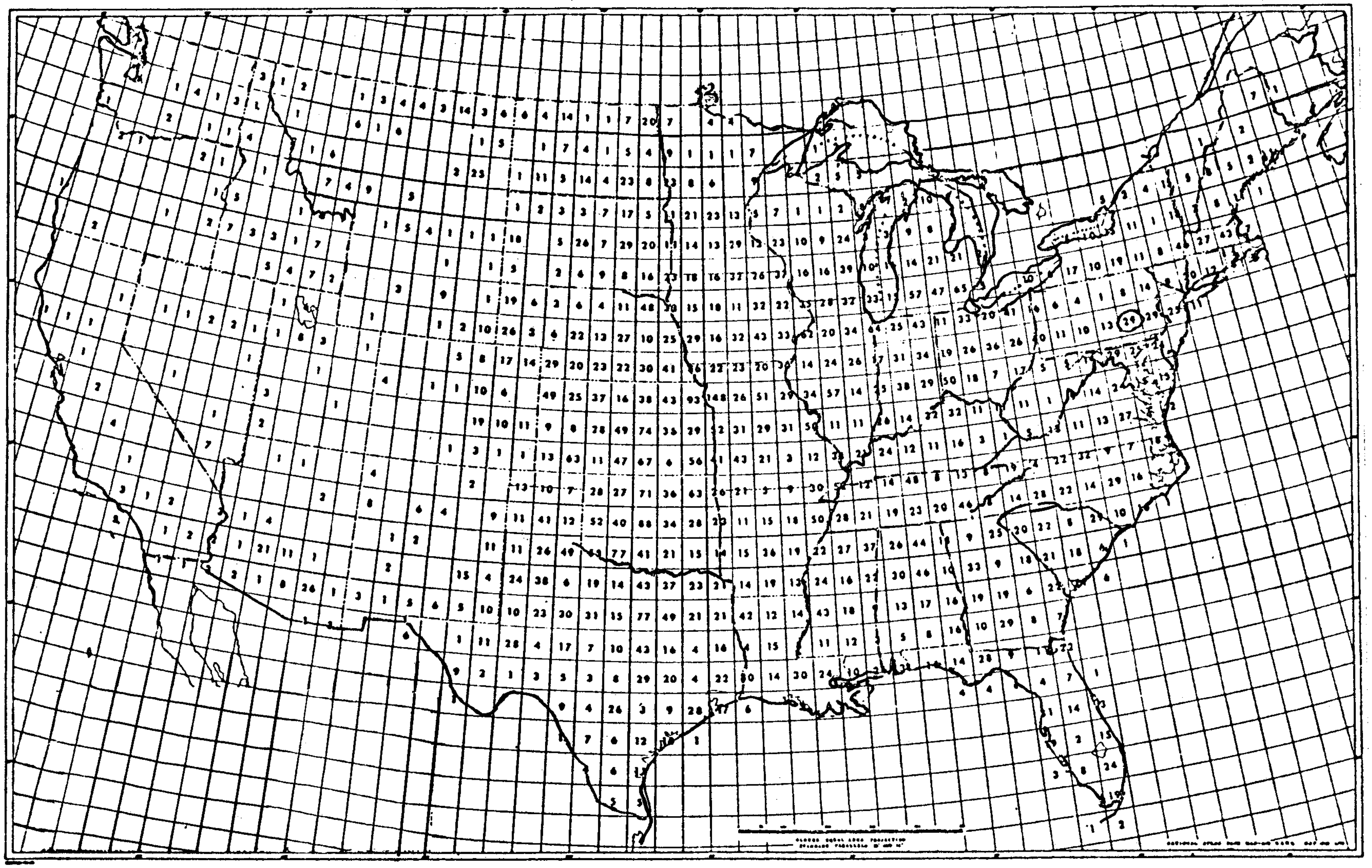
CALCULATED TORNADOES WINDSPEED BY 5 DEGREE SQUARES FOR 10^{-7} PROBABILITY PER YEAR

FIGURE 2.3-8



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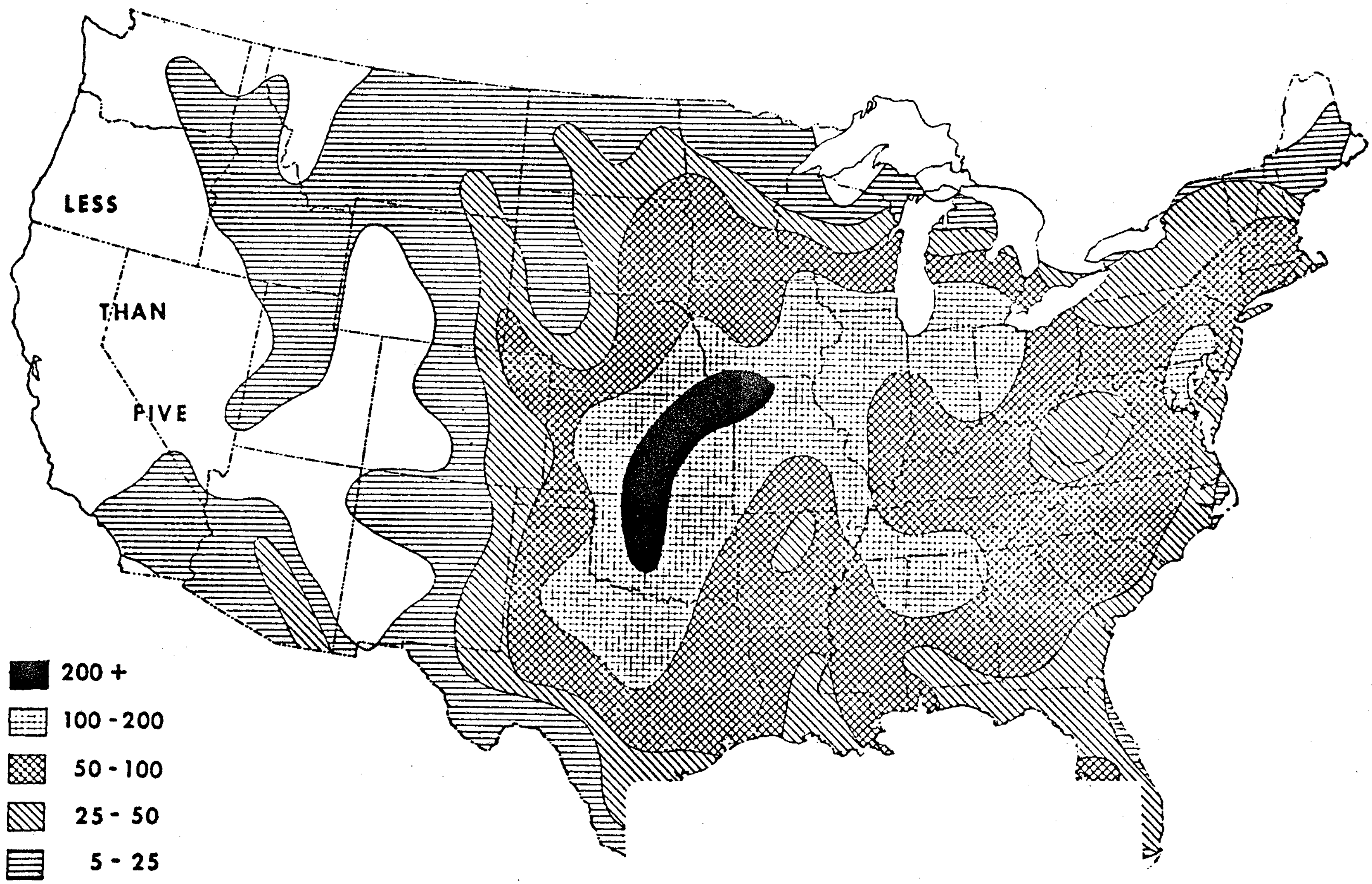
TOTAL WINDSTORMS, 50 KNOTS AND GREATER, 1055-1967, BY 1 DEGREE SQUARES (BASED ON SELS LOG)

FIGURE 2.3-9



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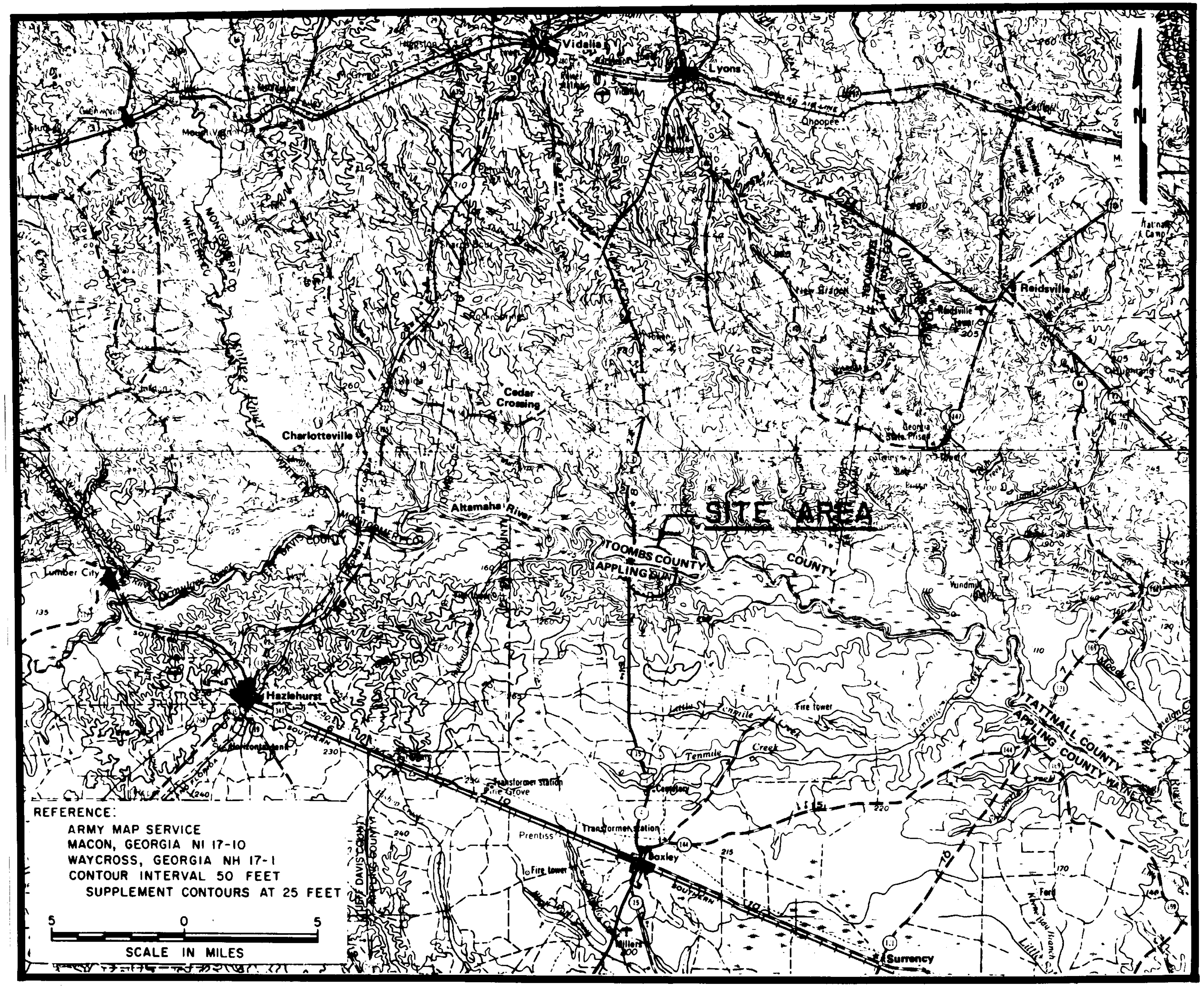
TOTAL WINDSTORMS, 50 KNOTS AND GREATER, 1055-1967, BY 2 DEGREE SQUARES (BASED ON SELS LOG)

FIGURE 2.3-10



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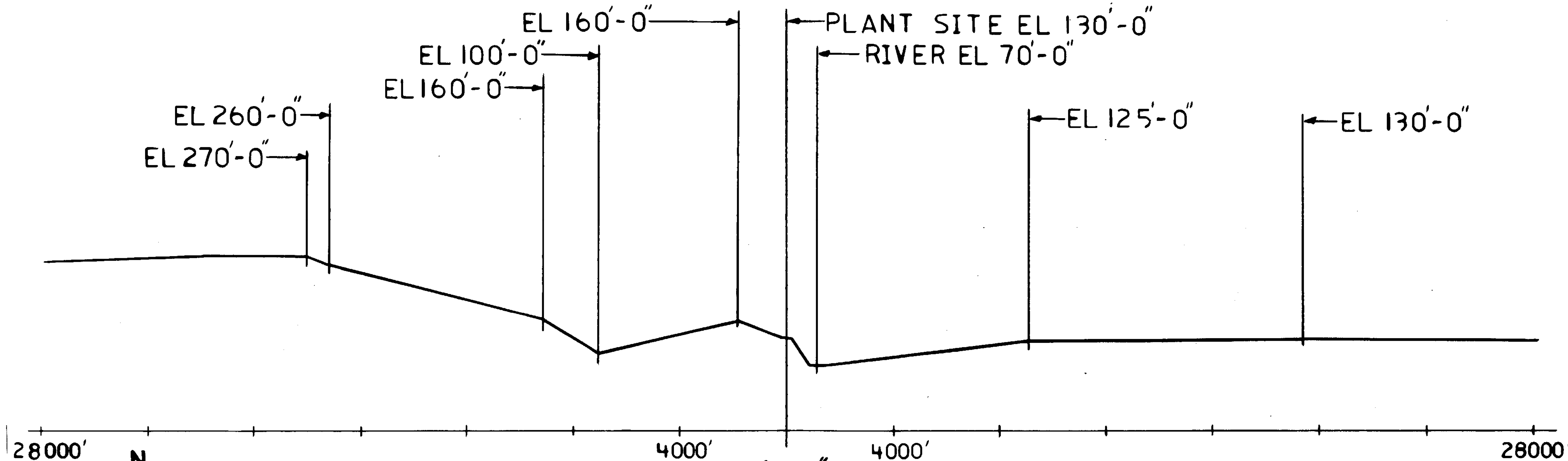
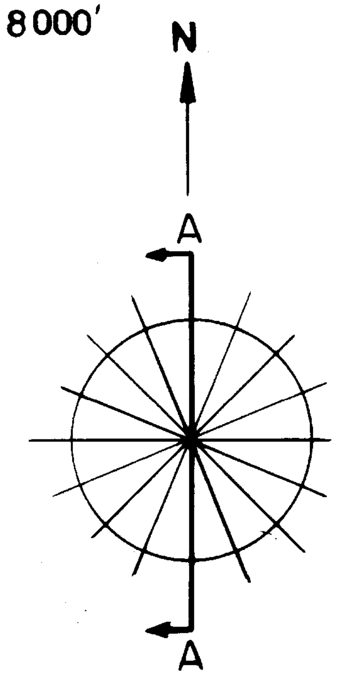
SOUTHERN COMPANY Energy to Serve Your World®	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 1 AND UNIT 2	REGIONAL TOPOGRAPHY FIGURE 2.3-11



PROFILE "A-A"

SCALE: 1" = 4000' HORIZ
1" = 200' VERT



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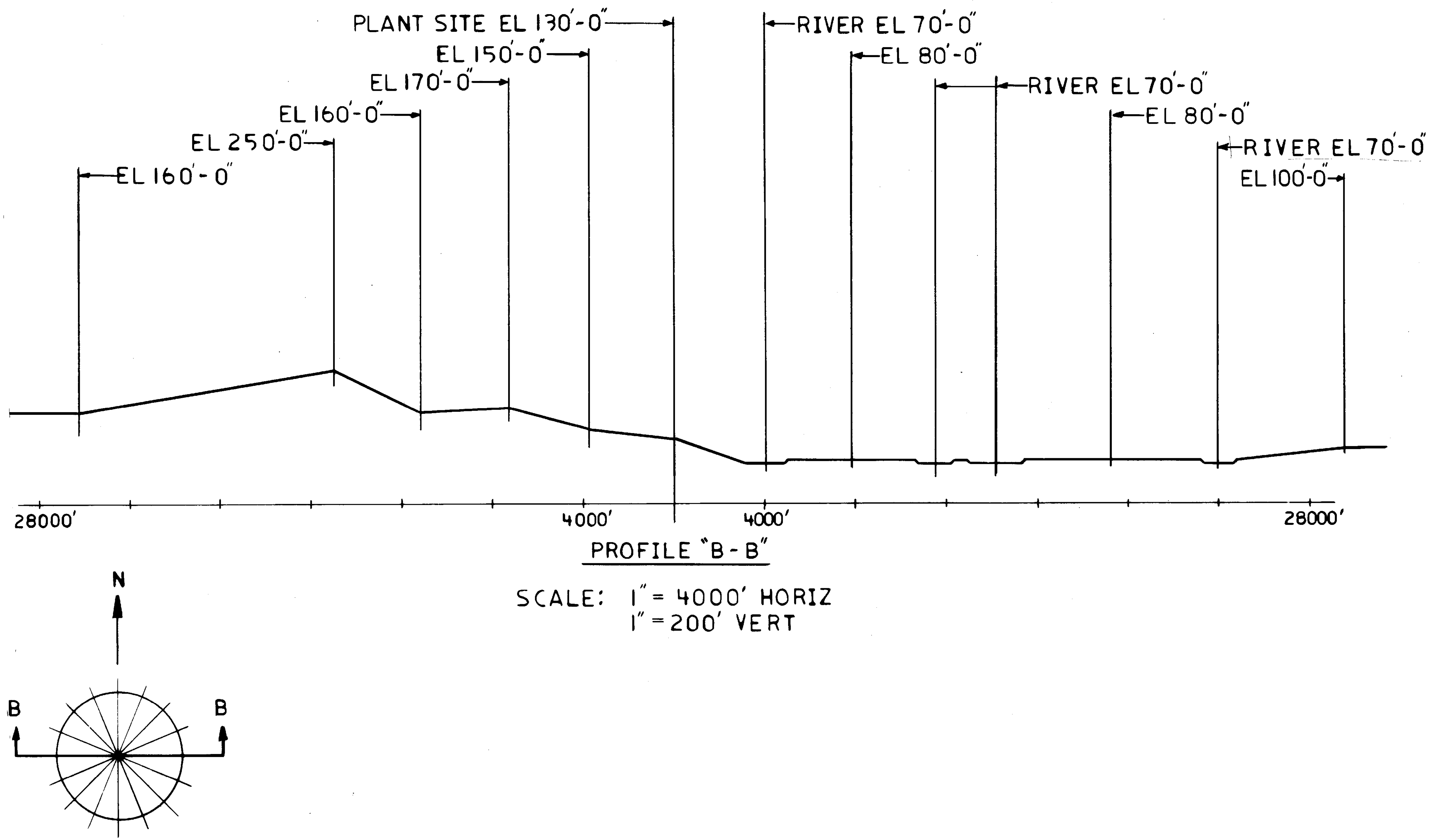
TOPOGRAPHICAL PROFILE

FIGURE 2.3-12 (SHEET 1 OF 8)



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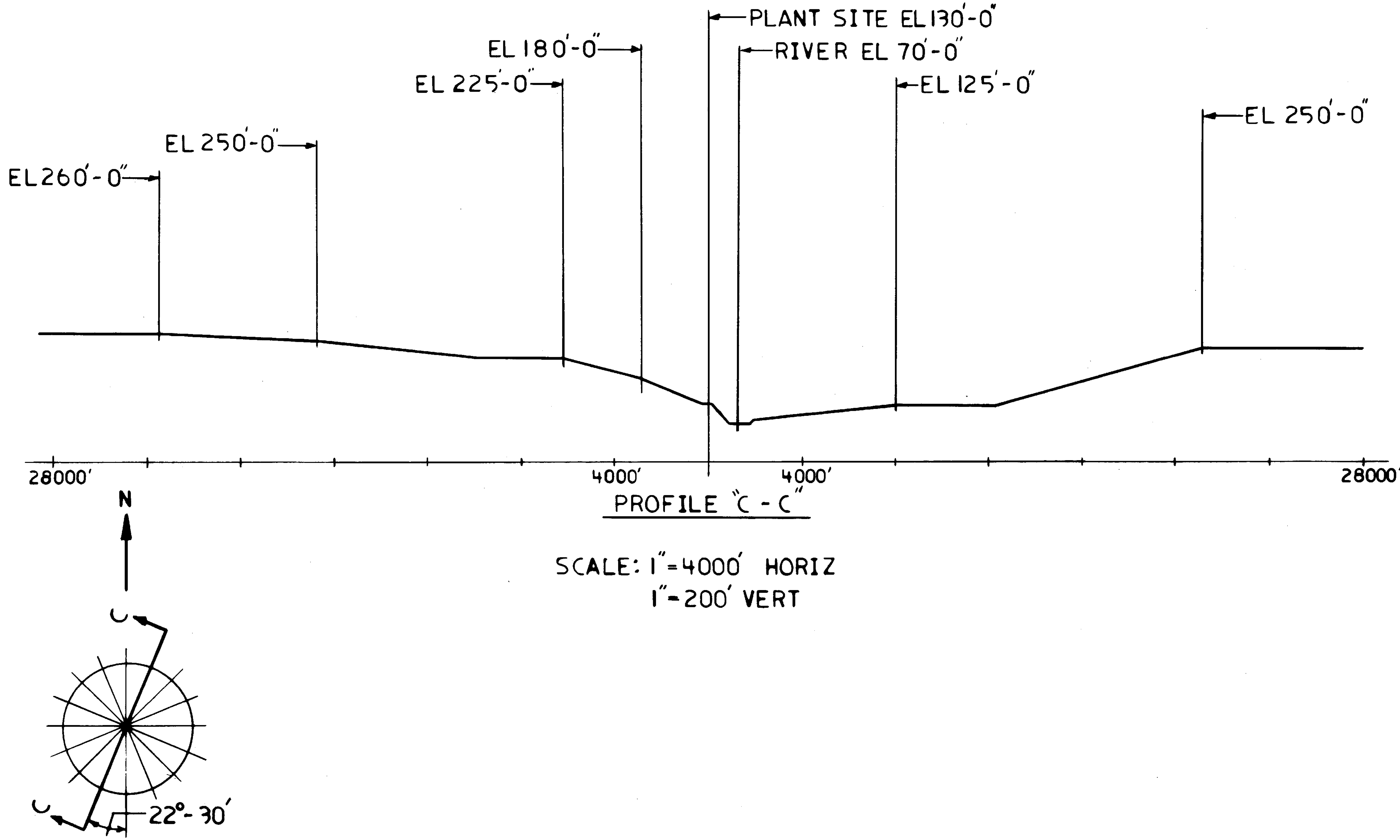
TOPOGRAPHICAL PROFILE

FIGURE 2.3-12 (SHEET 2 OF 8)



PROFILE "C - C"

SCALE: 1"=4000' HORIZ
1"-200' VERT

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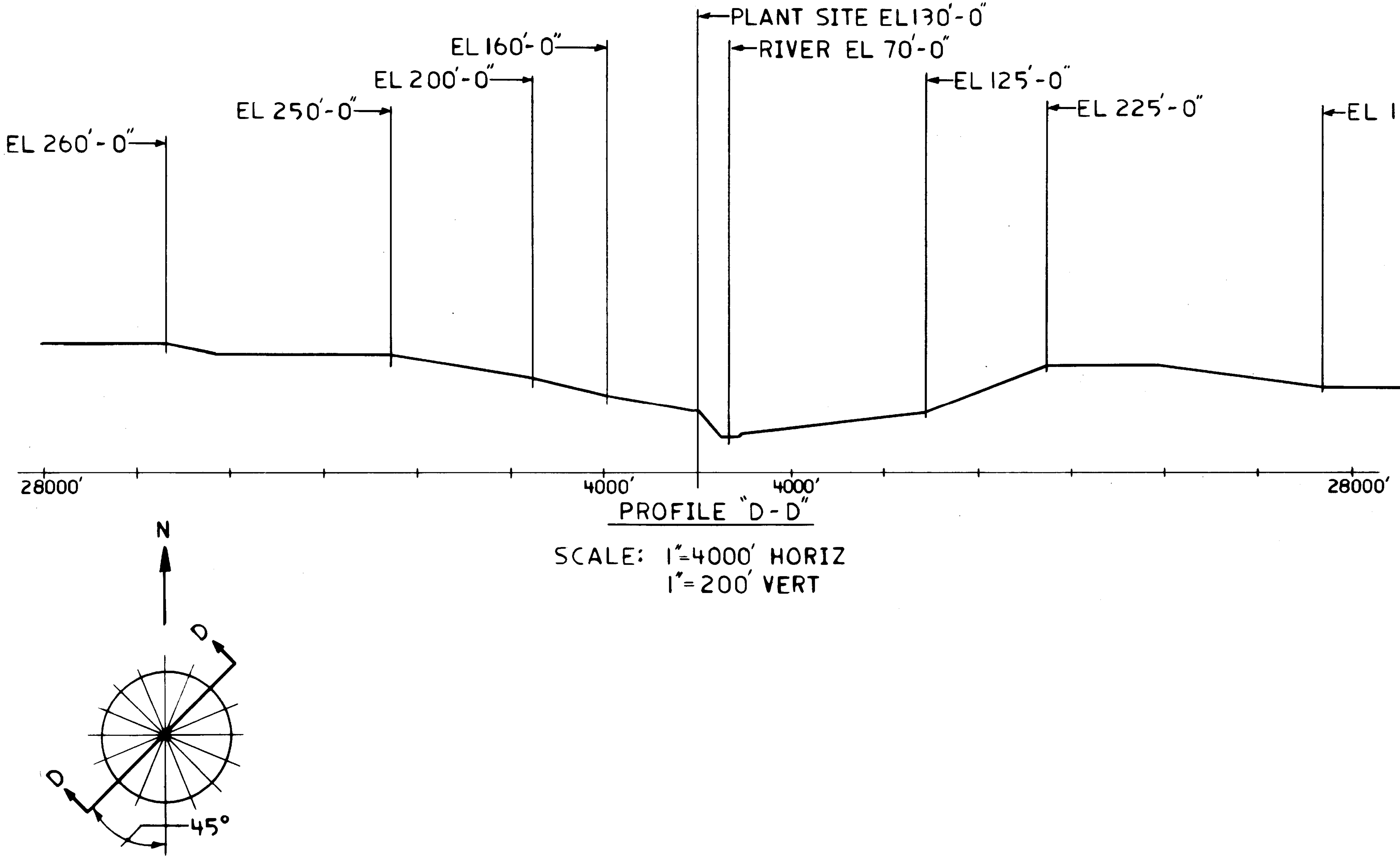
UNIT 1 AND UNIT 2

TOPOGRAPHICAL PROFILE

FIGURE 2.3-12 (SHEET 3 OF 8)



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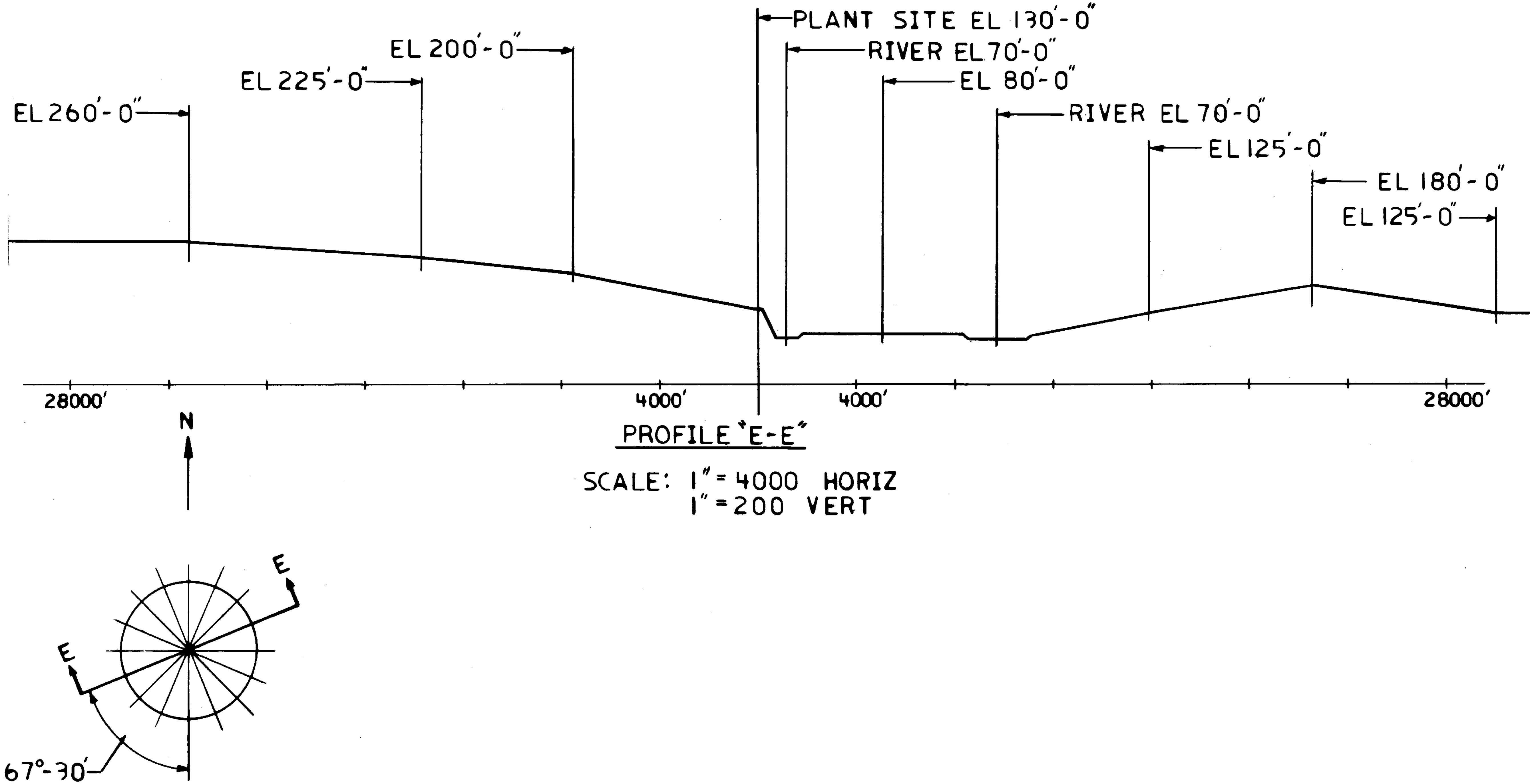
UNIT 1 AND UNIT 2

TOPOGRAPHICAL PROFILE

FIGURE 2.3-12 (SHEET 4 OF 8)



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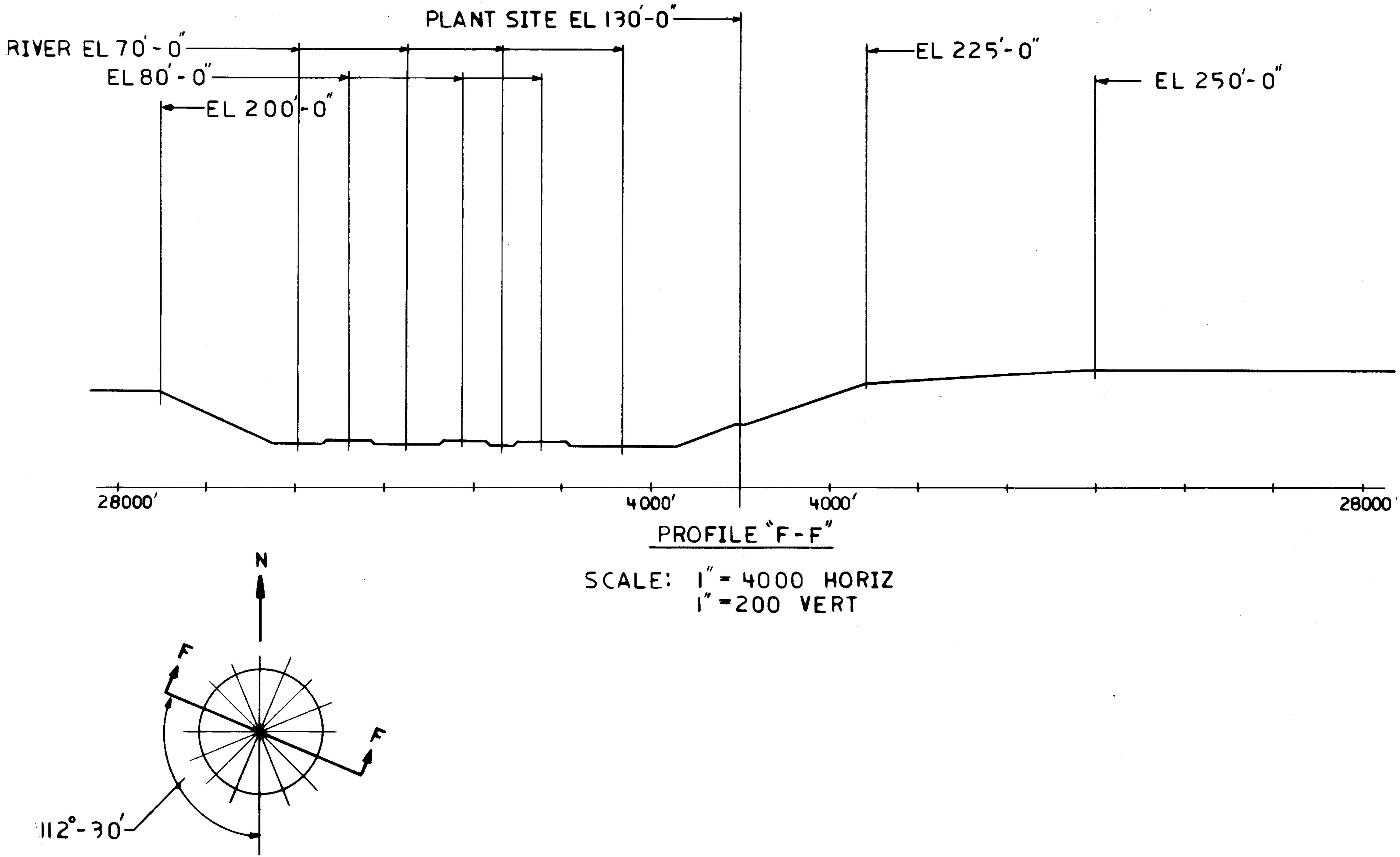
UNIT 1 AND UNIT 2

TOPOGRAPHICAL PROFILE

FIGURE 2.3-12 (SHEET 5 OF 8)



PROFILE "F-F"

SCALE: 1" = 4000 HORIZ
1" = 200 VERT

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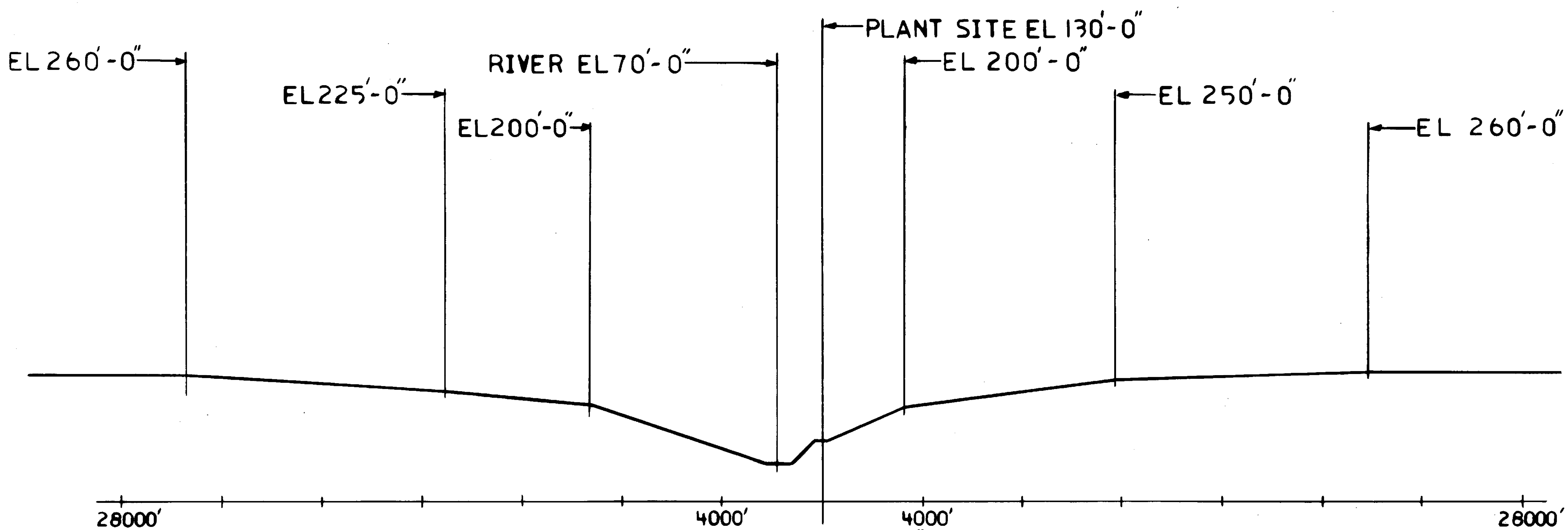
UNIT 1 AND UNIT 2

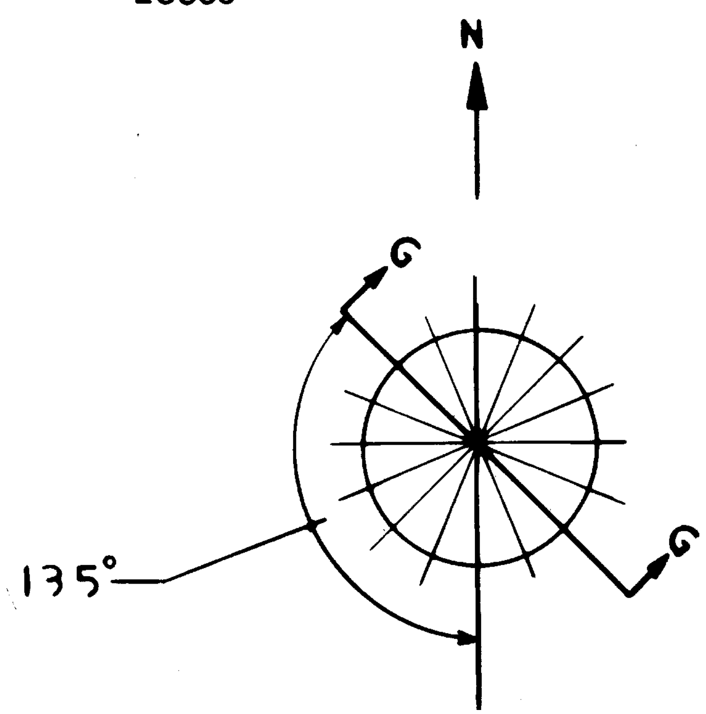
TOPOGRAPHICAL PROFILE

FIGURE 2.3-12 (SHEET 6 OF 8)



PROFILE "G - G"

SCALE: 1" = 4000' HORIZ
1" = 200' VERT



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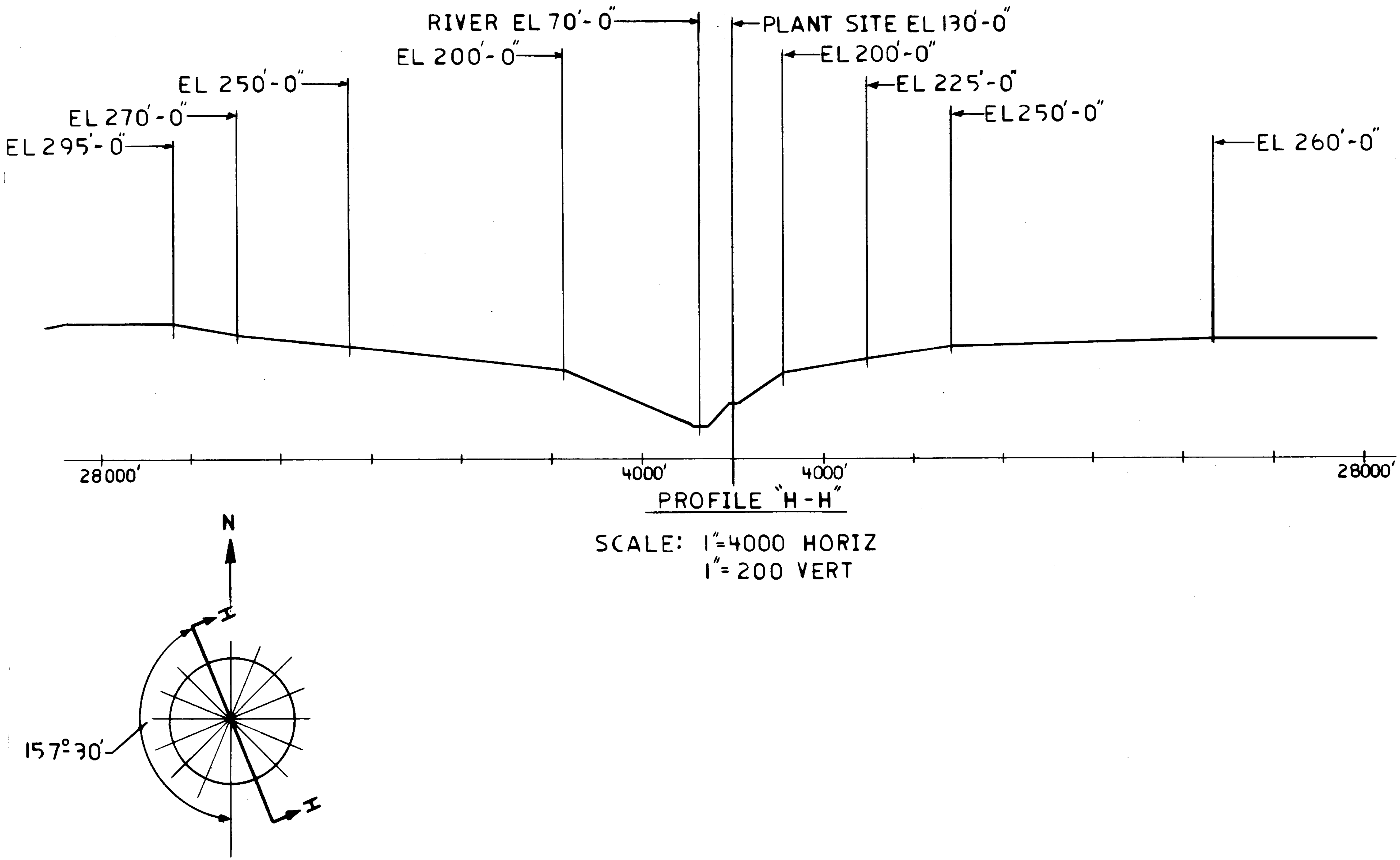
TOPOGRAPHICAL PROFILE

FIGURE 2.3-12 (SHEET 7 OF 8)



PROFILE "H-H"

SCALE: 1"=4000 HORIZ
1"=200 VERT

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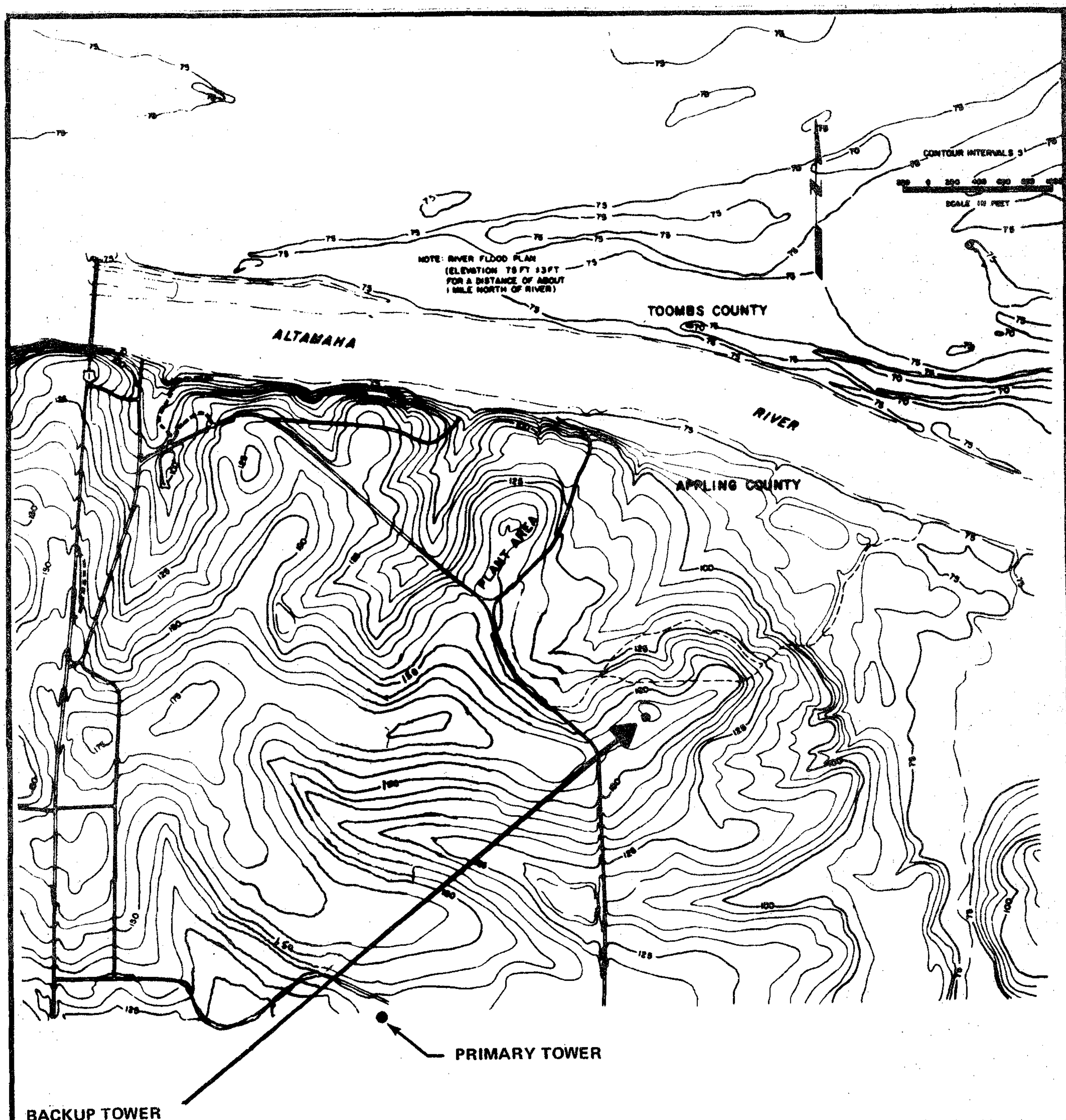
UNIT 1 AND UNIT 2

TOPOGRAPHICAL PROFILE

FIGURE 2.3-12 (SHEET 8 OF 8)



ACAD

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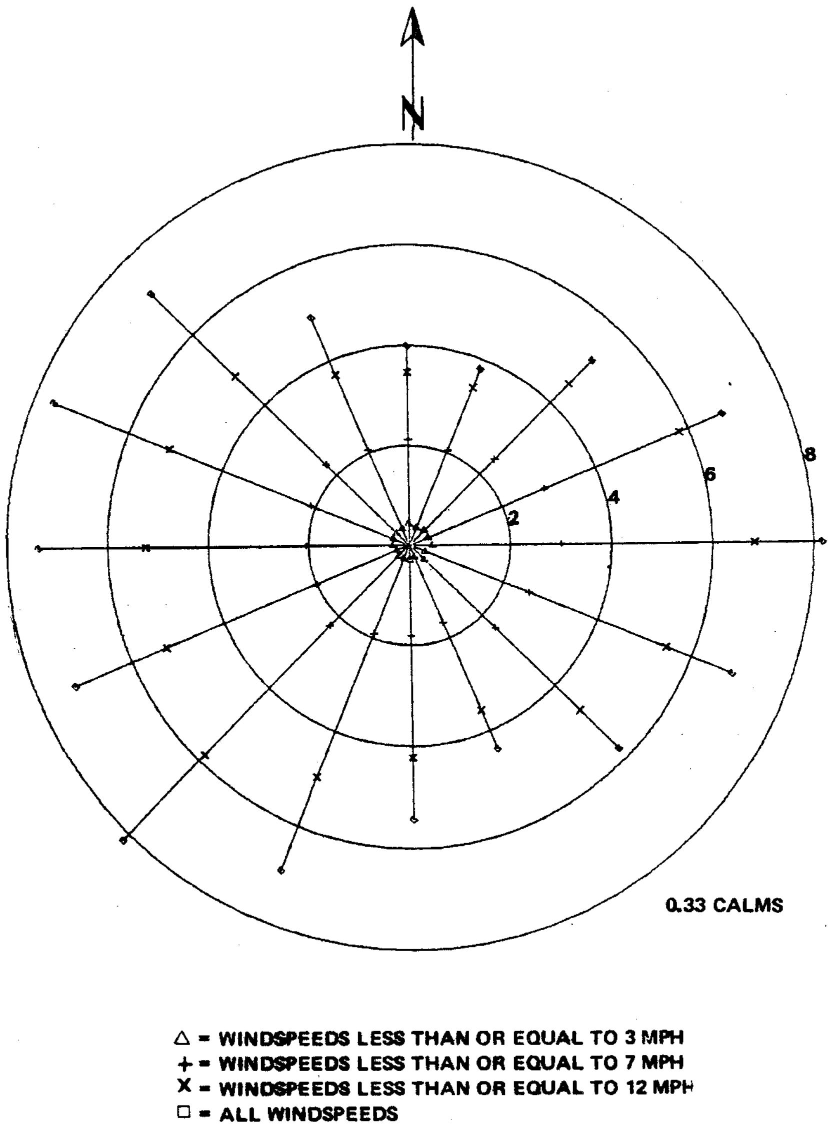
UNIT 1 AND UNIT 2

SITE TOPOGRAPHIC MAP

FIGURE 2.3-13



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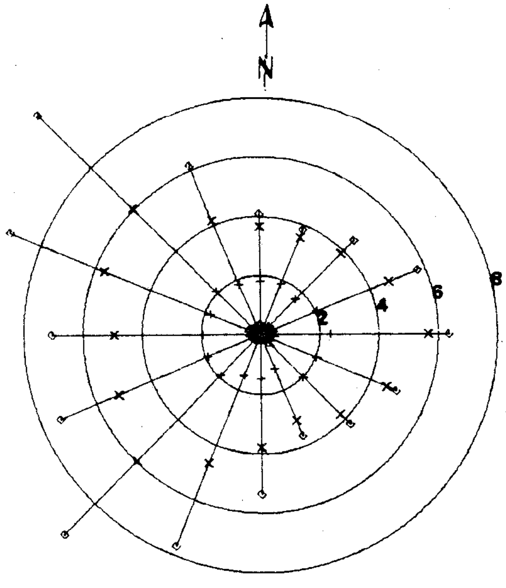
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

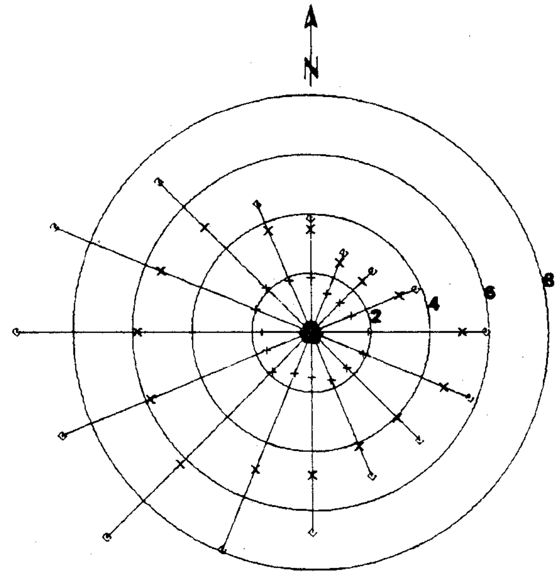
HATCH SITE 150-ft ANNUAL WIND ROSE
BASED ON 4 YEARS OF DATA

FIGURE 2.3-14



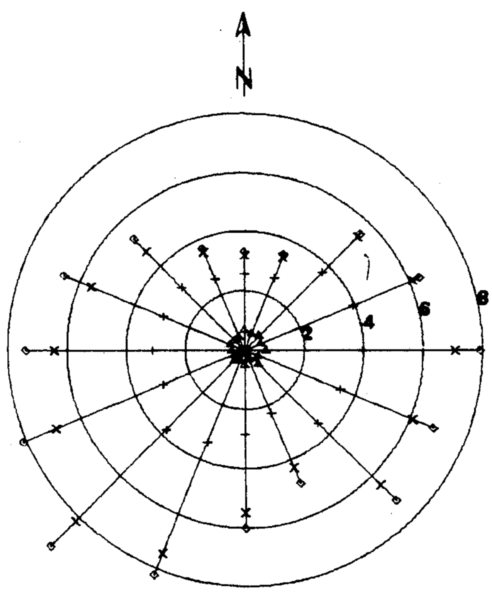
WINTER

0.10-PERCENT CALMS



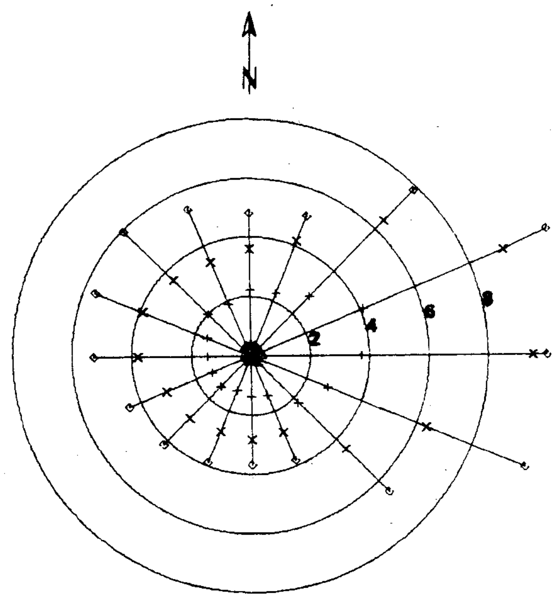
SPRING

0.27-PERCENT CALMS



SUMMER

0.72-PERCENT CALMS



FALL

0.20-PERCENT CALMS

△ = WINDSPEEDS LESS THAN OR EQUAL TO 3 MPH
\+ = WINDSPEEDS LESS THAN OR EQUAL TO 7 MPH
X = WINDSPEEDS LESS THAN OR EQUAL TO 12 MPH
□ = ALL WINDSPEEDS

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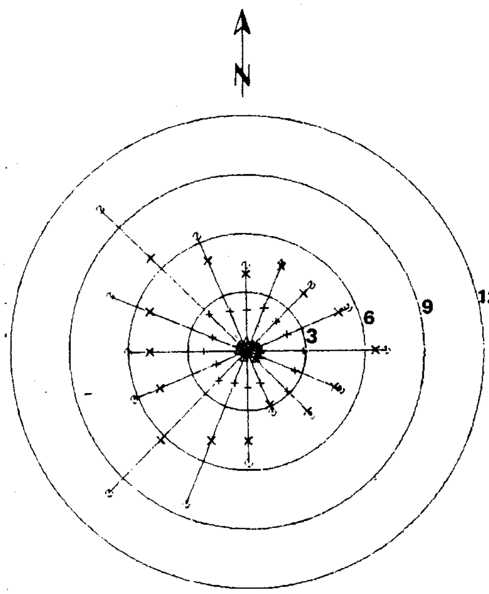
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

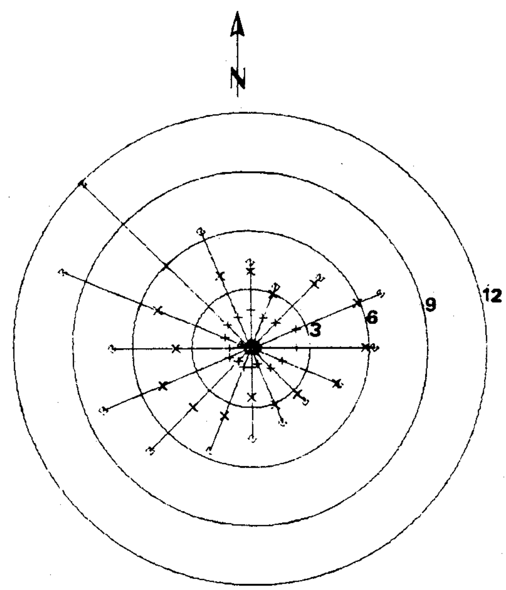
HATCH SITE 150-ft MONTHLY AND SEASONAL WIND ROSES BASED ON 4 YEARS OF SITE DATA

FIGURE 2.3-15 (SHEET 1 OF 4)



JANUARY

0.03-PERCENT CALMS



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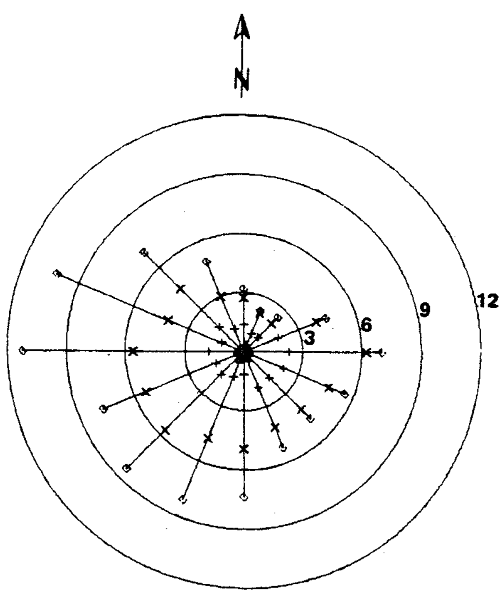
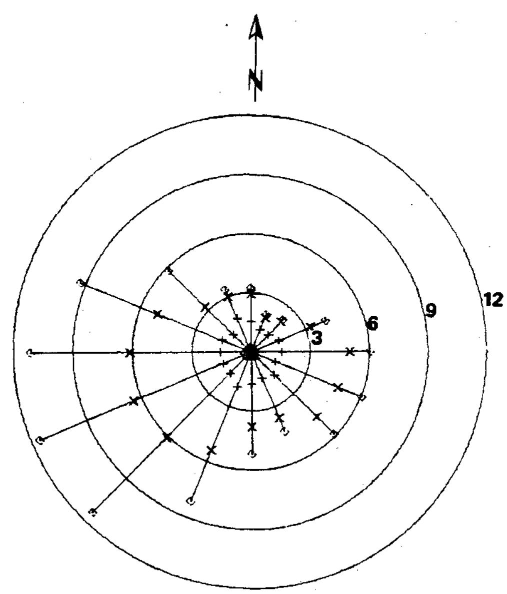
0.11-PERCENT CALMS



MARCH

0.03-PERCENT CALMS



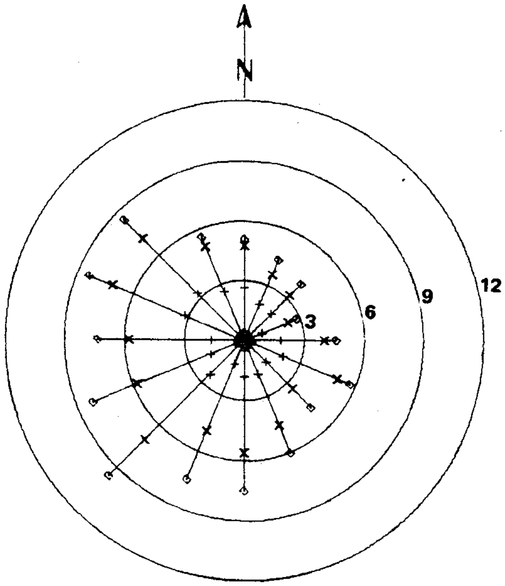
APRIL

0.57-PERCENT CALMS

ACAD

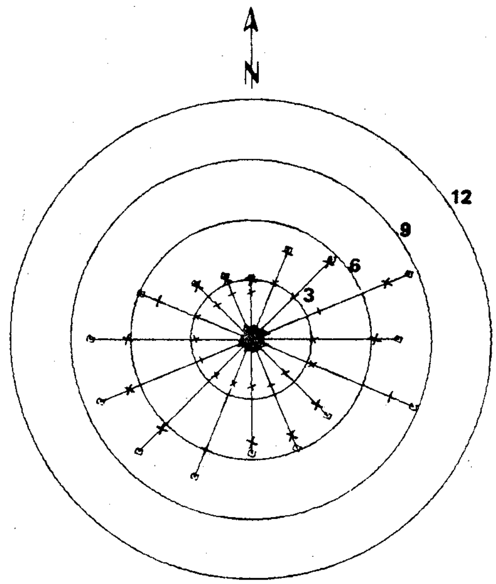
REV 19 7/01

SOUTHERN COMPANY *Energy to Serve Your World®* SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 1 AND UNIT 2	HATCH SITE 150-ft MONTHLY AND SEASONAL WIND ROSES BASED ON 4 YEARS OF SITE DATA
	FIGURE 2.3-15 (SHEET 2 OF 4)



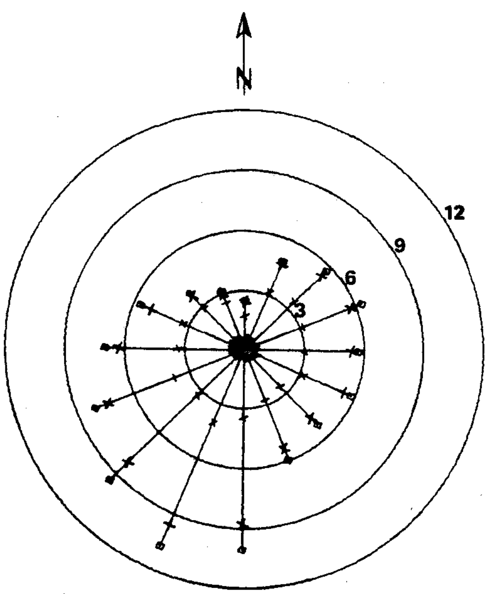
MAY

0.25-PERCENT CALMS



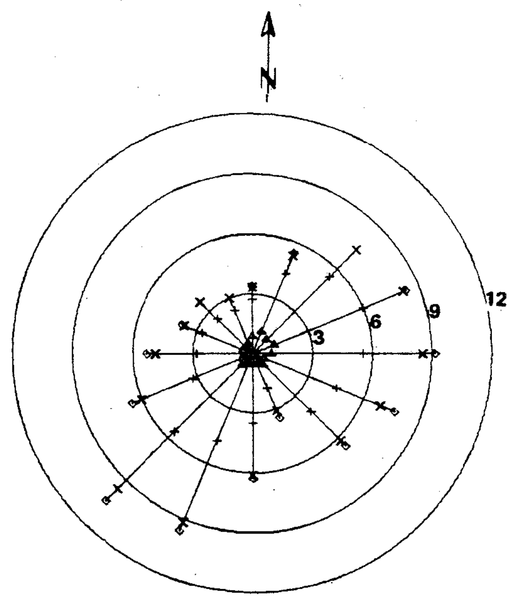
JUNE

0.52-PERCENT CALMS



JULY

0.93-PERCENT CALMS



AUGUST

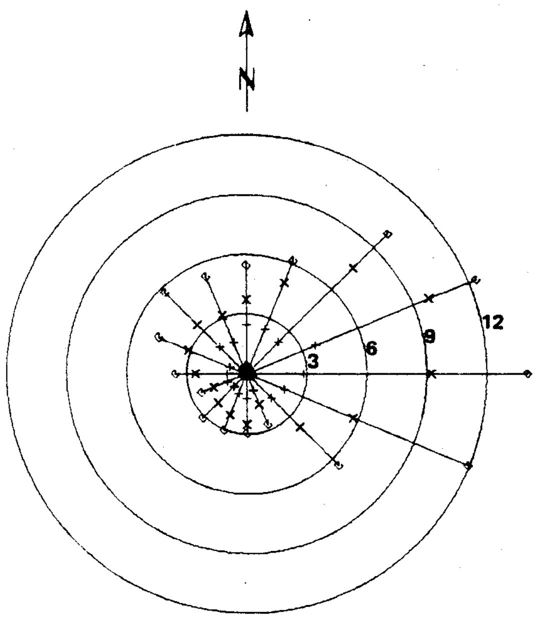
0.75-PERCENT CALMS

REV 19 7/01

SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

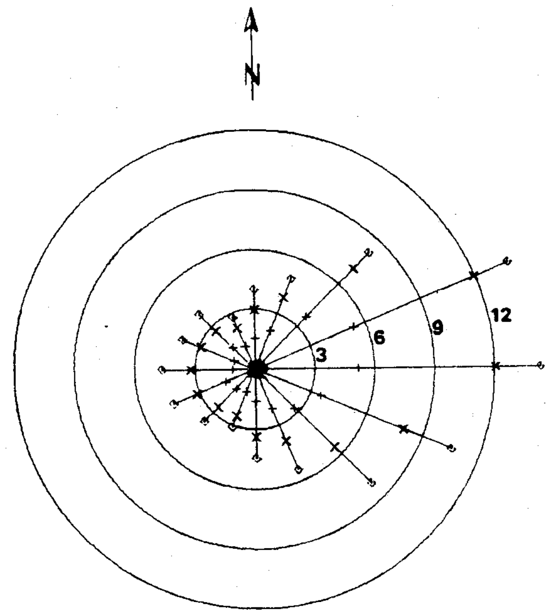
HATCH SITE 150-ft MONTHLY ANAD SEASONAL WIND ROSES BASED ON 4 YEARS OF SITE DATA

FIGURE 2.3-15 (SHEET 3 OF 4)



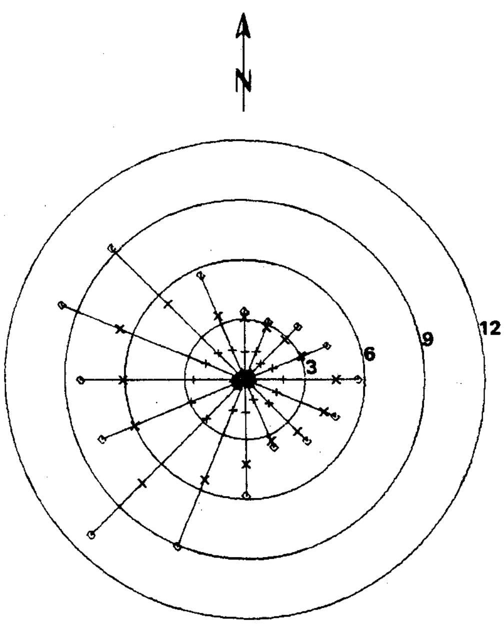
OCTOBER

0.00-PERCENT CALMS



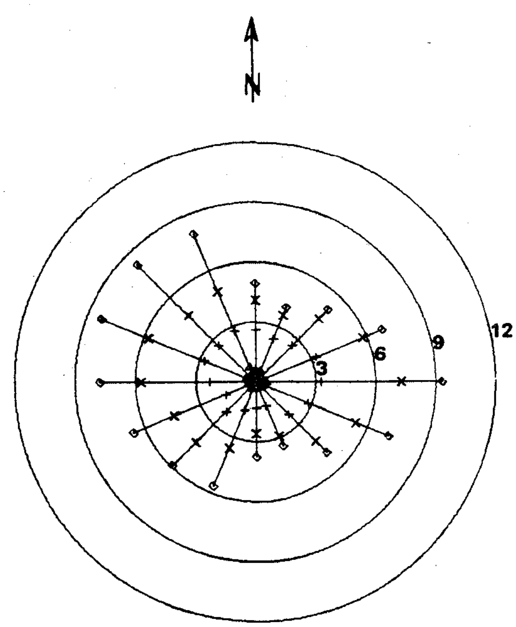
SEPTEMBER

0.07-PERCENT CALMS



DECEMBER

0.15-PERCENT CALMS



NOVEMBER

0.56-PERCENT CALMS

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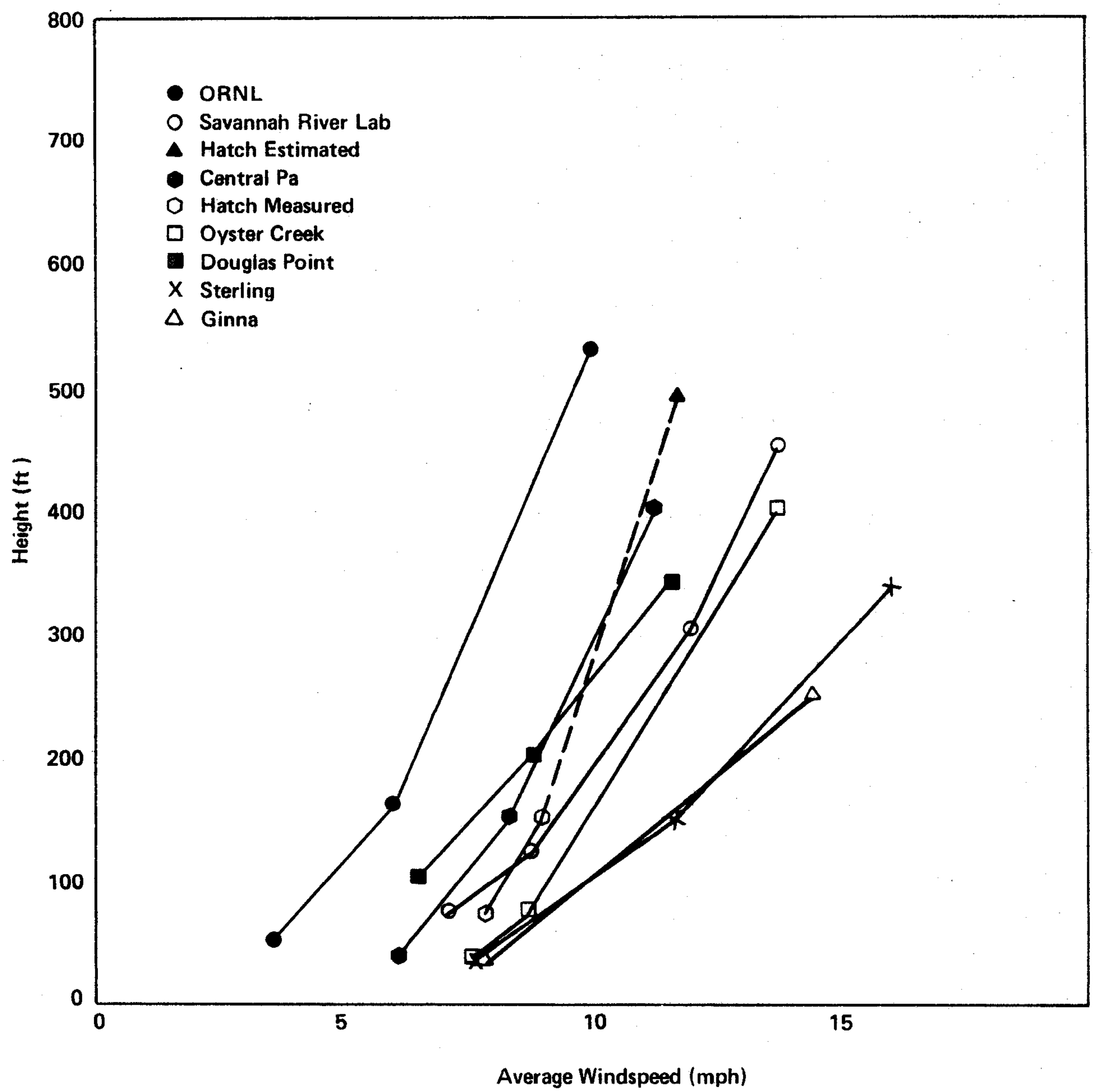
HATCH SITE 150-ft MONTHLY AND SEASONAL WIND ROSES BASED ON 4 YEARS OF SITE DATA

FIGURE 2.3-15 (SHEET 4 OF 4)



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COMPARISON OF MEASURED WINDSPEED VS. HEIGHT

FIGURE 2.3-16

2.4 HYDROLOGIC ENGINEERING

2.4.1 HYDROLOGIC DESCRIPTION (HNP-1 AND HNP-2)

2.4.1.1 Site and Facilities

The site of the Edwin I. Hatch Nuclear Plant (HNP), which is owned by Georgia Power Company (GPC), consists of about 2244 acres. Figures 2.4-1 and 2.4-2 characterize the site environs and show the approximate site boundaries. The site is located on the south side of the Altamaha River southeast of the intersection of the river with U.S. Highway No. 1 as shown on figures 2.4-1 and 2.4-2. It is in the northwestern sector of Appling County just across the river from Toombs County, ~ 98 miles southeast of Macon and 73 miles northwest of Brunswick. Figure 2.4-3 shows a topographic map of the site area and figure 2.4-4 is a plot plan of the plant area which shows the finished grade. As shown in figure 2.4-3, the natural site grade varies from ~ 175 ft to < 75 ft at the river. The natural grade in the plant area varies from about 150 ft to < 75 ft at the river. The major safety-related structures are listed in table 2.4-1. Grade elevation at the intake structure is 110 ft; the grade elevation at the control building, reactor building, and diesel building is 129.5 ft.

2.4.1.2 Hydrosphere

The dominant surface hydrological feature of the site region is the Altamaha River and its contributory streams, the Oconee and Ocmulgee, which join to form the Altamaha ~ 20 river miles upstream and west of the site. The location of these rivers and their drainage basins with respect to the site is shown on figures 2.4-5 and 2.4-6. The regional topography is shown in figure 2.3-11.

The area of the drainage basin affecting the Altamaha at the site is about 11,700 mi^2. The average flow in the river at the site is ~ 13,000 ft^3/s. The maximum historical flow, based on 58 years (1913 to 1970) of stage record taken by the U.S. Weather Bureau's Charlotte gage, ~ 19.0 river miles upstream from the site, was about 170,000 ft^3/s during the flood of January 1925. Estimates of the maximum flood of record range from 170,000 ft^3/s to 200,000 ft^3/s. This discrepancy is apparently due to differences in converting stage measurements at the Charlotte gage and high-water marks observed near the Baxley gage (by the Georgia Department of Transportation) to discharges in ft^3/s. A discharge of 200,000 ft^3/s at the plant site corresponds to an elevation of 91.3 ft mean sea level (msl). The finished plant grade is at el 129 ft msl.

During 34 years of measurement at the Doctortown gage(1) ~ 57.5 river miles downstream from the plant site, the minimum daily flow, which occurred in November 1954, was ~ 1430 ft^3/s corresponding to an elevation at the plant site estimated at 62.8 ft msl. The monthly average, maximum, and minimum river temperatures for a 17-year period, 1962 to 1978, from Doctortown are shown in table 2.4-2.

The waters of the Altamaha downstream from the site are not used for municipal or industrial water supplies, probably because wells provide a better source. There are no known uses of river water for human consumption or irrigation downstream of the site. There is sport fishing on the river with catches

including pickerel, large-mouth bass, channel catfish, red-breasted sunfish, and bluegill. American shad and striped bass migrate upstream as far as the dams that form Jackson Lake and Lake Sinclair. These dams are shown on figure 2.4-6. There is commercial fishing near the mouth of the river, about 115 river miles downstream from the plant site. Catches consist principally of oysters, crabs, shrimp, shad, king whiting, flounder, and fresh water catfish.

Should an accidental spillage of radioactive liquid on the surface of the ground at the plant occur, the liquid would either run off into the river because of the topographical configuration of the site (figure 2.4-7) or would enter the ground and then slowly move to the river. Since the soils at the site are relatively impermeable, it is expected that such movement would be very slow. The possibility of affecting offsite wells by spills into the ground water at the site is very remote.

There are three major dams in the Altamaha River basin above the site. The Sinclair Dam on the Oconee River is the largest with a reservoir storage capacity of 330,000 acre-ft at el 340 ft. The drainage area is 2910 mi^2. It is 169 river miles upstream from the plant site. The Wallace Dam, which is at a site at the upper end of Sinclair Reservoir, reduces its probable maximum flood (PMF) spillway outflow. Lloyd Shoals Dam is located on the Ocmulgee River 268 river miles above the HNP-2 site and has a drainage area of 1400 mi^2. The reservoir volume is 107,000 acre-ft at el 530 ft. Table 2.4-3 provides pertinent information about the upstream dams and reservoirs.

2.4.2 FLOODS (HNP-1 AND HNP-2)

2.4.2.1 Flood History

Maximum annual flows for the Charlotte gaging station are based on the stage discharge relation from 1913 through 1980. The rating curve is shown in figure 2.4-8. These flows are given in table 2.4-4. In 1970, the United States Geologic Survey (USGS) established a stream-gaging station near Baxley. The maximum annual flows from the Baxley gaging station are also shown on table 2.4-4, for the years 1971 through 1979.

The maximum historical flood at the plant site based on 68 years (table 2.4-4) of record was estimated by USGS to be 200,000 ft^3/s, and occurred January 22, 1925. This flow corresponds to a stage at the site of el 91.3 ft msl (200,000 ft^3/s). This was determined by USGS from a high-water mark furnished by the Georgia Department of Transportation. From the flood frequency analysis, this has been estimated as a 250-year flood.

The flood discharge studies are summarized in table 2.4-5.

2.4.2.2 Flood Design Considerations

In order to evaluate the ability of safety-related structures and equipment to withstand floods and flood waves, an investigation was made to determine the maximum water levels due to hypothetical floods. This study included the effects of winds concurrent with the probable maximum discharge which

produced the maximum water level at the HNP site (of the wave crest) of 108.3 ft. This is below the grade of the diesel generator, reactor building, and control buildings (129.5 ft). Dam failures were also considered but resulting water levels were below that caused by the PMF concurrent with waves due to winds.

2.4.2.3 Effects of Local Intense Precipitation

As has been stated in paragraph 2.4.2.2, the plant grade is at ~ el 129.5 ft msl which is high enough to be unaffected by the flood stage from the river. In addition, the effects of severe local precipitation have been investigated. To evaluate the effects of local intense precipitation, the probable maximum precipitation (PMP) used was selected from the World Record Point Precipitation Curve shown on Figure 9-44 in reference 2. The equation of the curve is:

$$R = 15.3\, D^{0.486}$$

where:

R = *rainfall (in.).*

D = *duration of precipitation (h).*

The topography of the plant is such that the runoff of rainfall is directed away from the power block area as shown in figure 2.4-7, both by natural drainage and by a combined system of culverts along with open ditches to the natural drainage channels which subsequently go to the river. Therefore, the drainage system for the site precludes flooding safety-related structures. The plant drainage system is designed for a maximum precipitation of 6 in./h, which is estimated to occur about once in 100 years. The plant site was checked to ensure that flooding of safety-related equipment would not occur as a result of the PMP (15.3 in. in 1 h). In the calculations, an assumption was made that the underground storm drainage system was blocked and the PMP runoff was carried off on the surface. The runoff from local PMP across the plant area was checked using the rational method,

$$Q = C\, i\, A$$

where:

Q = *peak rate of runoff in* ft^3/s *at the check location.*

C = *weighted runoff coefficient expressing the ratio of rate of runoff to rate of rainfall.*

i = *average intensity of rainfall in in./h for PMP during the time of concentration.*

A = *area in acres that drains to the check location.*

In checking the plant area the i used above was taken from the World Record Envelope Curve given in reference 2 for the time of concentration determined for each area checked. The C to be used is

discussed in reference 3 where it stated: "Higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff." A discussion on estimating storm runoff from small areas is given in reference 4. Since the runoff areas around the safety-related structures are largely planted in grass, C = 0.6 was used for each area, including those that are roof areas. This is conservative since the roof drainage system on structures with outside parapet walls includes outside scupper holes. This system permits rainfall pondage on these roofs and gradual release of the pondage through the downspouts after the rainfall ends.

The depth of water at check locations was determined by using the Manning equation:

$$AR^{2/3} = Qn / 1.486\, S^{1/2}$$

where:

- A = *cross-sectional area of the flowing water in square ft taken at right angles to the direction of flow.*
- R = *hydraulic radius.*
- Q = *discharge as determined from rational method.*
- n = *Manning coefficient of channel roughness.*
- S = *slope of water surface in ft/ft.*

In ascertaining the surface water depth in the plant area, an n of 0.05 was used. This was obtained by assuming conservatively that grassed areas were adjacent to the doorways and openings.

The roofs of all safety-related structures are designed to pass the local PMP corresponding to time of concentration of flow. The design includes measures to guard against wind-induced seepage through roof penetrations, windows, and doors where safety-related equipment could be damaged.

Icing normally does not occur. The combination of icing followed by heavy local precipitation is not considered for determining the effects of local intense precipitation for site drainage.

2.4.3 PROBABLE MAXIMUM FLOOD (HNP-1 AND HNP-2)

The probable maximum discharge and stage of the Altamaha River at the Hatch site in the general vicinity of the crossing of U.S. Highway No. 1 (~ 20.0 river miles downstream from the confluence of the Oconee and Ocmulgee Rivers) have been determined from a detailed study. The primary consideration in determining the flood potential is the maximum possible depth of precipitation which can occur over the contributing drainage basin above the plant site aggregating ~ 11,700 mi^2*. The resulting peak discharge would produce the probable maximum stage at the plant location with the river channel and flood plain in the present condition. The following paragraphs outline the study, describe the procedures and techniques employed from storm transposition, rotation, and maximization, and present the findings.*

2.4.3.1 Probable Maximum Precipitation

2.4.3.1.1 Selection of Storm

A detailed study of the storm of March 11 through 16, 1929, with primary center near Elba, Alabama was made because it has been found, through many studies for spillway design floods for hydro projects in Alabama and Georgia, to produce the maximum design flood. After examination of the area-depth duration curves and storm rainfall pattern for a number of other southeastern storms, the hurricane storm of July 5 through 10, 1916, with the center of greatest depth near Bonifay, Florida was also selected for detailed study. The results of these studies are discussed later. The 1916 storm was found to give the greater volume of precipitation in the Altamaha River basin above the plant site and was used as a basis of design. Subsequent paragraphs describe the details of the study.

2.4.3.1.2 Transposition of Storm

The selected storm was positioned within meteorological limits over the basin above the plant site so as to produce the maximum volume of precipitation. The maximum position is determined by positioning the storm at several locations and finding the position for the maximum volume of precipitation by trial and error. For the 1916 storm the maximum position was with the primary storm center located ~ 8-miles N 27°W from Lumber City, Georgia with the storm axis rotated 20 degrees clockwise from its original bearing.

2.4.3.1.3 Maximization of Storm Precipitation

The PMP in the selected storm in its transposed position over the Altamaha River basin is determined by ensuring that the amount of precipitable water is proportional to moisture charge[a] *and the storm efficiency.*[b] *Maximum possible moisture charge at any location and time is related to the temperature contrast*[c] *for that location and time. Inasmuch as maximum possible dewpoint and temperature contrast vary by months, it was necessary to determine the month in which the resulting moisture charge is maximum. However, due to use of the 1916 storm which resulted from a hurricane, the months considered were limited to June, July, August, September, and October. The PMP is computed for each reporting station within the basin in the transposed position, with appropriate adjustments made in the moisture charge to account for the elevation of the inflow barrier. The computational procedures used to relate, by stations, the PMP resulting from the storm in its transposed position to the actual rainfall depths are shown on table 2.4-6. The computed probable maximum depth at each station was used to prepare an isohyetal map*[d] *of the storm in the transposed position described in the preceding paragraph. The resulting map of the maximum probable storm precipitation is shown on figure 2.4-9.*

2.4.3.1.4 Rainfall Volume

The total storm volume over the drainage basin above the plant site is computed from the maximized isohyetal map. The portion of the total volume within each Theissen polygon[e] *is distributed by*

6-h periods in the same proportion as the rainfall depths at the respective precipitation station. The average total depth of storm rainfall for 11,700 mi^2 area above the plant site amounts to 16.93 in. and is shown by 6-h increments on figure 2.4-10.

2.4.3.2 Precipitation Losses

The ground was assumed to be saturated at the start of the storm as the result of antecedent rainfall and, accordingly, no initial retention loss has been taken. A study of several historical storms and related floods indicated that an average infiltration rate equal to 0.05 in./h is reasonable. For each polygon the 6-h increments of rainfall excess are obtained by deducting from the respective 6-h volumes of rainfall the portions thereof required to satisfy infiltration. The volume of rainfall excess over the basin above the plant site for each 6-h period equals the sum of the volumes within each Thiessen polygon or portion thereof, for respective periods, and is presented on figure 2.4-10. The average depth of rainfall excess over the drainage basin amounts to 14.19 in.

2.4.3.3 Runoff Model

The nearest location to the plant site on the Altamaha River, at which discharge hydrographs[f] for historical floods can be developed, is at the Georgia-Florida Railway crossing about 1 mile below the confluence of the Oconee and Ocmulgee Rivers and about 19 miles upstream of the site. At that station, which the weather bureau identifies as Altamaha River at Charlotte, river stages have been measured daily since 1913. Of five flood events selected for study, the unit hydrographs developed from the floods of November and December 1948 and February and March 1961, which were found to have the shorter times of concentration and higher peak discharges, were adopted for making further study at the plant site. Data for the 1948 flood at Charlotte, Georgia is shown on figure 2.4-11. Shown are the observed storm hydrograph, the unit hydrograph, the observed storm hydrograph with the base flow removed, and the reconstituted hydrograph using the unit hydrographs. Similar data for the 1961 flood are shown on figure 2.4-12. The unit hydrographs at the plant site were patterned after the unit hydrographs at the Charlotte station for the respective floods with the volumes thereof increased in direct proportion to the drainage areas at the two locations and the peak discharges related to the square root of the drainage areas. The contributing drainage areas above the Charlotte station and the plant site are estimated to be 11,550 and 11,700 mi^2, respectively. The 6-h unit hydrograph developed from the 1948 flood has a more

---

a. Moisture charge is the precipitable water in a saturated atmosphere with pseudoadiabatic lapse rate in the column of air above sea level at the representative 1000-mb dewpoint.

b. Storm efficiency in computing the maximum probable precipitation is the optimum combination of moisture charge and convergence of the wind.

c. Temperature contrast is defined as the difference in temperature between cold and warm air masses which can be expected to interact at any location resulting in the energy needed to produce precipitation.

d. Isohyetal map is a map showing lines of equal depths of precipitation.

e. Theissen polygon is the figure bound by the perpendicular bisectors of the lines joining adjacent precipitation stations, and the whole area is used to determine the weighted rainfall amounts for the basin.

f. Discharge hydrographs are graphs showing rate of flow of water with respect to time.

critical distribution than that resulting from the 1961 flood and, accordingly, was used to obtain the probable maximum stage at the plant site. The adopted 6-h unit hydrograph is shown on figure 2.4-13.

2.4.3.4 PMF Flow

The 6-h increments of rainfall excess were applied to the adopted 6-h unit hydrograph to obtain the hydrograph without base flow. It was assumed that the base flow would correspond to the fifth-day flow following the peak of a preceding storm runoff. The floods of record were analyzed and a base flow of 75,000 ft^3/s adopted. Adding the base flow to the hydrograph results in a peak discharge of 612,000 ft^3/s as presented on figure 2.4-10.

2.4.3.5 Water-Level Determination

To determine the probable maximum stage which corresponds to the probable maximum discharge, a stage discharge curve for the site was developed by computational means from known data.

Flood stage data for this portion of the Altamaha River is very limited. The best available data was for the 1948 flood with a discharge of 79,900 ft^3/s. The stage at Charlotte was el 96 ft and at Baxley el 83.1 ft. A straight-line hydraulic gradient was assumed between the gages at Baxley and Charlotte and projected downstream. Six valley cross-sections were surveyed in the 28-mile stretch of the river downstream of the U.S. Highway No. 1 bridge near the site. The location of these cross-sections are shown on figure 2.4-14. The values of Mannings n to match the projected gradient are tabulated in the following table under natural n value. To obtain a conservative n value for the section below Baxley, the stage at Baxley was assumed 2 ft higher and the gradient projected downstream. The values of Mannings n required to match this assumed gradient at each section computed as shown were in the following table under adopted n values except at section 2, where the plant is to be built and the river channel improved. These n values were used in computing water surface profiles for various flows up to the probable maximum. The computed stage relationship at U.S. Highway No. 1 is shown on figure 2.4-8. This relationship indicates that the peak discharge of 612,000 ft^3/s corresponds to a stage of el 105 ft. Using figure 2.4-8, the stage hydrograph shown on figure 2.4-10 was developed.

Section	*Computed natural n value*	*Adopted n value*
2	*0.032*	*0.031*
A	*0.110*	*0.145*
3	*0.061*	*0.093*
B	*0.043*	*0.074*
C	*0.097*	*0.159*
4	*0.069*	*0.130*

As indicated in paragraph 2.4.3.1.1, after transposition and maximization, the 1916 storm was found to produce the greatest volume of precipitation in the Altamaha River basin above the plant site. Similar results of the study of the 1929 storm are shown in figures 2.4-15 through 2.4-17.

Figure 2.4-17 shows a peak discharge of 540,000 ft^3/s based on the 1929 storm compared to 612,000 ft^3/s based on the 1916 storm. Similarly, the peak flood stage based on the 1929 storm is el 103.1 ft compared to el 105 ft based on the 1916 storm.

2.4.3.5.1 Flood Frequency-Discharge-Stage Relationships

The probable average recurrence intervals at which normal to moderately large flood discharges and stages of the Altamaha River at the plant site are equaled or exceeded were useful in the planning and design of the plant as well as of the work required for river control during construction. The techniques employed in the development of the frequency relationships are hereinafter discussed.

2.4.3.5.2 Selection of Method

It is considered likely that recurrence intervals extending up to 1000 years may be needed, whereas continuous records of stage of the Altamaha River and tributaries at locations within the same physiographic region as the plant site cover periods < 100 years in length. In view thereof, Hazen's logarithmic probability factors,[(1)] which should best utilize the available basic data to produce the desired results, have been selected for development of the probable flood frequency relationships.

2.4.3.5.3 Mean Flood and Coefficients of Variation and Skew

Weighted basin coefficients of variation and skew have been determined by using the records at five stations situated along the Altamaha River and the lower reaches of the Oconee and Ocmulgee Rivers. These basin coefficients of variations and skew are 0.59 and 1.19, respectively. Station coefficients of variation and skew have been computed for the periods of record at two stations on the Altamaha River, one operated by the weather bureau near Charlotte from which 53 years of data were used, and the second operated for 41 years by the geological survey at Doctortown, ~ 57.5 river miles downstream from the plant site. Respectively, the coefficients of variation are 0.58 and 0.60, and coefficients of skew are 1.45 and 1.48. The close agreement of the coefficients at the two stations indicates that these coefficients are better suited than the weighted basin coefficients for application to the plant site. The mean annual peak discharge at the site has been obtained by applying the drainage area above the site (11,700 mi^2) to a log-log relationship of mean annual peak discharge to drainage area as defined by the 2 stations on the Altamaha River above and below the plant site. The mean annual peak discharge at the plant site approximates 58,000 ft^3/s.

2.4.3.5.4 Frequency-Discharge-Stage Relationships - Results

Using a coefficient of variation of 0.59, a coefficient of skew of 1.45, and an annual mean peak discharge of 58,000 ft^3/s, a frequency-discharge relationship has been developed at the plant site with present channel conditions. Applying to this relationship the stage-discharge relationship previously described, a frequency-stage curve at the plant site has been developed and is shown on figure 2.4-18.

The findings from studies outlined herein relating to flood discharges and stages of the Altamaha River at the plant site are summarized in table 2.4-5.

2.4.3.6 Coincident Wind/Wave Activity

The possible maximum wave height would result from a 45-mph wind concurrent with the probable maximum discharge. The wave-height study was based on the procedure described in reference 5. The maximum fetch was directly up the river for a distance of 18 miles starting at the State Highway No. 121 bridge. The maximum sustained wind velocity with a duration of more than an hour was taken as 45 mph. The significant wave height that could be developed at the site was computed to be 6.5 ft (crest to trough). The wave crest at maximum discharge would be el 108.3 ft. This is safely below the plant grade of el 129 ft.

The pump intake is the one structure that could be affected by wave runup. This structure is of reinforced concrete with walls designed for an impact load of 4000 lb at 50 mph on an area of 25 ft^2. The floor in the pump room is at el 111 ft with the pump motors mounted above this. The floor is drained into the pump well beneath. Two doors are the only wall openings into the building, and these are placed in labyrinth offsets. The roadfill to the building is at el 110 ft. The wave crest, at maximum discharge, is at 108.3 ft and the waves could splash water onto the roadway. This could potentially lead to water draining into the valve pit. Two submersible pumps located in the valve pit sump are available to pump the water out, should this occur. Thus, the intake structure is protected against waves.

If wind-generated waves caused the water level inside the intake structure to rise temporarily to the wave crest of the design basis flood, there would be no flooding in the pump room or in the valve pit. Water rising in the intake structure well to el 108.3 ft would not reach the bottom of the pump room floor (drawing no. H-12192). The only concern would be leakage from the well into the valve pit through the residual heat removal service water (RHRSW) pump discharge line sleeves which penetrate the wall between the pump well and the valve pit (drawing no. H-21102). Each sleeve is sealed to the RHRSW pump discharge line specifically to prevent such leakage. Also, if any water did seep into the valve pit, it would be handled by two redundant, submersible sump pumps located in a small sump inside the valve pit.

2.4.4 POTENTIAL DAM FAILURES

2.4.4.1 Reservoir Description

There are three major dams in the Altamaha River Basin above the site. Sinclair Dam on the Oconee River is the largest with a reservoir storage capacity of 330,000 acre-ft at el 340 ft. The drainage area is 2910 mi^2. It is 169 river miles upstream of the Hatch site. This project has 1596 ft of earth dike and 1392 ft of concrete structure consisting of nonoverflow walls, powerhouse, intake, and spillway as shown in figures 2.4-19 and 2.4-20. The spillway has 24 gates on a crest at el 319 ft. Flood flows are controlled by gates to hold the reservoir at el 340 ft until all gates are fully open. The Elba storm of

1929 transposed, rotated, and optimized produces the PMF for this dam. The PMF results in a

maximum reservoir level of el 346 ft and a peak discharge of 389,000 ft^3/s, as shown on figure 2.4-21. This dam passes the PMF with 9 ft of freeboard, thereby providing adequate capacity to pass the flood without overtopping. The freeboard provides additional capacity for contingencies. The structures are capable of withstanding higher heads. Wallace Dam, at the upper end of Sinclair Reservoir, reduces the PMF spillway outflow. The Sinclair Reservoir normal elevation is 340 ft msl with normal daily drawdown at 1.8 ft. The surface area of the reservoir is 15,000 acres at normal elevation.

Wallace Dam, completed in 1980, is located on the Oconee River at river mile 172.7, which is 1.5 miles north of Georgia Highway 16 between Eaton and Sparta, Georgia, at the headwaters of Lake Sinclair and 22 miles above Sinclair Dam. It has a drainage area of 1830 mi^2.

The project consists of a reservoir; earth and concrete gravity dam, a 5-gate spillway, and appurtenant facilities; a semi-outdoor-type power-house integral with the dam, housing two conventional units at 56.25 MW each, and four reversible units at 52.2 MW each, for a total installed capacity of 321.3 MW; an excavated tailrace into Sinclair Reservoir for pumped storage operation of the project; a 230-kV substation; and recreation facilities.

The reservoir area is 19,050 acres at full-pond el 435 ft and extends ~ 40 miles upstream into the counties of Putnam, Hancock, Greene, and Morgan. The shoreline length of the reservoir is 374 miles. The existing 15,000-acre Sinclair Reservoir serves as the tailpond and is the source of water during pumping operations.

The powerhouse is a semi-outdoor-type reinforced concrete structure integral with the dam and is ~ 530 ft long.

Components of the dam are earth dam embankment sections ~ 1070 ft long, concrete nonoverflow sections with a total length of ~ 526 ft, and a concrete gravity spillway 266 ft in length containing 5 radial gates 43 ft wide x 48 ft high. Spillway piers are shifted downstream on the crest to increase the effective spillway length. The concrete dam sections, excepting the intake and spillway piers which are reinforced, are essentially unreinforced concrete gravity sections. All concrete dam sections are founded on sound rock and are designed for maximum reservoir flood level of el 440, with earthquake loadings applied at normal reservoir level of el 435.

Spillway discharge capacity has been model tested and rated at ~ 30,000 ft^3/s at flood pool level of 400. Each gate can release ~ 35,000 ft^3/s, when fully open, with normal full reservoir of 435. For this reason, the gates are operated in sequence and opened only one hoist increment (~ 1 ft) at a time. Sequence of operation is automatic. Rating tables are provided for various gate openings. The dam is 117 ft high and consists of 2700 ft of earth dikes and 1323 ft of concrete structures as shown on figures 2.4-28, 2.4-29, and 2.4-30. At normal full-pond elevation of 435 ft msl, the full reservoir storage capacity is 37,000 acre-feet with a surface area of 19,050 acres. The normal daily drawdown is about 1.5 ft. The shoreline length is ~ 374 miles. Area capacity curve is given in figure 2.4-31. The tailpond (Sinclair Reservoir) normal elevation is 340 ft msl with normal daily drawdown of 1.8 ft. The surface area of the Sinclair Reservoir is 15,000 acres.

Lloyd Shoals Dam is located on the Ocmulgee River 268 river miles above the HNP site and has a drainage area of 1400 mi^2. It has 1070.0 ft of concrete structure, 530 ft of earth dam, and 500 ft of auxiliary spillway as shown on figures 2.4-22 and 2.4-23. Flashboards are provided on the concrete spillway to maintain the reservoir at el 530 ft. When flood flows occur, the flashboards are overtopped and collapse to provide more discharge over the spillway. The spillway crest is at el 525 and el 527 ft. The auxiliary spillway flashboards are designed to collapse at pool el 526 ft. The reservoir volume is 107,000 acre-ft at el 530 ft. The normal head on the project is 102.2 ft, while the head at the time of the SPF would be 73 ft and at the time of the PMF would be 68.4 ft. At Lloyd Shoals, the 500-ft-long auxiliary spillway was added in 1971 to increase spillway design flood capacity. The Elba storm of 1929 when transposed, rotated, and optimized produces the PMF for this basin. This storm would result in the overtopping of Lloyd Shoals Dam at a peak outflow of 290,000 ft^3/s and a maximum pool elevation of 543.3 ft, as shown on figure 2.4-24. The resulting elevation at HNP is el 101.8 ft based on the Elba storm on the Altamaha Basin positioned for PMF at Lloyd Shoals, and without considering the failure of Lloyd Shoals.

2.4.4.2 <u>Dam Failure Permutations</u>

The design flood for HNP is based on the most severe precipitation event (PMP) in combination with upstream dam failures. There are two upstream dams which can affect the river's flood stage at HNP site:

- *Sinclair Dam, located 189 river miles upstream of the site on the Oconee River.*
- *Lloyd Shoals Dam, located 288 river miles upstream of the site on the Ocmulgee River.*

There are two approaches which can be used to analyze the effects of an upstream dam failure. The first is the conventional routing method and the second is the numerical solution of unsteady flow equations.

The numerical solution of the unsteady flow equations uses the finite difference approach. Use of this method required a prohibitively large number of river cross-sections in order to obtain credible results. A reduction in the number of river cross-sections would have reduced the number of computations required, but would also reduce the accuracy and hence the credibility of such computations. It was therefore concluded that the conventional routing (or graphic) method would give the best results for the effort required. The method adopted for use is fully discussed in reference 6. The values of lag time (L) and the storage coefficient (K) are based on actual experimental releases of water from both dams. By use of these two parameters in combination with a stage discharge curve at the HNP site (figure 2.4-8), the flood stage at the HNP site corresponding to any flood discharge can be determined.

Under SPF events, a failure of Sinclair Dam would result in a higher stage at the site than would Lloyd Shoals Dam because of its greater volume, dam length, and closer proximity to the Hatch site. Assuming instantaneous removal of the earth dike sections at Sinclair Dam during the peak of a SPF, a 27-ft-high wave would be created just below the dam with a discharge of about 3,000,000 ft^3/s. It should be noted that this disregards a concrete core wall section above normal pool level in the earth dike section which is unlikely to go out suddenly. Routing was done by graphic method as shown in technical memorandum WBTM HYDRO-4, "Elements of River Forecasting," dated October 1967. This instantaneous flow was

routed from Sinclair Dam to the site with a lag L of 72 h and a storage factor K of 39 h. The lag L[(6)] used in the routing was determined from Sinclair releases. Investigations were made during 1970 to determine the effect of releases from Lloyd Shoals and Sinclair hydro plants on the stages at and below HNP. Bihourly gage readings were taken at the HNP site. Below Sinclair Dam, continuous recording gages were available at Milledgeville (3.75 miles below Sinclair Dam), at Dublin, below HNP at Doctortown, Georgia, and below Lloyd Shoals Dam at Lumber City. The results of the tests from Sinclair are shown on figures 2.4-25 through 2.4-27. The time lag as shown was ~ 72 h (3 days). The dam failure could cause an additional flow of 100,000 ft^3/s at the Hatch site. Adding this additional flow of 100,000 ft^3/s to an assumed simultaneous SPF at the Hatch site would result in about a 4-ft increase in stage to el 100 ft. This is well below el 105 ft obtained from the PMF. This increase in stage has been verified by a wave decay curve which indicated that the wave, after traveling 169 river miles, would be ~ 15% of the starting wave or also ~ 4 ft.

In a similar analysis for Lloyd Shoals Dam, it was assumed that the entire concrete spillway would fail. This created a 24-ft-high wave with a discharge of ~ 800,000 ft^3/s. The study of the Sinclair releases described in the paragraph above was used to determine the lag value. The gage at Lumber City did not indicate any distinct wave after releases from Lloyd Shoals plant of 3000 ft^3/s. Since the Ocmulgee and Oconee Rivers have similar channels, the lag for Lloyd Shoals was determined as being proportional to the river miles above HNP using the lag obtained from Sinclair releases. This gave L of 130 h as a reasonable value for routing Lloyd Shoals failure wave. A storage factor K of 72 h was used. A wave of ~ 1 ft could be expected at the Hatch site after traveling 268 river miles. This failure could cause the water surface at the site to be at el 97 ft during the peak of the SPF at the site. When all the turbines are opened, a 2.5-ft-high wave is created in the tailrace with a 3000 ft^3/s surge. This surge cannot be detected at the Lumber City gage 237 river miles below the dam. It should be noted that these wave analysis approximations have been deliberately handled to produce wave heights of conservatively high values.

As Lloyd Shoals Dam would be overtopped at the time of its PMF, the effect of this failure at the site was considered. This would involve considering the concrete spillway vanishing at the peak of the PMF. The result of this failure considered simultaneously with the PMF and HNP is shown on figure 2.4-24 as a dashed line with cross marks above the PMF hydrograph for the HNP. The outflow hydrograph assumed at failure is conservatively high in total volume released, although the peak could be different depending on extent and mode of failure. The Lloyd Shoals Dam failure results in an artificial flood wave at HNP of 20,000 ft^3/s. This increases the stage 0.3 ft to el 105.3 ft. Consideration was given to whether there was a combination of storm and storm location which would fail Lloyd Shoals Dam and result in a more adverse effect at HNP than the site PMF. By definition, the highest flood stage at the site comes from the PMF. If we combine a Lloyd Shoals failure due to its PMF with the site PMF stage, we would have the upper limit of possible effect at the site as shown on figure 2.4-24. Thus, the upper limit of stage at HNP considering the failure of Lloyd Shoals Dam results in a stage of el 105.3 ft.

At Wallace Dam flood flows are controlled by gates at reservoir el 435 ft until all gates are open; then free overflow occurs over the spillway, based on the rating curve as shown on figure 2.4-32. Assuming instantaneous loss of all earth dikes coincident with the SPF for this dam, an artificial flood wave of 29 ft at the dam would result. This wave would be attenuated to 23 ft at Sinclair Dam and would overtop the dam by 8 ft. Assuming conservatively that this overtopping of the earth dikes resulted in their instantaneous failure, an artificial flood wave of 33 ft would be increased. This wave height would decay

to 5 ft at the Hatch site. The maximum stage at the Hatch site would be el 101 ft, assuming the stage at the Hatch site as that corresponding to the SPF. The Elba storm of 1929 transposed, rotated, and optimized produces the PMF for Wallace Dam. The PMF results in a maximum reservoir level of el 441.1 ft and a peak discharge of 316,800 ft^3/s, as shown on figure 2.4-33. This dam is designed to pass the PMF with 3.9 ft of freeboard, thereby having adequate capacity to pass the flood without overtopping. The dam structures are being designed to withstand the floodhead.

2.4.4.3 Unsteady Flow Analysis of Potential Dam Failure

See paragraph 2.4.4.2.

2.4.4.4 Water Level at Plant Site

See paragraph 2.4.4.2.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHES FLOODING

Not applicable.

2.4.6 PROBABLE MAXIMUM TSUNAMIS FLOODING

Not applicable.

2.4.7 ICE FLOODING

There is no record, in modern times, of the Altamaha River freezing over. Based on the river temperature data at Doctortown, Georgia, the minimum temperature of record is 37.4°F (3°C, table 2.4-2) and is safely above the freezing temperature. Therefore, the formation of fragile ice is unlikely and the ice blockage of the intake structure is not considered possible.

2.4.8 COOLING-WATER CANALS AND RESERVOIRS

Not applicable.

2.4.9 CHANNEL DIVERSIONS

The plant site is located ~ 1/2 mile below U.S. Highway No. 1. The river channel is relatively straight for a distance of 1.5 miles below U.S. No. 1. Thus, there are no meanders at the plant site which could be cut across to divert the flow. The U.S. No. 1 bridge and highway fill serve to control the channel

alignment to its present location. Thus, the channel alignment from the bridge to the plant site is relatively stable. The Altamaha River was surveyed by the U.S. Army Corps of Engineers in about 1900. It is reported that no major changes in channel alignment have occurred in subsequent years. Corps of Engineers personnel estimate that an oxbow meander is cut off about once every 100 years. Observation of river patterns shows that meanders develop very slowly. Thus, any possible effect on water supply to the river intake from channel changes should come from extremely slow changes which can be remedied as they occur.

2.4.10 FLOODING PROTECTION REQUIREMENTS

The topography of the HNP is such that none of the safety-related facilities are exposed to river flooding by the most severe flood at the site. The probable maximum discharge concurrent with a severe wind of 45 mph would develop a wave height of 6.5 ft (crest to trough). The elevation of the wave crest due to a sustained wind of 45 mph and at a PMF would be at el 108.3 ft msl. This is well below the plant grade of el 129 ft msl and, therefore, flooding is improbable from this source.

The intake is a safety-related structure. The pumps and motors are housed above the most severe flood stage. The floor elevation of the two outside access doors is at el 111.0 ft msl (table 2.4-1). There are no safety-related systems or components located below the design maximum flood elevation that are not protected against flooding. The foundation slabs and exterior walls of safety-related structures are designed to resist upward and lateral pressures caused by the maximum flood level.

The reinforced concrete intake pump structure walls which may be affected by the wave runup are designed for an impact load of 4000 lb at a windspeed of 50 mph over an area of 25 ft^2 (section 3.4).

The plant is located on high ground adjacent to the river. Minor surface area drainage into the plant is carried by the yard drainage. The nearest adjacent tributary drains the area south and east of the plant and has an area of 21 mi^2. The flowline of this drainage at the point nearest the plant is at el 75 ft. The crest of the ridge between the plant and flowline is el 125 ft. The flow area is adequate to remove the probable maximum rainfall on the drainage area without flooding the plant.

Another possible source of flooding is severe local precipitation. The power block area is not in the path of any watershed drainage; moreover, the location of the plant is such that the runoff from local precipitation is directed away from the power block both by natural drainage and by a combined system of culverts to the natural drainage channel (paragraph 2.4.2.3).

The plant drainage system is designed for a maximum precipitation of 6 in./h and has been checked to ensure that flooding of safety-related equipment does not occur as a result of local intense precipitation equal to the PMP. The roofs of all safety-related structures are designed to pass the local PMP corresponding to the time of concentration.

2.4.11 LOW-FLOW CONSIDERATION

2.4.11.1 Low Flow in the Altamaha River

Minimum stream flows and related stages may influence water supply to the intake pumps and the plan of operation of the plant. For these reasons, a very low-flow river stage-discharge relationship (discharge rating curve) was developed for the Hatch intake structure location as outlined below.

The Hatch intake structure is located on the Altamaha River at approximately river mile 116.4. The nearest USGS gage is: 02-2250, Altamaha River Near Baxley, Georgia. The Baxley gage is located on the south bank of the river (same side as the intake structure) ~ 400 ft downstream from the bridge on U.S. Highway No. 1, and 2750 ft upstream from the Hatch intake structure. The discharge rating curve at the intake structure was developed by making appropriate adjustments to the low-flow discharge rating curve at the Baxley gage. The USGS performs bathmetric surveys of the river cross-section at the Baxley gage and measures a river-stage relationship on the average every 6 weeks. From time to time, as needed, the USGS revises the rating table at the Baxley gage when adequate additional data are collected, and the new data show the river bottom has stabilized. Rating Table No. 11 was developed in October 1994. The discharge rating curve shown in figure 2.4-34 was developed using the USGS measurement data taken since Rating Table No. 11 was published. The drop in water surface level from the Baxley gage to the Hatch intake depends on the quantity of discharge in the river. A higher discharge would result in a steeper hydraulic gradient and, therefore, in a greater drop between the two sites, and vice versa. A drop of 0.1 ft was determined by level survey, from the Baxley gage to the Hatch intake, when the river elevation at the Baxley gage was 62.0 ft msl. The discharge rating curve at the Hatch intake structure was developed by adjusting to the Baxley discharge rating curve, as shown in figure 2.4-34. At the Hatch intake structure, the river level would be 61.4 ft msl for 1200 ft^3/s, which is the low flow of record at Charlotte gage and 60.9 ft msl for the hypothetical minimum flow of 950 ft^3/s at the intake structure.

In accordance with plant procedure, the river stage-discharge rating curve at the Hatch intake structure is determined at least twice per 12 months, including a low-flow extension to el 60.0 ft msl. The rating curve is verified to ensure that adequate water supply to the intake pumps is available for at least 30 days (required for a safe plant shutdown operation) when the river level falls to el 60.8 ft msl, which corresponds to 60.7 ft msl in the pump well of the intake structure.

In accordance with plant procedures, the USGS discharge-stage measurements taken since the last verification are analyzed, and the Hatch rating curve is adjusted every 6 months (at the middle and end of the year). The rating curve is evaluated to determine whether there is any adverse effect on water supply to the intake pumps. The rating curve is then used to estimate the number of days for which water supply would be available between river el 60.8 and 60.0 ft msl. Therefore, for up-to-date information on the Plant Hatch rating curve for low flows, refer to the most recent calculation, "Plant Hatch River Stage-Discharge Curve Verification."

2.4.11.2 Low Water Resulting from Surges, Seiches, or Tsunamis

Not applicable.

2.4.11.3 Historical Low Water

Stream flow data near the site are available from several gages as shown below:

Gage	*Agency*	*River Mile*	*Drainage Area (mi^2)*	*Period of Record Available*	*Type of Record*	*Lowest Flow of Record*
Charlotte	*USWB*[(a)]	*135.7*	*11,550*	*1925-*	*Daily gage*	*1200 ft^3/s (1925)*
Baxley	*USGS*	*116.9*	*11,600*	*1949-1951* *1970 -*	*Daily gage* *Recording*	*1620 ft^3/s (1986)*
Doctortown	*USGS*	*59.4*	*13,600*	*1931-*	*Recording*	*1430 ft^3/s (1954)*
	USWB			*1925-1931*	*Daily gage*	

Analysis of the records for these stations showed that 1925, 1954, 1986, and 1988 were periods of drought. The only reservoir in existence in 1925 was Lloyd Shoals.

In this 1925 period, the Charlotte gage had 23 consecutive days of low flow. In 1954 both Lloyd Shoals and Sinclair Reservoirs were operative. In that year, the Charlotte gage had 31 consecutive days of flow between 1300 and 1400 ft^3/s, while at Doctortown there were 23 days of low flow. In 1954 the low-flow period occurred in the latter part of October. During that month, there was no generation at Lloyd Shoals Dam except for 1700 kWh on October 22. The Lloyd Shoals reservoir level was constant at el 507.4 ft from October 22 to October 29, indicating inflow was equal to evaporation and leakage. At Sinclair Dam, the reservoir was drawn down 0.8 ft during the period from October 15 to October 31. This decrease in storage is equivalent to an average of 300 ft^3/s from the reservoir. Estimated evaporation losses from the 2 reservoirs, Lloyd Shoals and Sinclair, were 75 to 100 ft^3/s. Thus, the net addition to river flows due to Lloyd Shoals and Sinclair Dams was ~ 200 ft^3/s. Data from Lloyd Shoals operation for a similar analysis of 1925 flows are not available, but based on the similar period in 1954 were probably minimal.

During the past 48 years of measurement (1931 to 1979) at the Doctortown gage 57.5 river miles downstream from the plant site, the minimum daily flow occurred in November 1954 and was 1430 ft^3/s, corresponding to an elevation at the plant site estimated at 62.8 ft msl.

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a. USWB - United States Weather Bureau.

2.4.11.4 Future Control

For large rivers such as the Altamaha, there is some minimum base flow that can be sustained from ground water and aquifer flows. This is apparent by the number of days of essentially steady flow in the low-flow periods. An analysis of annual minimum flows at Charlotte indicate that this extrapolated hypothetical minimum natural (without reservoir supplementation) low flow is 950 ft^3/s and that the low flows are approaching this limit asymptotically. This is considered a very conservative low-flow estimate.

2.4.11.5 Plant Requirements

The minimum low flow is important because of its effect on the operation of plant service water (PSW) and RHRSW pumps. The RHRSW pumps at rated flow conditions require for net positive suction head (NPSH) a river stage of only 59.0 ft which corresponds to a flow of less than 100 ft^3/s. Thus, no further consideration is required on river stage with regard to submergence of these pumps.

The PSW pumps at rated conditions of about 8500 gal/min-pump require a stage in the pump well of 61.2 ft. Normal operation requires about 7840 gal/min for each of three pumps. Shutdown or emergency conditions require only 1 pump with a discharge of 4428 gal/min. Therefore, the applicant provides means for local measurement of level in the intake pump well. If the level in the well should reach the low level of 61.2 ft (~ 1200 ft^3/s river flow) the applicant throttles discharge flow from PSW (unless previously accomplished) such that maximum flow does not exceed 7000 gal/min. Means are provided for measurement of service water flow. The ability to achieve the pump flow rate for minimum required NPSH of 35.5 ft was demonstrated during the preoperational testing of the HNP-1 service water pumps, which are the same as those of HNP-2. The rating curve (figure 2.4-34) is based on records obtained from the USGS Altamaha River Gage No. 02-2250 just downstream of the U.S. No. 1 bridge near Baxley, Georgia. No additional work has been done at this station or at the U.S. Highway No. 1 bridge except the erection of a temporary weir across the river downstream of the intake structure during a period of low flow. This measure was taken to increase the effective water level at the intake structure but was later removed. The rating curve is verified at regular intervals. If any shift occurs in a manner which would adversely affect the water supply to the pumps, appropriate action is taken to maintain the water-supply capability under low-flow conditions.

2.4.11.6 Heat Sink Dependability

Technical Specification requirements relative to the ultimate heat sink assure that Plant Hatch is protected against essentially incredibly low flows. In these analyses, credit is not taken for Sinclair Dam discharge, since the HNP site was not given credit for Sinclair Dam's minimum instantaneous flow requirements imposed by the Federal Energy Regulatory Commission license.

Also, close surveillance is given to maintaining the depth of the approach channel in the river during periods of low river flows to ensure that water is available to the pumps. In this respect, the Altamaha River on the centerline of the intake structure was first sounded on July 18, 1974. GPC continues to monitor the bottom conditions on a yearly basis in late spring or early summer and take appropriate

action to maintain water supply under all conditions. Figure 2.4-43 shows river bottom profiles for the years as indicated. It shows that some siltation has taken place but is not considered excessive or necessarily a trend.

2.4.12 ENVIRONMENTAL ACCEPTANCE OF EFFLUENTS

The liquid radioactive releases occurring during the full range of operating conditions, the dilution factors used in evaluating these releases, and the doses resulting from these releases are discussed in subsections 11.2.6, 11.2.8, and 11.2.9, respectively.

See paragraphs 2.4.1.2 and 2.4.13.2 for locations and users of surface and ground waters, respectively.

The ultimate heat sink design parameters are covered in subsection 9.2.5.

There are no known safety-related effects of normal or accidental releases of radionuclides and heated water on surface and ground waters. Should an accidental spillage of radioactive liquid on the surface of the ground at the plant occur, the liquid would enter the ground and then slowly move to the river. The soils at the site are relatively impermeable and it is expected that such movement would be very slow (permeabilities are given in table 2.4-9). The possibility of affecting offsite wells by spills into the ground water at the site is very remote.

Liquid plant wastes suitable for release to the environment are discharged into the Altamaha River. The discharge structure is located ~ 1300 ft downstream of the intake structure to prevent any recirculation of liquid waste. Even in the case of low river flows, the flow is sufficient to carry the waste downstream and prevent recirculation to the plant's intake structure.

2.4.13 GROUND WATER

This section presents the results and conclusions of the ground water investigations for HNP. The investigations were performed by Law Engineering Testing Company, GPC, and Bechtel Corporation.

2.4.13.1 Description and Onsite Use

The site is within the coastal plain province of Georgia which extends from the fall line on the north to Florida on the south and from the Savannah River on the east to the Chattahoochee River on the west (figure 2.5-1). Sediments underlying the coastal plain consist of alternating beds of sand, gravel, clay, limestone, and marl that dip southeastward slightly more than the regional ground surface. These strata outcrop in belts nearly parallel to the present Atlantic coastline. Water enters the permeable sand, gravel, and limestone aquifers principally by direct infiltration of precipitation in their outcrop areas, and migrates downdip. The permeable strata lie between relatively impermeable layers of clay, marl, and silty or clayey sand. This configuration results in artesian conditions downdip from the aquifer recharge areas.[7]

Most of the large ground water supplies withdrawn from the coastal plain sediments are provided by the artesian or confined aquifers described in the previous paragraph. Ground water supplies may also be obtained from shallow, unconfined aquifers under water-table conditions. The unconfined aquifers consist of surficial sand and gravel deposits that underlie terraces in upland areas and floodplains adjacent to larger streams. Perched water zones, which are discontinuous lenses or layers of permeable material overlying relatively impermeable strata, locally provide minor and unreliable supplies of ground water. Recharge to the perched water zones and unconfined aquifers occurs locally where the water-bearing units are exposed to infiltration of precipitation.[(7)]

A small amount of ground water is used for plant operations (paragraph 2.4.13.1.3). This water is withdrawn from two high-capacity wells that are open to very permeable limestone. The characteristics of the regional and local aquifers and a description of plant and local water usage are provided in the following paragraphs.

2.4.13.1.1 Regional Aquifers

The major aquifer underlying the region in which the site is located is referred to as the principal artesian (or limestone) aquifer. In the southeast Georgia area, the aquifer ranges from 450- to 1500-ft thick.[(8)] *It consists chiefly of limestone, with occasional layers of dolomite, sand, silt, clay, and marl.*[(9)] *Geologic units comprising the aquifer range in age from Middle Eocene to Miocene and include, in ascending order, limestones of the Claiborne group, the Ocala limestone, the Suwanee formation, and the Tampa formation.*[(8)] *In some areas, sandy limestones in the lower part of the Hawthorn formation (Miocene) are considered part of the principal artesian aquifer. These limestones act as a hydrologic unit and provide over 70% of the ground water used in Georgia.*[(8)] *Above and below the aquifer are low-permeability beds that confine the water in the limestones. The upper confining bed consists of clay of the Hawthorn formation of Miocene age. Middle Eocene age clay and limestone of the McBean formation in eastern Georgia, and the lower Lisbon and Tallahatta formations elsewhere in Georgia provide the lower confining beds. The aquifer and confining beds dip gently to the southeast, with resultant artesian conditions occurring a short distance downdip from the recharge areas.*[(8)]

The outcrop areas serve as the source of recharge for the principal artesian aquifer.[(7)] *The main recharge area, over 60 miles northwest of the site, contains nearly 8500 mi*^{2} *in a northeast-southwest trending belt that extends from south of Augusta, Georgia, to the vicinity of Dothan, Alabama. Other areas of recharge include about 1000 mi*^{2} *in and to the south of the Okefenokee Swamp, and ~ 500 mi*^{2} *in the area of Valdosta, Georgia. Precipitation, which provides most of the recharge, averages 44 to 55 in. annually in the outcrop areas.*[(7)] *Recharge occurs downward through the exposed rock and from sinkholes in the limestone. Adjacent to the main outcrop belt, where the aquifer is underlain and overlain by sands, recharge takes place upward and downward from these sand units. These units include portions of the Hawthorn formation overlying the aquifer, and sands of Cretaceous and Paleocene ages underlying the aquifer.*[(9)]

The characteristics of the principal artesian aquifer vary widely in southeast Georgia. The southeastward slope of the potentiometric surface ranges from 1.5 to 15 ft/mi for a distance of ~ 25 miles downdip from the recharge area (figure 2.4-35). About 10 miles northwest of the site, the slope of the potentiometric surface changes to < 2 ft/mi and continues at that gradient almost to the coast. These

lower gradients appear to be due to an increase in the thickness and permeability of the aquifer downdip, enabling the limestone to transmit under much lower gradients the water supplied to it from the northwest.[10] Values for transmissivity are between 72,000 and 90,000 gal/day/ft where the gradients are steep, while from Appling County southeastward, transmissivity values range from 130,000 to 2,000,000 gal/day/ft.[9] The average transmissivity of the aquifer in this area is 220,000 gal/day/ft. Specific capacities of 47 wells penetrating the aquifer in different parts of the Georgia coastal plain range from 1.1 to 240 gal/min/ft of drawdown. Most of the wells penetrated over 100 ft of aquifer and had specific capacities > 50 gal/min/ft of drawdown. In general, the specific capacity of wells within the aquifer increases with increased exposure to the more permeable zones in the aquifer.[8]

Water-table conditions occur in the sediments overlying the principal artesian aquifer. The amount of water within these unconfined aquifers is largely dependent on their water-bearing characteristics, although large supplies are generally not available. Within the site region, the unconfined aquifers range in age from Late Miocene to Holocene and include sandy portions of the Hawthorn formation, the Pleistocene terrace deposits, and alluvium of Holocene age adjacent to the major rivers.[7]

The Hawthorn formation consists of clay, silt, sand, limestone, and dolomite. Some of the sandy limestone and dolomite beds are minor aquifers and furnish water for local rural supplies. Individually, the waterbearing lithologies are thin, discontinuous, and comprise only a small percentage of the total volume of the formation. Yields in the most permeable sections are generally < 200 gal/min. In some areas, water in the lower Hawthorn formation may exhibit artesian conditions. The most important function of the Hawthorn formation is to confine water in the underlying principal artesian aquifer.[9] The Pleistocene terrace deposits are similar to material in the Hawthorn formation and consist of varying amounts of sand, gravel, silt, and clay. They generally occupy upland areas between the major river valleys. Owing to rapid horizontal and vertical facies changes within the terrace deposits, they are unreliable as sources of ground water except for small rural supplies. Sand and gravel layers at the base of the deposits may provide over 10 gal/min per well, while perched lenses usually yield a smaller quantity of ground water.[8] The alluvium adjacent to the larger streams consists of sand, clay, and gravel and could potentially provide large supplies where deposits are hydraulically connected to streams. The Holocene alluvium is not generally used to supply ground water.[8]

2.4.13.1.2 Local Aquifers

Ground water in the vicinity of the site is obtained from five water-bearing strata. These are, in ascending order, the principal artesian aquifer, the middle and upper parts of the Hawthorn formation, the Brandywine formation, and the Altamaha River alluvium. The characteristics of these strata have been obtained from published reports and from data obtained during testing and operation of wells drilled into the water-bearing units.

The principal artesian aquifer beneath the site consists of the Lisbon formation of Middle Eocene age, the Ocala formation of Late Eocene age, the Suwanee formation of Oligocene age, and the Tampa and extreme lower Hawthorn formations of Miocene age. The Ocala and Suwanee formations are chiefly limestones, while the Lisbon, Tampa, and extreme lower Hawthorn formations are sandy limestone and calcareous clayey sand. These formations are overlain and confined by fine sand and sandy clay in the lower part of the Hawthorn formation. The lower confining bed consists of sandy clay of the Tallahatta

formation. The principal artesian aquifer is ~ 1000-ft thick beneath the site, with the top at approximate el -105 ft msl (figure 2.4-35). Recharge to the aquifer occurs at the outcrop area ~ 60 miles northwest of the site.

Eleven wells within 2 miles of the site extend into but do not completely penetrate the principal artesian aquifer (table 2.4-7). The bottoms of these wells range from el -117 ft msl to el -570 ft msl, with the intervals exposed to the aquifer known for seven of the wells. For the two wells on which tests were made (site wells 1 and 2), the specific capacity of the aquifer ranged from 100 to 125 gal/min/ft of drawdown. Transmissivity in the site vicinity is ~ 130,000 gal/day/ft. The effective permeability is between 0.1 and 0.2 ft/min, based on published transmissivity and thickness data.[(8)(9)] Properly designed individual wells drilled into the aquifer can safely yield over 1100 gal/min.

The water levels in wells drilled into the principal artesian aquifer are discussed in paragraph 2.4.13.2.2. The potentiometric surface of the aquifer in the vicinity of the site is generally between el 49 ft msl and el 60 ft msl, sloping gently to the southeast.

The upper part of lithologic unit 1 and all of unit 2 of the Hawthorn formation, as defined in paragraph 2.5.1.2.2, form an aquiclude between the principal artesian aquifer and an overlying confined aquifer within the Hawthorn formation. The aquiclude consists of fine sandy clay and is between 100- and 110-ft thick. The top of this zone is near sea level (el 0 ft msl). Published permeability values for the aquiclude are < 1×10^{-7} ft/min.[(8)(9)]

The middle portion of the Hawthorn formation, between approximate el 65 ft msl and sea level, contains a minor confined aquifer. The vertical limits of the aquifer are within lithologic units 3 and 4 of the Hawthorn formation (paragraph 2.5.1.2.2). The aquifer consists of fine to coarse sand and clayey sand. It is confined between sandy clay and clay in the upper Hawthorn formation (upper confining bed) and sandy clay in the lower Hawthorn formation, described above. The thickness of the aquifer under the site is ~ 65 ft. Recharge occurs locally to the southwest of the site, where this part of the Hawthorn formation is exposed. Natural discharge occurs where material in the aquifer is in contact with the alluvium underlying the Altamaha River floodplain, generally below el 60 ft msl.

Field tests conducted at the site indicate that the permeability of the minor confined aquifer ranges from 2.5×10^{-4} to 4.1×10^{-4} ft/min. The permeability generally increases with increasing depth in the aquifer and decreasing silt content of the water-bearing sand. Water levels of the aquifer, discussed in detail in paragraph 2.4.13.2.2, range between el 67 ft msl and el 85 ft msl. The potentiometric surface of the aquifer has a gradient of ~ 23 ft/mi to the north, toward the Altamaha River. Although no data on maximum safe yield are available for the site area, the low permeability values indicate that individual wells yield < 10 gal/min.

The confining bed overlying the minor confined aquifer corresponds with lithologic unit 5 and the upper part of lithologic unit 4 of the Hawthorn formation (paragraph 2.5.1.2.2). It consists chiefly of sandy clay and clay, with locally cemented sand layers. The confining bed is ~ 40- to 50-ft thick, with the irregular top generally at el 100 ft msl to el 120 ft msl. Permeabilities determined from field tests in sandy zones of the confining bed were < 2.0×10^{-6} ft/min.

Unconfined ground water exists within the upper part (lithologic unit 6) of the Hawthorn formation. This is the surface unit over most of the site south of the Altamaha River. The base of this aquifer corresponds with the irregular top of the confining bed described above. The base is at approximate el 120 ft msl, although in the southeastern part of the site and near the Altamaha River, it is at approximate el 100 ft msl or less. The aquifer consists of clayey sand, about 45- to 50-ft thick, that becomes less clayey with depth. The high clay content near the top of the aquifer and at the ground surface locally forms discontinuous, relatively impermeable zones. Recharge to the aquifer occurs by infiltration of precipitation through and around the leaky clay zones.

The permeability of the unconfined aquifer, as determined by field tests on a site borehole extending to el 132 ft msl, was ~ 1.5×10^{-3} ft/min. Tests in two pits exposing the aquifer between el 132 ft msl and el 128 ft msl yielded permeabilities of 1.1×10^{-3} ft/min and 1.7×10^{-3} ft/min. No data on the maximum yield from this aquifer in the site area are available. However, 33 wells within 2 miles of the site utilize ground water from the unconfined aquifer for domestic purposes throughout the year (see paragraph 2.4.13.2.1). The low permeability values indicate that a yield of less than 10 gal/min per well may be expected.

Water levels in the unconfined aquifer are discussed in detail in paragraph 2.4.13.2.2. In summary, they range from el 148 ft msl west of the site to < el 100 ft msl east of the plant area. The water table reflects the topography of the site area, with high water levels underlying hills and low water levels near valleys. The flow direction is north and east toward the Altamaha River floodplain, along gradients ranging from 14 to 80 ft/mi.

The relatively impermeable clay zones at the top of the Hawthorn formation cause perched water conditions in the overlying Brandywine formation. The Brandywine is Pliocene(?) to Pleistocene in age and caps the hills at and around the site above approximate el 165 ft msl to el 170 ft msl. Maximum known thickness in this area is ~ 20 ft. The perched aquifer consists of poorly sorted sand and gravel and is recharged by precipitation in the site area. Perched ground water is found only locally in the deposits, although ten wells near the site (table 2.4-7) dug or drilled into the aquifer contain water throughout the year. A few springs occur at the base of the Brandywine about 1.5 miles southwest of the plant. During drier seasons, these springs are dry, owing to lack of recharge. No values for permeability or maximum safe yield have been recorded for the Brandywine formation in the site area. A discussion of the perched water levels is included in paragraph 2.4.13.2.2.

Pleistocene(?) to Holocene age alluvium underlying the Altamaha River floodplain contains ground water under water-table conditions. The alluvium consists of up to 55 ft of poorly sorted sand, gravel, and clay. The top of these deposits is generally below el 75 ft msl. Recharge to the alluvial aquifer is provided mainly by infiltration of local precipitation. Recharge is also provided by discharge from the Altamaha River during high stages and by the minor confined aquifer in the Hawthorn formation, to which the alluvium is hydraulically connected. The alluvium is a potential source of large quantities of water,(8) although only two shallow dug wells near the site are open to the aquifer (table 2.4-7).

2.4.13.1.3 Plant Wells and Ground Water Requirements

Two onsite water wells provide ground water for plant usage. The locations of the wells are shown on figures 2.4-36 and 2.4-37. The wells draw water from the Ocala limestone and the Suwanee formation which are part of the principal artesian (limestone) aquifer. Well No. 1 has a surface elevation of 109.5 ft msl and is 680-ft deep. The top 455 ft are cased and sealed with cement grout to prevent seepage of water into the well bore from higher water-bearing strata. The interval from a depth of 455 ft to 680 ft (el 345 ft msl to el 570 ft msl) is open to the limestone aquifer. The static water level from this interval was at a depth of 59 ft (el 50 ft msl) on November 21, 1969. Well No. 2 has a surface elevation of 151.9 ft msl and is 711-ft deep. The well bore is cased and sealed above a depth of 490 ft with cement grout. Between 490 ft and the bottom (el 339 ft to el 559 ft msl) the well is open to limestone. On the test date, September 3, 1969, the static water level was at a depth of 103 ft (el 48.9 ft msl). As-built drawings at the two site wells are included as figure 2.4-38.

Each well is equipped with a 100-hp electric suction pump. Well No. 1 was pumped for 9 h at 752 gal/min. Drawdown was 5 ft, indicating a specific capacity of 125 gal/min/ft of drawdown. Well No 2 was pumped for 9 h at 797 gal/min. Drawdown stabilized at 8 ft, indicating a specific capacity of nearly 100 gal/min/ft of drawdown. The maximum rate of withdrawal in well No. 1 was 1120 gal/min, with 10 ft of drawdown. No attempt was made to determine the maximum rate of withdrawal for well No. 2.

The initial use of ground water from these wells is to fill the following storage tanks:

	Total Capacity (gal)
Fire protection tanks (2)	*600,000*
Demineralized water storage tank	*100,000*
Sanitary water storage tank	*20,000*
Filtered water storage tank	*100,000*

After the tanks are filled, normal plant operation requires 374,880 gal/day of ground water. Of this amount, 364,800 gal/day is supplied as makeup to the demineralizer at a rate of 320 gal/min for 19 h/day. The remaining 10,080 gal/day is routed through the sanitary water storage tank to the sanitary water system at a rate of 7 gal/min for 24 h/day. Ground water is not used for emergency cooling. However, ground water is used as makeup for the fire protection system as described in the HNP Fire Hazards Analysis and Fire Protection Program.

2.4.13.2 Sources

2.4.13.2.1 Present and Projected Ground Water Use

To determine the use of ground water in the site region, a survey of water users within ~ 2 miles of the plant was conducted. The results of the survey are presented on table 2.4-7, with the well locations shown on figure 2.4-36. Of the 61 wells located, 5 are abandoned, 9 are open to the principal artesian aquifer, two are screened in the minor confined aquifer, 33 draw water from the unconfined aquifer, 10 draw from the perched (Brandywine) deposits, and 2 are open to the Altamaha River alluvium. The two site wells which obtain water from the principal artesian aquifer are also listed in table 2.4-7. The primary use of water from the local wells is for domestic needs, with a limited amount for livestock. Well water is not used for irrigation. Most wells are equipped with pumps, although a few dug wells use bucket lifts. There are no large tanks near the site for storage of ground water; however, there are several storage ponds southwest of the site which utilize surface runoff for stock and crop watering. Requirements and usage of the two site wells are presented in paragraph 2.4.13.1.3.

At present, there is no industrial demand for ground water within the site area. The nearest appreciable amount of ground water withdrawal is 10 miles south of the site, where the town of Baxley has three wells which withdraw a total of 250,000 to 300,000 gal/day from the principal artesian aquifer. The wells are slightly down-gradient from the site. Water storage for the town is provided by two 60,000-gal tanks.

An estimated 115 people live within 2 miles of the plant (figure 2.1-4). The suggested normal per capita use for this area is ~ 65 gal/day.[7] Based on these figures, the total present usage from all aquifers is estimated to be 7475 gal/day, or ~ 5.2 gal/min. The population within the same area is expected to increase to about 125 by the year 2012 (figure 2.1-4). By conservatively assuming that per capita use will increase to 100 gal/day, the total projected ground water usage by the year 2012 is estimated to be 12,500 gal/day, or ~ 8.7 gal/min.

2.4.13.2.2 Piezometer Installations and Piezometric Levels

Fifty-six piezometers have been installed at and near the site to allow monitoring of water levels in the unconfined and minor confined aquifers in the Hawthorn formation. Five additional piezometers are open to the sandy clay separating these two aquifers to detect the presence and levels of water in the aquiclude. The locations of the piezometers are shown on figure 2.4-37, with data concerning the piezometers presented on table 2.4-8. Forty-four of these piezometers were destroyed during construction activities. There are currently 17 active piezometers. No piezometers are used to monitor water levels in the principal artesian aquifer or the Brandywine (perched) aquifer. Water levels in the principal artesian aquifer were noted during tests on the two site water wells (paragraph 2.4.13.1.3) and during the local well survey. The potentiometric surface in the site vicinity was at approximate el 70 ft msl in 1944.[10]

As of 1968, the potentiometric surface had declined to elevations ranging from 49 ft msl to 60 ft msl. The decline in the site area accompanied a general decline of the potentiometric surface in the entire

southeast Georgia area. This decline has been attributed to excessive pumping from the aquifer along the Atlantic coast.[9] The range in the water levels in the site vicinity represents withdrawal from different portions of the principal artesian aquifer. Wells completed in the upper part of the aquifer generally have higher water levels than those drawing from deeper parts of the aquifer. The potentiometric surface of the aquifer may fluctuate as much as 10 ft seasonally or following heavy rain in the recharge areas.[8]

Water levels in the perched (Brandywine) aquifer were measured during the local well survey. The levels ranged from el 161 ft msl to el 174 ft msl. Local residents stated that springs discharging from the base of the perched aquifer went dry during periods of extended drought, although no dug wells drawing from the aquifer had gone dry.

Contours of the natural water level in the unconfined aquifer, shown on figure 2.4-39, illustrate the correlation of the water surface with the configuration of the terrain. The highest water levels occur beneath the crest of the hill, west of the plant site, and along the spurs radiating from the hill. Flow direction is downslope toward the river and small tributary drainage channels. Gradients range from 0.0026 to 0.015. The underlying aquiclude precludes significant downward percolation, so ground water in the aquifer moves laterally to the stream channels. The top of the aquiclude is irregular, ranging in elevation from ~ 100 ft msl to 120 ft msl. Where the tributary channels have cut below the top of the aquiclude, the unconfined water is discharged at the ground surface through springs.

The natural potentiometric surface of the confined water within the minor confined aquifer underlying the aquiclude is shown on figure 2.4-40. The contours indicate that the configuration of the potentiometric surface is not related to surface topography. The predominant direction of ground water movement within the aquifer is north, toward the Altamaha River. The gradient is about 0.0043. The river channel has cut below the base of the aquiclude under the flood plain, providing hydraulic contact between the river alluvium and the minor confined aquifer.

Since periodic monitoring of site piezometers began in late 1969, the levels in the unconfined and minor confined aquifers have shown little fluctuation (figure 2.4-41). Other than one weekly series of isolated measurements (which are believed in error) and the effects of the dewatering operation, water levels of both aquifers have fluctuated less than 10 ft. Future ground water levels should remain essentially where they are.

Fluctuations in the potentiometric surface of the minor confined aquifer respond closely to fluctuations of the river level (figures 2.4-41 and 2.4-42). Potentiometric levels prior to April 24, 1970, were controlled primarily by the dewatering operations for the construction excavation. After recovery of the levels following deactivation of the dewatering system, the hydrographs (figures 2.4-41 and 2.4-42) demonstrate that levels in the minor confined aquifer are higher than the river level most of the time. During high flood flows, a temporary flattening of the hydraulic gradient may occur. Reversals in the normal gradient due to pumping are discussed in paragraph 2.4.13.2.4.

2.4.13.2.3 Permeability and Porosity Values

Values of permeability and porosity for the relevant geologic formations beneath the site are shown on table 2.4-9. Field permeability tests were conducted utilizing piezometer walls as inflow wells. The field tests were falling-head type in sealed piezometers and constant-head permeameter type. These tests were conducted in accordance with the NAVDOCKS and Bureau of Reclamation (E-18) procedures.(11)(12) Permeability values for formations not tested at the site were obtained from publications.(7)(8)(9)(10) Porosity values were determined from laboratory test data.

2.4.13.2.4 Reversibility Potential and Withdrawal Effects

No reversal of the gradients of the unconfined and minor confined aquifers at the site should occur as a result of present or future offsite or onsite pumping. The closest area of concentrated ground water withdrawal is at Baxley, Georgia, 10 miles south of the site. The present rate of withdrawal from the three city wells at Baxley is relatively small (up to 300,000 gal/day). Water is extracted exclusively from the principal artesian aquifer, which is hydraulically isolated from the unconfined and minor confined Hawthorn formation aquifers. The city wells are slightly down-gradient from the site. Pumpage for the Baxley municipal water system, therefore, has no effect on water levels in the Hawthorn formation aquifers underlying the site.

A water usage survey conducted in August and September 1967 revealed that there were 61 wells within 2 miles of the site, of which 5 were abandoned (table 2.4-7). Thirty-three of the remaining wells (59% of the total wells in use) obtained water from the unconfined aquifer, serving an estimated 68 people (59% of the total population within 2 miles of the plant). At the per capita rate of 65 gal/day(7) ~ 4420 gal/day is withdrawn from the unconfined aquifer. This represents an average rate of withdrawal of less than 0.1 gal/min per well. There is no apparent influence from the existing wells on the present unconfined water surface (figure 2.4-39). Therefore, continued usage causes no reversal of the gradient of the unconfined aquifer.

The population within 2 miles of the plant is expected to increase to 125 by the year 2012 (figure 2.1-4), representing an increase of 10 people. If all 10 people use well water obtained from the unconfined aquifer, and the per capita usage by the year 2012 increases to 100 gal/day, then a total of 7800 gal/day will be withdrawn from the aquifer. Assuming the present ratio of people per well remains at ~ 2 to 1, a total of 39 wells will be pumping from the aquifer. The rate of withdrawal will be ~ 0.14 gal/min per well, or an increase of < 0.05 gal/min per well over the present withdrawal rate. This low increase of usage indicates that the possibility of a reversal of the present ground water flow in the unconfined aquifer due to offsite overdraft is extremely remote.

Two wells, serving an estimated four people, draw water from the minor confined aquifer within 2 miles of the site. They withdraw an estimated total of 260 gal/day, or < 0.1 gal/min per well. If the present 4 people and the 10 people expected to enter the area by 2012 use well water obtained from the minor confined aquifer at a per capita rate of 100 gal/day, the total withdrawal from the aquifer will be about 1400 gal/day. Assuming the present ratio of people per well remains at 2 to 1, a total of seven wells will be pumping from the aquifer. The rate of withdrawal will be ~ 0.14 gal/min per well, or an increase of

0.05 gal/min over the present withdrawal rate. This low increase of usage is not expected to alter the existing ground water flow in the minor confined aquifer.

There are no onsite wells drawing water from the two Hawthorn formation aquifers. Water for plant usage is supplied by two wells open to the principal artesian aquifer at a normal rate of 374,880 gal/day (paragraph 2.4.13.1.3 and figures 2.4-36 through 2.4-38). The wells are ~ 1780 ft apart. For 19 h/day, the rate of withdrawal is 327 gal/min; with only one well in operation, the corresponding maximum drawdown at the pumping well is < 4 ft. During the remaining 5 h, withdrawal is 7 gal/min, resulting in considerably less drawdown at the well. Because of the distance between wells and the low drawdown, there is no interference between the two wells. Normal plant operation does not significantly affect the natural southeastward slope of the potentiometric surface of the principal artesian aquifer.

If required, each site well is capable of producing 750 gal/min of ground water, resulting in a temporary maximum drawdown of 7.5 ft per well. There is no interference between wells at this rate of withdrawal and drawdown. Since the duration of any such pumping is short, there is no significant effect on the potentiometric surface of the principal artesian aquifer. Ground water is not used for emergency cooling.

2.4.13.2.5 Recharge Areas Within Plant Influence

There are no significant areas of recharge within the influence of the plant. Since the northern boundary of the plant site extends beyond the Altamaha River, the alluvium in the river valley at the site is not used as a source of ground water for local residents. All other local recharge areas are up-gradient from the plant and supply water to aquifers not used by the plant.

2.4.13.3 Accident Effects

Normal operation of the plant has no adverse effect on the ground water systems in the site vicinity. All ground water for plant usage is derived from wells in the deeper principal artesian aquifer.

In the unlikely event of an accidental release of radioactive contaminants into water-bearing strata at the plant, movement of the contaminants into the nearest potential potable water supply (the Altamaha River) would be affected by several factors. First, ion exchange and absorption properties of the soil would retard the migration of contaminants to some extent, and the concentration of the ions moving with the ground water would be reduced. Secondly, downward movement of the contaminants would be contained within the minor confined aquifer in the Hawthorn formation, the top of which is ~ 8 ft below the radwaste building foundation. This limitation of downward movement is due to the confining beds beneath the minor confined aquifer and to upward artesian pressures associated with the deeper principal artesian aquifer. Construction of the plant wells (completed in the principal artesian aquifer) includes a cement grout seal from the ground surface through the top of the aquifer to prevent seepage downward along the well bore.

For an analysis of the rate of movement of contaminated ground water, the most probable leak or spill location is the HNP-2 radwaste building. The most conservative (shortest) path for ground water

movement from the HNP-2 radwaste building to the nearest potable water supply was determined on the basis of the following assumptions:

A. *The nearest potential potable water supply is the Altamaha River.*

B. *Ground water moves down-gradient from the leak or spill location to the river along a straight path.*

C. *The shortest (and fastest) route of travel is within the minor confined aquifer, the top of which is ~ 8 ft below the HNP-2 radwaste building foundation.*

D. *The thin layer of aquiclude material between the radwaste building foundation and the minor confined aquifer does not prevent rapid downward infiltration of contaminants into the aquifer.*

E. *The upward hydraulic pressure within the minor confined aquifer does not prevent the leaked or spilled contaminants from entering the aquifer and traveling to the river.*

F. *Once the ground water reaches the river bank, it encounters Altamaha River water, independent of the river bed conditions.*

Based on these assumptions, the most conservative path to the Altamaha River from the HNP-2 radwaste building is to the north through the minor confined aquifer, a minimum distance of 1300 ft.

The travel time of ground water is dependent on the velocity of the water and the distance traveled. The velocity of the ground water is calculated using the Darcy equation:

$$\overline{V} = KI/n$$

where:

K = *coefficient of permeability of the aquifer.*

I = *gradient of the potentiometric surface.*

n = *effective porosity of the aquifer.*

The permeability coefficient K of the minor confined aquifer was determined in the field using both falling-head and constant-head permeability tests. In general, the permeability increased from 3.1×10^{-5} *ft/min (16.3 ft/year) in the upper portions of the aquifer to* 2.5×10^{-4} *ft/min (131.4 ft/year) in the lower portions of the aquifer over 40 ft below the plant foundations. However, a conservative value of* $K = 2.5 \times 10^{-4}$ *ft/min was used to determine* $\overline{V}$.

Along the most conservative path (a distance of 1300 ft), the maximum gradient of the potentiometric surface in the minor confined aquifer is 23 ft/mi toward the Altamaha River, based on preconstruction piezometer readings (figure 2.4-40). This conservative value of $I = 0.0043$ *was used to determine* $\overline{V}$.

The velocity of ground water within the aquifer is inversely proportional to the effective porosity n. A conservative value of total porosity N, which includes effective porosity n and specific retention, was calculated from laboratory test data on unit weights of samples of aquifer material. The calculated value for N from 72 samples was 0.50 (50%). A conservative value of effective porosity n, estimated to be 0.10 (10%) was used to calculate $\overline{V}$.[13]

From the above values, the velocity of ground water from the Unit 2 radwaste building to the Altamaha River through the minor confined aquifer is 5.65 ft/year.

The travel time is calculated from:

$$T = L/\overline{V}$$

where:

T = *travel time.*

L = *distance traveled.*

$\overline{V}$ = *velocity of ground water.*

Using the minimum value of L = 1300 ft and $\overline{V}$ = 5.65 ft/year, the travel time of ground water from the Unit 2 radwaste building to the Altamaha River through the minor confined aquifer is 230 years.

The general direction of ground water flow in the immediate plant area in the unconfined and minor confined aquifers is northward, toward the Altamaha River. There are no potential ground water recharge areas within the influence of the plant (paragraph 2.4.13.2.5), and domestic wells within the plant boundaries have been abandoned. Therefore, the possibility of contaminating existing or future ground water withdrawal systems following an accidental release of contaminants at the plant site is extremely remote.

Because the subsurface materials at the Edwin I. Hatch site are primarily sands and clays, with only partial cementation of a few sand horizons, it was necessary to use drilling mud to hold the exploratory holes open. The heavy drilling mud retained in the holes provide an effective barrier to ground water migration through the area of the hole, as the permeability of the mud is considerably lower than that of the surrounding sands and clays.

2.4.13.4 Monitoring or Safeguard Requirements

As discussed in paragraph 2.4.13.2.4, ground water users in the vicinity of the plant are not affected by the withdrawal of ground water for plant use. The plant uses a relatively small amount of water obtained from the principal artesian aquifer, which is hydraulically isolated from the aquifers supplying most of the offsite ground water users. The quantity of ground water required for plant use is much less than the maximum safe yield of the aquifer (paragraph 2.4.13.1.2). Plant usage does not adversely affect offsite wells open to the aquifer.

During extremely high river stages, a temporary flattening of the gradient of the minor confined aquifer may occur. The area affected by the PMF (el 91 ft msl) would be immediately adjacent to the Altamaha River and would not extend to the plant, owing to the low permeability of the minor confined aquifer and the short duration of the flood. Wells in the vicinity of the plant would not be affected by such a gradient change because of their distance from the river (figure 2.4-36). Site piezometers are monitored throughout the life of the plant.

In the event of an accidental spill of contaminants, the piezometers and observation wells, shown on figure 2.4-37, can be monitored periodically to detect the flow path and dispersion of contaminants.

Potable water from both surface and subsurface sources is available in the area surrounding the site. Analyses of water samples are shown on table 2.4-10. These analyses provide background data against which future quality tests may be compared.

2.4.13.5 Design Bases for Subsurface Hydrostatic Loading

Paragraph 2.4.13.1.2 describes the system of wells and piezometers that are used to define the ground water conditions in the areas of the HNP-1 and HNP-2 structures. Those studies, which are the basis of the design parameters, define two independent ground water zones or aquifers. An unconfined aquifer occurs within unit 6 (upper stratum) of the Hawthorn formation. The water table within the aquifer trends parallel to the natural ground surface at depths of 10 to 15 ft below the ground surface. A confined aquifer occurs within the sandy (middle) portions of the Hawthorn formation. The confined aquifer corresponds to unit 3 and the lower part of unit 4 of the Hawthorn formation, below approximate el 65 ft msl. The piezometric level of water within the confined aquifer is at or below approximate el 80 ft msl. A cemented sandstone and sandy clay zone, corresponding to unit 5 and the upper part of unit 4 of the Hawthorn formation, comprises an aquiclude separating the two aquifers.

The post-construction behavior of the ground water level in the unconfined aquifer was anticipated to be consistent with the ground water level prior to construction; the ground water level reflected the ground surface contours. During construction, the ground surface elevation in the yard area was lowered from approximate el 143 ft msl to approximate el 129 ft msl. A corresponding reduction in the elevation of the unconfined ground water level resulted. To provide a conservative basis for basement wall design, a design ground water level 7 ft below the general yard level, or el 122 ft msl, was selected.

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TABLE 2.4-1

ACCESS TO SAFETY-RELATED STRUCTURES

Structures	Access	No. of Access	Floor El (ft)
Intake	Outside doors	2	111
Control building	Outside door	1	130
	Elevator door	1	130
Reactor building	Outside door (Airlocked)	1	130
Diesel building	Outside doors	6	130
	Air intakes	2	130
Main stack	Outside doors	2	120
	Freight door	1	120
	Outside door	1	145

TABLE 2.4-2 (SHEET 1 OF 6)

AVERAGE, MAXIMUM, MINIMUM TEMPERATURE OF ALTAMAHA RIVER WATER TEMPERATURE (°C)

	1962			1963			1964			1965		
Month	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min
Oct	22.0	25.0	16.0	22.0	24.0	19.0	19.0	27.0	17.0	21.0	24.0	16.0
Nov	14.0	16.0	12.0	15.0	18.0	12.0	17.0	18.0	15.0	16.0	18.0	13.0
Dec	9.0	13.0	8.0	8.0	12.0	6.0	12.0	15.0	11.0	11.0	13.0	10.0
Jan	8.0	11.0	6.0	9.0	11.0	7.0	7.0	10.0	5.0	11.0	14.0	6.0
Feb	14.0	18.0	11.0	9.0	11.0	7.0	9.0	11.0	9.0	11.0	14.0	8.0
Mar	14.0	19.0	10.0	15.0	20.0	10.0	15.0	17.0	10.0	14.0	19.0	11.0
Apr	18.0	20.0	16.0	20.0	24.0	18.0	18.0	21.0	14.0	20.0	23.0	17.0
May	25.0	29.0	21.0	23.0	26.0	20.0	22.0	26.0	19.0	25.0	29.0	22.0
Jun	22.0	29.0	27.0	26.0	29.0	24.0	28.0	31.0	25.0	25.0	28.0	24.0
Jul	30.0	31.0	28.0	27.0	28.0	25.0	27.0	29.0	25.0	28.0	29.0	26.0
Aug	30.0	31.0	27.0	29.0	31.0	27.0	26.0	28.0	26.0	29.0	31.0	28.0
Sep	27.0	30.0	23.0	26.0	30.0	21.0	25.0	27.0	23.0	27.0	29.0	23.0
Annual Average	30.0			19.0			19.0			20.0		

TABLE 2.4-2 (SHEET 2 OF 6)

	1966			1967			1968			1969		
Month	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min
Oct	21.0	25.0	18.0							23.0	27.0	17.0
Nov	15.0	18.0	18.0	Data not			13.0	18.0	11.0	16.0	18.0	14.0
Dec	11.0	14.0	9.0	available			12.0	14.0	11.0	14.5	16.0	13.0
Jan	10.0	14.0	3.0				8.0	11.0	6.0	12.0	16.0	10.0
Feb	9.0	14.0	4.0				9.0	10.0	8.0	10.0	12.0	8.0
Mar	13.0	17.0	11.0				14.5	18.0	9.0	11.0	14.0	9.0
Apr	23.0	24.0	16.0				21.0	23.0	18.0	18.0	19.0	14.0
May	24.0	25.0	21.0				24.0	26.0	22.0	21.5	24.0	18.0
Jun	24.0	26.0	22.0				27.0	30.0	23.0	27.0	31.0	24.0
Jul	28.0	30.0	25.0				29.0	31.0	28.0	30.5	32.0	29.0
Aug	27.0	29.0	26.0				21.0	33.0	26.0	26.0	29.0	24.0
Sep	26.0	29.0	24.0				27.0	29.0	26.0	25.0	26.0	23.0
Annual Average	19.0						18.0			19.5		

TABLE 2.4-2 (SHEET 3 OF 6)

	1970			1971			1972			1973		
Month	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min
Oct	22.1	24.0	19.0	23.7	26.0	21.5	24.2	27.5	22.5	22.8	27.0	21.0
Nov	15.7	19.0	14.0	17.1	21.5	13.5	18.7	23.0	14.0	18.9	22.5	14.5
Dec	10.8	14.5	9.0	13.8	15.0	12.0	14.4	16.5	13.0	14.5	17.5	13.0
Jan	8.3	11.0	5.5	11.3	12.5	9.5	13.5	14.5	12.5	12.0	14.0	10.0
Feb	10.42	11.0	9.5	10.9	14.0	9.5	11.5	13.0	10.0	9.9	11.5	9.0
Mar	14.6	16.0	11.0	14.4	16.0	13.5	15.8	17.5	13.0	14.1	17.0	9.0
Apr	18.9	23.5	15.5	18.2	21.5	14.5	19.4	22.0	17.0	16.2	19.0	12.5
May	24.5	26.5	22.0	21.8	23.5	20.0	22.5	23.5	21.5	21.3	23.5	19.0
Jun	27.1	30.5	24.5	26.5	29.0	23.5	24.6	26.0	22.5	24.1	24.5	23.5
Jul	29.7	30.0	29.0	28.51	29.5	28.0	27.0	29.0	24.5	26.6	27.0	24.5
Aug	28.6	30.5	26.5	27.6	28.5	26.0	29.0	29.5	28.5	26.5	28.0	24.5
Sep	28.1	28.5	26.0	27.4	28.0	27.0	28.2	29.0	27.0	27.8	28.0	25.0
Annual Average	19.9			20.1			20.7			19.8		

TABLE 2.4-2 (SHEET 4 OF 6)

	1974			1975			1976			1977		
Month	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min
Oct	23.75	26.5	21.0	23.25	25.5	21.0	20.0	23.5	16.5	19.25	25.0	13.5
Nov	19.5	21.0	18.0	19.5	22.0	17.0	15.75	22.0	9.5	12.25	14.5	10.0
Dec	15.5	18.5	12.5	15.75	17.0	14.5				9.5	11.5	7.5
Jan	15.25	17.0	13.5	14.5	15.5	13.5				6.0	9.0	3.0
Feb	15.75	17.5	14.0	15.0	15.5	14.5	13.25	17.5	9.0	9.75	14.5	5.0
Mar	15.75	17.5	14.0	15.75	16.5	15.0	16.5	19.0	14.0	16.5	20.0	13.0
Apr	18.5	20.0	17.0	18.0	19.5	16.5	21.25	25.0	17.5	20.75	23.0	18.5
May	22.5	25.0	20.0	21.75	24.0	19.5	22.25	24.0	20.5	25.0	28.0	22.0
Jun	26.0	27.0	25.0	25.25	26.5	24.0	25.0	28.0	22.0	29.5	32.0	27.0
Jul	27.5	28.5	26.5	26.5	27.0	26.0	27.5	30.5	24.5	30.25	32.5	28.0
Aug	28.5	29.0	28.0	27.75	29.0	26.5	27.5	30.5	24.5	29.0	30.5	27.5
Sep	27.5	29.0	26.0	25.0	29.0	21.0	24.75	28.0	21.5	27.75	29.5	26.0
Annual Average	21.34			20.67			21.38			19.63		

TABLE 2.4-2 (SHEET 5 OF 6)

Month	1978 Avg	Max	Min
Oct	22.25	27.5	17.0
Nov	18.0	22.0	14.0
Dec	12.25	16.5	8.0
Jan	8.0	10.5	5.5
Feb	7.25	9.5	5.0
Mar	14.25	20.0	9.0
Apr	21.0	24.0	18.0
May	23.75	28.0	19.5
Jun	29.25	32.5	26.0
Jul	30.0	32.0	28.0
Aug	29.0	30.5	27.5
Sep	27.0	30.5	23.5
Annual Average	20.19		

TABLE 2.4-2 (SHEET 6 OF 6)

Year[a]	*Maximum Temperature (°C)*
1979	*31.0 (B)*
1980	*32.0 (J)(G)*
1981	*31.0 (J)(G)*
1982	*29.5 (B)*
1983	*30.0 (B)(J)*
1984	*30.0 (B)(J)*
1985	*31.0 (G)*
1986	*31.5 (G)*
1987	*31.0 (G)*
1988	*30.0 (B)*
1989	*29.5 (G)*
1990	*32.0 (J)*
1991	*29.0 (J)(G)*
1992	*31.0 (J)(G)*
1993	*31.0 (J)(G)*

Note: 1962-1978 maximum temperature values recorded were taken at the Doctortown, Georgia, gaging station.

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a. 1979-1993 maximum temperature values recorded were taken at the Baxley, Georgia, gaging station (B); the Jesup, Georgia, gaging station (J); or the Gardi, Georgia, gaging station (G) as noted. No monthly maximum, average, or minimum values are available.

TABLE 2.4-3

SUMMARY OF DATA ON DAMS

Name of Dam and Reservoir	Sinclair	Lloyd Shoals	Wallace
Owner	GPC	GPC	GPC
River miles from site	189	189	211
Drainage area (mi)	2910	1400	1830
Construction completed (year)	1952	1910	1980
Stream bed elevation	258	435	325
Top-of-dam elevation	355	540	445
Normal pool elevation	340	530	435
Usable (conservation) storage (acre-feet)	214,600	78,000	32,000
Floor-control allocation (acre-feet)	None	None	None
Type of spillway	24T - 30x21	FB: 308.5x2 420x5	5T - 42x45
Crest elevation	319.0	528.0 525.0	391
Earth or rockfill dike Conc power house N_2O wall Conc spillway	1596 1392	530 1570	2700 1000
Seismic design	--	--	0.05 hor - 0.0333 vert on concrete structure

a. All elevations approximately msl.
b. All dimensions in feet.
c. T - tainter gates.
d. FB - flash boards.

TABLE 2.4-4

GAUGING STATION RECORDS - ALTAMAHA RIVER BASIN ALTAMAHA AT CHARLOTTE AND BAXLEY, GEORGIA ANNUAL FLOOD PEAKS

Charlotte Gauge				Baxley Gauge	
Water (year)	Discharge (ft^3/s)	Water (year)	Discharge (ft^3/s)	Water (year)	Discharge (ft^3/s)
1913	98,000	1949	44,000	1971	97,500
1914	29,000	1950	21,400	1972	67,800
1915	37,500	1951	20,900	1973	53,800
1916	52,900	1952	66,800	1974	46,500
1917	45,100	1953	66,000	1975	91,300
1918	32,300	1954	31,800	1976	53,400
1919	90,800	1955	20,600	1977	45,000
1920	46,400	1956	30,500	1978	73,000
1921	38,500	1957	30,500	1979	76,000
1922	82,000	1958	49,000		
1923	68,500	1959	37,000		
1924	33,600	1960	79,100		
1925	170,000	1961	81,000		
1926	29,800	1962	49,000		
1927	20,400	1963	50,900		
1928	108,000	1964	79,100		
1929	130,000	1965	60,200		
1930	39,000	1966	93,800		
1031	32,300	1967	40,500		
1932	34,100	1968	24,500		
1933	40,000	1969	34,000		
1934	39,500	1970	73,000		
1935	21,400	1971	100,000		
1936	145,000	1972	66,200		
1937	45,100	1973	54,100		
1938	66,000	1974	45,100		
1939	86,000	1975	89,000		
1940	31,000	1976	55,000		
1941	33,600	1977	47,300		
1942	93,000	1978	79,250		
1943	76,200	1979	77,000		
1944	108,000	1980	84,000		
1945	36,500				
1946	61,900				
1947	64,300				
1948	123,000				

TABLE 2.4-5

SUMMARY OF FLOOD DISCHARGE STUDIES
(HNP-1 AND HNP-2)

Condition	Discharge (ft^3/s)	Stage Equaled or Exceeded (ft msl) With Present Channel
Bankfull	20,000	74.0
Recurrence interval (years)		
2	52,000	79.8
5	80,000	83.5
10	102,000	85.3
20	122,000	86.9
50	150,000	88.6
100	172,000	89.7
250	200,000	91.3
1000	243,000	93.0
PMF	612,000	105.0

TABLE 2.4-6

PROBABLE MAXIMUM PRECIPITATION ADJUSTMENT FACTOR ALTAMAHA RIVER ABOVE NUCLEAR PLANT SITE STORM OF 5-10 JULY 1916 (GM 1-19) TRANSPOSED FOR MONTH OF OCTOBER (HNP-1 AND HNP-2)

Thiessen Polygon	Barrier Elevation (ft)	1	2	3	4	5	6
Greensboro	1000	73.6	2.70	0.24	2.46	7.3	1.06
Uniontown	900	73.8	2.73	0.22	2.51	7.4	1.07
Selma	750	73.9	2.75	0.19	2.56	7.6	1.10
Benton	750	74.0	2.76	0.18	2.58	7.6	1.10
Thomasville	850	74.0	2.76	0.21	2.55	7.5	1.09
Fort Deposit	550	74.2	2.79	0.16	2.63	7.8	1.13
Highland Homes	500	74.5	2.83	0.12	2.71	8.0	1.16
Greenville	600	74.3	2.81	0.15	2.66	7.8	1.13
Evergreen	650	74.9	2.89	0.17	2.72	8.0	1.16
Bermuda	750	74.7	2.86	0.19	2.67	7.7	1.12
Troy	450	74.8	2.88	0.12	2.76	8.1	1.17
Molino	450	75.3	2.94	0.12	2.82	8.3	1.20
Carniers	450	75.9	3.03	0.12	2.91	8.6	1.25
Defuniak Springs	450	75.6	2.98	0.12	2.86	8.4	1.22
Bonifay	400	75.8	3.01	0.11	2.90	8.6	1.25
Ozark	450	75.2	2.93	0.12	2.81	8.3	1.20
Marianna	350	76.0	3.04	0.09	2.95	8.7	1.26
Wausau	350	76.0	3.04	0.09	2.95	8.7	1.26
Alaga	400	75.8	3.01	0.11	2.90	8.6	1.25
Pushmataha	950	73.8	2.73	0.24	2.49	7.3	1.07
Demopolis	950	73.5	2.69	0.22	2.47	7.3	1.06
Milllry	900	74.2	2.79	0.22	2.57	7.6	1.10
Clanton	750	73.7	2.72	0.18	2.54	7.5	1.09
Pratville	650	74.0	2.76	0.16	2.60	7.7	1.12
Montgomery	650	74.1	2.77	0.16	2.61	7.7	1.12

Explanatory Notes

Storm's 12-h 1000-MB dewpoint
- Actual throughout (Dp)s = 76.0°F
- Maximum at selected location and time
- $(Dp)'_M$ per station __ __ __ column 1

Storm's moisture charge adjusted for elevation of inflow barrier

Actual total at (Dp)s dewpoint ___ (Wp)s = 3.04 in.
Actual inflow barrier reduction at (Dp)s __ __ __ (Wp)s = 0.04 in.
Net actual above inflow barrier $(Wp)s - (\overline{W}p)s = 3.00$ in.

Maximum corresponding to $(Dp)_M$
- $(Wp)'_M$ per station __ __ __ column 2

Maximum inflow barrier reduction at $(Dp)_M$
- $(Wp)'_M$ per station __ __ __ column

Maximum above inflow barrier
- $(Wp)'_M - (\overline{W}p)'_M$ per station ___ column 4

Temperature contrast
- Actual storm center (Tc)s = 5.3
- Storm center transposed in location and time $(Tc)_T = 8.7$

Moisture charge adjusted for inflow barrier and storm efficiency
- Actual storm location $(Tc)_S^{1/2}\left[(Wp)s - (\overline{W}p)s\right] = 6.9$
- Maximum at selection location and time
- $(Tc)_T^{1/2}\left[(Wp)'_M - (\overline{W}p)'_M\right]$ per station $= 8.7^{1/2}$ x column 4 = column 5

Maximum probable precipitation factor Ratio of maximum at station in transposed location to actual for respective station in original location =

$$\frac{(Tc)_T^{1/2}\left[(Wp)'_M - (\overline{W}p)'_M\right]}{(Tc)_S^{1/2}\left[(Wp)_S - (\overline{W}p)_S\right]} = \frac{\text{column 5}}{6.9} = \text{column 6}$$

TABLE 2.4-7 (SHEET 1 OF 2)

RESULTS OF LOCAL WELL SURVEY

Well Number	Owner	Surface Elevation (ft)	Type of Well	Elevation of Bottom of Well (ft)	Elevation of Water Surface (ft)	Aquifer	Remarks
1	Buck Dunn, Baxley	11	Drilled	98	108	Unconfined	
2	Lambert Miles, Baxley	117	Dug	98	102	Unconfined	Water rushed into well from aquifer
3	Rube Beacher, Lyons	123	Drilled	(-)234	60	Principal artesian	160-ft casing (4 in.) sulphur water
4	Willis	136	Dug	-	124	Unconfined	
5	Branch	124	Drilled	56	67	Minor confined	32-ft casing
5A		125	Dug	108	113	Unconfined	
6	Deen	74	Dug	45	54	Alluvium	
7	Emmanuel	144	Dug	130	132	Unconfined	Abandoned
8	Hutcheson	145	Dug	127	138	Unconfined	Water spurted in when dug
8A		162	Dug	-	148	Unconfined	
9	King	142	Dug	130	132	Unconfined	Water spurts from bottom
10	Leona Hutcheson	158	Dug	131	135	Unconfined	30 years old, water comes from sides
11	Henry Hutcheson	152	Dug	131	136	Unconfined	7 years old
12	J. C. Mosley	131	Drilled	44	81	Minor confined	57-ft casing
12A		124	Dug	109	110	Unconfined	Abandoned
13	E. E. Mosley	99	Dug	85	87	Unconfined	
14	Beecher	115	Dug	100	108	Unconfined	Water spurted in when dug
15	Tom Lawrence	131	Drilled	(-)196	-	Principal artesian	Casing 265 ft; well 10 ft into aquifer; sulphur water
16	Crosby	114	Drilled	(-)300	-	Principal artesian	Sulphur water, 260-ft casing
16A	Crosby	100	Dug	84	92	Unconfined	
17	Robertson	142	Drilled	(-)213	52	Principal artesian	Slight sulphur, 355-ft casing
18	J. O. Beasley	174	Dug	155	161	Perched	
19	Melr	169	Dug	125	128	Unconfined	Was at 41 ft and went dry, deepened to 44 ft and water rushed in
20	Coleman	143	Drilled	(-)212	52	Principal artesian	Possibly same as well No. 17
21	V. Cannon	80	Dug	55	-	Alluvium	Sealed
22		148	Dug	123	138	Unconfined	
23	Braswell	159	Dug	141	147	Unconfined	
24	Calloway	157	Dug	142	146	Unconfined	
25	Covington	172	Drilled	(-)328	-	Principal artesian	207-ft casing
26	Ansley	150	Dug	130	136	Unconfined	
28	Sellers	156	Dug	139	-	Unconfined	

TABLE 2.4-7 (SHEET 2 OF 2)

Well Number	Owner	Surface Elevation (ft)	Type of Well	Elevation of Bottom of Well (ft)	Elevation of Water Surface (ft)	Aquifer	Remarks
29	Collins	167	Dug	148	-	Unconfined	Covered over, abandoned
30	Sellers	151	Dug	129	138	Unconfined	
31	Sellers	134	Drilled	105	128	Unconfined	Well drilled 9/12/67
32	J. W. Adams	146	Drilled	135	140	Unconfined	
33	Moody	176	Dug	161	170	Perched	
34	Williams	184	Drilled	151	166	Perched	
35	Thigpen	188	Dug	156	173	Perched	Muddy water
35A		188	Drilled	139	164	Unconfined	
36	Sellers	164	Dug	134	152	Unconfined	
37		166	Dug	155	159	Unconfined	
38	Dewnann	165	Dug	151	157	Unconfined	
39		160	Dug	144	148	Unconfined	Abandoned
40	Hucheson	180	Dug	157	171	Perched	
41	Branch	187	Dug	147	157	Unconfined	
42	Deen	203	Drilled	157	174	Perched	
43	Branch	195	Dug	155	170	Perched	
44	Hardee	190	Drilled	155	168	Perched	
44A	Hardee	190	Dug	165	168	Perched	
45	Hutcheson	158	Dug	120	138	Unconfined	
46	Hutcheson and Setters	146	Drilled	(-)256	-	Principal artesian	Unable to measure, sealed
47	Baker	134	Drilled	(-)251	-	Principal artesian	
48		160	Drilled	(-)117	80(1940) 72(1956)	Principal artesian	
49	Baker	150	Dug	110	-	Unconfined	Abandoned, sealed
51	Williamson	156	Dug	136	148	Unconfined	
52	Williamson	143	Dug	123	135	Unconfined	
53		149	Dug	134	138	Unconfined	
54	Britt	130	Dug	116	121	Unconfined	
55		181	Dug	158	166	Perched	
56	Lawrence	162	Dug	151	153	Unconfined	
57		153	Dug	137	147	Unconfined	
Site Well 1		109.5	Drilled	(-)570.5	50.5	Principal artesian	As-built drawing, figure 2.4-38
Site Well 2		151.9	Drilled	(-)559.1	48.9	Principal artesian	As-built drawing, figure 2.4-38

TABLE 2.4-8 (SHEET 1 OF 4)

PIEZOMETER DATA

OBS Pt Number	Year Constructed	Surface Elevation	Location Coordinates North	Location Coordinates East	Piezometer Range or Bottom Elevation	Remarks
P-1	1969	128	703+935	444+640	50	Destroyed 1971
P-2	1969	128.4	703+640	444+625	50	Destroyed 1971
P-3	1969	128.4	703+350	444+720	50	Destroyed 1971
P-6	1969	129.1	703+930	444+100	50	Destroyed 1971
P-7	1969	130.4	703+590	445+100	50	Destroyed 1971
P-8	1969	128.8	703+590	445+100	50	Destroyed 1971
P-9	1969	119.7	703+930	445+420	50	Destroyed 1973
P-10	1969	120.8	703+280	445+439	50	Destroyed 1974
P-11	Sept 71	130.0	702+989	445+418	50	Destroyed 1974
P-12	Oct 71	132.6	703+038	444+844	50	
P-13	Oct 71	131.2	703+038	444+584	50	
P-14	Oct 71	130.1	704+081	444+469	50	
P101A	1967	138.0	700+270	441+240	53.0	Destroyed 1969
P101B	1967	138.0	700+270	441+240	118.0	Destroyed 1969
P102A	1967	141.3	703+950	441+470	114.3	
P102B	1967	142.1	703+950	441+470	56.3	
P103A	1967	156.4	703+030	442+750	51.4	Destroyed 1968
P103B	1967	156.4	703+030	442+750	136.4	Destroyed 1968
P104A	1967	142.6	700+090	443+800	124.0	
P104B	1967	142.6	700+090	443+800	66.6	
P105A	1967	135.5	701+240	444+140	116.5	
P105B	1967	135.5	701+240	444+140	53.5	
P106A	1967	128.9	704+230	443+380	53.0	Destroyed 1969
P106B	1967	128.9	704+230	443+380	98.9	Destroyed 1969
P107A	1967	131.0	700+260	445+590	51.0	Destroyed 1974
P107B	1967	131.0	700+260	445+590	101.0	Destroyed 1974
P108A	1967	103.9	700+590	447+550	88.9	
P108B	1967	103.9	700+590	447+550	69.9	
P109A	1967	96.3	698+960	448+010	84.7	
P109B	1967	96.3	698+960	448+010	59.7	
P110A	1967	103.9	697+920	445+530	43.9	Destroyed 1969
P110B	1967	103.9	697+920	445+530	83.9	Destroyed 1969
P111A	1967	159.0	700+590	451+140	137.0	

TABLE 2.4-8 (SHEET 2 OF 4)

OBS Pt Number	Year Constructed	Surface Elevation	Location Coordinates North	Location Coordinates East	Piezometer Range or Bottom Elevation	Remarks
P111B	1967	159.0	700+590	451+140	128.4	
P112A	1967	76.6	707+372	441+869	31.3	
P112B	1967	76.6	707+372	441+869	66.3	
B-402-1	1967	120.3	703+550	445+650	78-83	Destroyed 1968
B-402-2	1967	120.3	703+550	445+650	52-58	Destroyed 1968
B-402-3	1967	120.3	703+550	445+650	28-40	Destroyed 1968
B-402-4	1967	120.3	703+550	445+650	13-22	Destroyed 1968
B-402-surficial	1967	120.3	703+550	445+650	100-120	Destroyed 1968
B-404-1	1967	117.5	702+950	445+940	77-84	Destroyed 1968
B-404-2	1967	117.5	702+950	445+940	61-66	Destroyed 1968
B-404-3	1967	117.5	702+950	445+940	45-55	Destroyed 1968
B-404-4	1967	117.5	702+950	445+940	23-30	Destroyed 1968
B-404-5	1967	117.5	702+950	445+940	7-17	Destroyed 1968
B-404-surficial	1967	117.5	702+950	445+940	98-115.5	Destroyed 1968
B-407.1	1967	110.3	702+820	447+060	73-82	
B-407-2	1967	110.3	702+820	447+060	63-68	
B-407-3	1967	110.3	702+820	447+060	45-55	
B-407-4	1967	110.3	702+820	447+060	27-35	
B-407-5	1967	110.3	702+820	447+060	5-15	
B-407-surficial	1967	110.3	702+820	447+060	90-110	Destroyed 1968
B-411-1	1967	114.0	703+750	444+950	70-76	Destroyed 1969
B-411-2	1967	114.0	703+750	444+950	52-58	Destroyed 1969
B-411-3	1967	114.0	703+750	444+950	20-50	Destroyed 1969
B-411-4	1967	114.0	703+750	444+950	20-30	Destroyed 1969
B-411-5	1967	114.0	703+750	444+950	6-15	Destroyed 1969
B-411-surficial	1967	114.0	703+750	444+950	118-114	Destroyed 1969
B-434-1	1967	120.0			80	Destroyed 1968[(a)]
B-434-2	1967	120.0	704+065	444+000	50	Destroyed 1968[(a)]
B-434-3	1967	120.0	704+065	444+000	28-33	Destroyed 1968[(a)]
B-434-4	1967	120.0	704+065	444+000	8-13	Destroyed 1968[(a)]
P-1009A	Sept 72	139.0	702+635	444+360	44.0	
P-119B	Sept 72	139.0	702+635	444+360	105.0	

TABLE 2.4-8 (SHEET 3 OF 4)

Piezometer	Year Constructed	Reference Point Elevation (ft msl)	Plant Grid Location Coordinates North	East	Depth Below Surface (ft)	Bottom Elevation (ft msl)
N1A	1977	131.7	52+79	42+34	83.6	45.7
N1B	1976	131.0	52+80	42+33	26.5	102.9
N2A	1976	132.0	52+60	44+98	81.1	47.8
N2B	1976	131.1	52+40	44+98	26.9	102.0
N3A	1976	131.0	49+49	46+71	84.3	44.4
N3B	1976	130.6	49+49	46+51	27.0	101.7
N4A	1976	135.8	45+91	46+94	83.5	50.1
N4B	1976	135.7	45+71	46+92	27.0	106.8
N5B	1977	131.3	48+35	51+63	25.0	103.3
N7A[(b)]	1976	131.1	53+14	51+74	82.6	46.4
N8A	1976	131.9	56+12	52+78	79.6	49.9
N8B	1976	131.3	55+28	51+19	25.9	103.5
N9B	1977	131.6	55+26	49+44	17.0	112.4
N10B	1977	132.0	54+57	51+18	20.0	109.5
N11B	1977	131.7	52+72	51+25	20.0	109.6
N12B	1977	131.8	52+53	51+15	15.0	114.7
N13B	1977	129.5	52+65	37+64	25.0	103.0
N14B	1977	133.2	39+96	53+41	25.0	106.6
N15B	1977	120.1	46+91	62+12	25.0	94.2
P13A	1976	130.5	47+38	46+84	69.8	59.2
P13B	1977	129.9	47+40	46+80	23.2	105.8
P15A	1976	130.6	53+80	47+61	75.0	53.9
P15B	1976	131.3	53+90	47+61	19.3	109.7
P16	1976	131.4	53+14	51+87	14.6	115.2
P17A	1976	131.7	56+36	48+99	77.7	52.1
P17B	1976	131.7	56+46	48+99	14.8	115.3

TABLE 2.4-8 (SHEET 4 OF 4)

OBS Pt Number	Year Constructed	Reference Point Elevation (ft msl)	Plant Grid Location Coordinates North	East	Depth Below Surface (ft)	Bottom Elevation (ft msl)
NW2A	2006	131.2	50+40	53+30	25.0	106.2
NW2B	2006	130.8	49+40	54+50	25.0	105.8
NW3A	2006	131.2	47+90	53+00	25.0	106.2
NW3B	2006	130.7	47+80	54+40	25.0	105.7
NW4A	2006	130.5	48+00	47+60	25.0	105.5
NW5A	2006	130.4	49+50	45+80	25.0	105.4
NW5B	2006	130.2	50+20	42+80	25.0	105.2
NW6	2006	131.3	56+50	46+90	25.0	106.3
NW7A	2006	130.5	56+10	43+50	25.0	105.5
NW8	2006	131.2	58+40	48+40	25.0	106.2
NW9	2006	131.8	57+00	52+70	25.0	106.8
NW10	2006	131.2	50+60	51+40	25.0	106.2
R1	2008	130.5	48+00	47+50	80.0	50.5
R2	2008	131.2	58+42	48+45	78.5	52.7
R3	2008	131.1	53+05	51+56	91.0	40.1
R4	2008	82.9	60+50	62+00	38.5	44.4
R5	2008	130.8	53+75	53+70	32.0	98.8
R6	2008	130.6	54+05	54+20	36.1	94.5

a. Destroyed shortly after installation.
b. Abandoned in 2008.

TABLE 2.4-9

PERMEABILITY AND POROSITY DATA

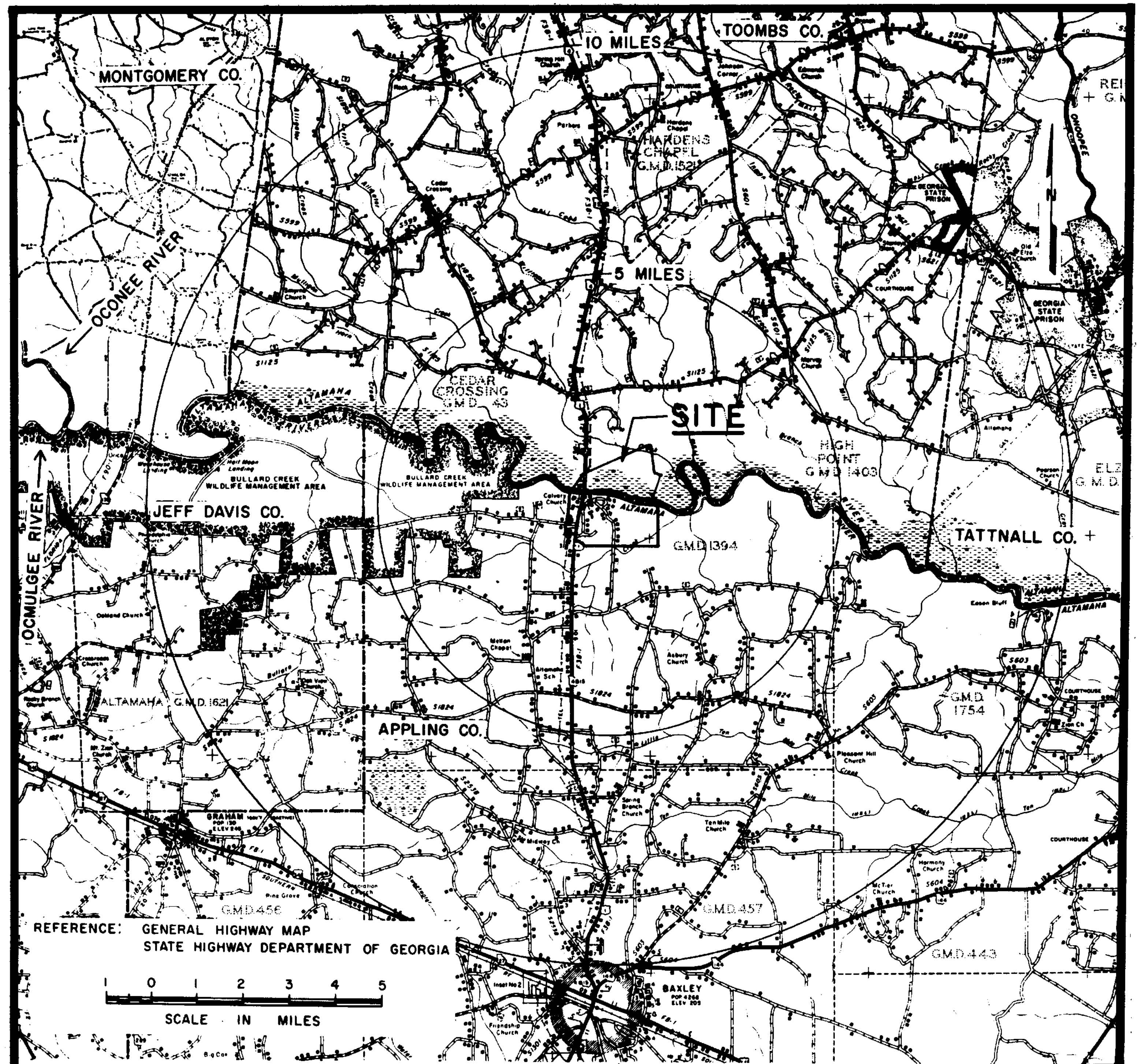
Formation	*Elevation of Tested Strata (ft msl)*	*Permeability (ft/min)*		*Porosity*		
		Min	*Max*	*Samples*	*Average n*	*Effective n (estimated)*
Hawthorn unit 6 (unconfined aquifer)	*128-132*	1.1×10^{-3}	1.7×10^{-3}	*(1)*	*.19*	*.12*
Hawthorn units 5 and 4 (aquiclude)	*70-117*	1.0×10^{-6}	4.4×10^{-6}	*(7)*	*.31*	*.10*
Hawthorn units 4 and 3 (confined aquifer)	*20-72*	3.1×10^{-5}	2.5×10^{-4}	*(72)*	*.50*	*.15*

TABLE 2.4-10

WATER QUALITY ANALYSES

	Sample No. 1	Sample No. 2	Sample No. 3	Sample No. 4
Date of collection	--	--	7/19/67	11/15/67
Silica (SiO_2) (ppm)	46	45	10	8.1
Iron (Fe) (ppm)	--	0.11	3.1	0.11
Manganese (Mn) (ppm)	--	--	0.4	0.0
Calcium (Ca) (ppm)	28	36	10	24
Magnesium (Mg) (ppm)	9.2	10	2	0.0
Sodium (Na) (ppm)	21.0	15	8	33.4
Potassium (k) (ppm)	0.9	2.7	--	3.5
Bicarbonate (HCO_3) (ppm)	172	161	--	113.5
Carbonate (CO_3) (ppm)	0	0	--	0
Sulfate (SO_4) (ppm)	2.4	20	7	0
Chloride (Cl) (ppm)	9.0	10	7	3.0
Fluoride (F) (ppm)	0.6	0.7		7.2
Nitrate ($N0_3$) (ppm)	0	0.2	1	0
Dissolved solids				
Calculated (ppm)	202	219	--	164.4
Residue on evaporation at 180°C	--	--	--	258.0
Hardness as $CaCO_3$ (ppm)	180	131	--	60.0
Noncarbonate hardness as CaCO (ppm)	0	0	--	0
Alkalinity as $CaCO_3$ (ppm)	--	--	36	93
Specific conductance (micro-mhos at 25°C)	280	311	100	270
pH	7.9	7.6	6.6	7.4
Color	0	5	15	--

Sample No. 1 - Hutcheson's well, north of Baxley, drilled 277 ft; analysis furnished by Geological Survey Water Resources Division, Atlanta, Georgia.
Sample No. 2 - City of Baxley, drilled 849 ft, cased 564; analysis furnished by Geological Survey Water Resources Division, March 12, 1963.
Sample No. 3 - GPC sample - raw water from the Altamaha River near site; analysis furnished by GPC, July 19, 1967.
Sample No. 4 - Piezometer water, boring 411-5, el 6 ft to 15 ft; analysis, November 20, 1967.



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SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 1 AND UNIT 2	EDWIN I. HATCH NUCLEAR PLANT SITE FIGURE 2.4-1

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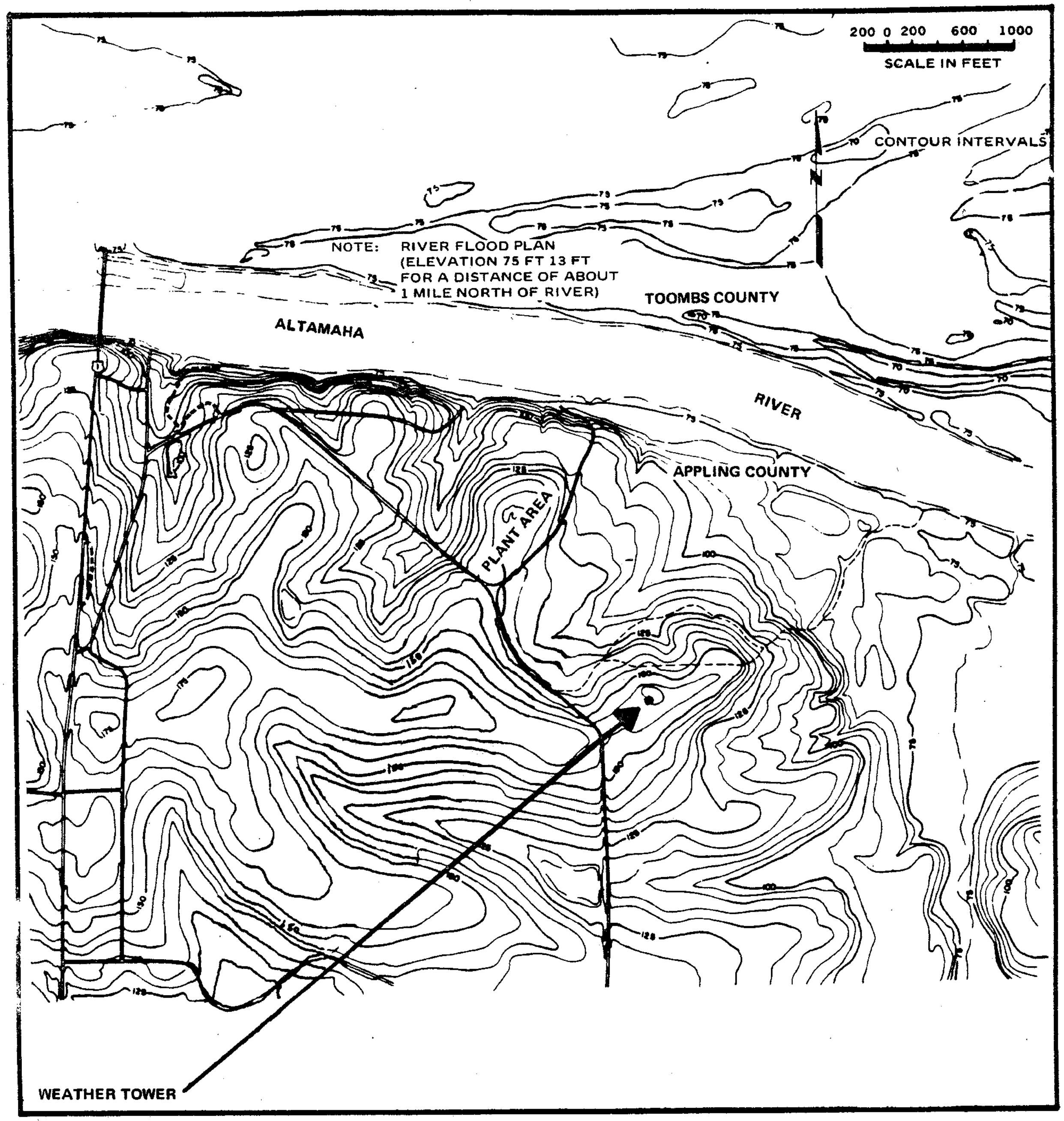
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 1 AND UNIT 2	LOCAL SITE ENVIRONS FIGURE 2.4-2



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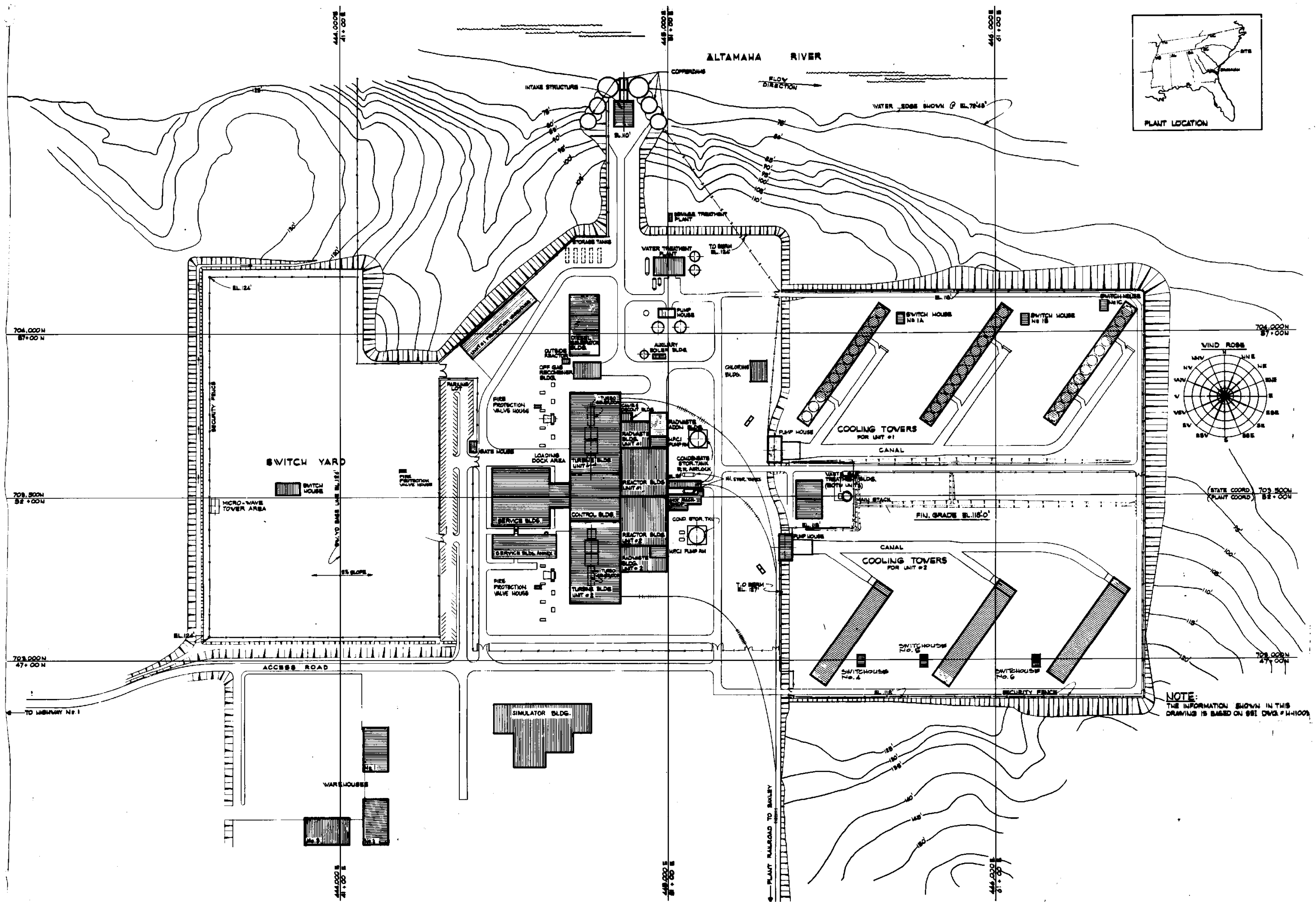
UNIT 1 AND UNIT 2

SITE TOPOGRAPHIC MAP

FIGURE 2.4-3



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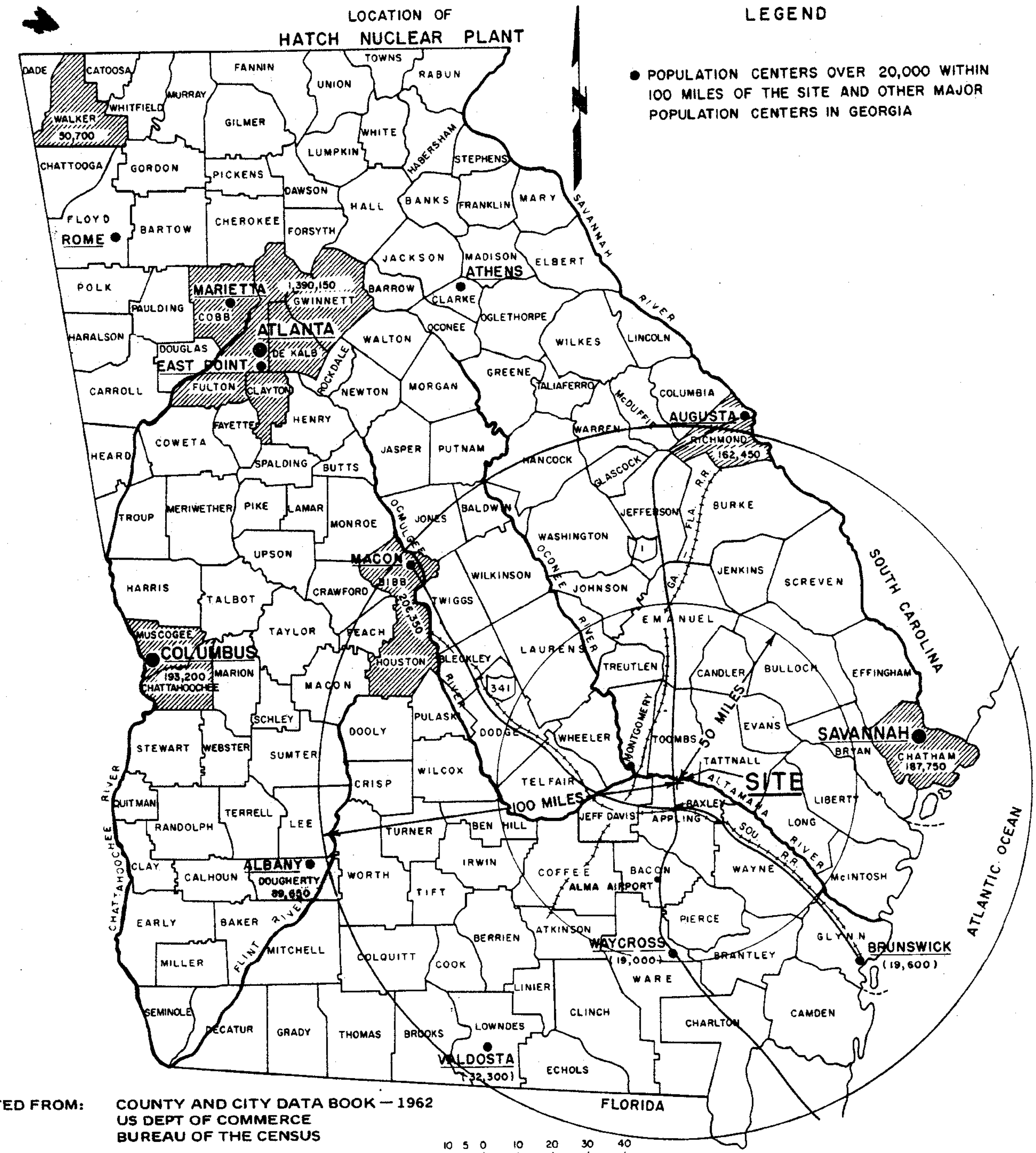
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

HATCH STRUCTURES GENERAL ARRANGEMENT PLAN

FIGURE 2.4-4

STATE OF GEORGIA
LOCATION OF
HATCH NUCLEAR PLANT



ADAPTED FROM: COUNTY AND CITY DATA BOOK — 1962
US DEPT OF COMMERCE
BUREAU OF THE CENSUS

UPDATED FROM: CENSUS OF THE POPULATION — 1970

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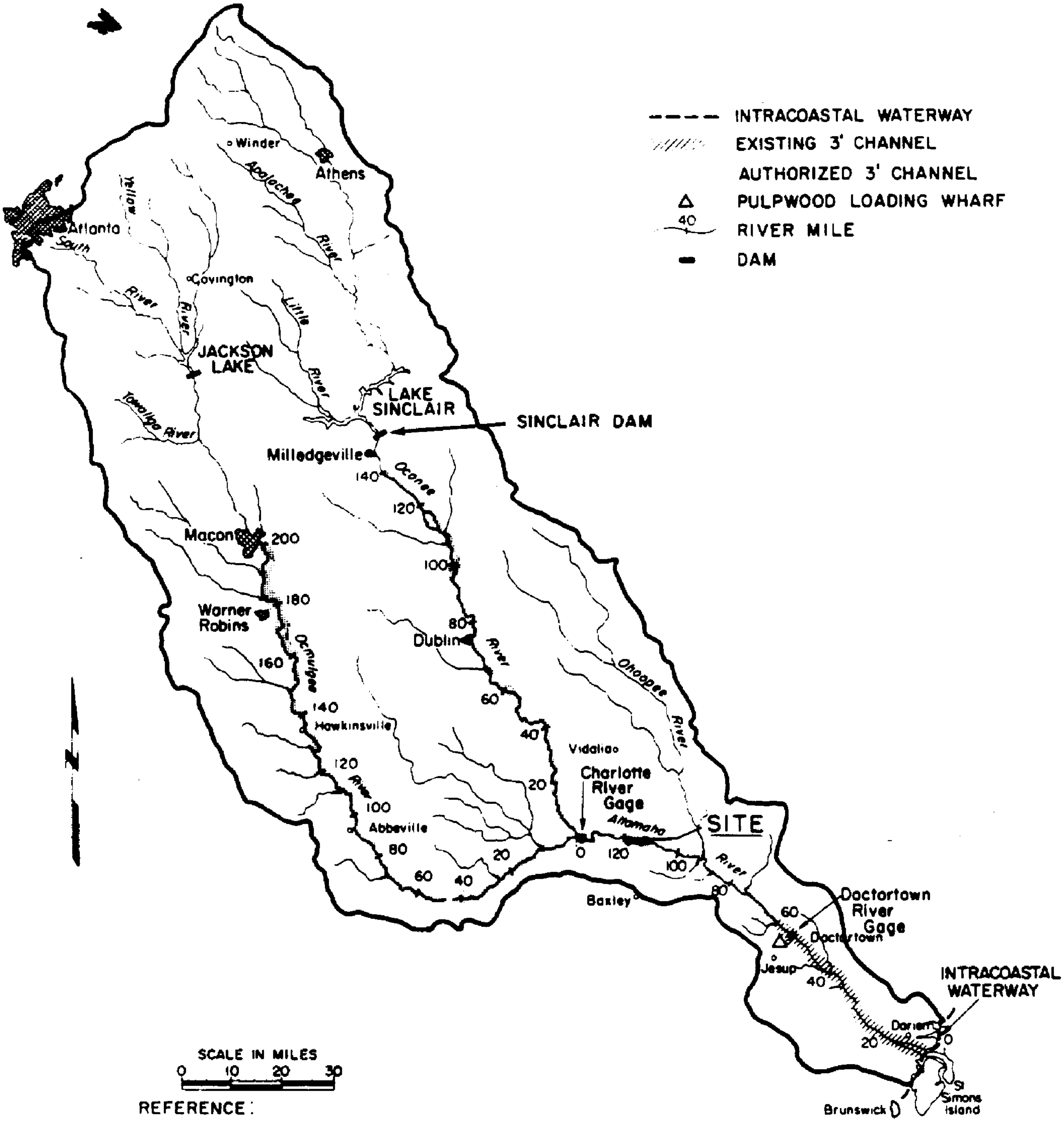
LOCATION OF HATCH NUCLEAR PLANT

FIGURE 2.4-5

ALTAMAHA DRAINAGE BASIN



REFERENCE:
UNITED STATES STUDY COMMISSION
SOUTHEAST RIVER BASINS
APPENDIX 3 1963

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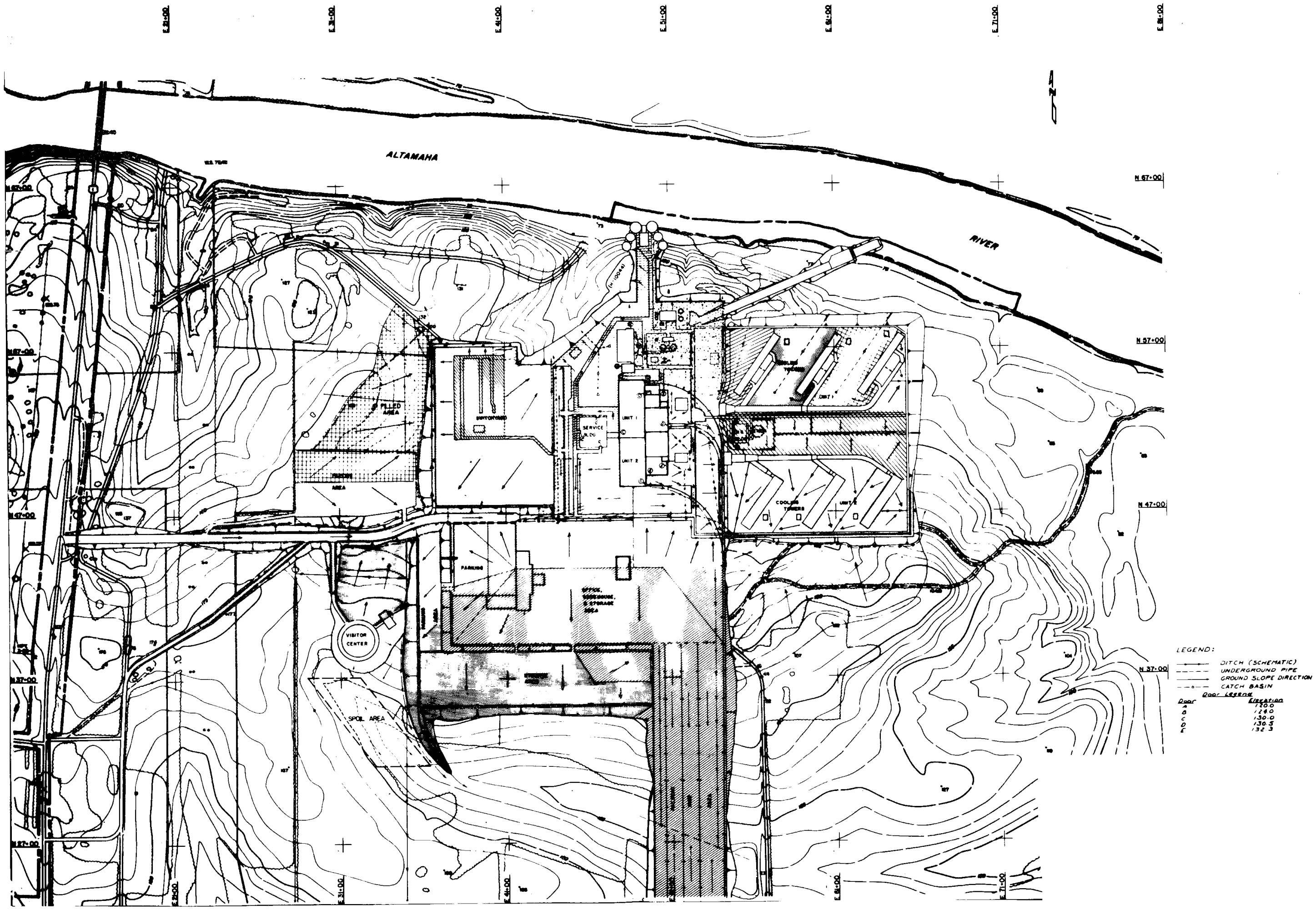
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

ALTAMAHA DRAINAGE BASIN

FIGURE 2.4-6



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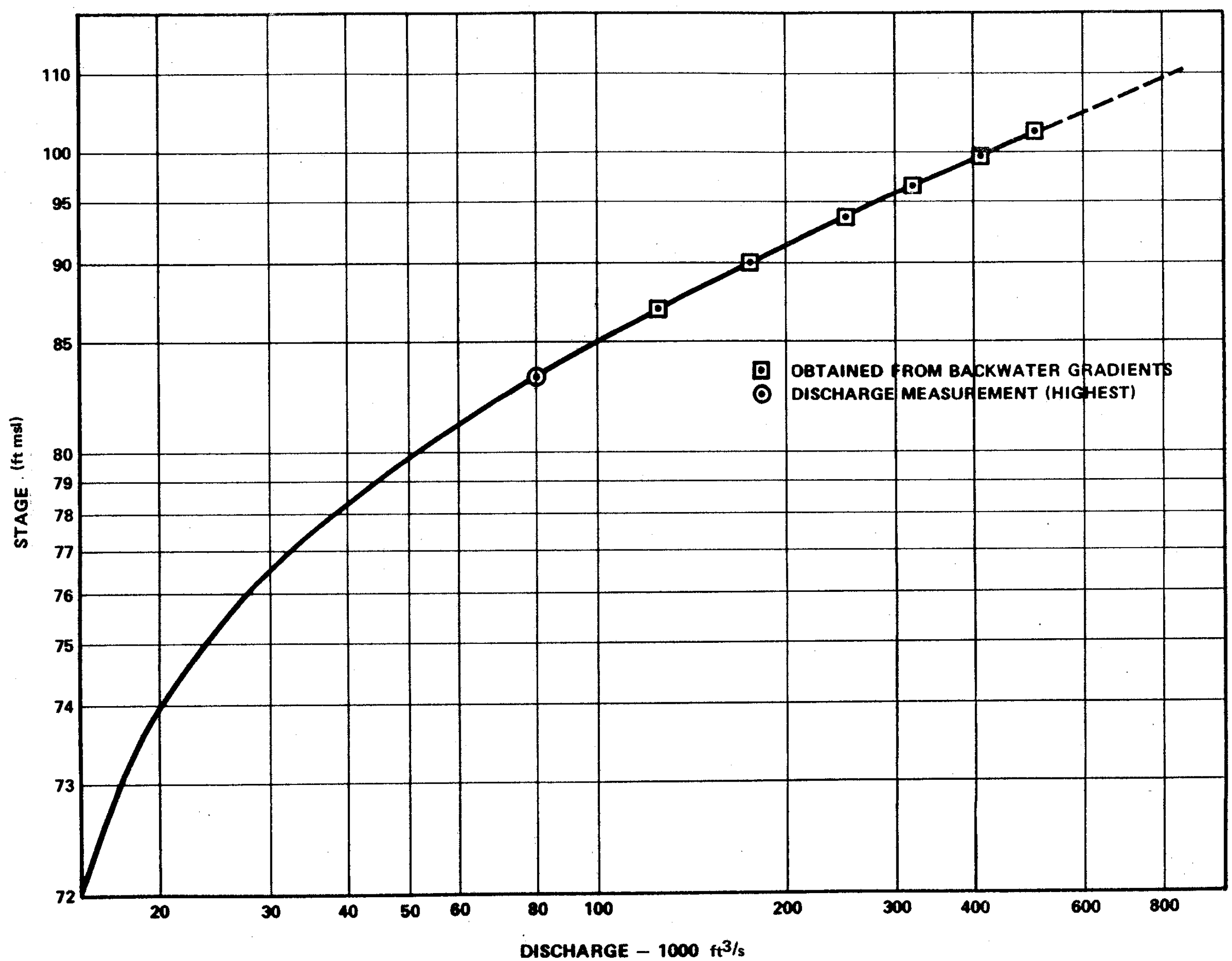
OVERALL SITE DRAINAGE PLAN

FIGURE 2.4-7



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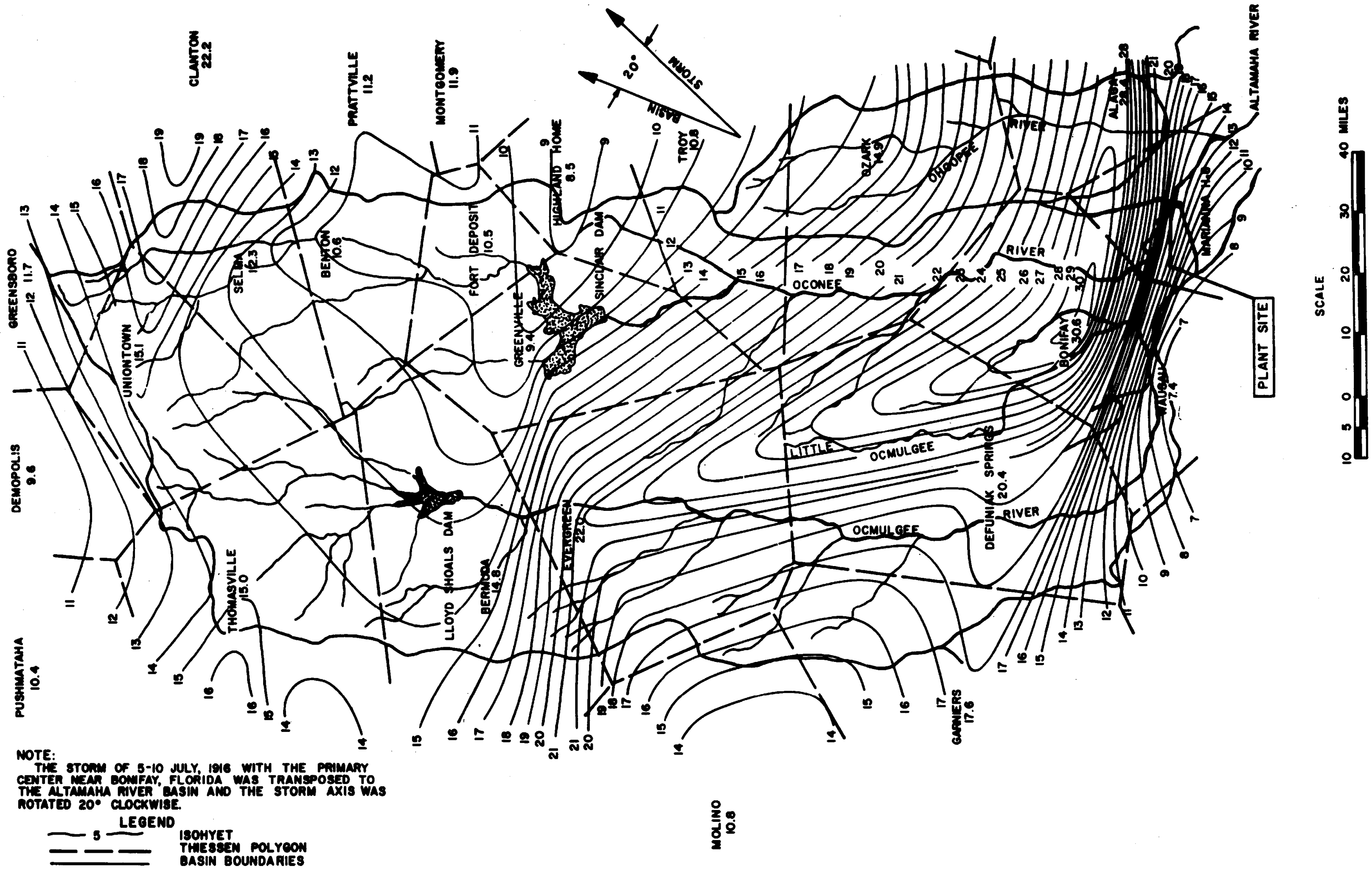
UNIT 1 AND UNIT 2

ALTAMAHA RIVER AT HNP STAGE DISCHARGE RELATION HIGH FLOWS

FIGURE 2.4-8



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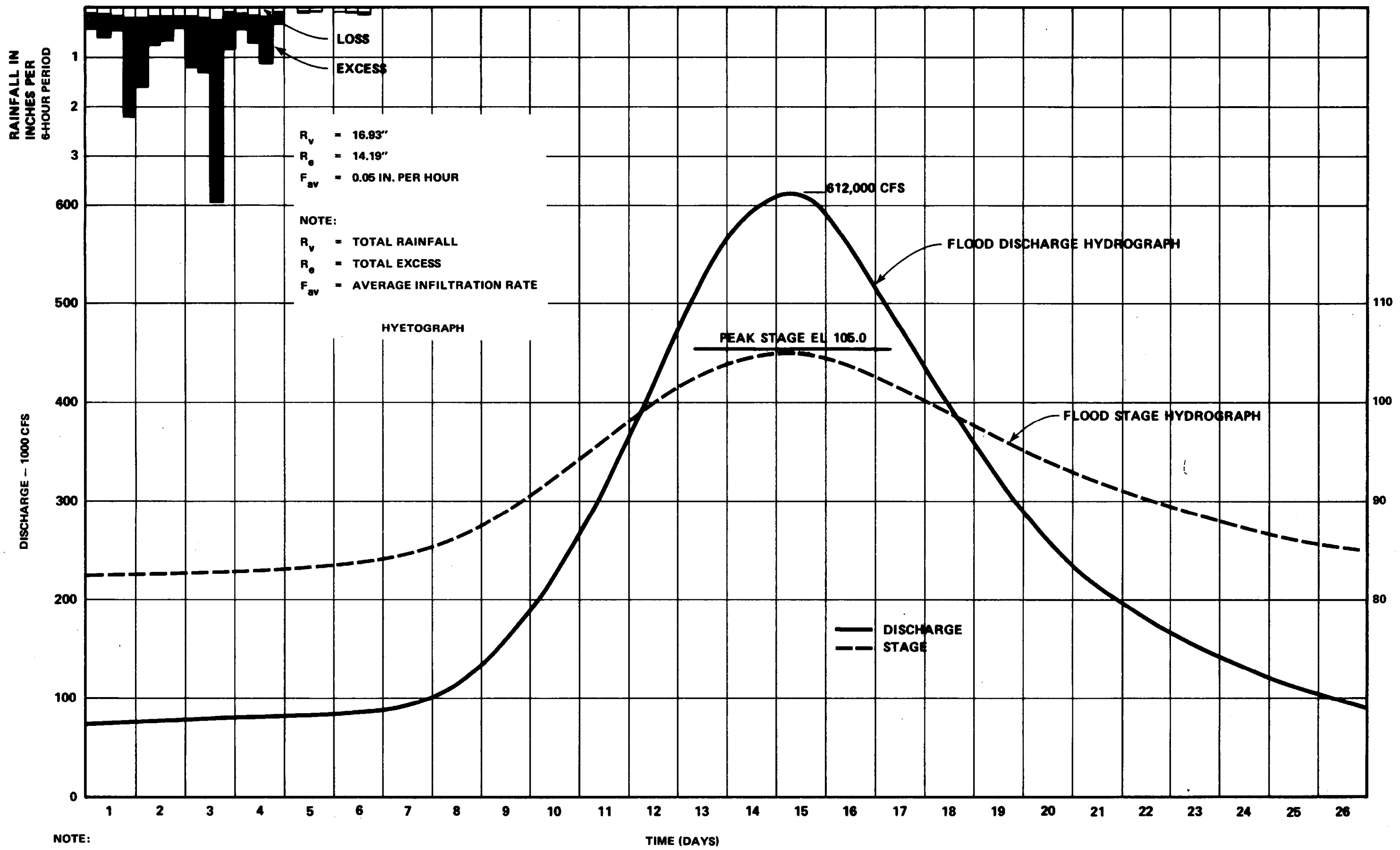
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 1 AND UNIT 2	ISOHYETAL MAP FOR PROBABLE MAXIMUM PRECIPITATION – ALTAMAHA RIVER ABOVE PLANT SITE FIGURE 2.4-9



NOTE:
RAINFALL BASED ON STORM OF JULY 5-10, 1961, (GMI-19)
WITH PRIMARY CENTER NEAR BONIFAY, FLORIDA

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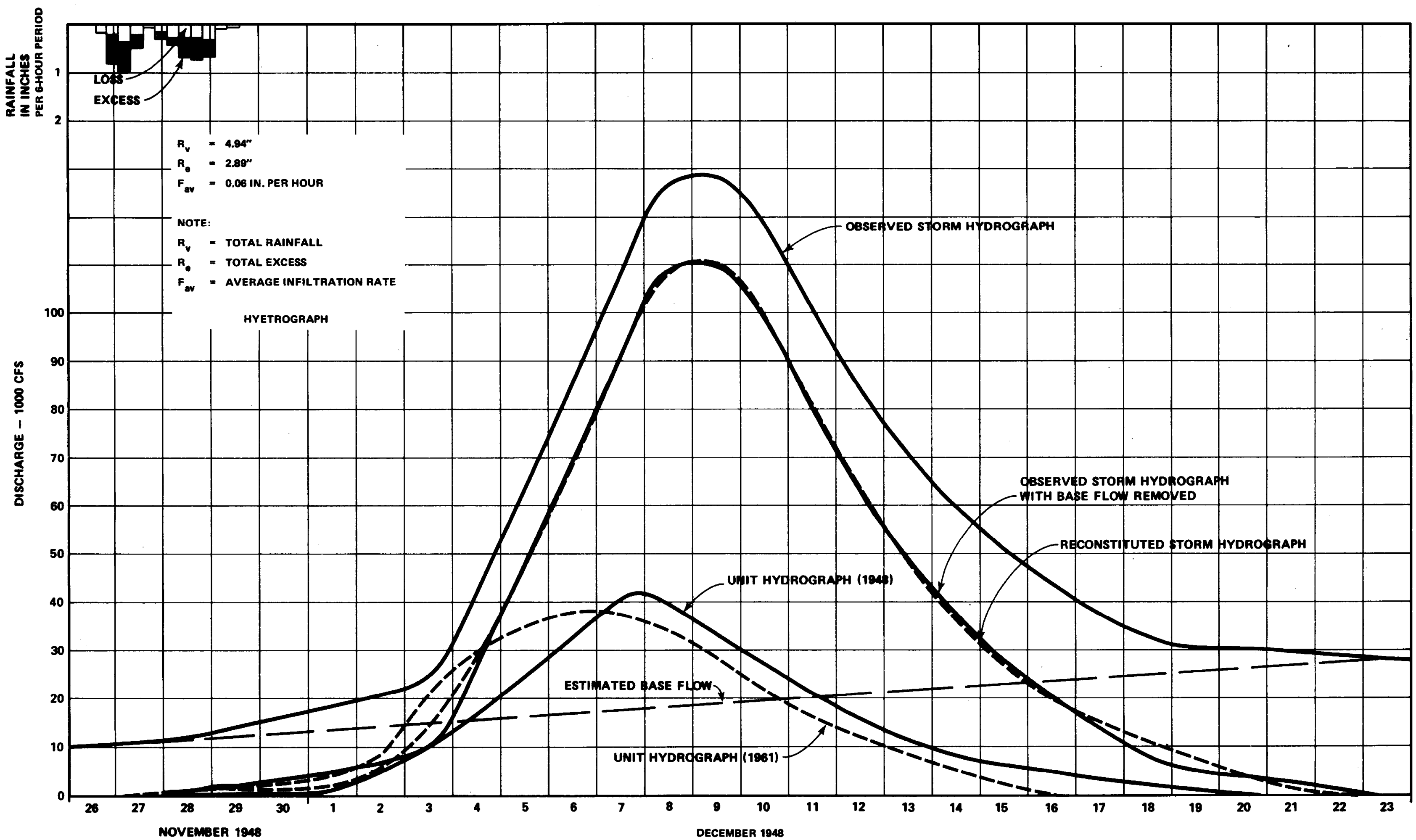
UNIT 1 AND UNIT 2

PROBABLE MAXIMUM FLOOD ALTAMAHA RIVER

FIGURE 2.4-10



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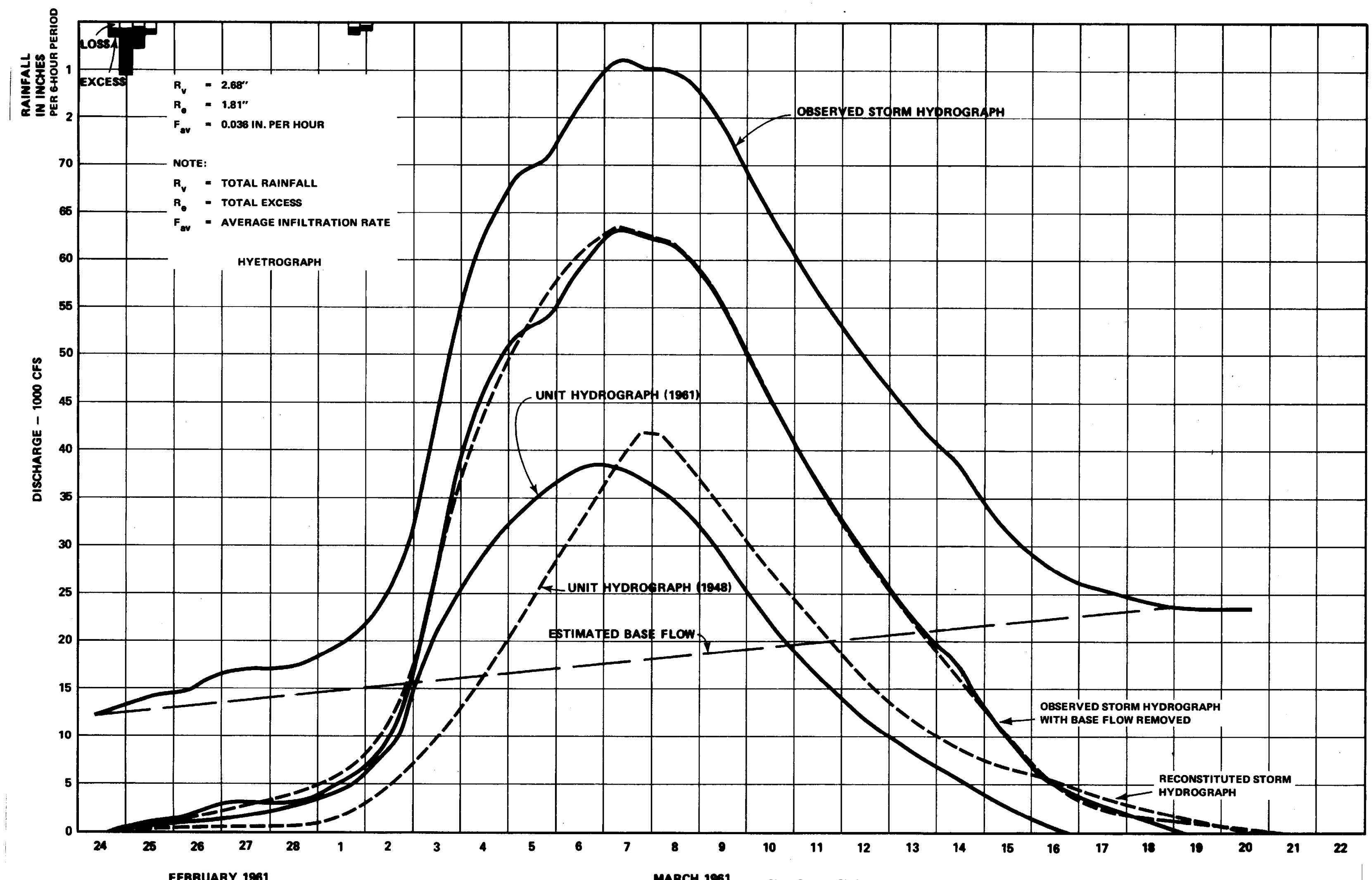
HYDROGRAPH – 1948 FLOOD

FIGURE 2.4-11



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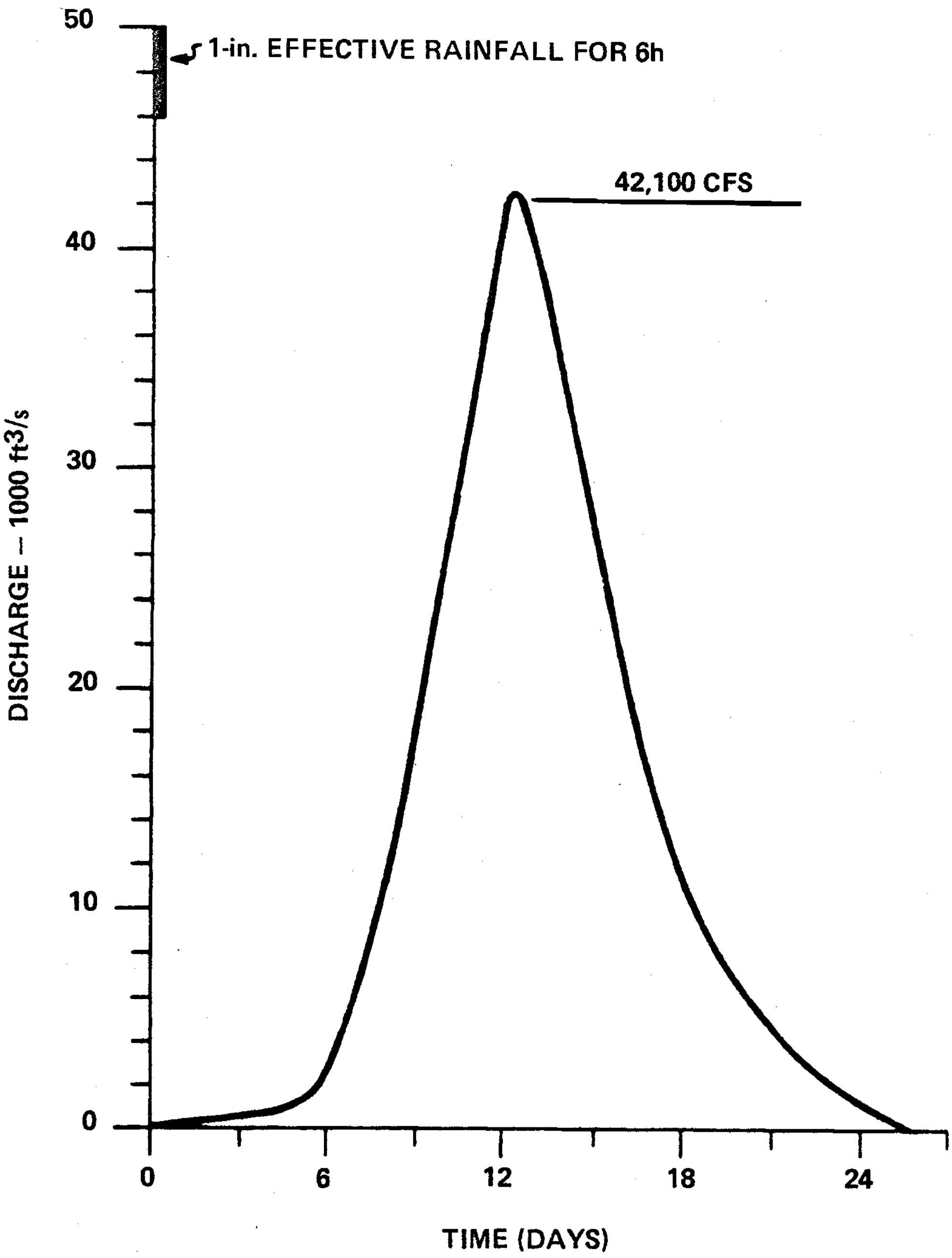
UNIT 1 AND UNIT 2

HYDROGRAPH – 1961 FLOOD

FIGURE 2.4-12



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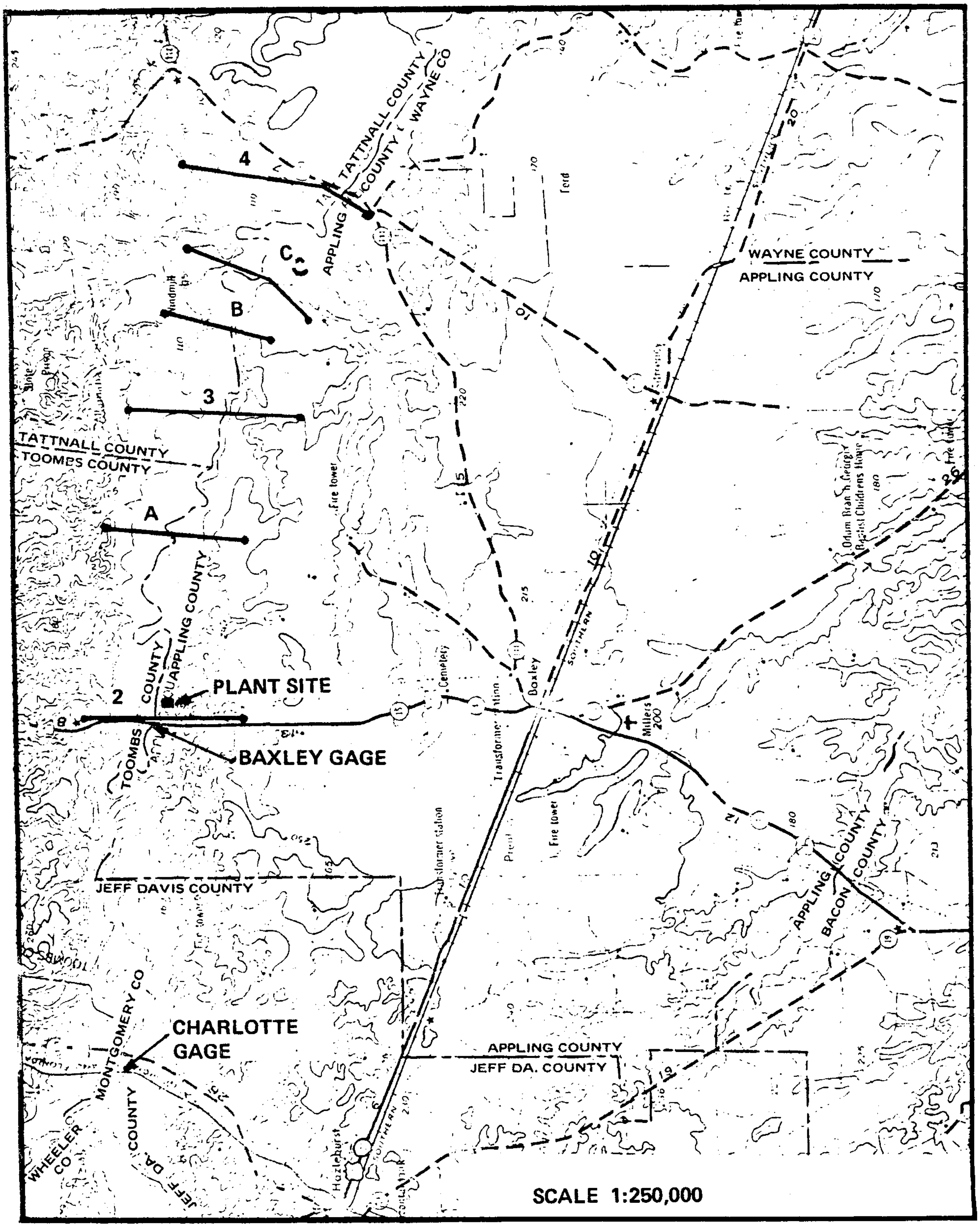
UNIT 1 AND UNIT 2

ADOPTED 6-h UNIT HYDROGRAPH ALTAMAHA RIVER AT NUCLEAR POWER PLANT SITE

FIGURE 2.4-13



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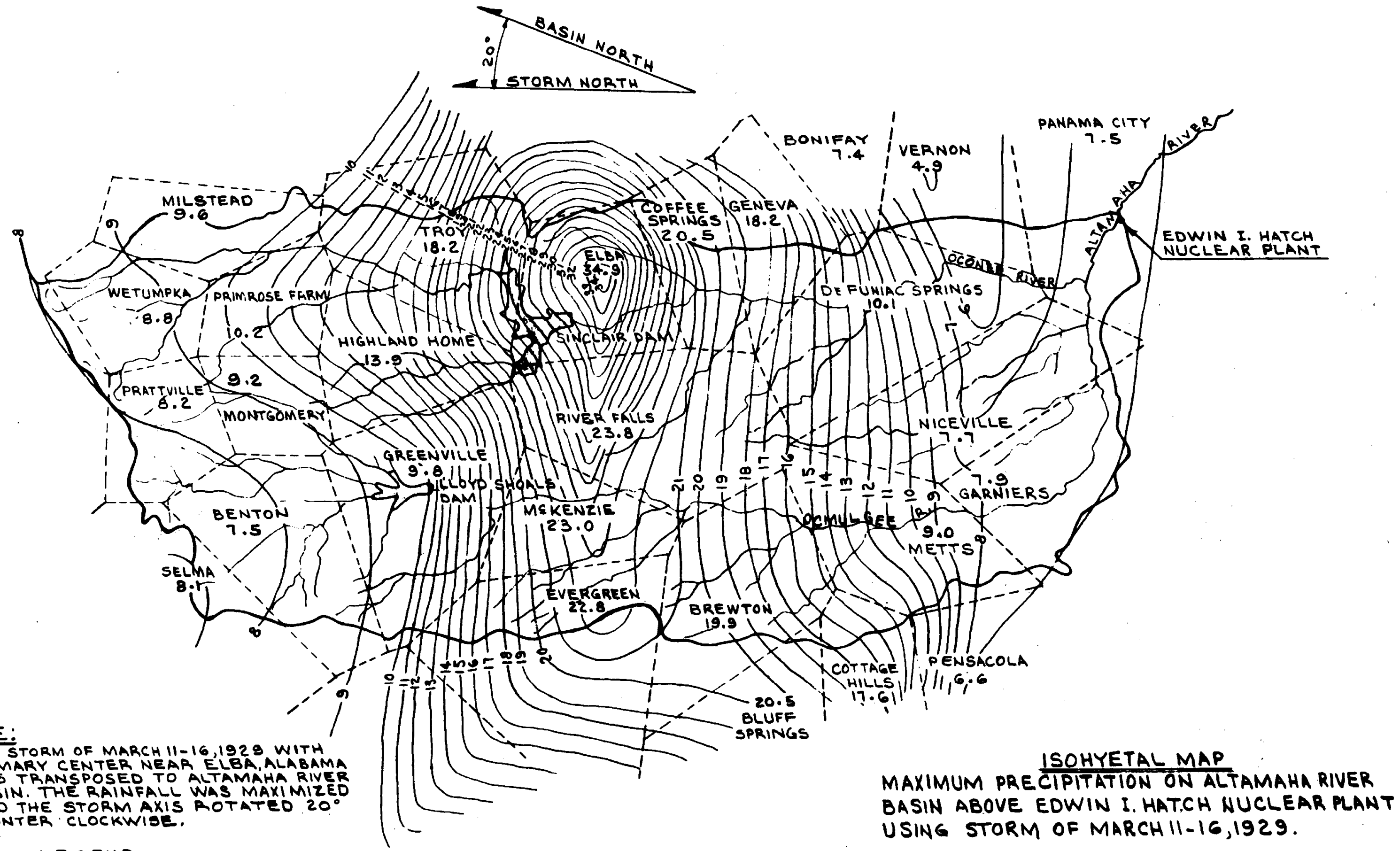
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

LOCATIONS OF SECTIONS RUN IN 1967 FOR BACKWATER GRADIENT STUDY

FIGURE 2.4-14



NOTE:
THE STORM OF MARCH 11-16, 1929 WITH PRIMARY CENTER NEAR ELBA, ALABAMA WAS TRANSPOSED TO ALTAMAHA RIVER BASIN. THE RAINFALL WAS MAXIMIZED AND THE STORM AXIS ROTATED 20° COUNTER CLOCKWISE.

LEGEND

—5— ISOHYET
- - - - THIESSEN POLYGON
—·—·— BASIN BOUNDARIES

ISOHYETAL MAP

MAXIMUM PRECIPITATION ON ALTAMAHA RIVER BASIN ABOVE EDWIN I. HATCH NUCLEAR PLANT USING STORM OF MARCH 11-16, 1929.

SCALE
10 5 0 10 20 30 40 MILES

HISTORICAL

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EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

ISOHYETAL MAP

FIGURE 2.4-15

PROBABLE MAXIMUM PRECIPITATION ADJUSTMENT FACTOR
ALTAMAHA RIVER BASIN ABOVE EDWIN I. HATCH NUCLEAR PLANT

STORM OF MARCH 11-16, 1929, (LMV 2-20) TRANSPOSED FOR MONTH OF MAY

THIESSEN POLYGON	BARRIER ELEVATION (FEET)	1	2	3	4	5	6
WETUMPKA	1000	74.4	2.82	0.24	2.58	7.8	1.10
MILSTEAD	750	74.5	2.83	0.20	2.63	8.0	1.12
PRATTVILLE	900	74.4	2.82	0.23	2.59	7.9	1.11
SELMA	950	74.4	2.82	0.24	2.58	7.8	1.10
BENTON	900	74.6	2.84	0.23	2.61	7.9	1.11
MONTGOMERY	750	74.5	2.83	0.19	2.64	8.0	1.13
PRIMROSE FARM	750	74.5	2.83	0.19	2.64	8.0	1.13
TROY	650	74.8	2.87	0.17	2.70	8.2	1.15
HIGHLAND HOME	700	74.7	2.86	0.18	2.68	8.1	1.14
GREENVILLE	750	74.8	2.87	0.19	2.68	8.1	1.14
EVERGREEN	700	75.2	2.92	0.18	2.74	8.3	1.17
MCKENZIE	700	75.1	2.91	0.18	2.73	8.3	1.16
RIVER FALLS	600	75.2	2.92	0.15	2.77	8.4	1.18
ELBA	600	75.1	2.91	0.15	2.76	8.4	1.18
COFFEE SPRINGS	450	75.2	2.92	0.11	2.81	8.5	1.20
GENEVA	450	75.2	2.92	0.11	2.81	8.5	1.20
BREWTON	450	75.5	2.97	0.12	2.85	8.6	1.22
BLUFF SPRINGS	450	75.5	2.97	0.12	2.85	8.6	1.22
COTTAGE HILLS	450	75.7	3.00	0.12	2.88	8.7	1.23
METTS	450	75.7	3.00	0.12	2.88	8.7	1.23
GARNIERS	400	75.9	3.03	0.11	2.92	8.9	1.25
NICEVILLE	450	75.7	3.00	0.12	2.88	8.7	1.23
PANAMA CITY	350	76.0	3.04	0.10	2.94	8.9	1.25
DEFUNIAC SPRINGS	450	75.5	2.97	0.11	2.86	8.7	1.22
VERNON	400	75.7	3.00	0.11	2.89	8.8	1.23

EXPLANATORY NOTES

STORM'S 12-HOUR 1000-MB DEWPOINT
- ACTUAL THROUGHOUT $(Dp)_s = 67^{\circ}$
- MAXIMUM AT SELECTED LOCATION AND TIME $(Dp)'_M$ PER STATION – – – – – COLUMN 1

STORM'S MOISTURE CHARGE ADJUSTED FOR ELEVATION OF INFLOW BARRIER
- ACTUAL TOTAL AT $(Dp)_s$ DEWPOINT – $(Wp)_s = 1.96''$
- ACTUAL INFLOW BARRIER REDUCTION AT $(Dp)_s$ $(\overline{Wp})_s = 0.04''$
- NET ACTUAL ABOVE INFLOW BARRIER $(Wp)_s - (\overline{Wp})_s = 1.92''$

MAXIMUM CORRESPONDING TO $(Dp)'_M$
- $(Wp)'_M$ PER STATION – – – – – COLUMN 2

MAXIMUM INFLOW BARRIER REDUCTION AT $(Dp)'_M$
- $(\overline{Wp})'_M$ PER STATION – – – – – COLUMN 3

MAXIMUM ABOVE INFLOW BARRIER
- $(Wp)'_M - (\overline{Wp})_M$ PER STATION – – – – – COLUMN 4

TEMPERATURE CONTRAST
- ACTUAL STORM CENTER – – – – – $(Tc)_s = 13.7$
- STORM CENTER TRANSPOSED IN LOCATION AND TIME – – – – – $(Tc)_t = 9.2$

MOISTURE CHARGE ADJUSTED FOR INFLOW BARRIER AND STORM EFFICIENCY
- ACTUAL STORM LOCATION $(Tc)_s^{1/2} \times [(Wp)_s - (\overline{Wp})_s] = 7.1$
- MAXIMUM AT SELECTED LOCATION AND TIME $(Tc)_t^{1/2} \times [(Wp)'_M - (\overline{Wp})'_M]$ PER STATION = $9.2^{1/2}$ X COLUMN 4 = COLUMN 5

MAXIMUM PROBABLE PRECIPITATION FACTOR
- RATIO OF MAXIMUM AT STATION IN TRANSPOSED LOCATION TO ACTUAL FOR RESPECTIVE STATION IN ORIGINAL LOCATION =

$$\frac{(Tc)_t^{1/2} \times [(Wp)'_M - (\overline{Wp})'_M]}{(Tc)_s^{1/2} \times [(Wp)_s - (\overline{Wp})_s]} = \frac{\text{COLUMN 5}}{7.1} = \text{COLUMN 6}$$

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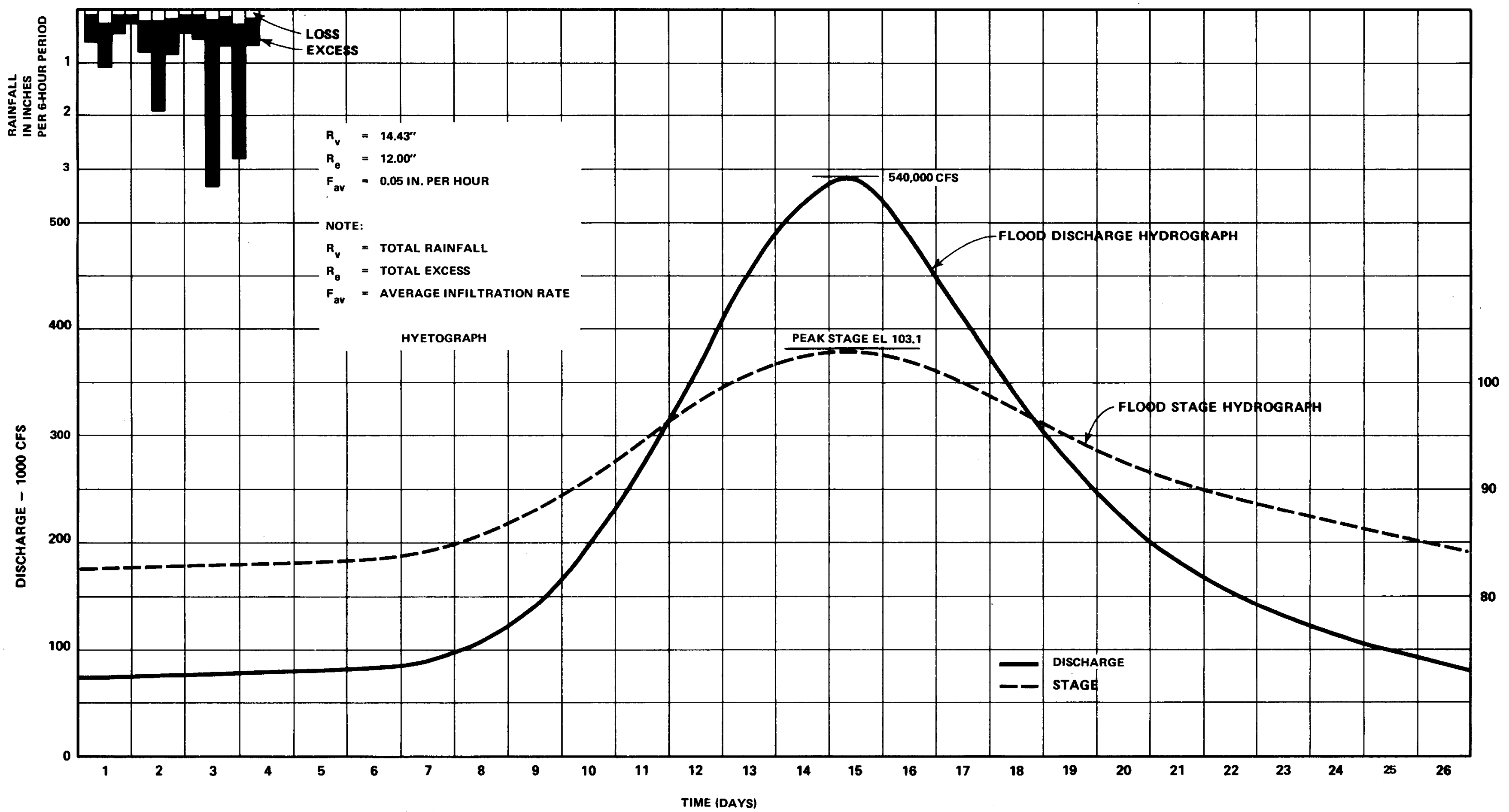
PROBABLE MAXIMUM PRECIPITATION ADJUSTMENT FACTOR

FIGURE 2.4-16



STORM OF MARCH 11-16, 1929, TRANSPOSED AND MAXIMIZED TO OBTAIN MAXIMUM PRECIPITATION IN ALTAMAHA BASIN ABOVE EDWIN I. HATCH NUCLEAR PLANT

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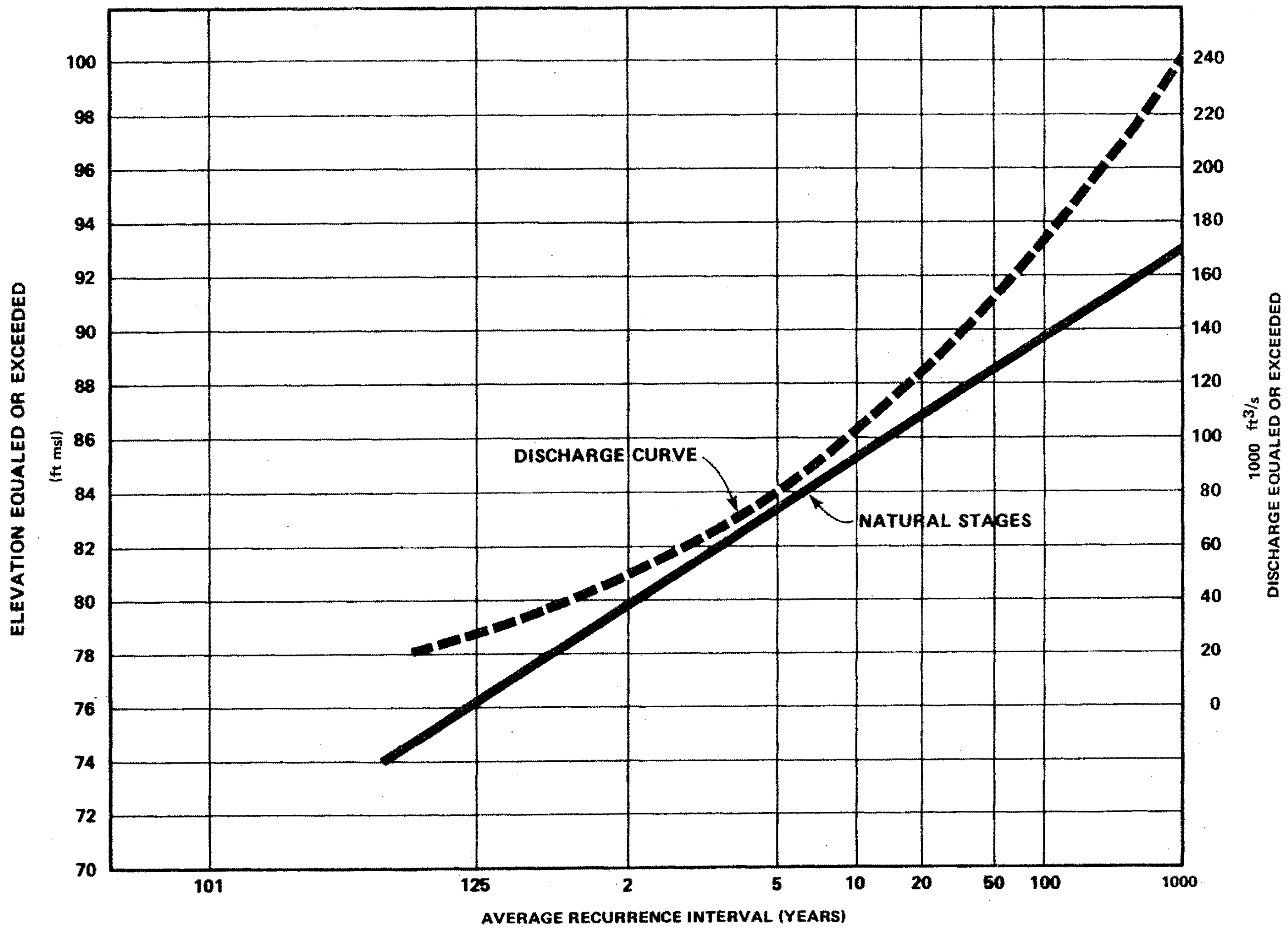
STORM HYDROGRAPH

FIGURE 2.4-17



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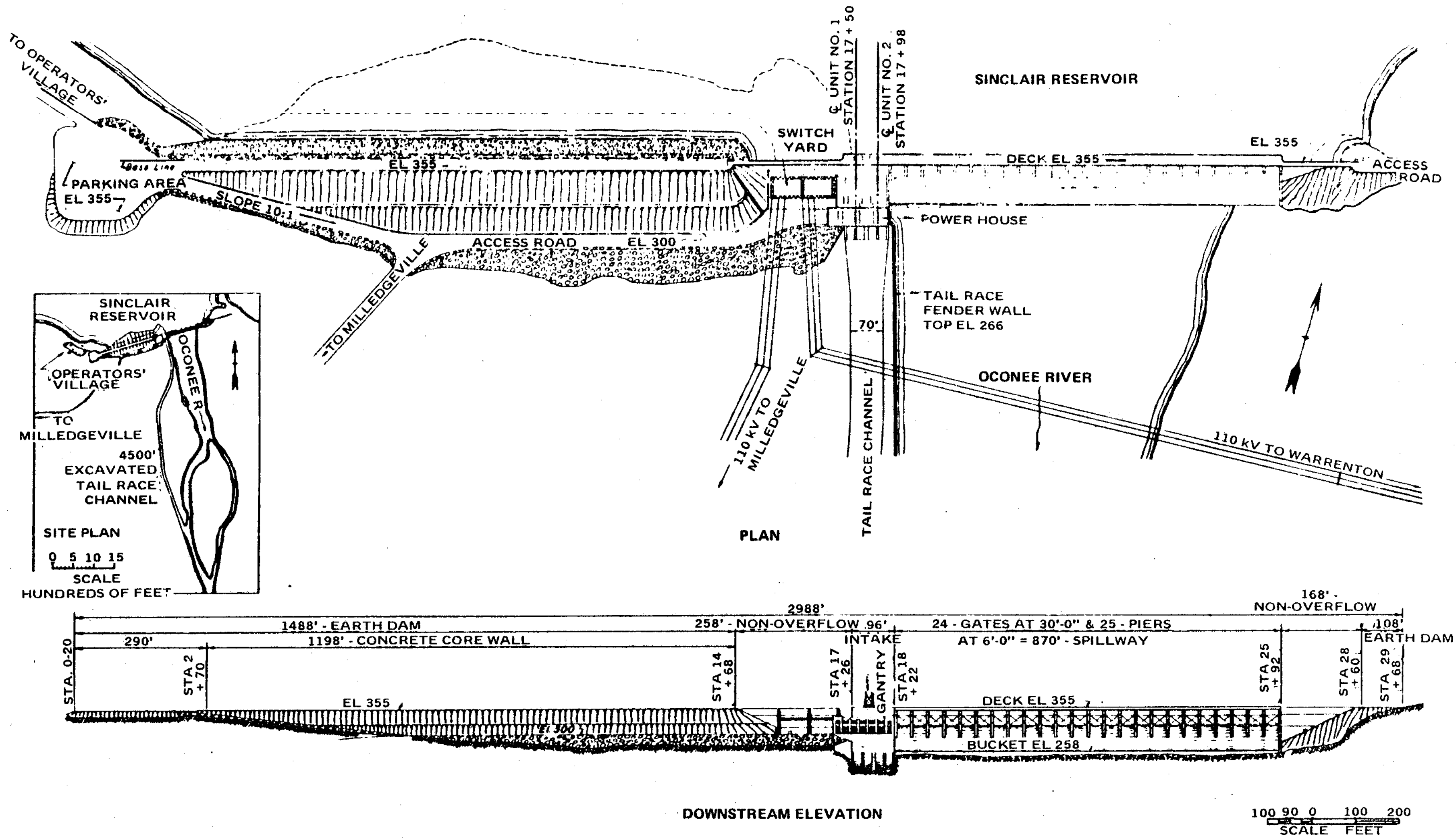
DISCHARGE STAGE FREQUENCY CURVES – ALTAMAHA RIVER AT U.S. HIGHWAY 1 CROSSING GEORGIA

FIGURE 2.4-18



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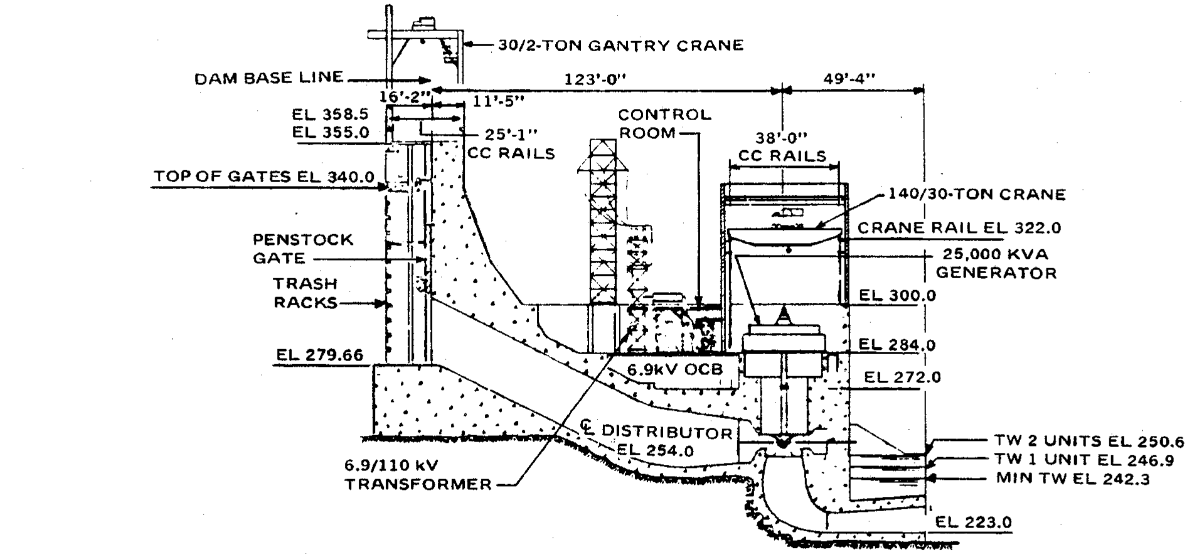
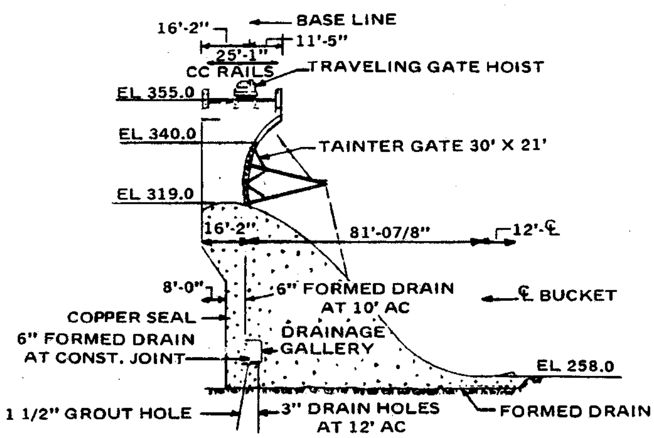
SINCLAIR DAM PLAN AND ELEVATION

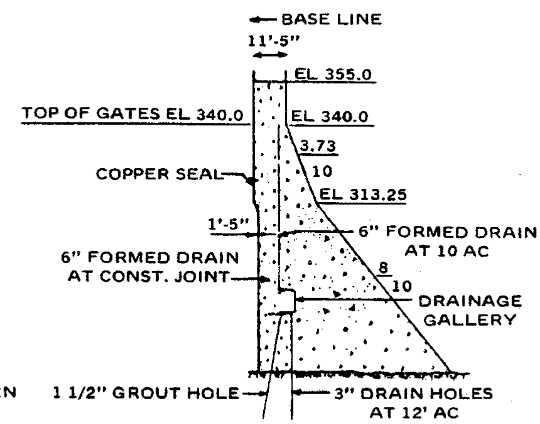
FIGURE 2.4-19



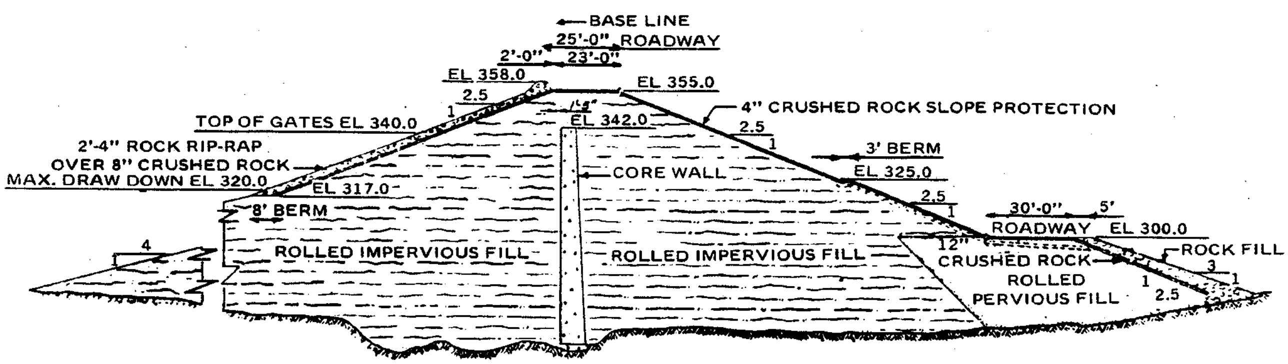
SECTION THRU POWERHOUSE & INTAKE



SECTION THRU SPILLWAY



SECTION THRU NON-OVERFLOW



SECTION THRU EARTH DAM

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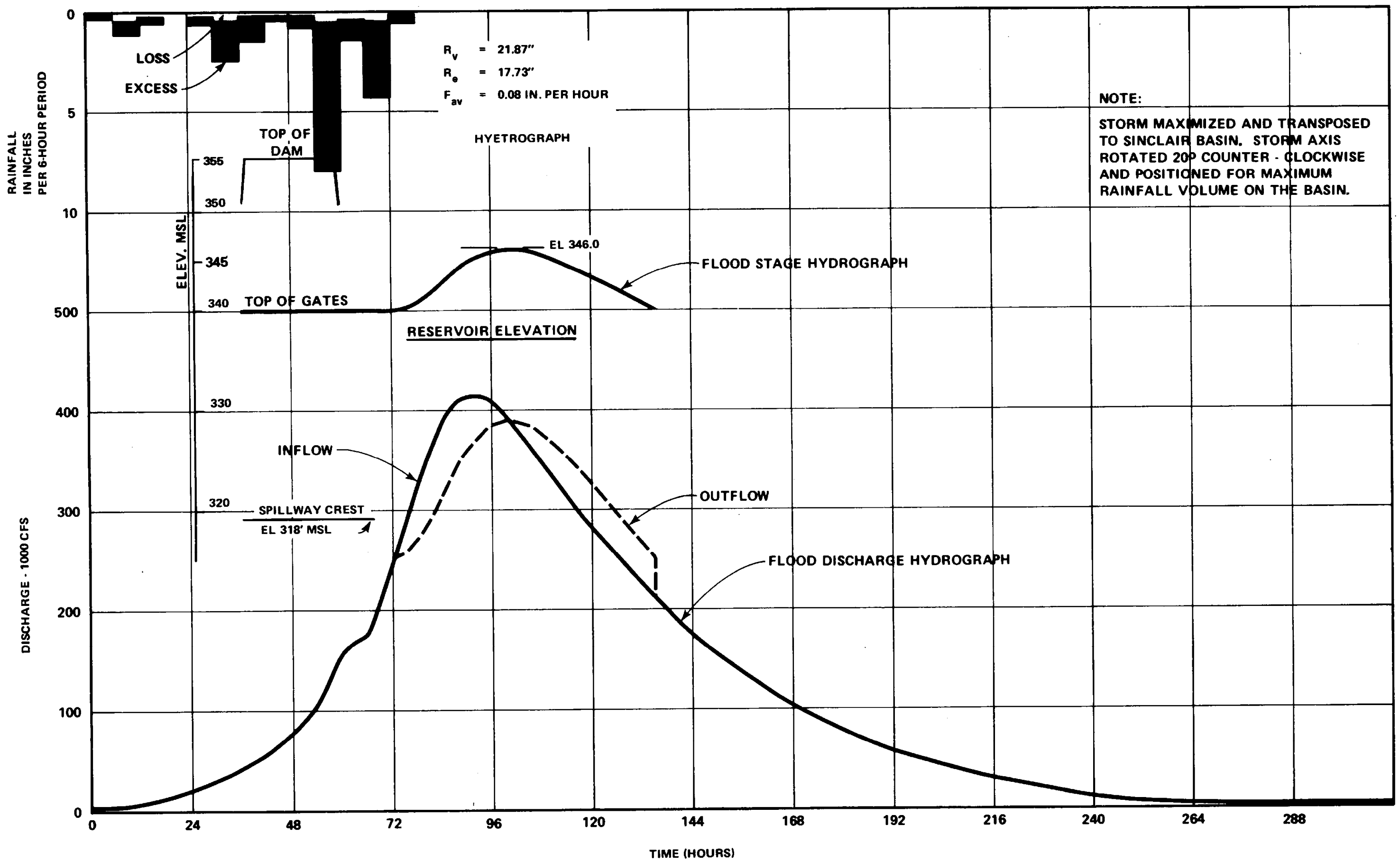
SINCLAIR DAM SECTIONS

FIGURE 2.4-20



NOTE:

RAINFALL BASED ON STORM OF MARCH 11-16, 1929, (LMV 2-20)

STORM MAXIMIZED AND TRANSPOSED TO SINCLAIR BASIN; STORM AXIS ROTATED 20° COUNTER-CLOCKWISE AND POSITIONED FOR MAXIMUM RAINFALL VOLUME ON THE BASIN

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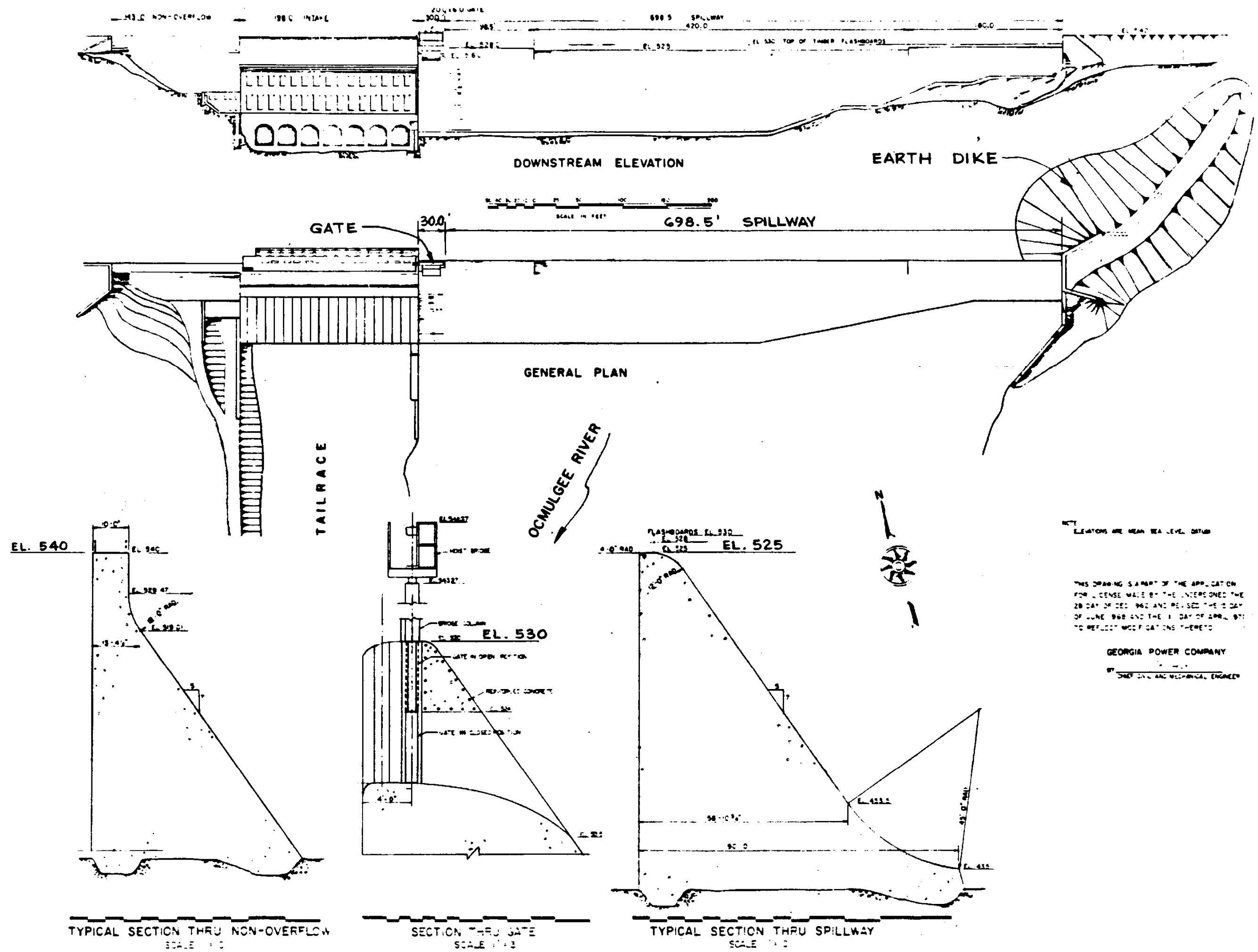
SINCLAIR DAM HYDROGRAPH DURING SPILLWAY DESIGN FLOOD

FIGURE 2.4-21



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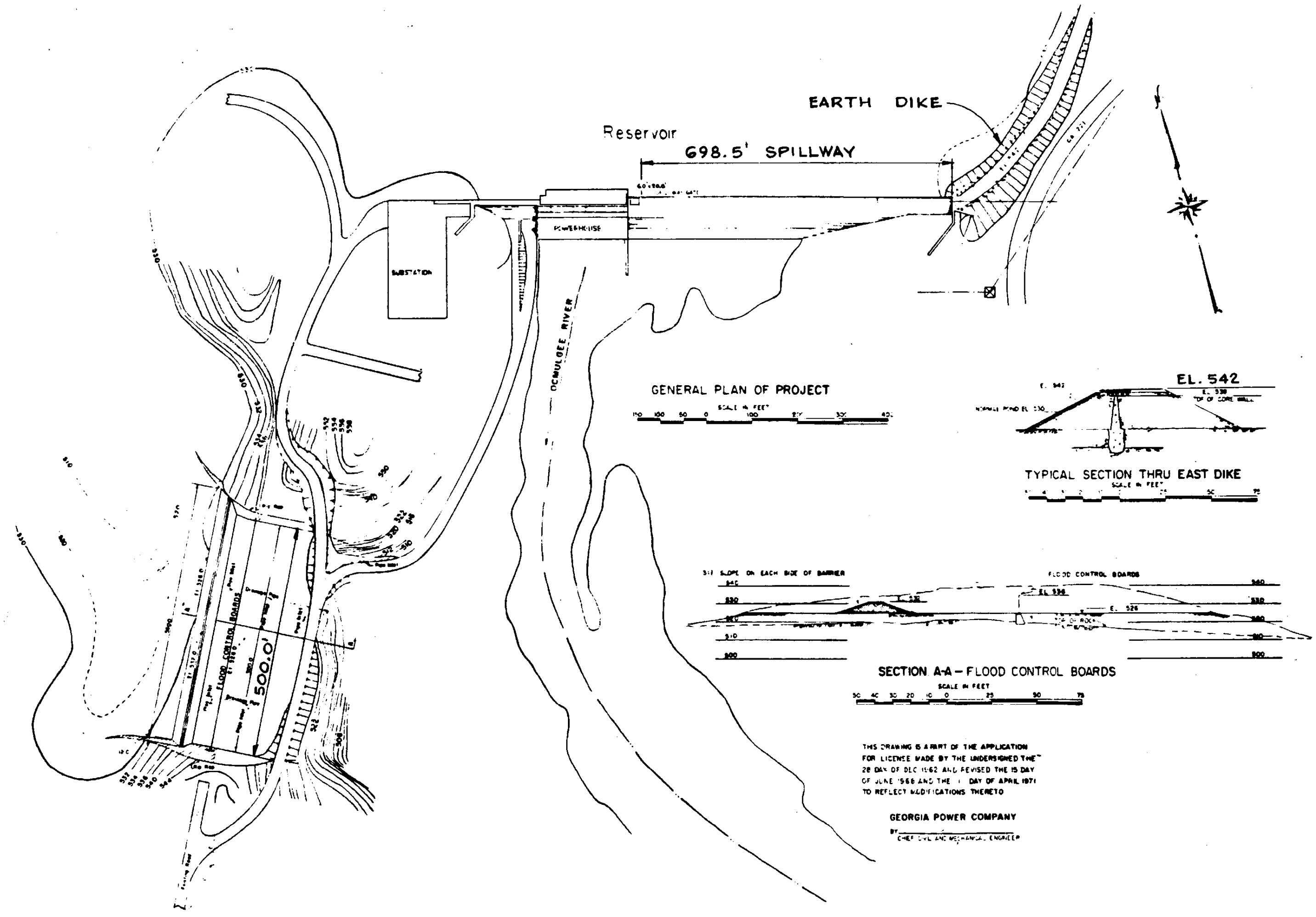
LLOYD SHOALS DAM PLAN, ELEVATION, AND SECTIONS

FIGURE 2.4-22



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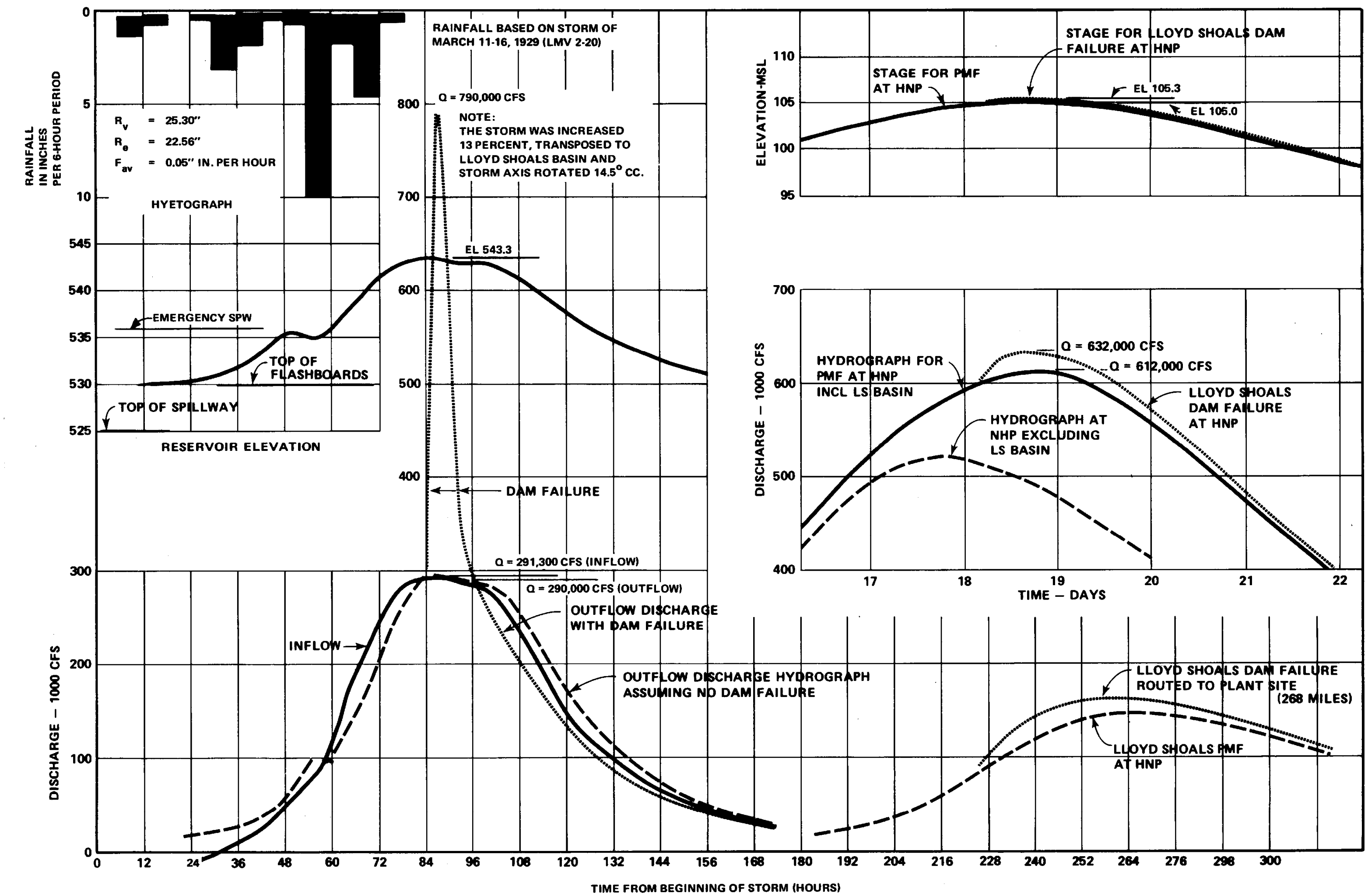
LLOYD SHOALS PROJECT GENERAL PLAN AND SECTIONS

FIGURE 2.4-23



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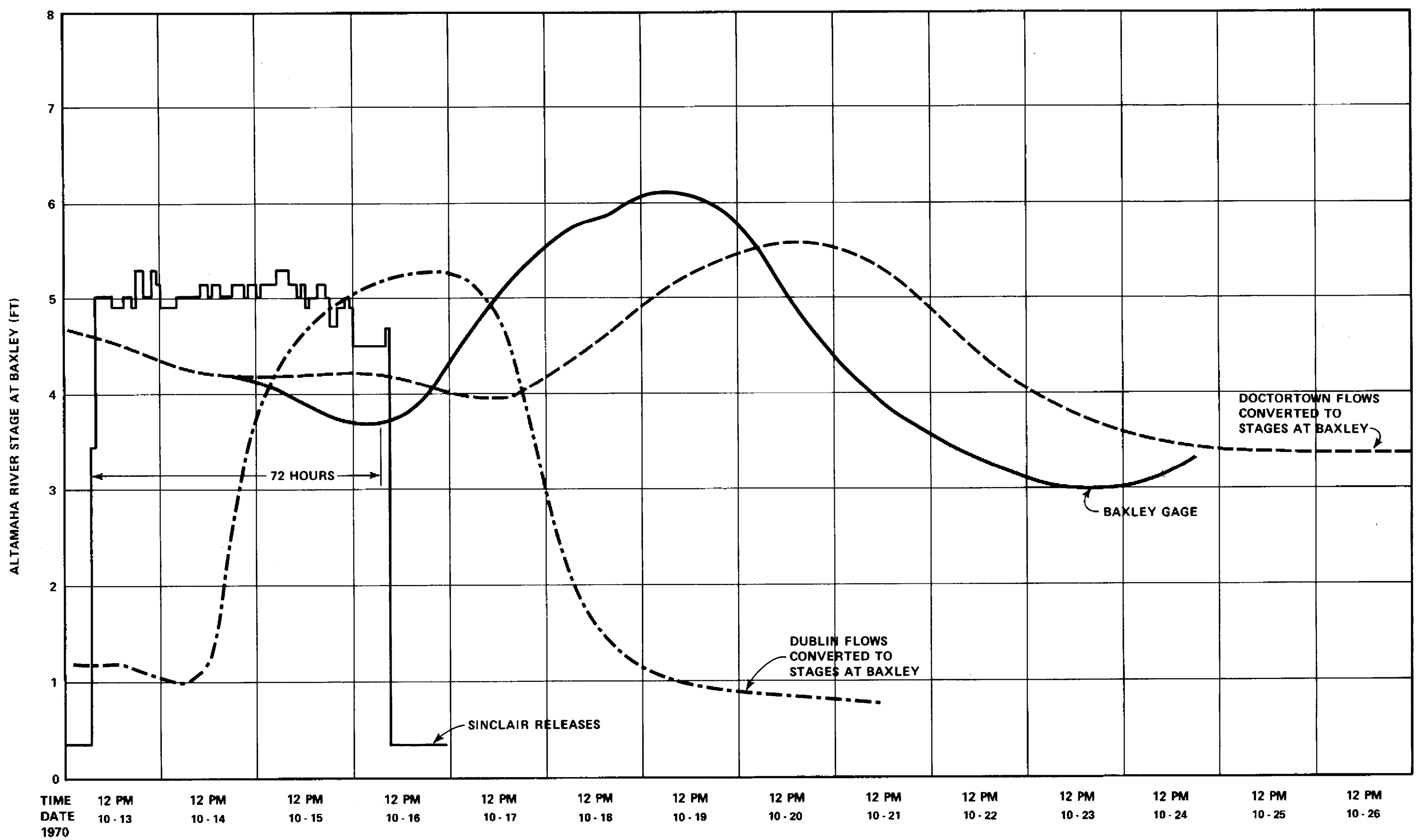
LLOYD SHOALS DAM FAILURE STUDY

FIGURE 2.4-24



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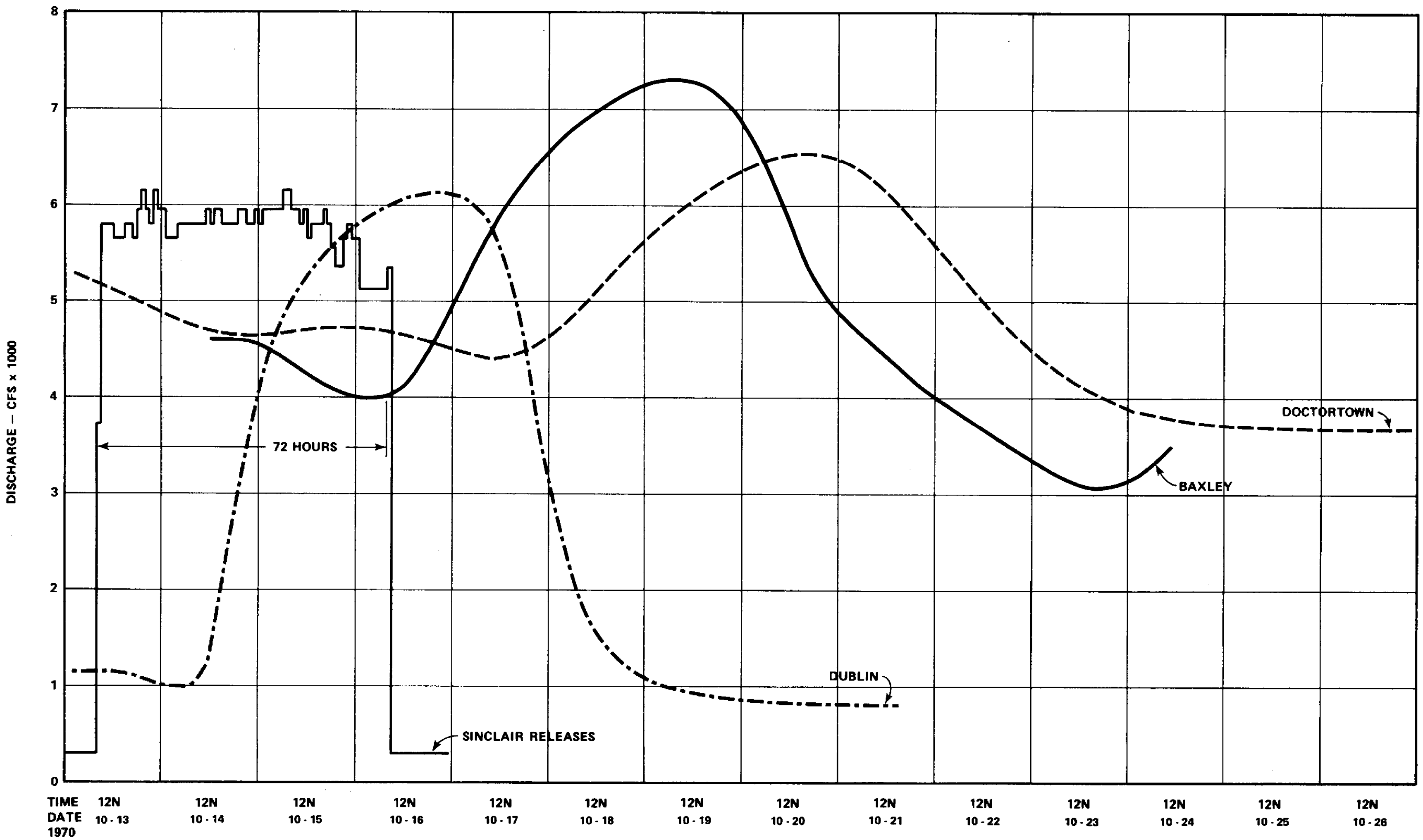
SINCLAIR DAM GENERATION TEST STAGE CURVES

FIGURE 2.4-25



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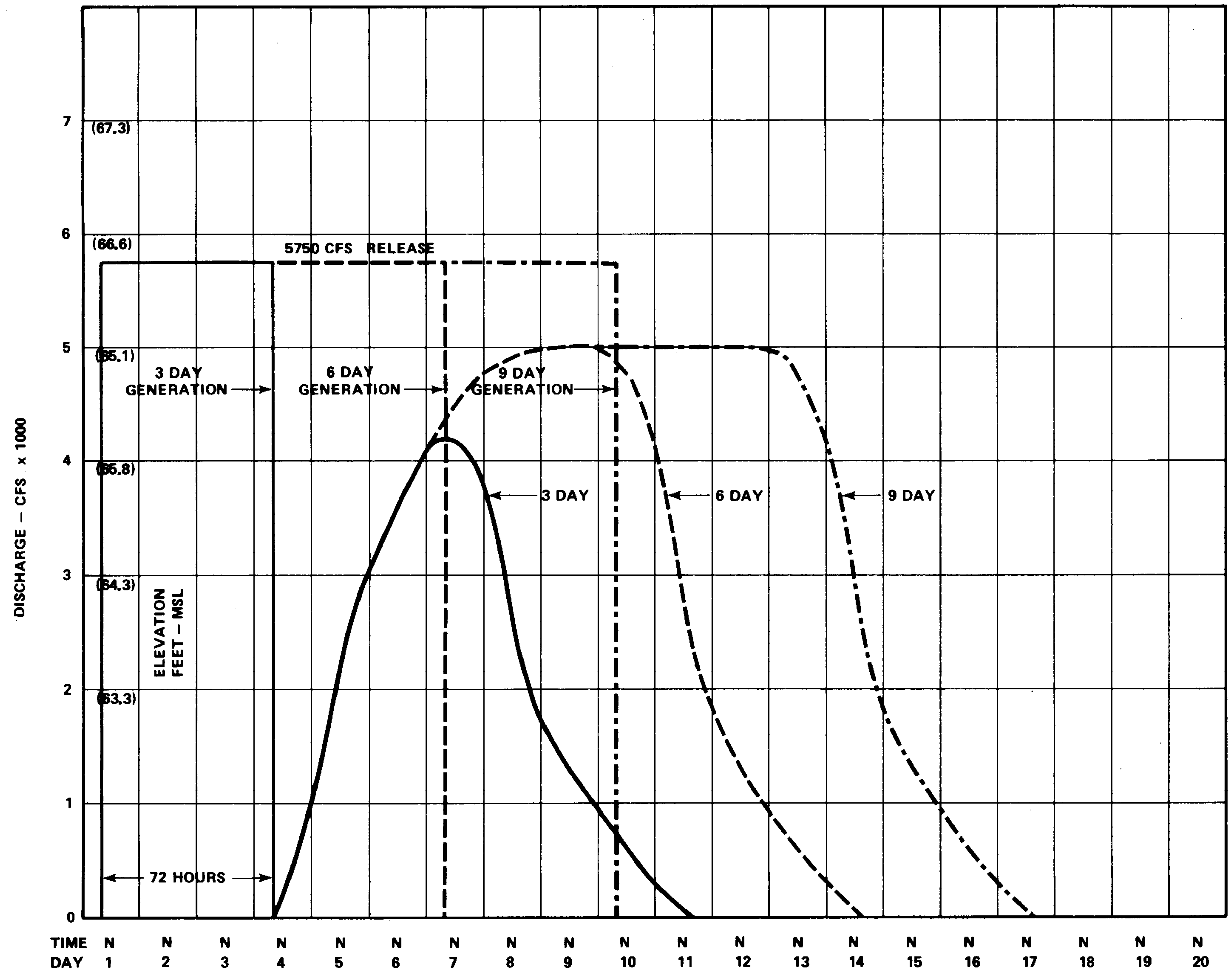
SINCLAIR DAM GENERATION TEST DISCHARGE CURVES

FIGURE 2.4-26



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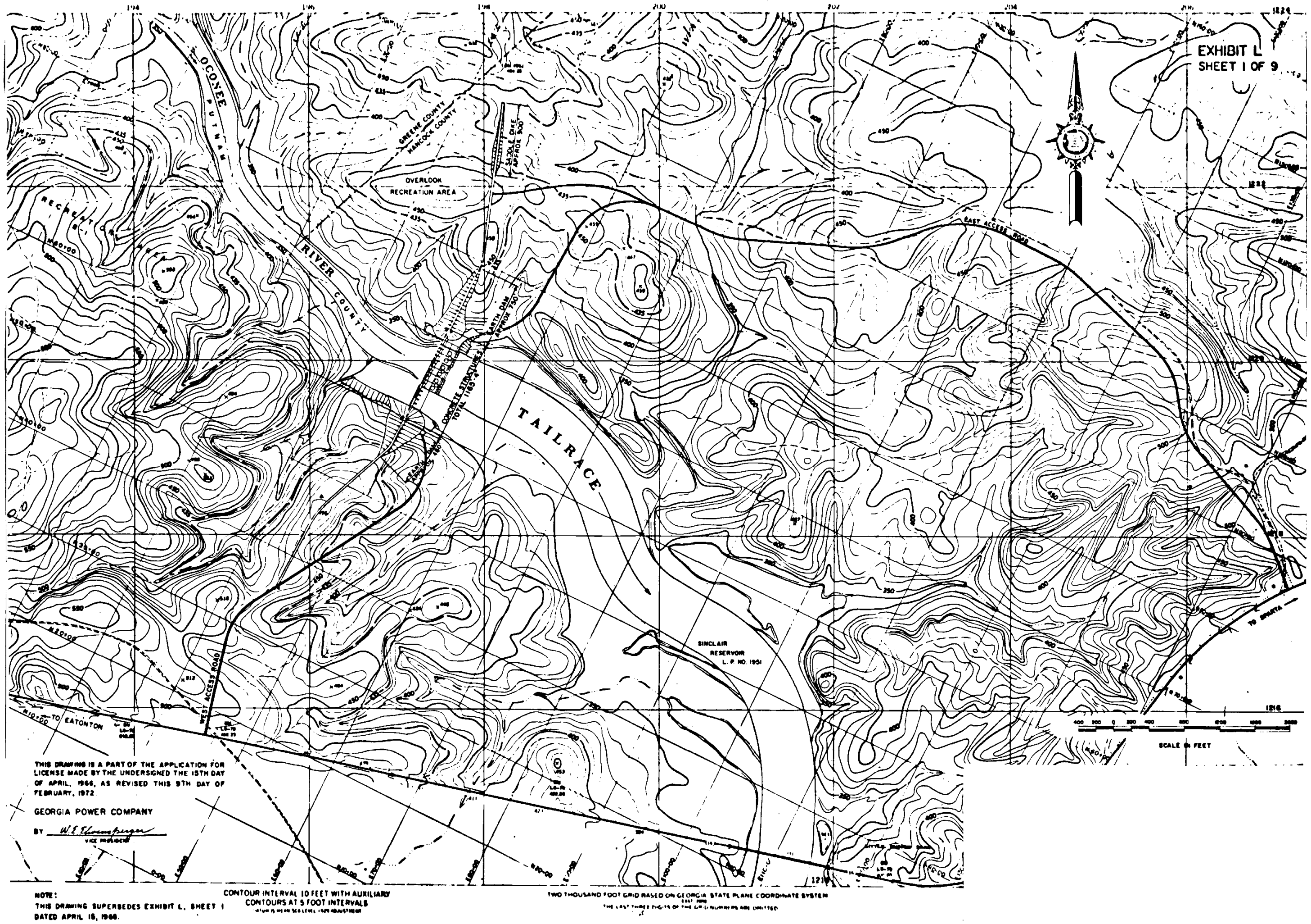
HYDROGRAPH AT BAXLEY, GEORGIA FROM SINCLAIR TURBINE RELEASE (UNITS 1 AND 2)

FIGURE 2.4-27



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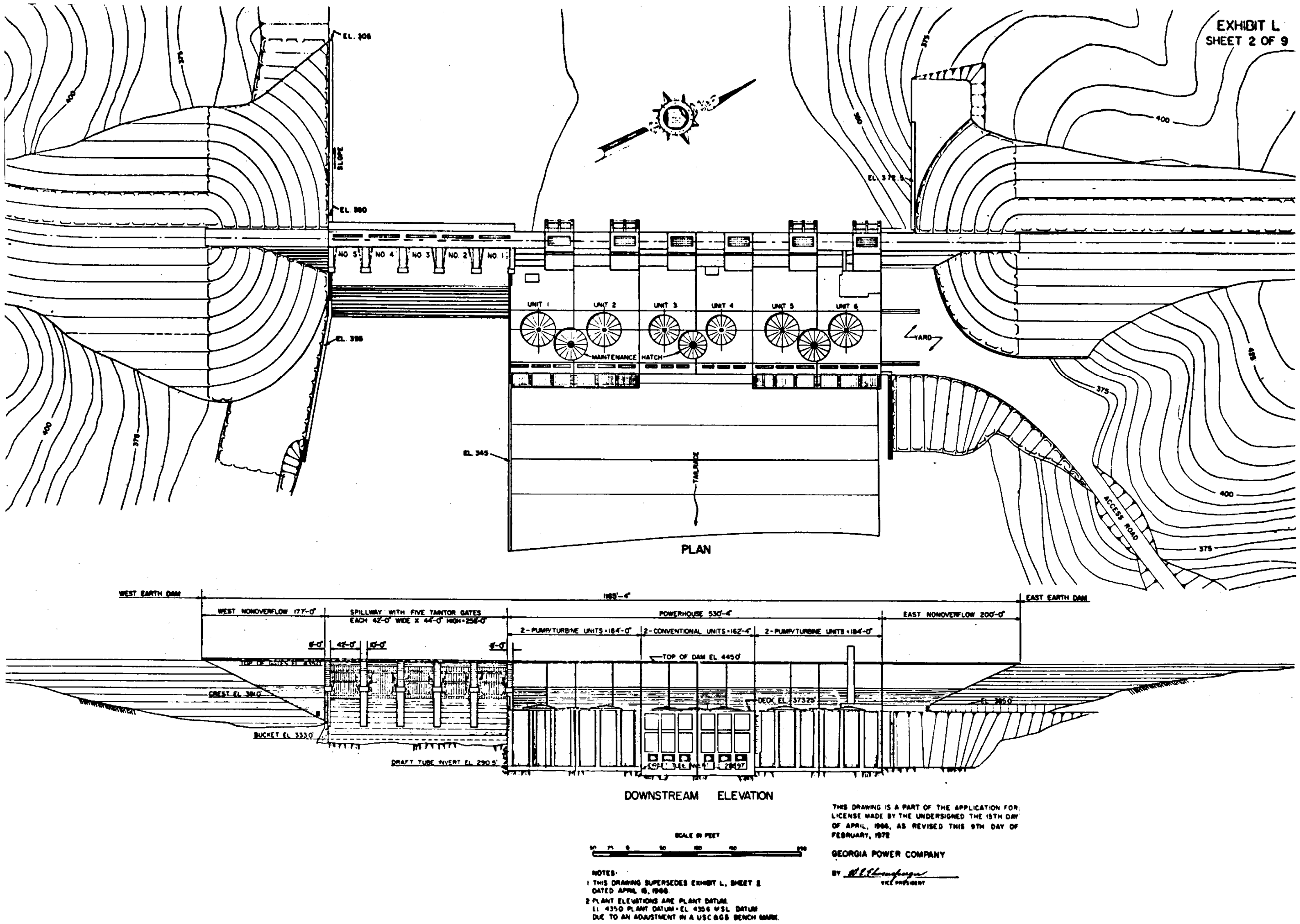
WALLACE DAM GENERAL PLAN OF DEVELOPMENT

FIGURE 2.4-28



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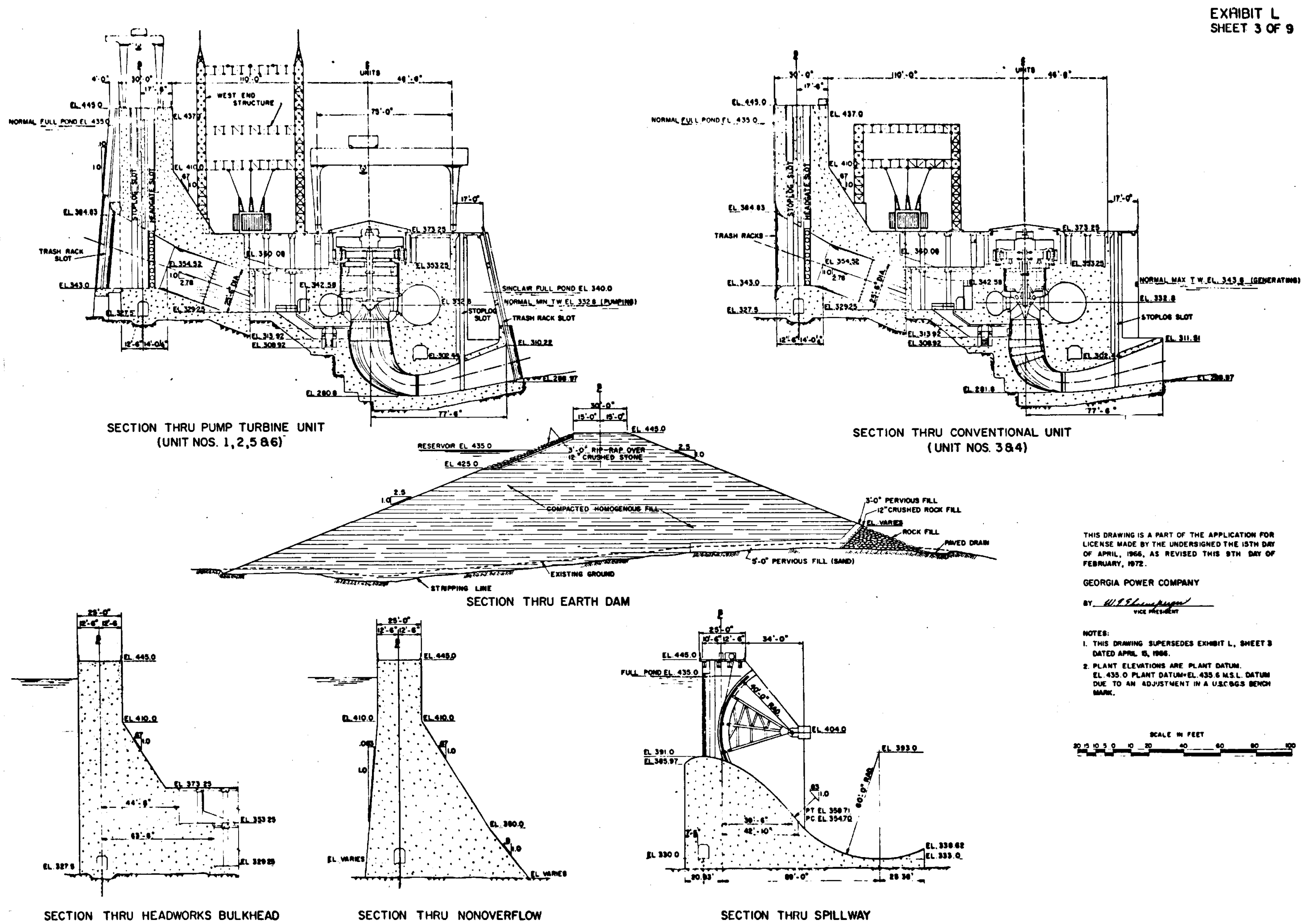
WALLACE DAM PLAN AND DOWNSTREAM ELEVATION

FIGURE 2.4-29



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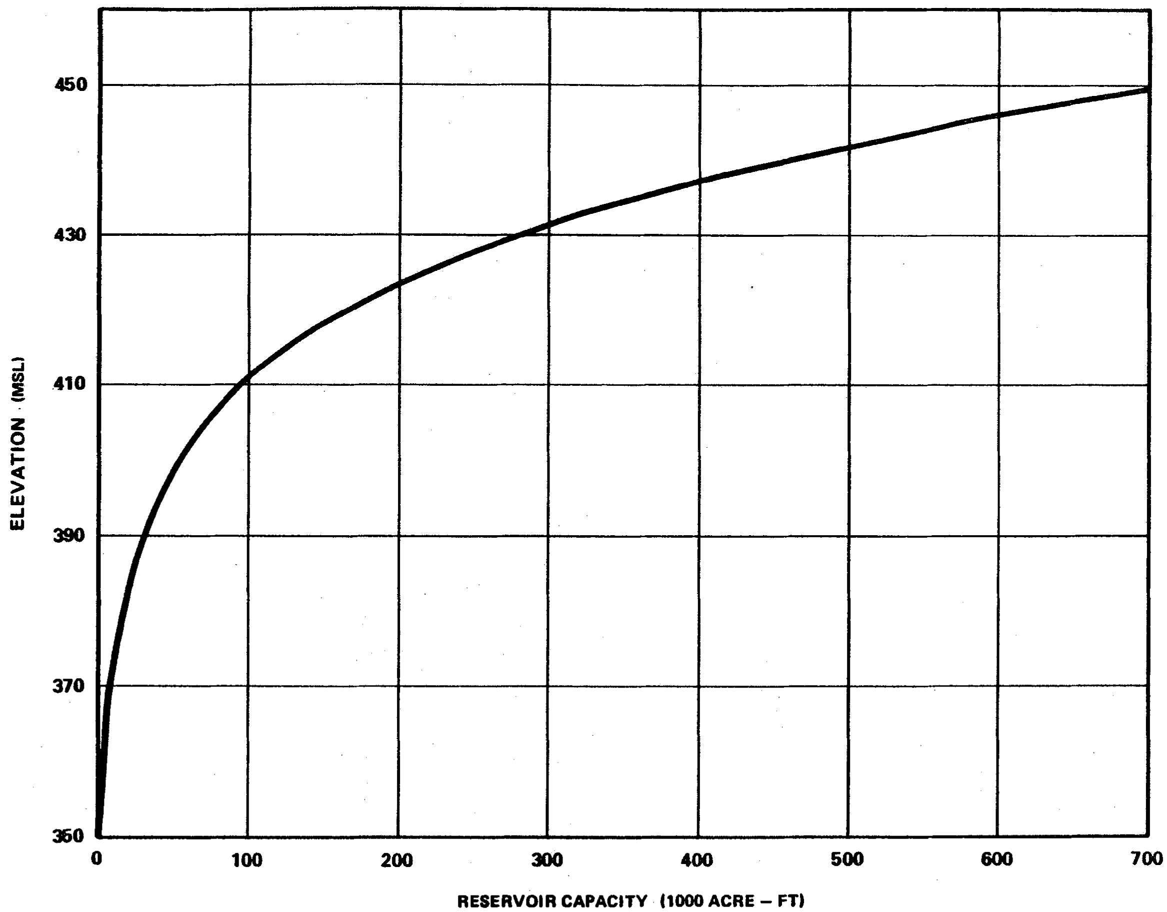
WALLACE DAM TYPICAL SECTIONS

FIGURE 2.4-30



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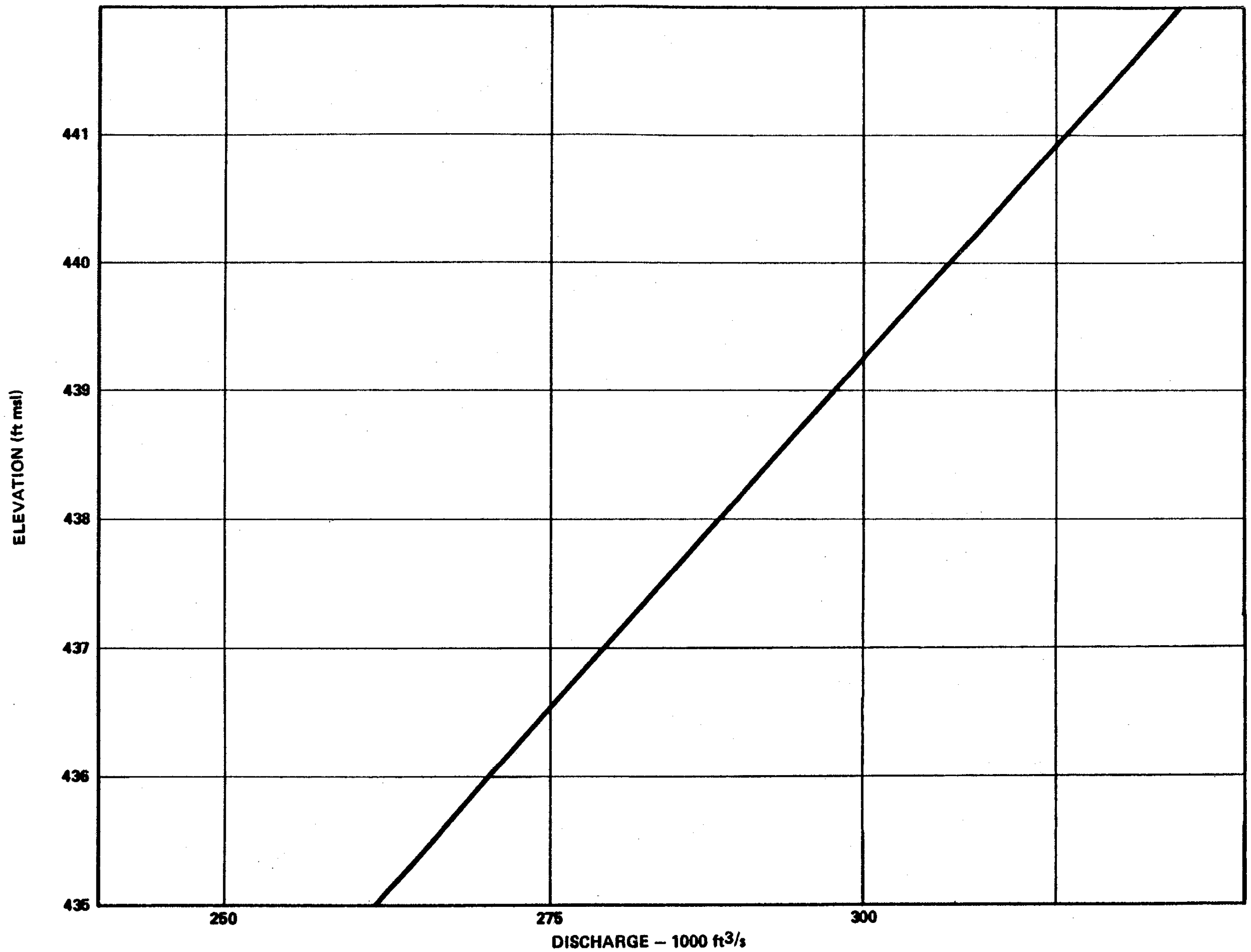
WALLACE DAM RESERVOIR CAPACITY CURVE

FIGURE 2.4-31



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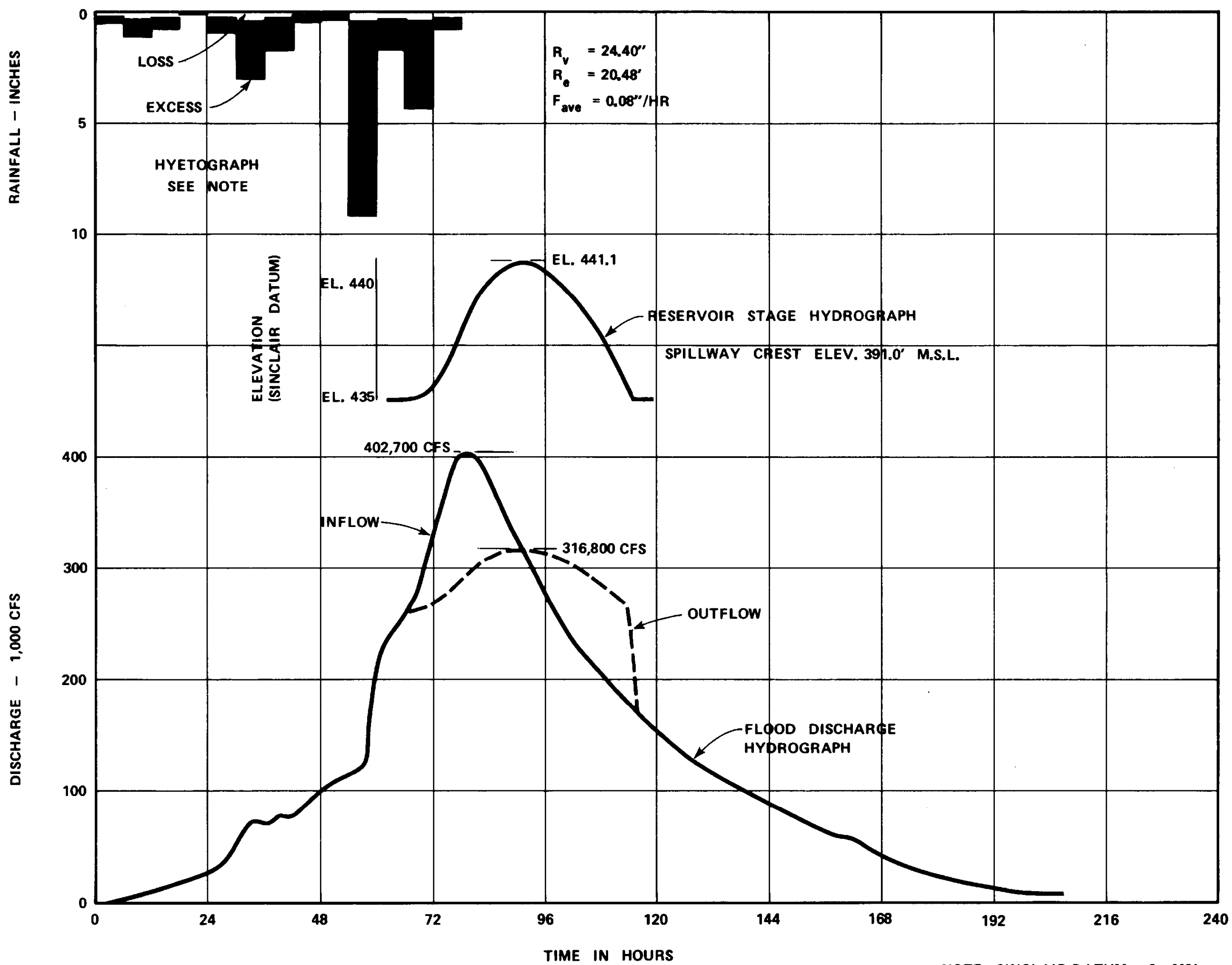
WALLACE DAM SPILLWAY RATING CURVE USED IN 1972

FIGURE 2.4-32



NOTE:
MARCH 11-16, 1929 ELBA STORM (LMV 2-20) MAXIMIZED AND TRANSPOSED TO WALLACE BASIN, STORM AXIS ROTATED 20° CLOCKWISE AND POSITIONED FOR MAXIMUM RAINFALL VOLUME ON THE BASIN WITH CENTER AT LAT. 33° 54.9', LONG. 83° 23.8'

NOTE: SINCLAIR DATUM + .6 = MSL

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WALLACE DAM SPILLWAY DESIGN FLOOD

FIGURE 2.4-33

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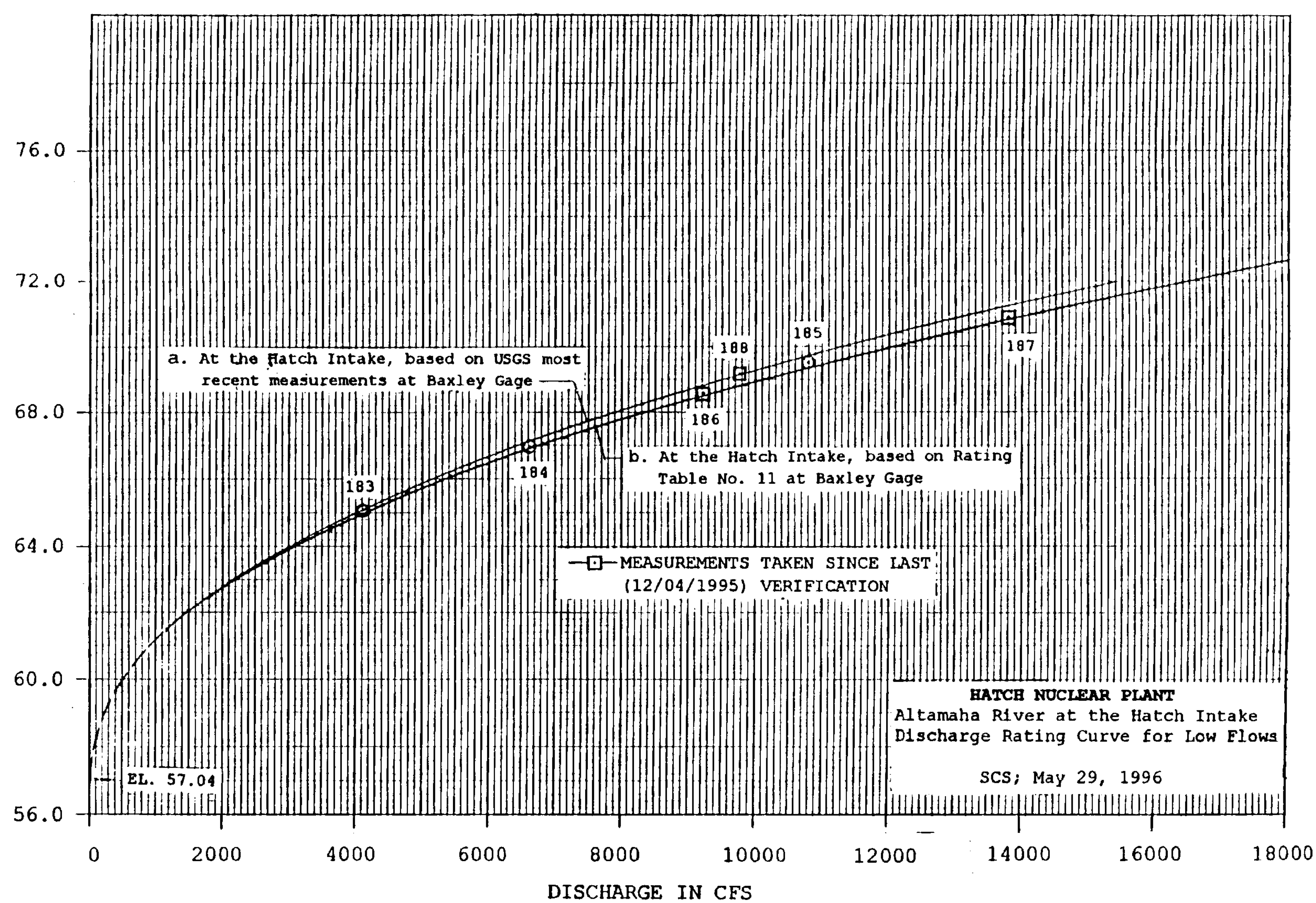

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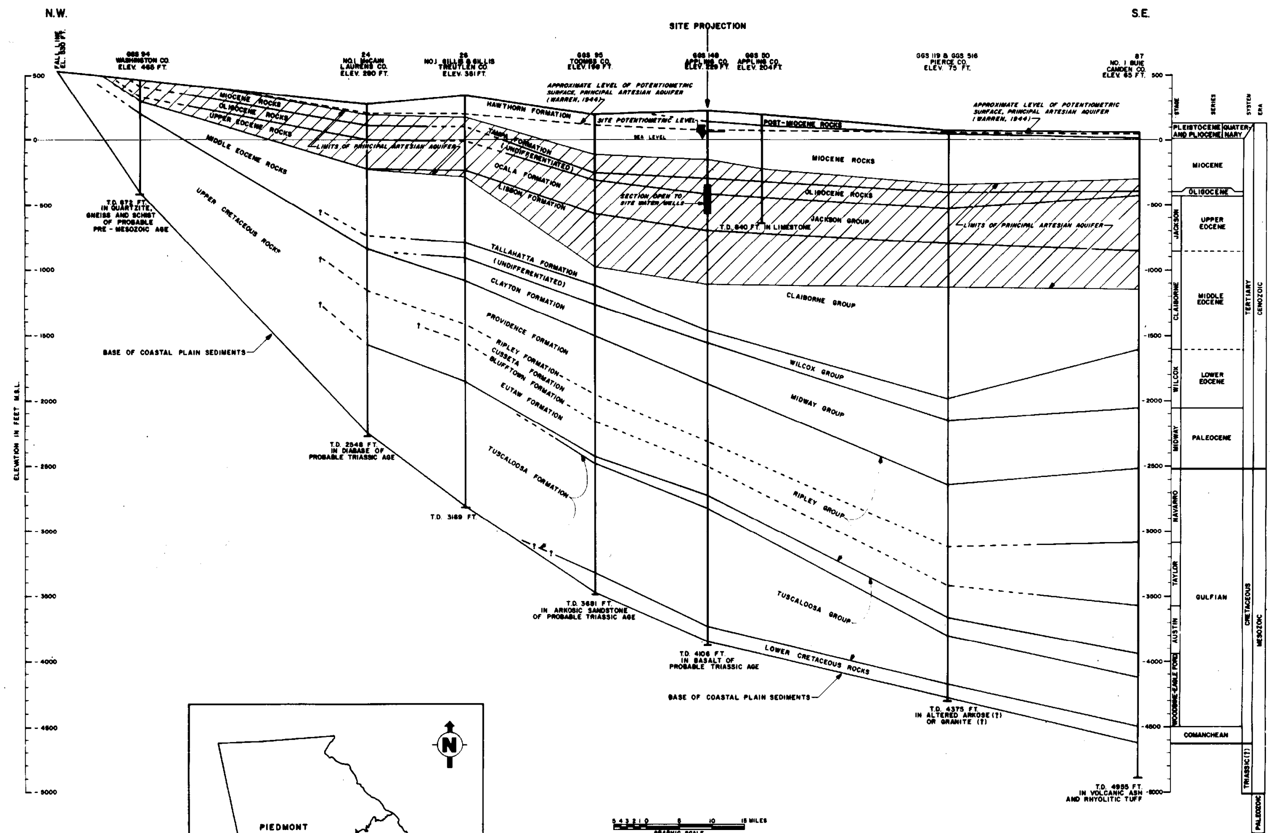
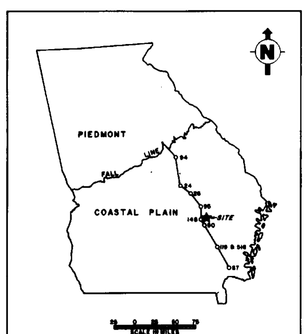
ALTAMAHA RIVER AT HNP STAGE DISCHARGE RELATION LOW FLOWS

FIGURE 2.4-34





INDEX MAP OF GEORGIA SHOWING LINE OF CROSS SECTION

NOTES

MARINE EQUIVALENT OF TUSCALOOSA FORMATION REFERRED TO AS ATKINSON FORMATION IN EXTREME SOUTH GEORGIA.

HAWTHORN FORMATION INCLUDED IN PLIOCENE ("Neogene Undifferentiated") BY GEORGIA GEOLOGICAL SURVEY (APRIL, 1976).

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MAHER, J.C., and APPLIN, E.R., 1968, CORRELATION OF SUBSURFACE MESOZOIC AND CENOZOIC ROCKS ALONG THE EASTERN GULF COAST: A.A.P.G., CROSS SECTION PUBLICATION 6, 29p., 6 PLATES.

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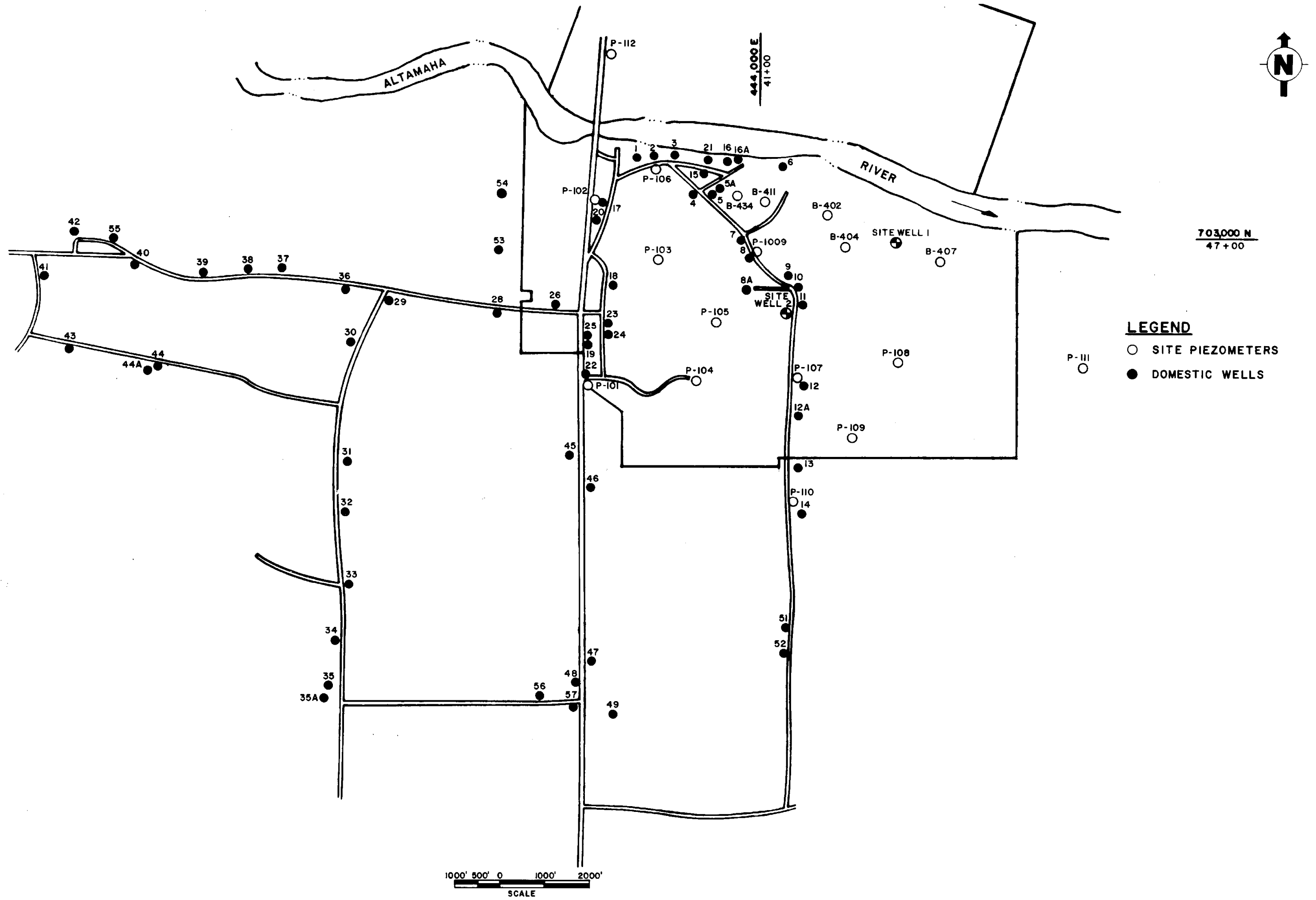
PROFILE OF PRINCIPAL ARTESIAN AQUIFER, NW-SE

FIGURE 2.4-35



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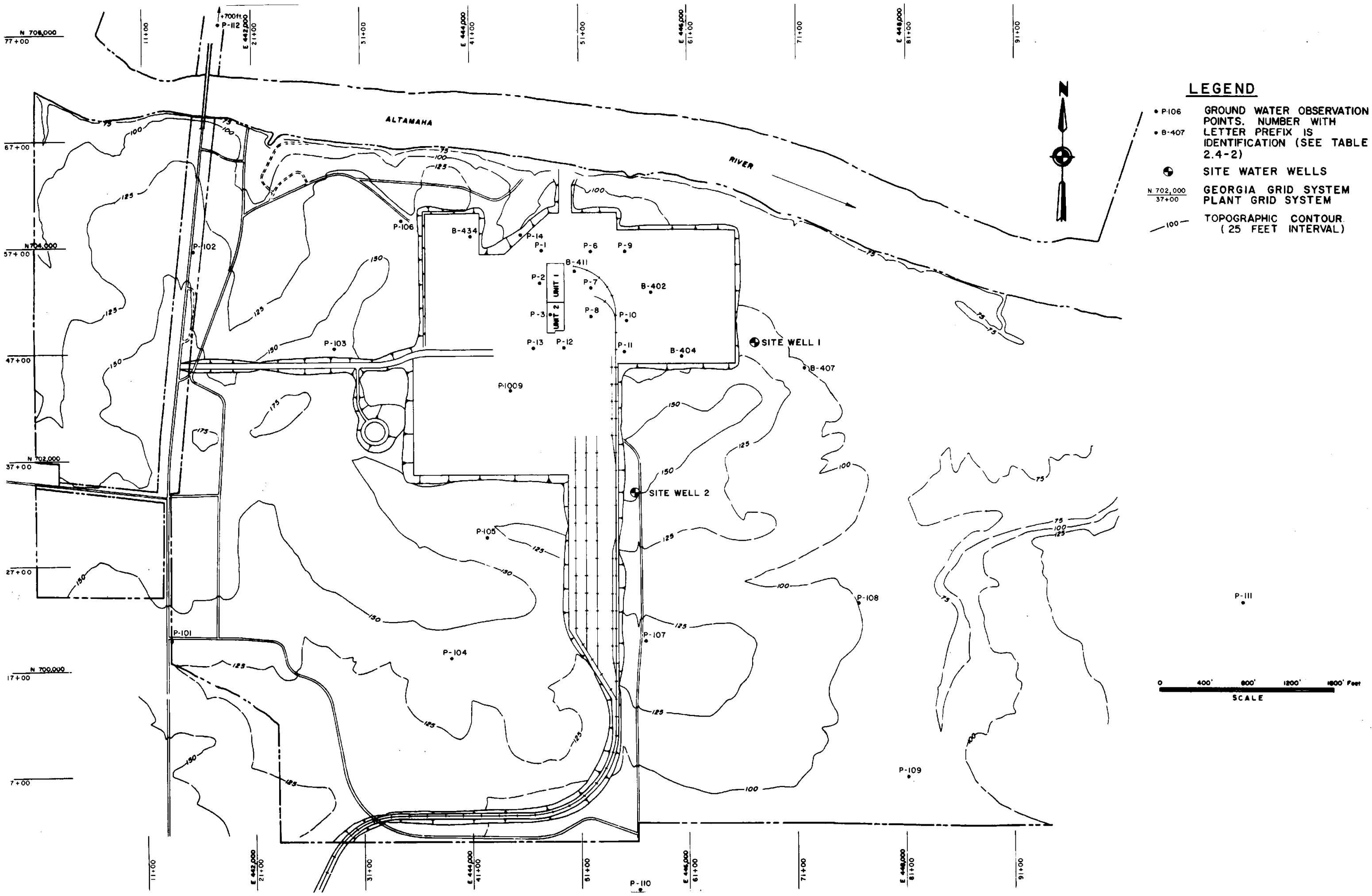
WELL LOCATIONS IN PLANT VICINITY

FIGURE 2.4-36



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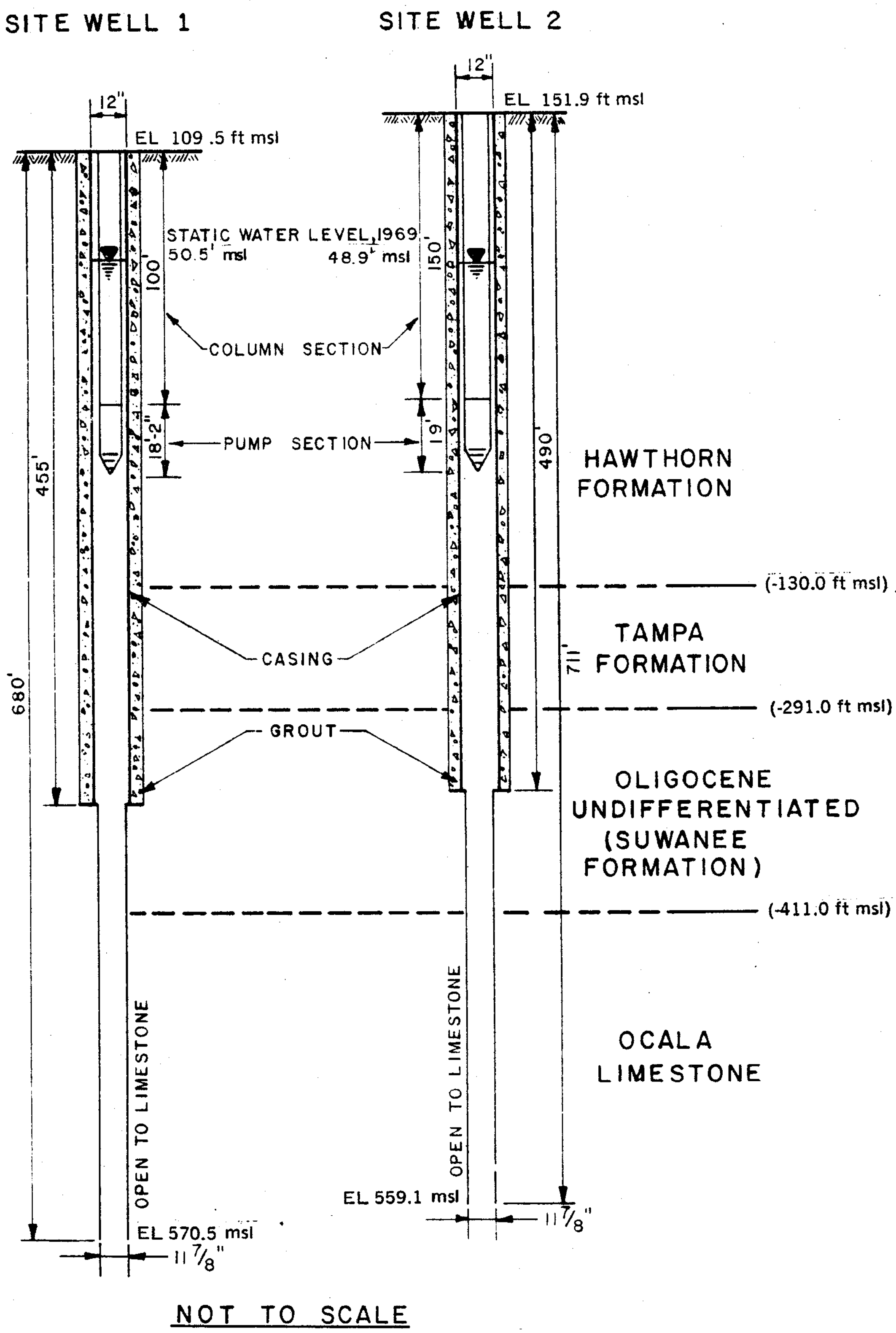
SIZE PIEZOMETER AND WELL LOCATIONS

FIGURE 2.4-37



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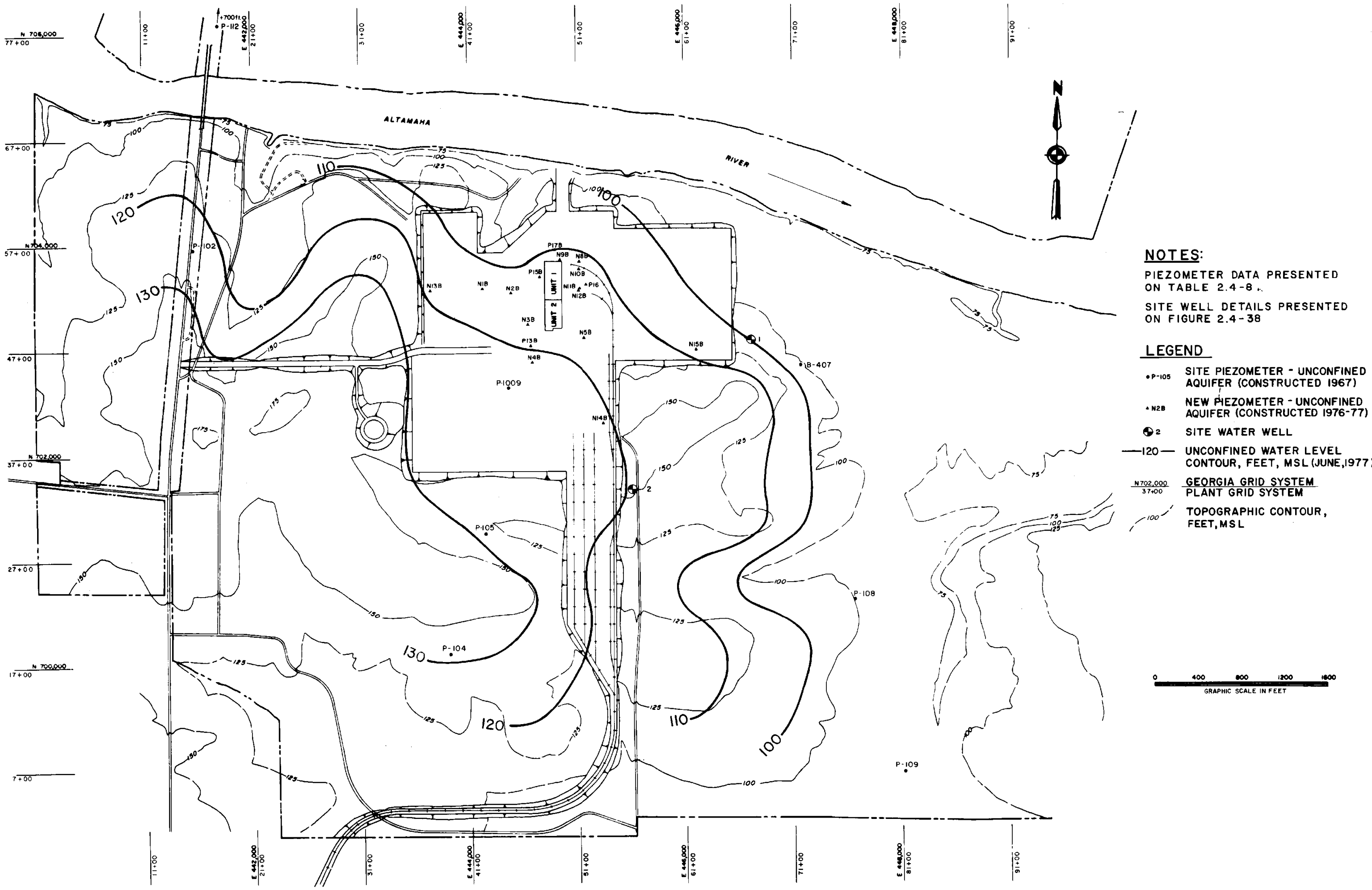
DETAIL OF SITE WELLS

FIGURE 2.4-38



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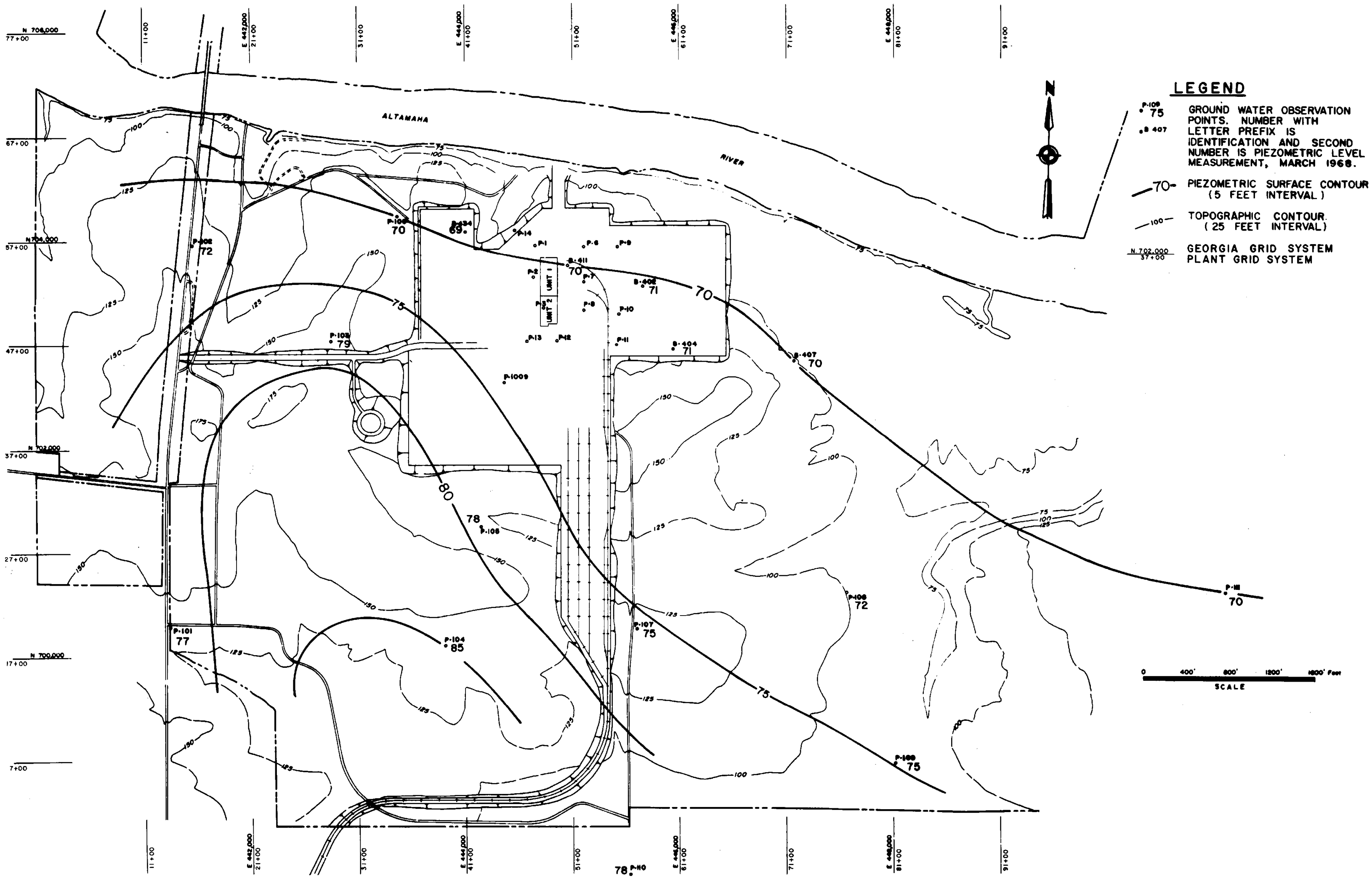
CONTOURS OF UNCONFINED WATER SURFACE

FIGURE 2.4-39



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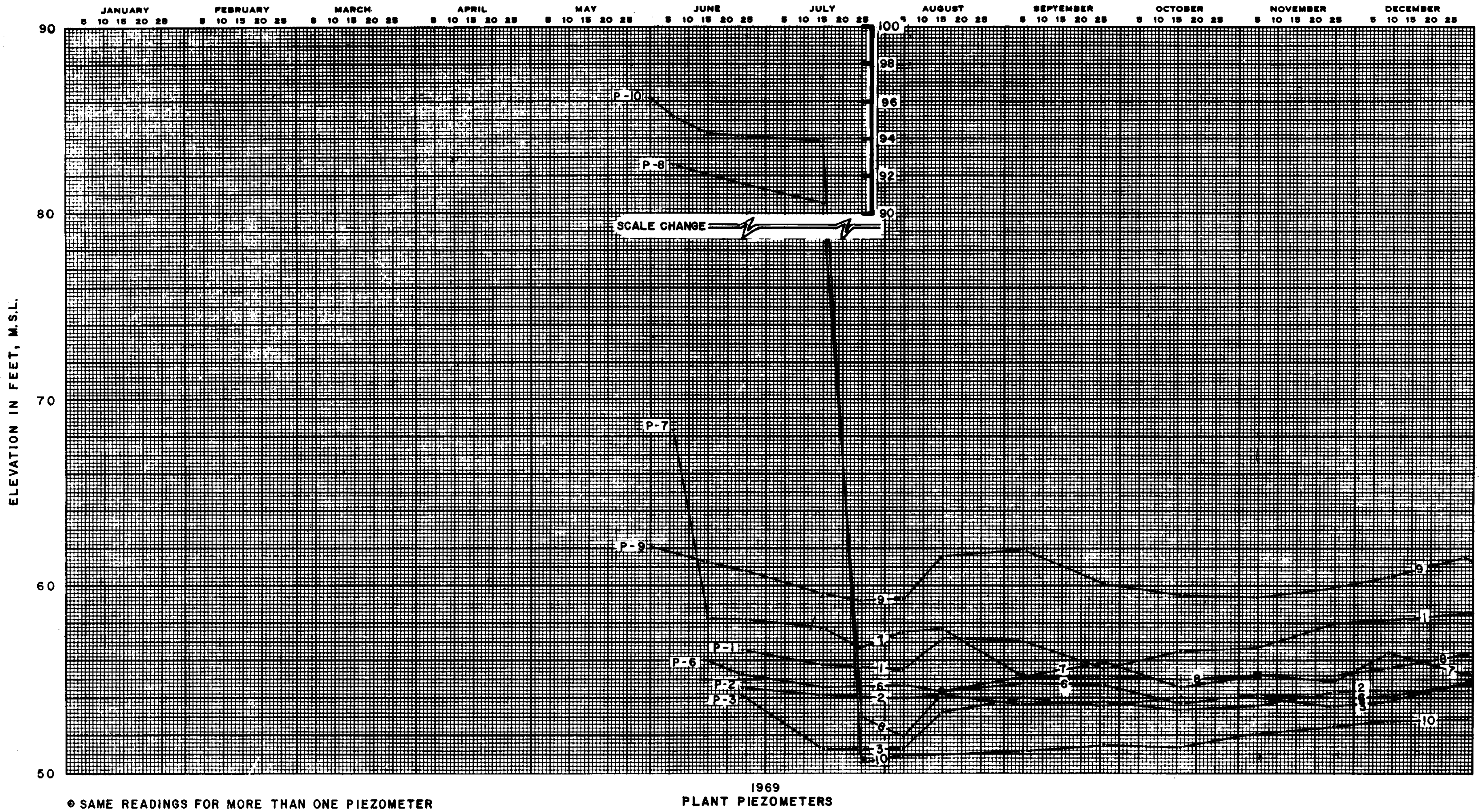
CONTOURS OF POTENTIOMETRIC SURFACE, MINOR CONFINED AQUIFER

FIGURE 2.4-40



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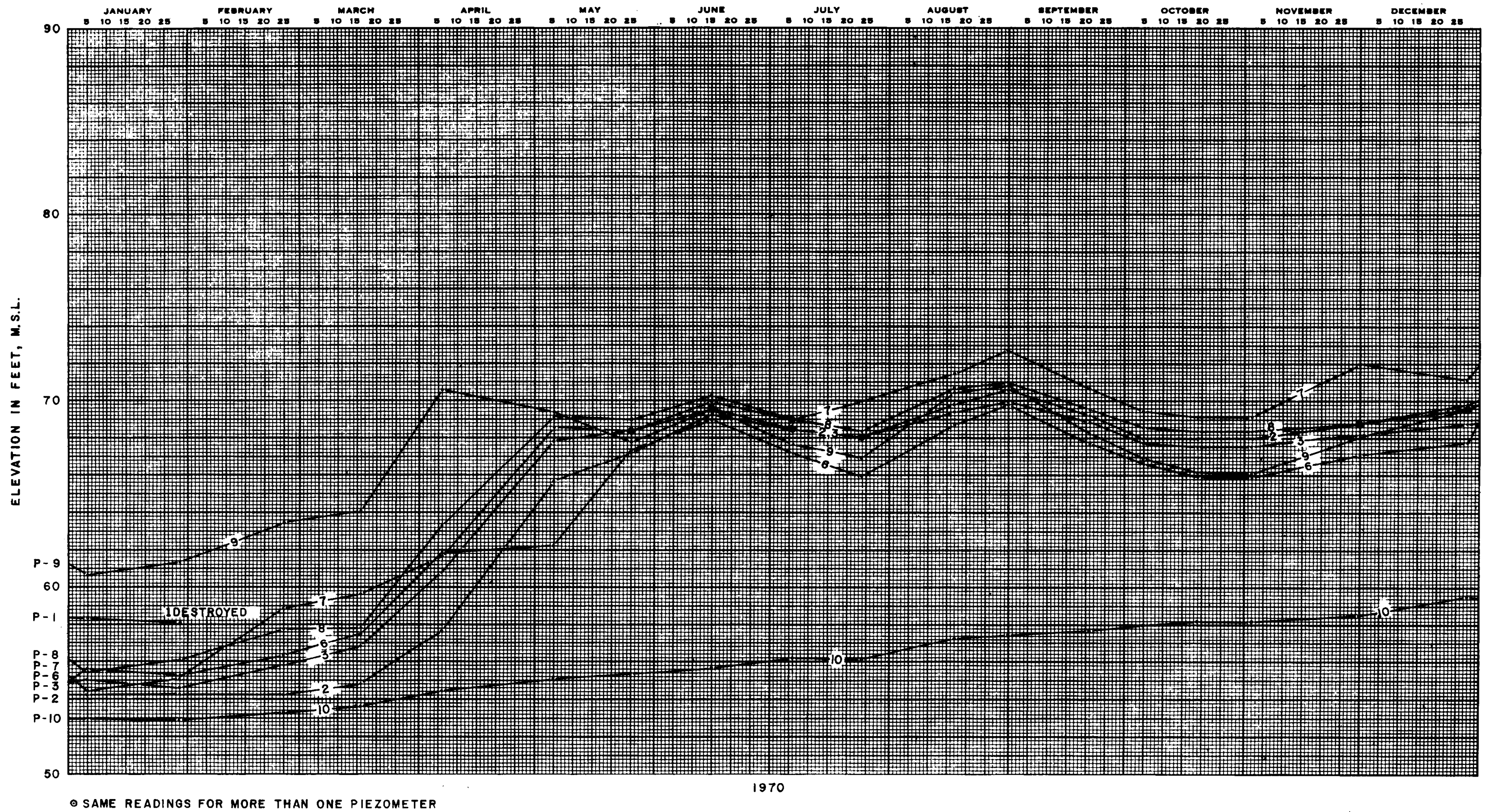
PIEZOMETER LEVELS

FIGURE 2.4-41 (SHEET 1 OF 6)



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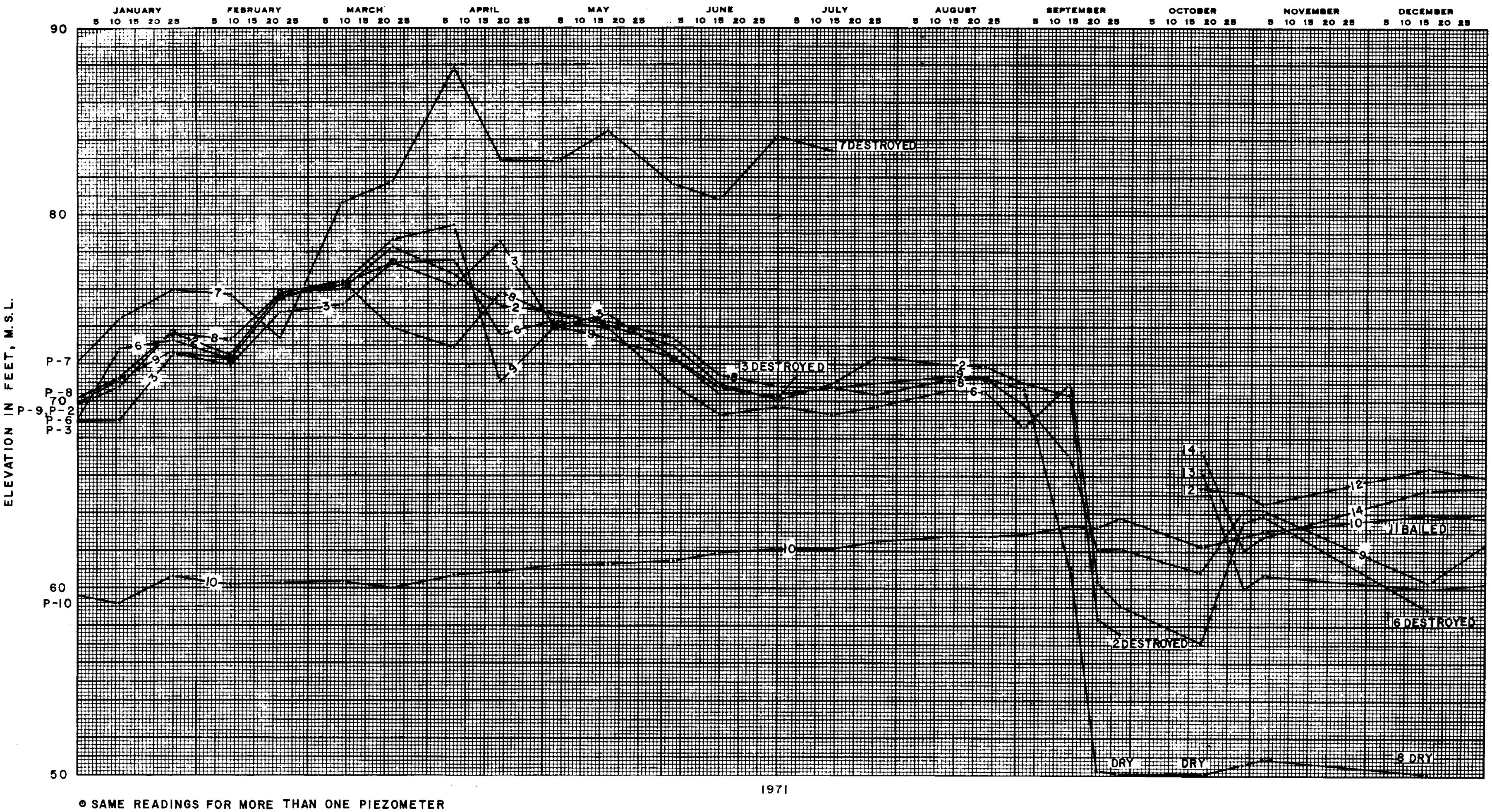
PIEZOMETER LEVELS

FIGURE 2.4-41 (SHEET 2 OF 6)



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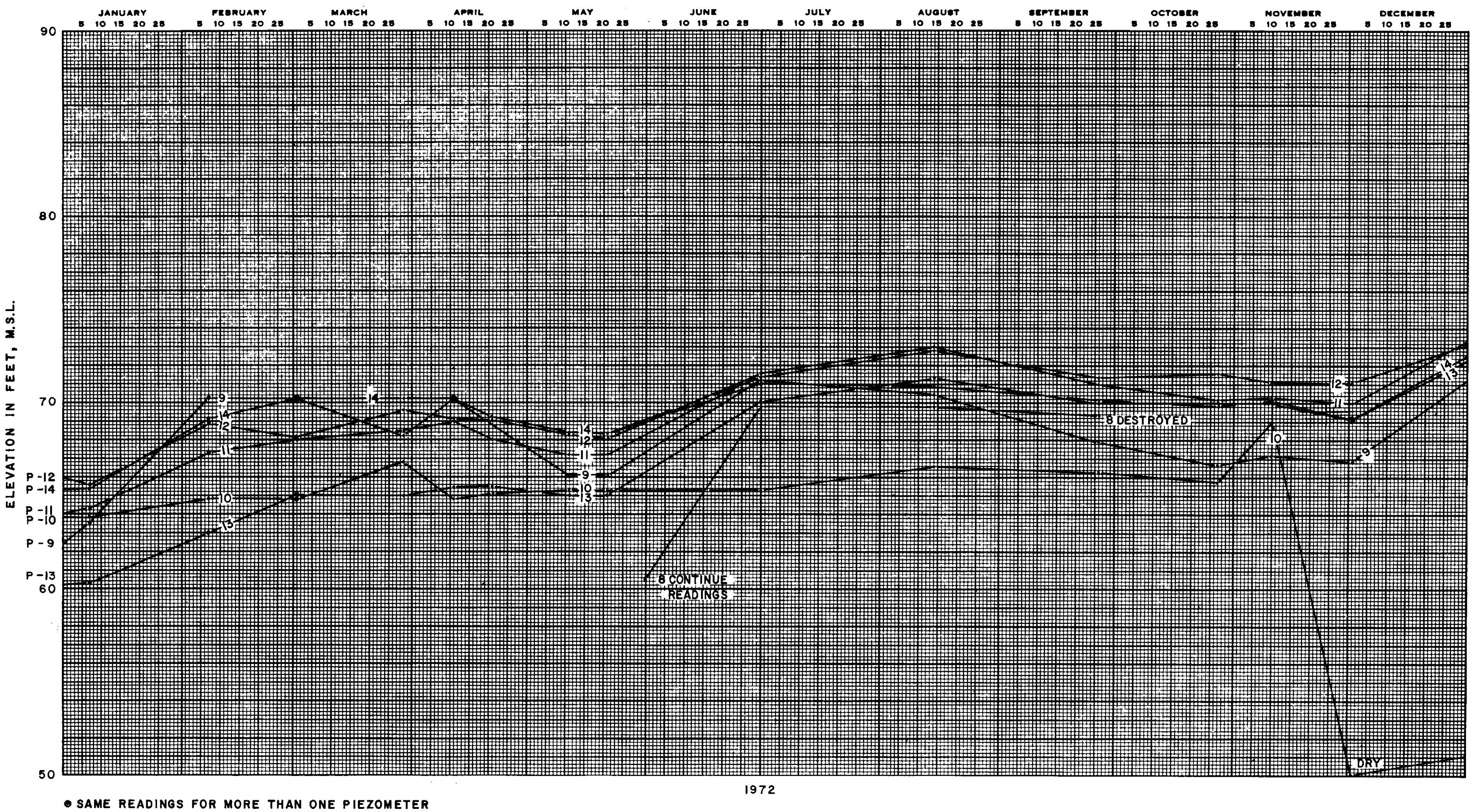
PIEZOMETER LEVELS

FIGURE 2.4-41 (SHEET 3 OF 6)



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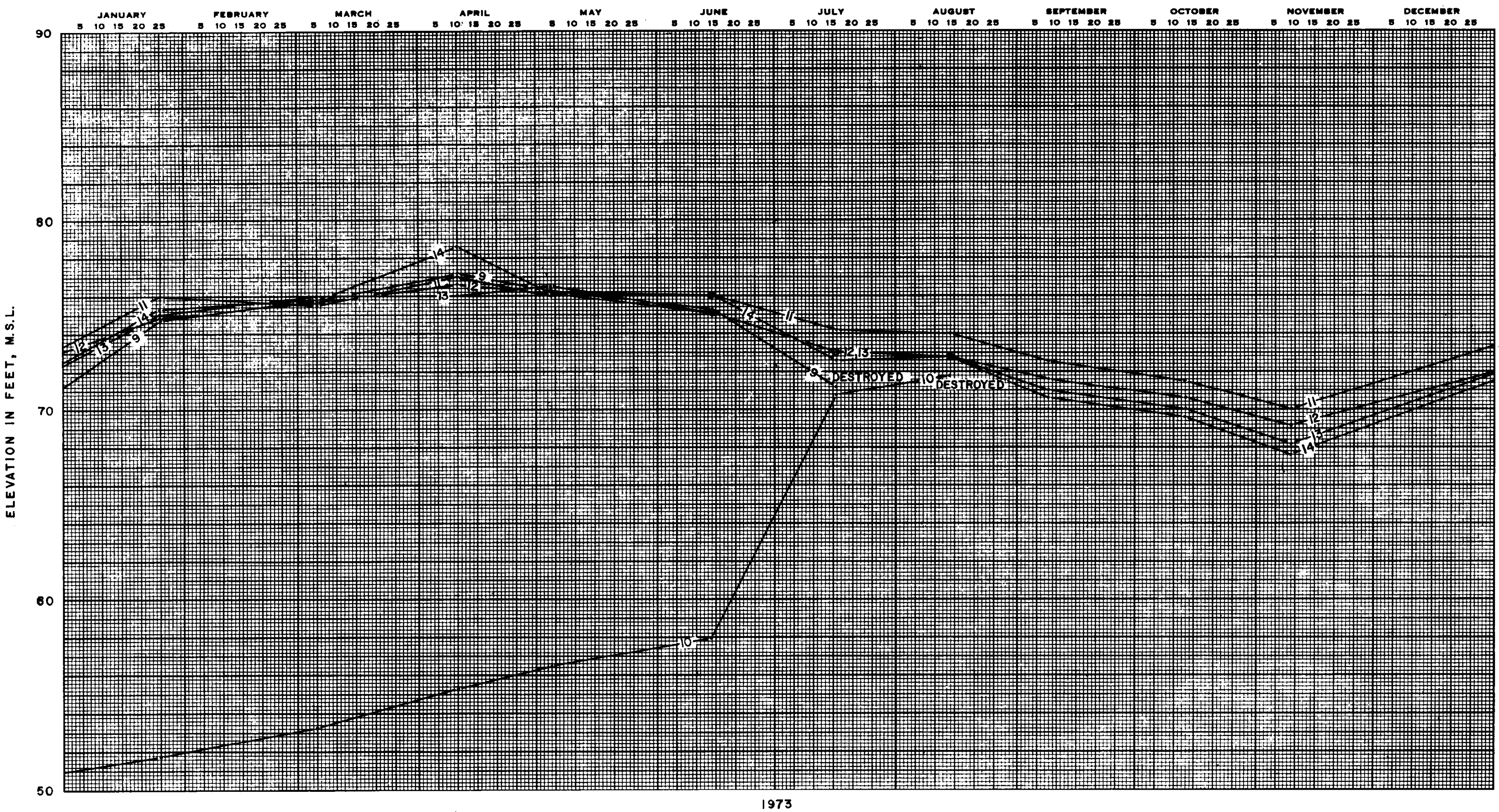
PIEZOMETER LEVELS

FIGURE 2.4-41 (SHEET 4 OF 6)



⊙ SAME READINGS FOR MORE THAN ONE PIEZOMETER

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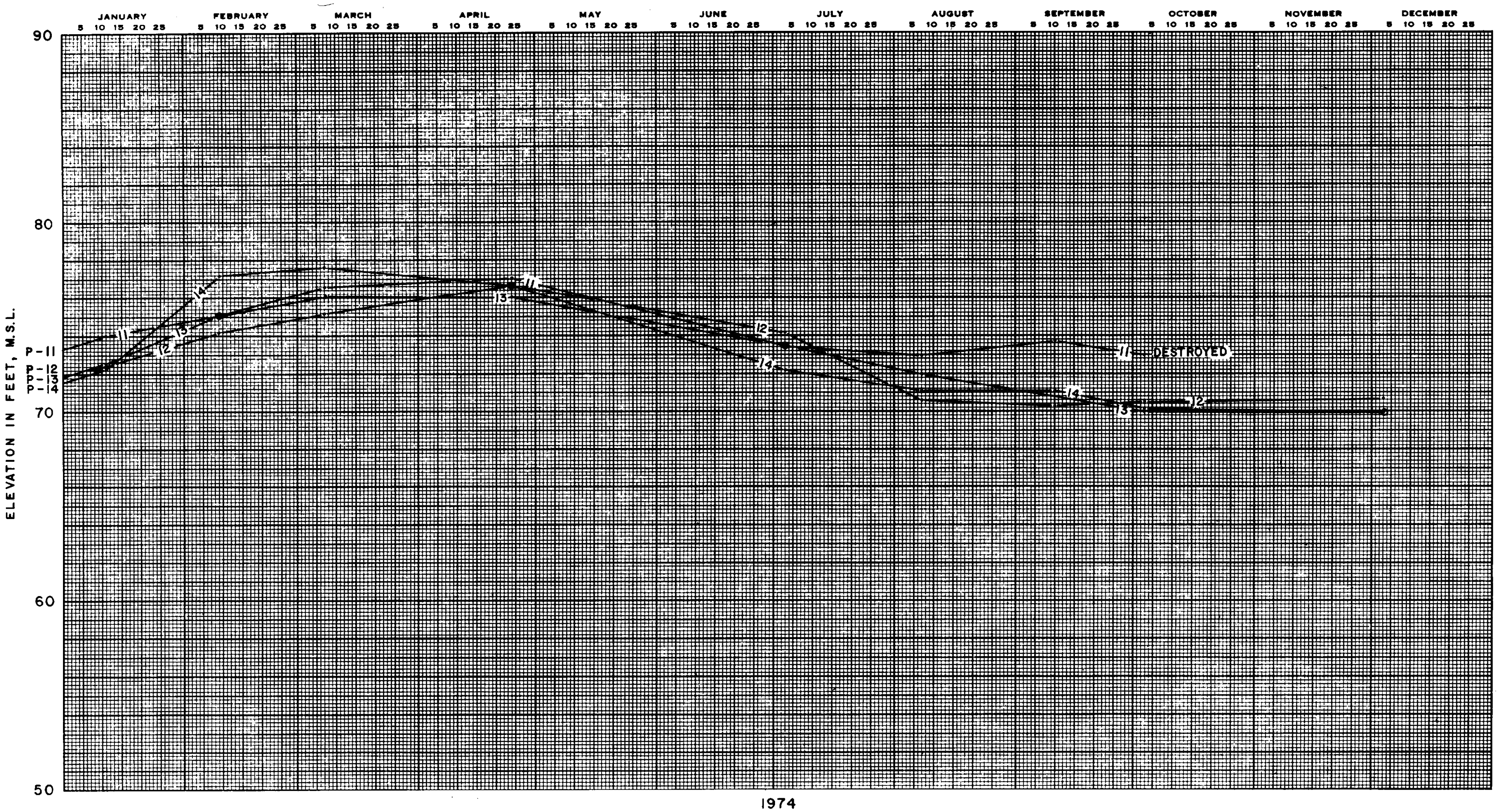
UNIT 1 AND UNIT 2

PIEZOMETER LEVELS

FIGURE 2.4-41 (SHEET 5 OF 6)



⊙ SAME READINGS FOR MORE THAN ONE PIEZOMETER

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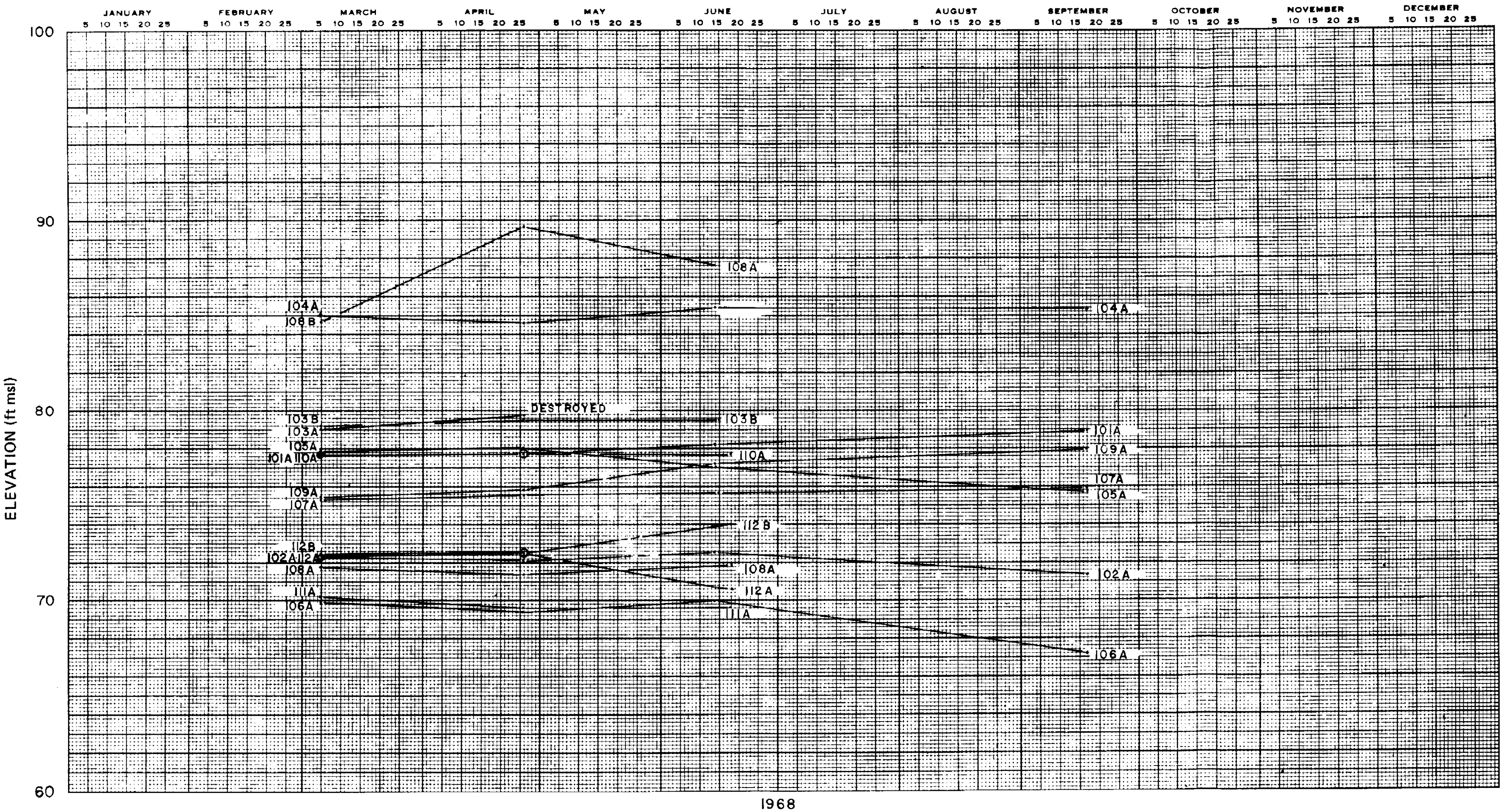
PIEZOMETER LEVELS

FIGURE 2.4-41 (SHEET 6 OF 6)



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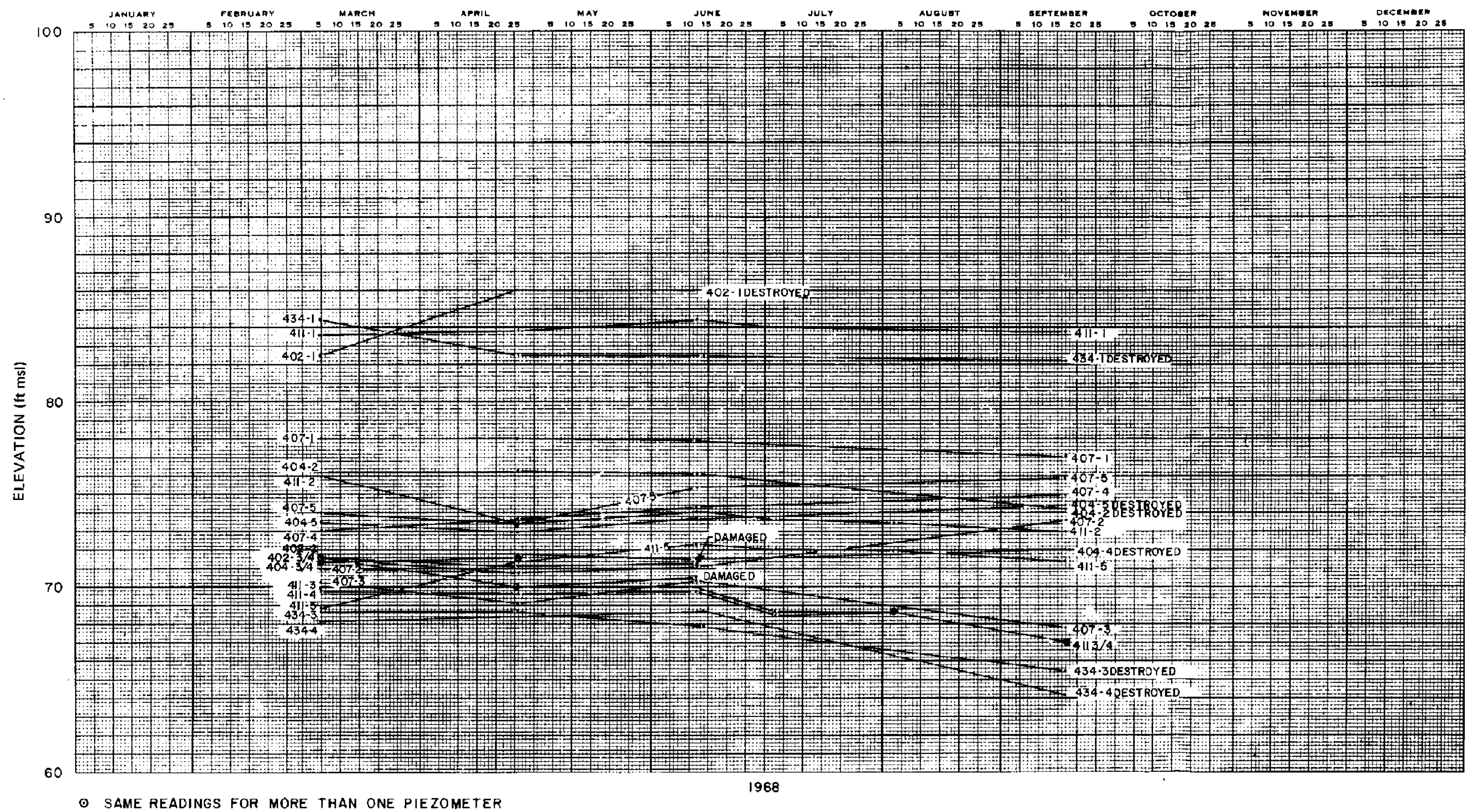
UNIT 1 AND UNIT 2

SITE PIEZOMETER LEVELS

FIGURE 2.4-42 (SHEET 1 OF 9)



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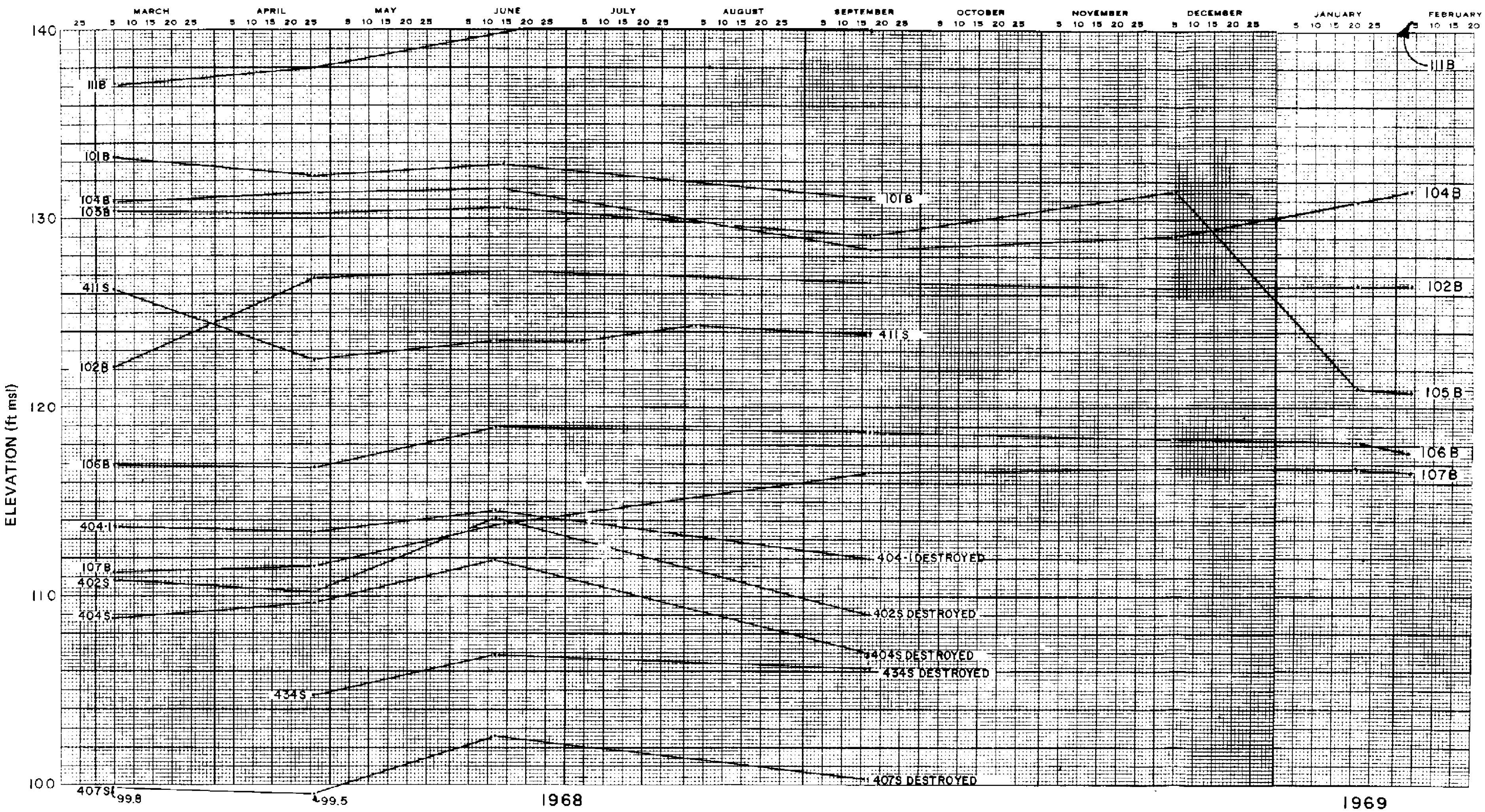
UNIT 1 AND UNIT 2

SITE PIEZOMETER LEVELS

FIGURE 2.4-42 (SHEET 2 OF 9)



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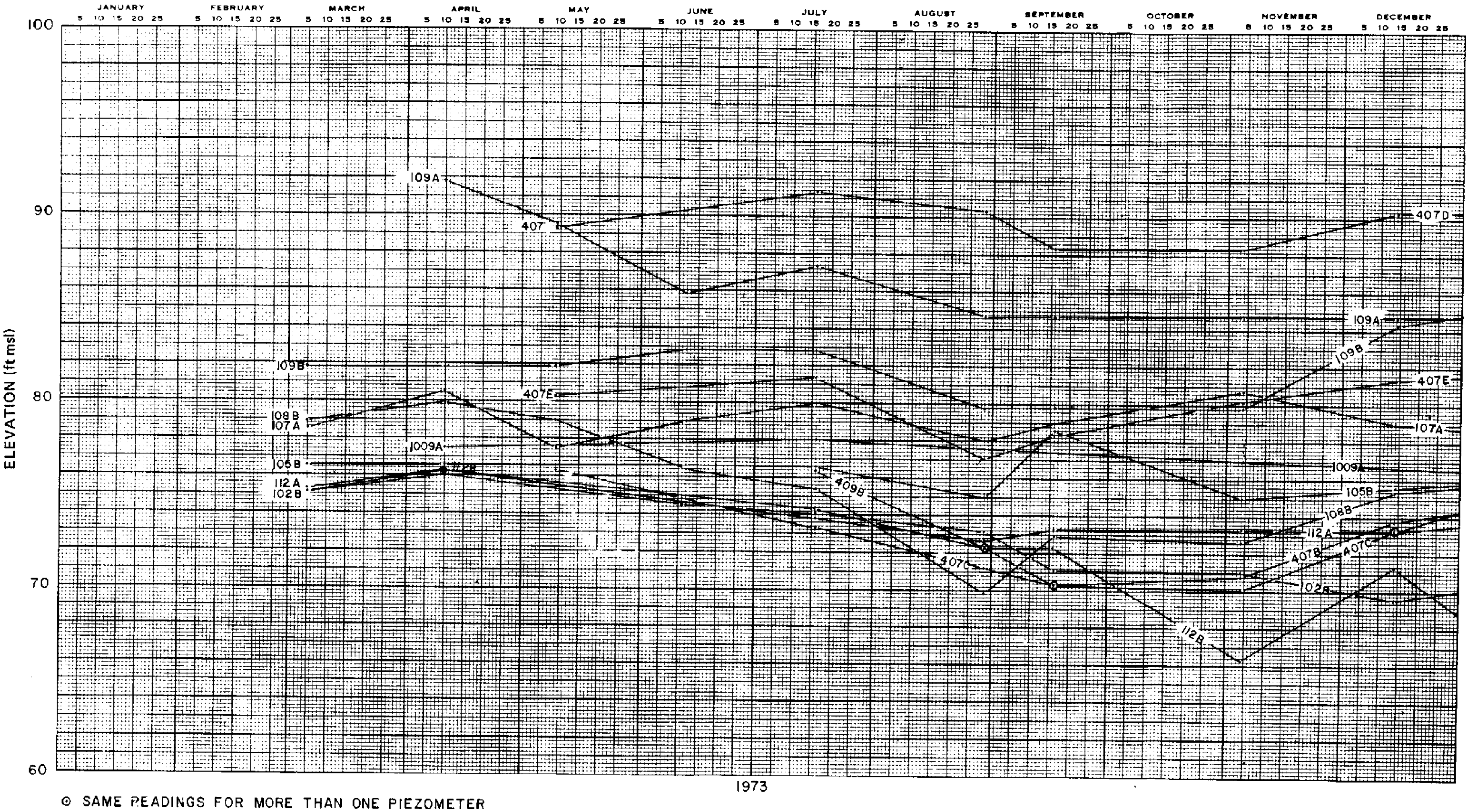
UNIT 1 AND UNIT 2

SITE PIEZOMETER LEVELS

FIGURE 2.4-42 (SHEET 3 OF 9)



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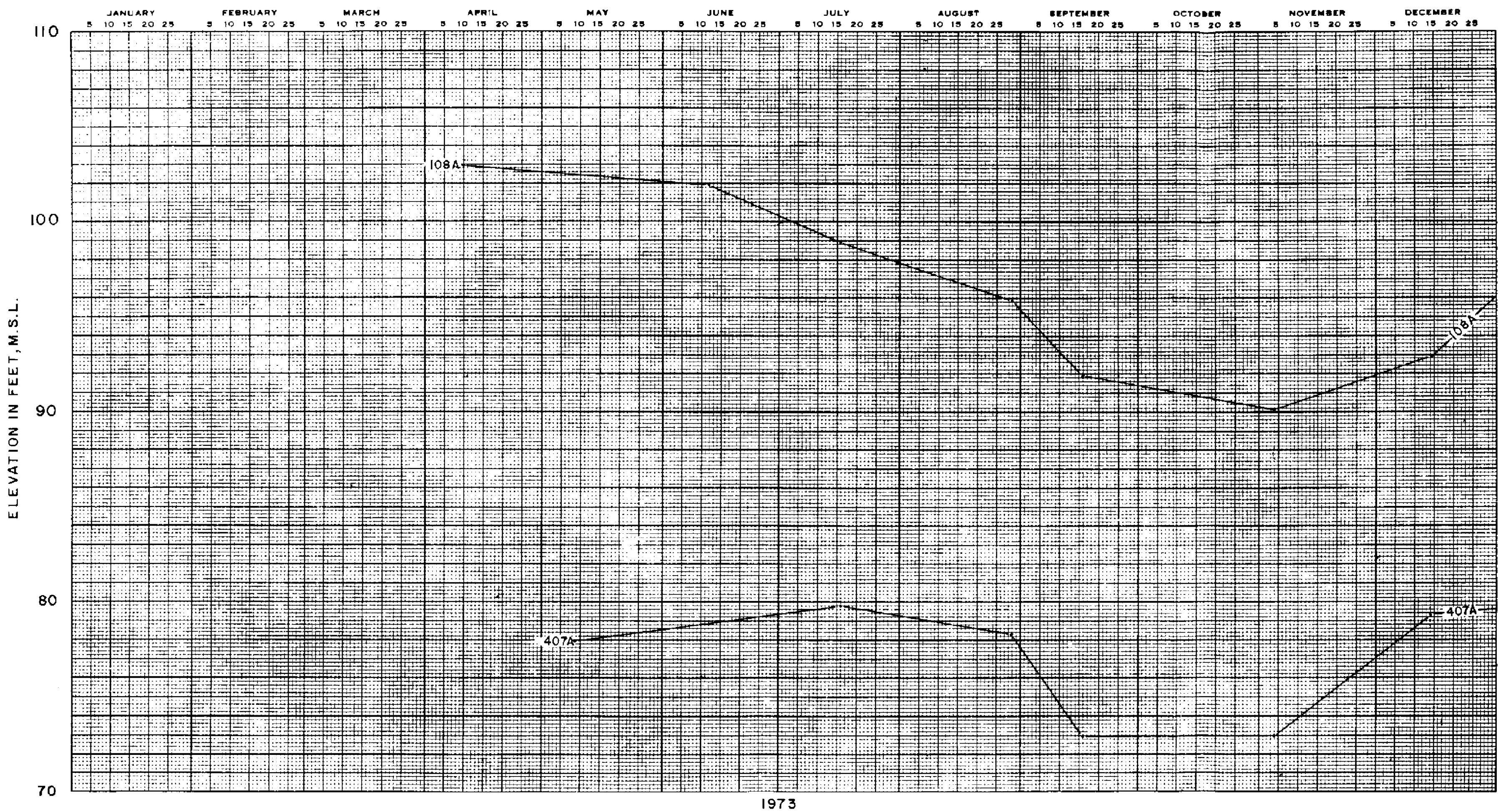
SITE PIEZOMETER LEVELS

FIGURE 2.4-42 (SHEET 4 OF 9)



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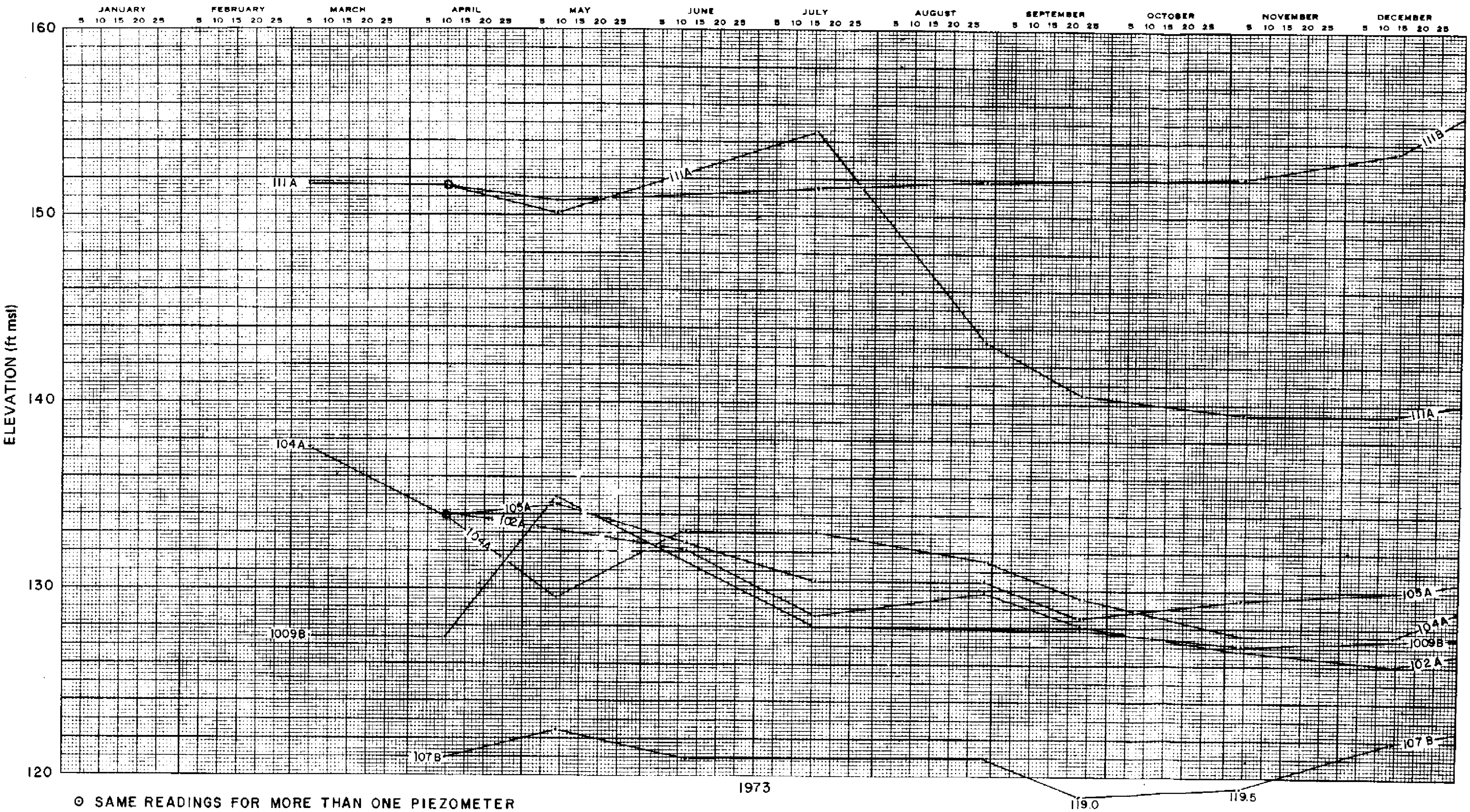
SITE PIEZOMETER LEVELS

FIGURE 2.4-42 (SHEET 5 OF 9)



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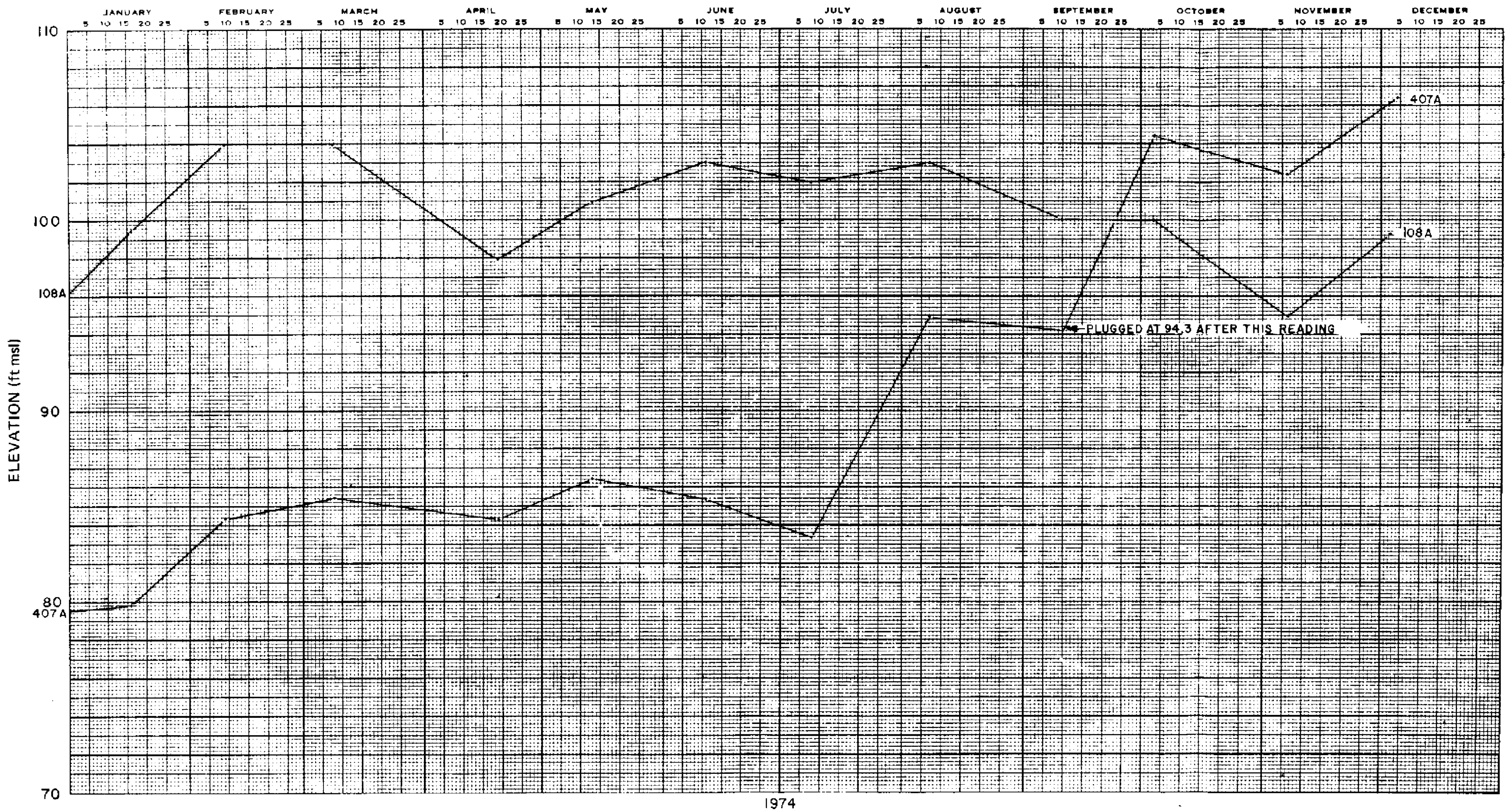
SITE PIEZOMETER LEVELS

FIGURE 2.4-42 (SHEET 6 OF 9)



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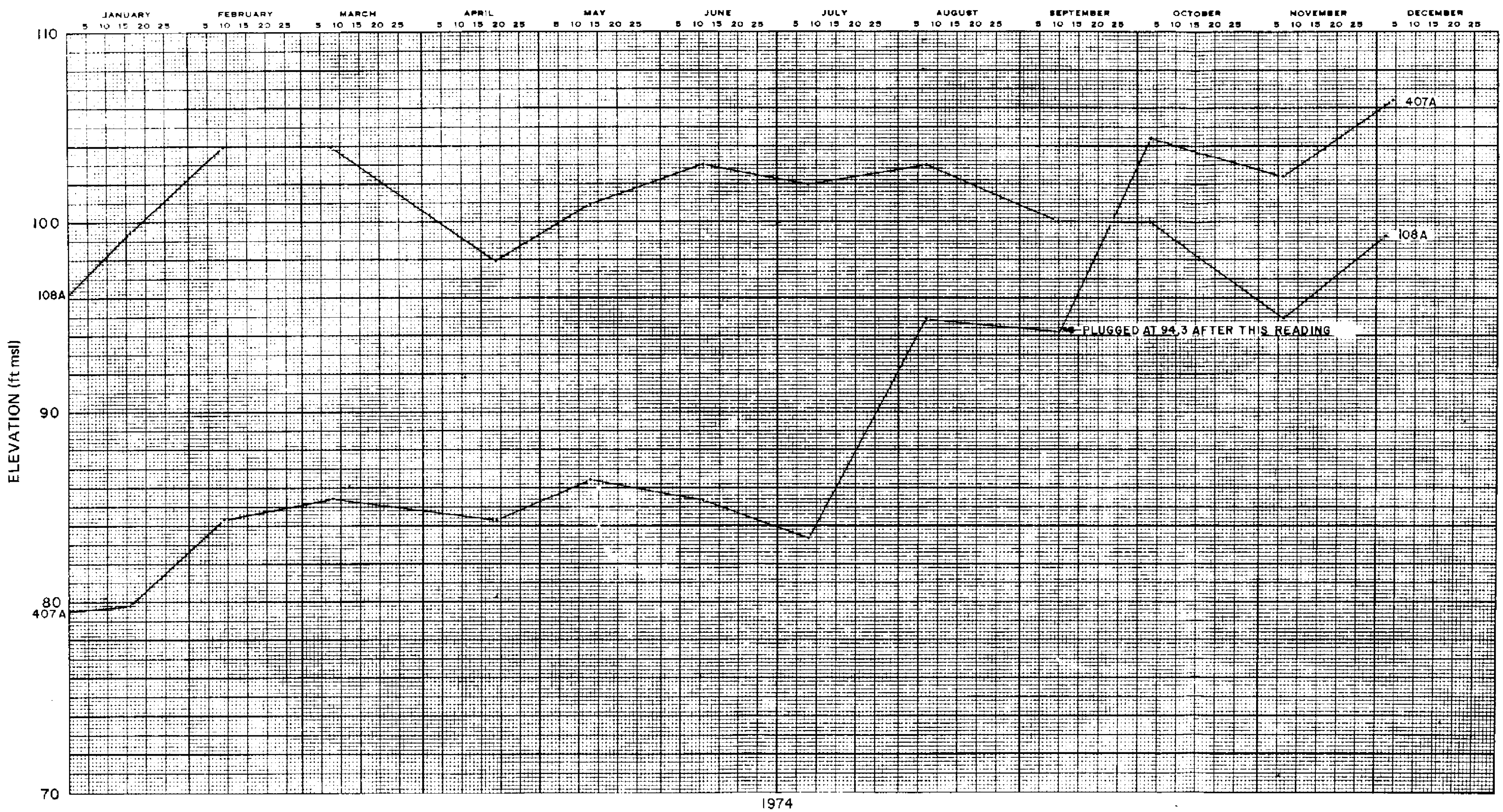
SITE PIEZOMETER LEVELS

FIGURE 2.4-42 (SHEET 7 OF 9)



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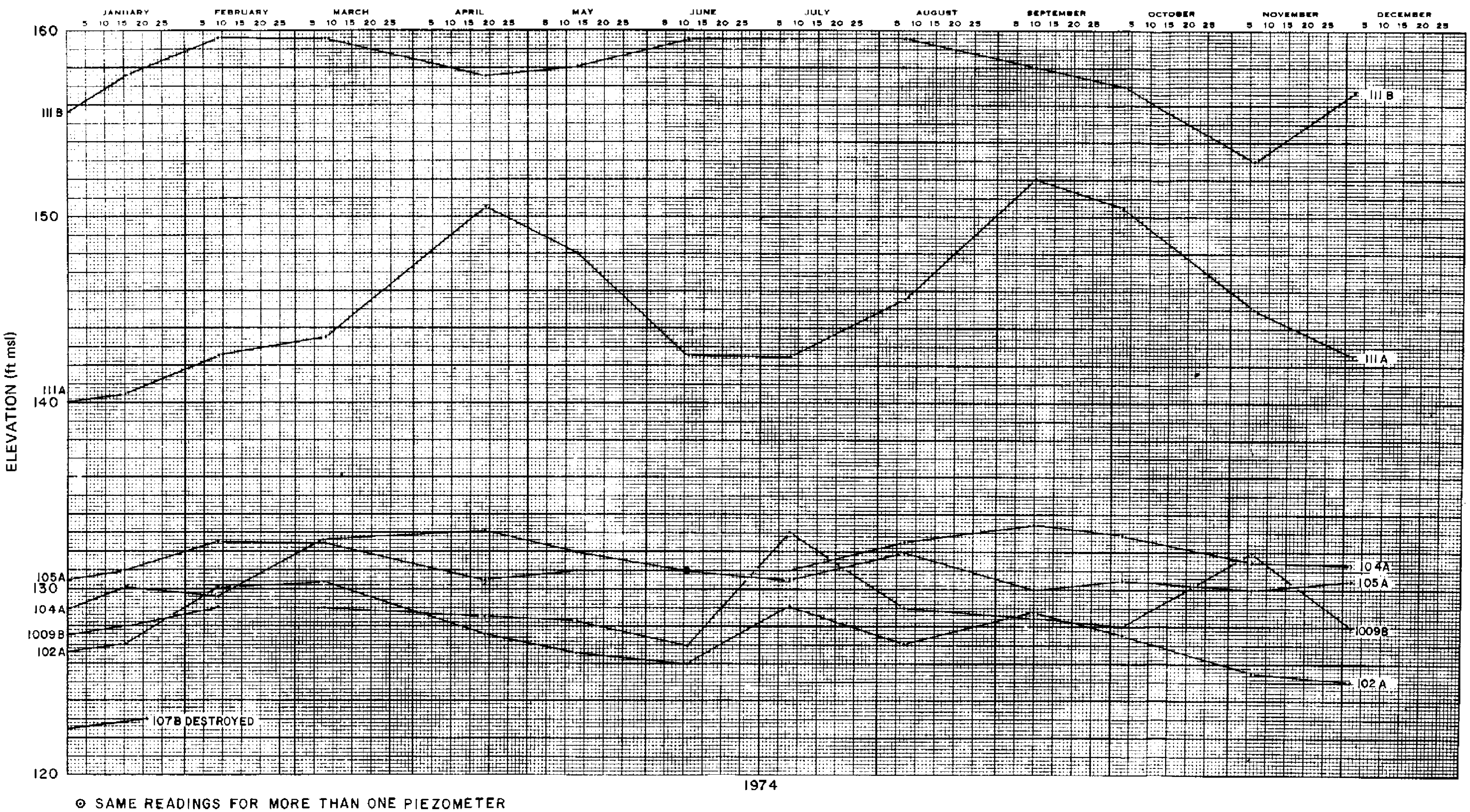
UNIT 1 AND UNIT 2

SITE PIEZOMETER LEVELS

FIGURE 2.4-42 (SHEET 8 OF 9)



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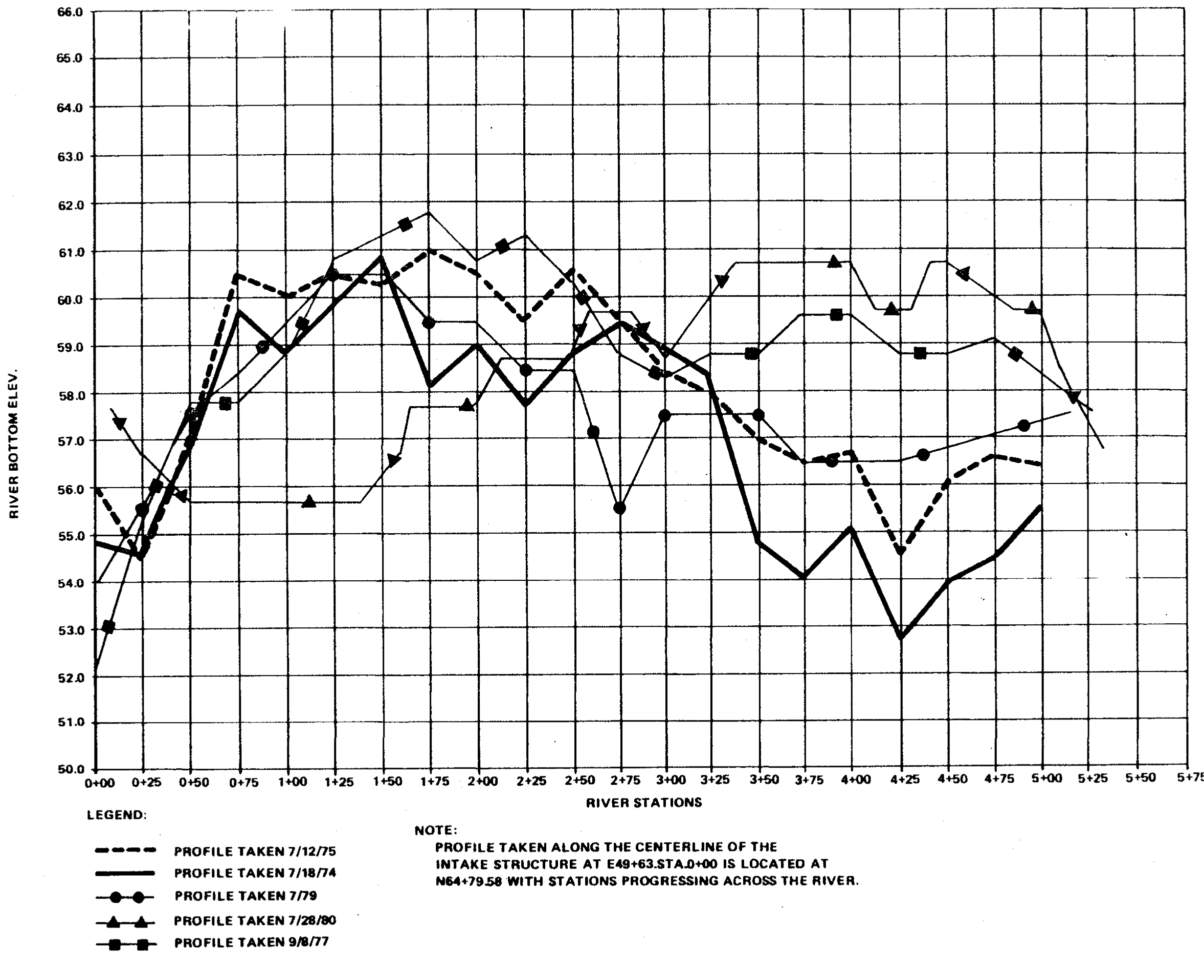
UNIT 1 AND UNIT 2

SITE PIEZOMETER LEVELS

FIGURE 2.4-42 (SHEET 9 OF 9)



NOTE:
PROFILE TAKEN ALONG THE CENTERLINE OF THE INTAKE STRUCTURE AT E49+63.STA.0+00 IS LOCATED AT N64+79.58 WITH STATIONS PROGRESSING ACROSS THE RIVER.

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ALTAMAHA RIVER PROFILE AT INTAKE

FIGURE 2.4-43

2.5 GEOLOGY AND SEISMOLOGY (HNP-1 AND HNP-2)

In compliance with the criteria provided in appendix A, Seismic and Geologic Siting Criteria for Nuclear Power Plants, of 10 CFR 100, this section provides information regarding the geologic and seismic characteristics of the site and the region surrounding the site.

The Edwin I. Hatch Nuclear Plant (HNP) is located on the south bank of the Altamaha River in Appling County, southeastern Georgia. This area is in the Coastal Terraces subprovince of the Atlantic Coastal Plain physiographic province. The site is underlain by ~ 4000 ft of relatively unconsolidated Mesozoic and Cenozoic sands, gravels, clays, marls, claystones, sandstones, and limestones. These strata, overlying basaltic basement rock of pre-Cretaceous age, dip and thicken seaward. No structural features affect the material underlying the site. No major or minor fault zones are near the site, nor were any local faults discovered during field mapping, exploratory drilling, and construction.

The site is within a region of infrequent seismic activity. No earthquakes within 200 miles of the site, including those in the Charleston area ~ 150 miles from the site, produced Modified Mercalli intensities at the site greater than VI. Historically, reported earthquakes occurring in other areas have not produced intensities greater than VI at the site. The design basis earthquake (DBE) is conservatively selected as Modified Mercalli Intensity VII.

The Hawthorn Formation of Miocene to Pliocene(?) age is the foundation-bearing stratum for the major plant structures. It consists primarily of sand, clay, and cemented sand and clay layers. There are no zones of deformation, alteration, or weakness within the Hawthorn Formation.

The site is underlain by both confined and unconfined aquifers. Local and regional ground water conditions have not been altered by construction and operation of the plant.

The scope of site investigations included the geologic, geohydrologic, and seismologic conditions of the area and evaluation of these conditions regarding their effects on the design, construction, and operation of a nuclear plant at the site. The purpose of the investigations was to determine the following:

- *The characteristics of the foundation materials, especially in regard to their suitability for supporting plant structures.*
- *The extent of geologic structures affecting the site.*
- *The seismicity of the area.*
- *The depth and configuration of the ground water table.*
- *The characteristics of soil and rock with respect to their effects on the migration of radioactive solutions, should such solutions come in contact with them.*

The purpose was accomplished by conducting programs of geological and geophysical field exploration, foundation analysis and evaluation, installation of a ground water monitoring system, and review of pertinent literature.

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

The following sections and subsections contain the results and conclusions of the regional and site geologic and seismic investigations. Information on regional and local ground water conditions is included in subsection 2.4.13, Ground Water, and is only summarized in the following geology subsections. The characteristics of soils with respect to the support of major plant structures are discussed in detail in supplement 2A and cross-referenced in this section.

The information presented in the following sections was obtained from the latest published sources and reflects presently accepted geologic interpretations. The Georgia Geological Survey prepared a new state geologic map, published in June 1975, which designates new terminology for some formations found at the Hatch site. These changes in nomenclature do not affect lithologic or structural relationships in the site area, nor do they have any significance in the selection of the DBE or in the description of site ground water conditions.

2.5.1.1 Regional Geology

2.5.1.1.1 Regional Physiography (HNP-1 and HNP-2)

The Edwin I. Hatch Nuclear Plant is on the south bank of the Altamaha River ~ 10 miles north of Baxley, Georgia, and ~ 73 miles northwest of Brunswick, Georgia. The site is within the Atlantic Coastal Plain physiographic province.[1] Within 200 miles of the site are parts of three other major physiographic provinces: the Blue Ridge, Piedmont, and East Gulf Coastal Plain. The first two provinces are associated with the Appalachian Mountain System. They are separated from the Coastal Plain province by the Fall Line, a break in slope represented by rapids in the major streams about 80 miles northwest of the site.[2] The regional physiography is shown in figure 2.5-1.

The Blue Ridge province trends northeastward across the northwest corner of Georgia. Elevations in the province range from a high of over 6000 ft in North Carolina to less than 2000 ft in Alabama where the Blue Ridge rocks dip below Coastal Plain sediments at the Fall Line. In Georgia, local relief approaches 200 ft, with rounded summits nearly 4000 ft above sea level. The summits and valleys in the province are distinctly nonlinear, reflecting both the lack of structural control and the erosive effects of the well-developed drainage system that flows generally transverse to the northeastern trend of the Appalachians.[3] The boundary with the Piedmont province to the southeast is the Brevard Fault Zone and is marked by a somewhat obscured, northeast-trending topographic lineation in Georgia.[4] The nearest approach of the Blue Ridge province to the site is ~ 185 miles.

The Piedmont is a rolling, southeast-sloping plain between the Blue Ridge on the northwest and the Coastal Plain provinces on the south and southeast. The plain's surface is broken by numerous hills and ridges that rise as monadnocks up to 1000 ft high, although local relief is generally < 200 ft. The seaward edge of the Piedmont province is marked by the Fall Line, where Piedmont rocks dip below the mostly unconsolidated material of the Coastal Plain provinces.[3] The Piedmont is ~ 80 miles northwest of the site.

The Atlantic and East Gulf Coastal Plain provinces extend from the Fall Line to the Atlantic Ocean and Gulf of Mexico, respectively. The Atlantic Coastal Plain is characterized by nearly flat-lying terrace

surfaces, underlain by limestone or unconsolidated sand and clay, that occur as narrow belts parallel to the coast. By contrast, the wider subprovince belts of the East Gulf Coastal Plain consist of numerous ridges or cuestas separated by low valleys or inner lowlands. Rocks underlying the East Gulf subprovinces vary in their resistance to erosion and range from sandstone, shale, and limestone to softer clay, sand, and marl. The transition zone between the two major provinces is in central Georgia, about 60 miles west of the site, between the eastward flowing Ocmulgee River and the southward flowing Flint River.[(2)]

The Fall Line Hills, Red Hills, Dougherty Plain, Tifton Upland, and Coastal Terraces subprovinces are common to both Coastal Plain provinces. The Fall Line Hills extend from the Tennessee River on the west to the central Carolinas on the east where they are known as the Sand Hills. The maturely dissected topography, with relief approaching 350 ft, is developed on predominantly sand-bearing formations of Cretaceous age. The Red Hills, with a similar topography, lie immediately seaward of the Fall Line Hills in Georgia, and are developed on Eocene rocks. In Alabama, the Red Hills become the Southern Red Hills, underlain by both Eocene and Oligocene rocks weathered bright red. The Dougherty Plain extends from southeastern Alabama and the Florida panhandle into central Georgia. This wide, largely flat plain contains shallow solution depressions developed in Eocene limestone. Low scarps separate the Dougherty Plain from the higher, landward Red Hills subprovince and the higher, seaward Tifton Upland. The Tifton Upland is a region of gently rolling hills with broad rounded summits underlain by Miocene-age sand. Relief is generally < 50 ft, but may approach several hundred feet near the wide, flat-bottomed river valleys. The Red Hills, Dougherty Plain, and Tifton Upland merge with the Upper Coastal Plain subprovince of South Carolina. The Coastal Terraces subprovince is seaward from and lower in elevation than the adjacent Tifton Upland. This subprovince, with maximum relief of ~ 100 ft near major stream valleys, can be subdivided into at least seven terraces, whose nearly flat surfaces constitute late Miocene to late Pleistocene sea floors. The landward limit of each terrace is marked by a low, often obscure scarp. The older, higher terraces merge with the Middle Coastal Plain of South Carolina, while the younger, lower terraces merge with the Lower Coastal Plain of South Carolina and the late Pleistocene terraces of Florida.[(2)]

The physiography of that part of Florida within 200 miles of the site has been determined by Pleistocene terracing, solution of underlying rocks, and stream erosion. The subprovinces affected by these factors are the late Pleistocene Terraces, East Florida Flatwoods, Lake Region, Lime-Sink Region, and Flatwoods and Hammock Lands.[(2)]

The late Pleistocene terraces, with highest surfaces of el 40 to 45 ft, 65 to 70 ft, and 95 to 100 ft, extend from the South Carolina Lower Coastal Plain into Florida without change of character. The surfaces are nearly flat, with a slight seaward dip, and are often swampy. This terrace belt merges with the East Florida Flatwoods Terraces near Jacksonville, Florida. The Lake Region to the west is higher in elevation (~ 100 ft above sea level) and is characterized by large, shallow lake basins enclosed primarily by sand. By contrast, the Lime-Sink Region further west has numerous small-solution depressions and lakes and as much as 50 ft of relief. The Flatwoods and Hammock Lands extend along the entire west coast of Florida to Mobile Bay. Between Tampa Bay and the Apalachicola River, low relief is typical, with old sinkholes discontinuously covered by more recent thin sand deposits.[(2)]

The site is adjacent to the valley of the Altamaha River, near the boundary between the Brandywine and Coharie Terraces of the Coastal Terrace subprovince.[(2)] From the Altamaha River flood plain to the ground surface at the site, relief is ~ 100 ft.

2.5.1.1.2 Regional Geologic Maps (HNP-1 and HNP-2)

The surficial geology of the region is characterized by Precambrian and early Paleozoic rocks inland from the Fall Line, and Cretaceous to Holocene sediments from the Fall Line to the Atlantic and Gulf coasts. The rocks within the Appalachian provinces are largely folded, faulted, and metamorphosed.(4) Those in the Coastal Plain provinces dip seaward at low angles and have undergone comparatively minor structural deformation.(1)

Regional maps depicting the surface geology and tectonic features are presented in figures 2.5-2 and 2.5-3, respectively.

2.5.1.1.3 Regional Geologic Setting (HNP-1 and HNP-2)

Since the physiography of a province is determined largely by the character of its underlying rocks, the names and boundaries of the geologic and physiographic provinces within 200 miles of the site are considered the same.(1)

The geology of the region within 200 miles of the site may be divided into two categories:

- *Areas in which Precambrian and Paleozoic rocks are exposed.*
- *Areas containing exposures of upper Mesozoic and Cenozoic sediments, underlain by Precambrian, Paleozoic, and lower Mesozoic rocks.*

The Blue Ridge and Piedmont provinces contain surface exposures of Precambrian and Paleozoic rocks. Mesozoic and Cenozoic sediments constitute the surface formations in the Atlantic and Gulf Coastal Plains.

The Blue Ridge province, included in the western Piedmont by some authors,(5) extends from the Cartersville Fault to the Brevard Fault Zone.(4) It has been interpreted as:

- *A synclinorium modified by doming and faulting subsequent to deposition and metamorphism of Middle Devonian to Early Mississippian sediments.(6)*
- *An anticlinorium consisting of middle Precambrian basement rocks flanked by younger rocks that were folded in the mid-Paleozoic and then broken and transported westward by later Paleozoic thrusting.*

Regardless of interpretation, the rocks are unmetamorphosed to highly metamorphosed and deformed by flexure, slip, flow folding, and thrusting.(4) The metamorphic grade generally increases southeastward. Granitic plutons and ultrabasic intrusives of Paleozoic age are common in the southeastern part of the province.(3) Two belts of low-grade metasediments, the Talladega and Murphy Belts, are found in the northwest part of the province in Georgia. (See figure 2.5-2.)

East and south of the Brevard Fault Zone is the Piedmont province. Rocks in the province range in age from middle Precambrian to early mid-Paleozoic and consist of granite and metamorphics of various

grades. On the northwestern edge, Chauga Belt low-grade metamorphics of late-Precambrian to Early-Cambrian age form a synclinorium separating the Blue Ridge from the Inner Piedmont. Rocks in the Inner Piedmont are mostly granitized, high-grade metamorphics.[4] They have been overturned and overthrust to the northwest,[1] forming a large anticlinal mass of northwest-directed nappes rooted to the southeastern side of the Inner Piedmont. The Kings Mountain Belt in Alabama and Georgia is an anticlinal belt of late Precambrian to Cambrian rocks that range from weak to low-grade metamorphics, similar to the Chauga Belt. The Charlotte Belt, southeast of the Kings Mountain Belt, consists of an isoclinally folded anticlinorium. The anticlinorium is cored by middle-Precambrian basement rocks and overlain by late-Precambrian high-grade metasediments and metavolcanics. A few early mid-Paleozoic plutons intrude the metamorphics. A low-rank assemblage of late-Precambrian to middle-Paleozoic metasediments and metavolcanics is found in the Carolina Slate Belt. The belt is interpreted as a synclinorium, with northeast trending folds that are either open or are tightly compressed and overturned.[4]

Rocks underlying the upper Mesozoic and Cenozoic coastal plain deposits vary in age and lithology. The basement rocks in Georgia consist of Precambrian and Paleozoic high-grade metamorphics, granite, diorite, and some volcanic rhyolites in southeast Georgia. In the tri-state area of southeastern Alabama, southwestern Georgia, and northern Florida, the coastal plain sediments are underlain by tightly consolidated, clastic sedimentary rocks. The rocks contain many fossils which range in age from Cambrian to Silurian. A well drilled in Appling County, Georgia, less than 5 miles from the site, ended in basalt of probable Triassic age at a depth of 4108 ft.[7]

The top of basement rock beneath the East Gulf and Atlantic Coastal Plains represents a portion of the erosional surface developed on deformed Appalachian Belt rocks prior to or during the Jurassic. The surface is exposed inland from the Fall Line where the overlapping wedge of younger coastal plain material terminates. Geophysical data suggest the presence of general north-to-northeasterly trends in the basement underlying the Atlantic Coastal Plain. These trends may be due to lithologic or structural variations in the basement rocks, or to topographic relief developed on a pre-Cretaceous erosion surface. Seismic surveys and well borings reveal an irregular surface with a general seaward slope for the top of basement rocks underlying the Coastal Plain provinces.[1]

Overlying the Paleozoic basement rocks, the Atlantic and East Gulf Coastal Plain sediments range in age from Triassic to Holocene. A regional geologic column showing these strata is shown in figure 2.5-4. These sediments generally consist of alternating layers of relatively unconsolidated sand, sandstone, shale, clay, and limestone. Triassic deposits in the form of red beds occur in isolated grabens underlying the Atlantic Coastal Plain. No Jurassic strata are known to exist in the Atlantic Coastal Plain, and pre-Cretaceous rocks are not exposed in either Coastal Plain province.[1]

Cretaceous through Holocene sediments are found at the surface in both coastal plain areas. The outcrop pattern, with bands of older strata lying landward of younger strata, reflects the gentle seaward dip of the deposits.[3] In Georgia, this dip is between 5 and 50 ft/mile.[8] Regionally, the dip increases with depth as a result of seaward thickening of the coastal plain deposits.[3] The sediments consist of gravels, sands, silts, clays, marls, and their consolidated equivalents, such as sandstone and limestone. Numerous transgressions and regressions of the sea have resulted in the interfingering of marine and nonmarine deposits.[9] The total thickness of these units ranges from a feather edge along the Fall Line to more than 7500 ft in southwestern Georgia.[8]

Geologic structure in the Coastal Plain provinces within 200 miles of the site is relatively simple. There are no known features affecting material younger than Miocene. The structural features of the region are discussed in paragraph 2.5.1.1.6, Regional Tectonic Structures.

2.5.1.1.4 Regional Geologic History (HNP-1 and HNP-2)

The geologic history of the region is characterized by mountain building and erosion in the Appalachian areas and by deposition of marine and nonmarine sediments in the Coastal Plain provinces.[1]

During the Precambrian and early Paleozoic, a large sedimentary basin, the Appalachian geosyncline, extended along the eastern portion of the United States.[3] *Subsidence within the geosyncline allowed great thicknesses of sediments to collect. In the middle and late Paleozoic, this basin sustained mountain-building forces that metamorphosed portions of the early Paleozoic and older sediments, injected plutonic masses into them, and raised them by folding and faulting. The metamorphosed area includes the Piedmont and Blue Ridge provinces.*[1]

In the Triassic, the eastern Appalachian provinces were again faulted and injected with northwest trending diabase dikes. Terrigenous deposition occurred in northeast trending, graben-like basins.[3] *Erosion of the Appalachian areas marginal to the present coastlines continued until the Cretaceous.*[1]

Deposition of marine and nonmarine sediments in the coastal plain areas began in the Cretaceous.[1] *The sediments were deposited in seas that originally invaded the margin of the continent up to the Fall Line.*[9] *During the Cretaceous, several rivers draining the Appalachian Highlands contributed vast amounts of material to the slowly subsiding continental margin.*[1] *After the Late Cretaceous, the seas began a persistent, although irregular, retreat with progressively younger marine and marginal marine sediments being deposited on older strata in belts generally parallel to the present coastline. As a result, a seaward-thickening wedge of coastal plain sediments was built up.*[9] *Cenozoic deposition of uniform thicknesses of material was probably modified by submarine erosion and intermittent, slow growth of fold structures in southern Georgia and Florida.*[1] *Material in the coastal areas younger than Miocene appears to be unaffected by geologic structures.*

2.5.1.1.5 Regional Geologic Conditions (HNP-1 and HNP-2)

The geologic conditions of the coastal plain within 200 miles of the site are related to the rate of subsidence of the buried Paleozoic fold belts during deposition of the coastal plain sediments and the source and lithology of the coastal plain deposits. The coastal plain begin to form after tilting and subsidence of the Appalachian Fold System. The truncated surface of this ancient system dips southward and southeastward beneath the coastal plain. Material eroded from the Paleozoic rocks was laid down in or on the margins of seas that overlapped inland from the Atlantic Ocean and Gulf of Mexico. Mesozoic and Cenozoic deposits that cover the Paleozoic surface dip and thicken toward the coast.[1] *The regional subsurface conditions are shown in figures 2.5-5 and 2.5-6, Regional Geologic Profiles.*

Mesozoic Conditions

The first coastal plain deposition within 200 miles of the site occurred in the Early Cretaceous when strata of the Comanche series were deposited. These materials have been identified in deep wells drilled

in seaward areas of Alabama and Georgia. They typically consist of red beds in updip areas and interbedded evaporites, shale, and carbonates downdip. No Lower Cretaceous deposits are exposed within the study area.[1]

Upper Cretaceous Gulf Series deposits overlying the Comanche Series include, in ascending order, the Tuscaloosa group, and the Eutaw, Blufftown, Cusseta, Ripley, and Providence Formations. These units underlie the coastal plain within 200 miles of the site and are exposed in southwestern Georgia and Alabama in belts seaward from the Fall Line. In addition, the Tuscaloosa group is exposed in central and eastern Georgia and South Carolina. The Tuscaloosa generally has a lower, terrigenous sand and gravel unit, a middle silt and clay sequence (mostly marine), and an upper terrigenous sand-to-gravel unit. The overlying formations are lithologically similar to the Tuscaloosa, but interfinger downdip with predominantly calcareous beds. Gulfian deposits rest in updip areas with angular unconformity on Comanchean and older strata, while in seaward areas the contact appears transitional.[1]

The preceding Mesozoic deposits were laid down during a time of transgression and submergence.[9] *Numerous landward unconformities indicate that submergence was interrupted by sporadic emergence. Deposition of Cretaceous strata was centered in areas of subsidence (depocenters) adjacent to the Appalachian Fold Belt. Subsidence contemporaneous with deposition was necessary to contain the great thickness of sediments. One such basin, the Apalachicola embayment, is in southwestern Georgia and northern Florida and contains thin, near-surface Quaternary and Tertiary rocks overlying thick deposits of Cretaceous and older Mesozoic strata. This basin was subsiding and receiving coarse deposits from the adjacent uplifted highlands during the Early Cretaceous. As the Cretaceous sea spread inland over the eroded fold belts, Gulfian (Late Cretaceous) marine sediments were deposited in the embayment, while the coarser sediments were deposited inland. The thin layers of overlying Cenozoic material indicate that subsidence had ceased before their deposition. The pattern of deposition was further modified by positive features, such as the Peninsular Arch. This subsurface arch extends from southcentral Georgia into eastcentral Florida. Lower Cretaceous strata are absent on the apex of the arch and pinch out against the flanks, indicating that the feature was positive and possibly forming during the Jurassic and Early Cretaceous. Erosion of the Paleozoic core supplied coarse material to the basin on the western flank of the arch during the Early Cretaceous. Development of the arch apparently had ceased by the Late Cretaceous, since Gulfian marine deposits are found undeformed on the arch.*[1]

Cenozoic Conditions

The Paleocene is represented by the Clayton formation of the Midway group. The Clayton is predominantly limestone and sandy marl in eastern Alabama and western Georgia. In Florida, Midway strata are characterized by limestones, oolitic beds, and evaporites of the Cedar Keys Formation.[1] *In South Carolina, Paleocene strata are represented by undifferentiated Midwayan clay and sand beds.*[10] *Midway beds in the area of study lie unconformably on Cretaceous strata. The Midwayan strata were probably deposited in near-shore and shallow-marine environments. Locally, Paleocene strata lack considerable thickness or are absent, owing to erosion or nondeposition or both.*[1]

Eocene strata, exposed in belts seaward of Cretaceous formations, lie disconformably on Paleocene and older deposits in both the Atlantic and Gulf Coast areas. (See figure 2.5-2.) The Eocene is represented by, in ascending order, the Wilcox, Claiborne, and Jackson groups.

Undifferentiated lower Eocene Wilcox deposits contain sandy material in updip areas and become finer-grained, more calcareous, and generally more marine seaward in the subsurface. Deposition probably occurred in deltaic, marginal marine, and shallow marine areas.[1]

Middle Eocene Claiborne Group deposits rest disconformably on the Wilcox Group. The Claiborne Group is represented by the Tallahatta and Lisbon Formations in southeastern Alabama and Georgia.[1] *Equivalent strata are the Congaree and McBean Formations in South Carolina,*[10] *and the Lake City and Avon Park limestones in northern Florida. The Tallahatta generally contains unconsolidated sand and lignitic, calcareous, and micaceous silty clay and limestone. The overlying Lisbon Formation consists of fossiliferous clay, marl, and calcareous sand. Claiborne sediments were deposited in warm, shallow seas.*[1]

The upper Eocene is represented by the Ocala Formation (and equivalents) of the Jackson Group. In downdip areas and in the subsurface, the Ocala is a highly fossiliferous, calcareous clay and limestone. Updip exposures are typically a deeply weathered sandy clay. Jackson Group deposits represent an extensive marine invasion of the continental margin in the late Eocene.[1]

Undifferentiated Oligocene deposits are mainly limestone and marl, with some calcareous clay and dolomite.[1] *The discontinuous outcrop pattern of Oligocene material, especially in western Georgia, is the result of solution of the calcareous strata.*[11] *The Oligocene Formations were deposited in a warm, shallow marine environment.*[1]

Miocene deposits in the coastal plain include shallow marine and nonmarine rocks of the Tampa, Alum Bluff, and Choctawhatchee stages. The basal stage (Tampa) in Georgia and northern Florida is a series of dolomitic limestones interbedded with clays and sands in updip areas.[12] *This stage is represented at the site by the Tampa Formation. The middle and upper stages (Alum Bluff and Choctawhatchee) are predominantly clastics, consisting of sandy micaceous clays and arkosic sands,*[8] *and include the Hawthorn Formation at the site.*

Sand and gravel deposits generally recognized as Pliocene occur near the Apalachicola River in northern Florida. Material identified as Miocene in Georgia and South Carolina, including the Hawthorn Formation, may be of Pliocene age. (This material is referred to as "Neogene Undifferentiated" by the Georgia Geological Survey). Other possible Pliocene material includes marl in downdip areas, and river terrace deposits along stream valleys.[1]

Pleistocene deposits along the coast consist of nonmarine, marginal, and marine sands and clays underlying seaward coastal terraces of terrace surfaces. They merge inland along the major river valleys with fluvial deposits.[1]

Cyclic advances and retreats of the sea determined depositional patterns in the Tertiary following a period of erosion at the end of the Cretaceous. During the Paleocene, Midway sediments were deposited to within 50 miles of the Fall Line over the eroded Cretaceous deposits. Erosion then resulted from a general lowering of sea level. This pattern of transgression and deposition, followed by regression and erosion, was repeated throughout the Tertiary. Each succeeding stage encroached inland to a lesser extent over the eroded remains of the previous stage.[9]

Various structural features modified Tertiary depositional patterns. From the Eocene into the Miocene, materials were thinly deposited and slightly folded in the area of the rising Ocala uplift. Accumulation of

great thicknesses of Miocene and earlier materials took place in negative areas, such as the Apalachicola embayment. The present outcrop pattern reflects the minor influence of these structural features on Tertiary deposition.[1]

2.5.1.1.6 Regional Tectonic Structures (HNP-1 and HNP-2)

Rocks in the Blue Ridge and Piedmont tectonic provinces within 200 miles of the site have been faulted and folded to varying degrees. Major deformation of these rocks occurred prior to the Mesozoic, although there is evidence for Triassic displacement of Piedmont rocks. By contrast, the Cenozoic coastal plain sediments within 200 miles of the site are not displaced by major faults.

The major structures within the Blue Ridge and Piedmont provinces are the Brevard, Towaliga, and Goat Rock Fault Zones. (See figure 2.5-3.) These fault zones extend from the Fall Line in Alabama northeast and eastward into Georgia. The Brevard Zone, more than 180 miles northwest of the site, follows a relatively straight trace through Georgia, South Carolina, and into North Carolina.[38] *This fault marks the boundary between the Blue Ridge province to the northwest and the Piedmont province to the southeast. Fault planes within the zone dip steeply to the southeast. It has been variously interpreted as a right-lateral strike-slip fault with at least 135 miles of displacement;*[39] *a major fold complicated by trough faulting;*[40] *a zone of simultaneous thrusting and left-lateral strike-slip movement with less than 10 miles of displacement;*[38] *and the sole of a great overthrust.*[5] *The Towaliga and Goat Rock Fault Zones form the northwest and southeast sides, respectively, of the Kings Mountain Belt in Alabama and western Georgia. The northern portion of the Towaliga Zone also forms the northwest boundary of the Charlotte Belt in eastern Georgia. Movement along the Towaliga Fault Zone, which is more than 125 miles northwest of the site, has been variously interpreted as strike-slip with a minor dip-slip component;*[41] *high-angle thrusting toward the southeast;*[4] *and northwestward relative displacement along the sole of an overthrust.*[5] *Fault planes within the zone dip steeply to the northwest.*

The Goat Rock Fault Zone is exposed in the Piedmont province more than 100 miles north of the site. The fault zone has been mapped northwestward from the Coastal Plain - Piedmont border near Salem, Alabama, into western Jones County, central Georgia. In western Georgia, the fault zone is the intensely sheared portion of the Uchee Block[55] *and forms the southern boundary of the Pine Mountain series.*[63] *The main fault zone is ultramylonite, mylonite, and blastomylonite bordered on each side by mylonite gneisses.*[43][54] *Based on the presence of discontinuous shear zones and regional magnetic characteristics and lineaments, various workers have proposed to extend the fault zone beyond central Georgia as far as the vicinity of Columbia, South Carolina. (See references 4, 42, 44, 46, 50, 53, 54, 57, 60, 61, 62, 68, 70, and 72.) Total length of the mapped and proposed portions of the fault zone is ~ 250 miles. The Geologic Map of Georgia acknowledges the presence of northeast-trending shear zones up to 10 miles long in east-central Georgia, but does not indicate a continuous fault trace northeastward beyond central Georgia.*[82] *Two of the shear zones are coincident with proposed splays off an extended Goat Rock Fault Zone in east-central Georgia termed the Flat Rock and Morton Fault Zones.*[45] *A zone of cataclastic rock forming the boundary between the Kiokee and Carolina Slate Belts northeast of Columbia, South Carolina, has been attributed to another proposed extension of the Goat Rock Fault Zone terminating near Laurinburg, North Carolina.*[56] *The sense and magnitude of displacement of these shear zones has not been reported. Recent speculation based on magnetic data extends the Goat Rock Fault into Virginia.*[74]

The width of the fault zone is nearly 10 miles in Alabama and adjacent parts of Georgia.[5] The proposed extension in South Carolina is believed to be 2 miles wide (oral communication, D. E. Howell). Some workers contend that the Goat Rock Fault Zone is the southern boundary of a folded thrust sheet which is bounded on the northern edge by the Brevard Fault. (See references 5, 48, 52, and 77.) The amount of displacement is not known. Dip of the fault in west-central Georgia is 10° to 50° southeast.[43][44][55] The reported type of displacement includes thrusting,[5][48][54] and right lateral strike-slip.[43] A mapped high-angle fault, trending southwestward from Columbia, South Carolina, into Georgia,[42] has been associated with the Goat Rock Fault Zone. However, the dip of that fault plane (northwestward) and sense of displacement (normal) is opposite to the dip and displacement of mapped portions of the Goat Rock Fault in eastern Alabama and western Georgia. (See references 5, 43, 44, 48, 54, and 55.)

Apparently, the Goat Rock Fault Zone is one of many structures created during the collision of southeastern North America and Africa between Late Devonian and Permian time.[4] Potassium/argon dates for rock from the Goat Rock Fault Zone give an approximate 300-million year (Early Mississippian) age for the time of major fault movement.[69] An upper time limit for surface displacement on the Goat Rock Fault Zone can be established as pre-Cretaceous. Near Salem, Alabama, and Columbia, South Carolina, undeformed Cretaceous sediments of the Coastal Plain cover the fault zone.[54][68] Several diabase dikes are mapped across the trace of the Goat Rock Fault Zone and show no offset.[47][59] The diabase dikes are part of a large dike system which extends from Alabama to Massachusetts.[58] The dikes are Late Triassic[58] or Jurassic,[49] the age of the dikes being determined by stratigraphic and paleomagnetic methods, respectively, by these workers.

Historic epicenter location maps for eastern Alabama and most of Georgia show no geographical distribution of events which suggests the Goat Rock Fault Zone is active in these areas.[36][37][51] The Goat Rock Fault obliquely crosses the "South Carolina-Georgia Seismic Zone" in which two-thirds of all historic activity in the southeastern United States has occurred.[37][67]

The number of reported seismic events in South Carolina has increased significantly in recent years. This is due to increased instrumentation in this region, and the increase of detection capability this implies. Consequently, the increase in the number of recorded events is due to the detection of small events that previously went undetected, and not to an overall increase in seismicity. This contention is supported by cumulative strain release and cumulative frequency of occurrence for events of Modified Mercalli IV or greater over the time intervals 1776-1973 and 1872-1974, respectively, in the southeastern United States.

If the Charleston event of 1886 is assigned a maximum Modified Mercalli intensity of X, then ~ 5000 times more energy was released during this event than all other known events in the southeastern United States during the 1776 to 1973 period. Approximately 85% of all remaining strain released during this period occurred during the years 1905 through 1916. Therefore, the strain release pattern is dominated by a few large events which occurred more than 60 years ago. No increase in the rate of occurrence of these events is evident. The frequency of occurrence of events from the above data set between 1957 and 1974 is comparable to a period between 1911 and 1930. Between these two time periods, there is a definite lull in recorded activity. A similar lull in the frequency of events occurred between 1911 and the swarm of events associated with the 1886 Charleston events. Therefore, historic data does not suggest that the southeastern United States is in or is entering a period of unusual seismic activity in either a qualitative or quantitative sense.

Several recent studies have linked recent and historical seismicity to the eastern extension of the Goat Rock Fault in South Carolina. (See references 63, 65, 66, and 68.) In the absence of corraborative

evidence such as focal mechanism solutions, any seismic implications of this fault's activity must depend on some observed alignment of epicenters. This type of evidence is the sole basis for proposing current activity along the Goat Rock Fault. In particular, the 8 epicenters listed in table 2.5-3 and shown in figure 2.5-15 are proposed by Talwani[66] to occur along the Goat Rock Fault. An indication of the method of epicentral determination is also included in table 2.5-3. An S indicates that the epicenter is derived from an isoseismal map, while an F implies that felt data is the only source of an epicentral location. An I implies that some instrumental location was computed.

There are several reasons to doubt a simple interpretation of the apparent alignment as given by Talwani.[66] The uncertainty in locating individual, small epicenters makes their extremely close alignment probably fortuitous. In figure 2.5-15, a coordinate system is superposed on the epicentral locations of the 8 events as given by Talwani.[66] The orientation and scale of this coordinate system are such that the data may be treated as univariant and Gaussian with a zero mean and a standard deviation of ±8 km. That is, it was assumed that the fault trace is the true location for all the events of table 2.5-3; however, due to univariant Guassian measurement noise, the events do not appear to fall on this line. The assigned standard deviation of ±8 km is believed to be a conservatively small average value for these epicenter locations. Under these conditions, the error criterion defined to be the sum of the squares of the residuals may be shown to be a chi-square variable with 6 degrees of freedom.[75] The best fit for the data using the least squares method is also shown in figure 2.5-15. The sum of the squares of the residuals of each individual epicenter from the least square regression line is 0.218 compared to an expected error criterion value of 6.[71] The probability of the error criterion value, 0.218, occurring under the above assumptions is < 2 in 10,000. It is concluded that either the average standard deviation given above is too large or that the striking coincidence of these eight epicenters with the trace of the Goat Rock Fault is fortuitous.

There are also questions as to the pertinence of assigning several of the epicenters to a NE to SW trending structure. Two of the events, January 4, 1974, and December 8, 1974, of table 2.5-3 and figure 2.5-15, are so small (magnitude ≤ 1) that it is unnecessary to associate them with any structure. Furthermore, both NW to SE and NE to SW structural trends occur in the area of the January 4, 1974, epicenter[73] so that it is not clear with which trend, if any, this event is best associated. In the case of the 1879 event, there is disagreement about its actual location. All sources found[76][78][79] place this event 25 to 50 km north of the position shown in figure 2.5-15 as derived from Talwani. (See figure 2.5-16.)

Figure 2.5-16 and table 2.5-4 show that the historical seismicity of South Carolina has been rather diffuse with the exception of the Charleston area events which are listed in table 2.5-5. This conclusion is not altered by recent detailed work of the United States Geological Survey (USGS)-USC seismic network.[80] Lineations anywhere in the state are at best poorly supported by epicenter location data. In view of all these difficulties, it is not reasonable to consider the Goat Rock Fault seismically active in any but a speculative sense.

Sediments of the Atlantic and East Gulf provinces are only slightly modified by post-Paleozoic structural movements. The significant tectonic structures underlying the coastal plain region within 200 miles of the site include the Peninsular arch, Ocala uplift, Apalachicola embayment, and Southeast Georgia embayment. Several structures of questionable existence or having only minor effects may also underlie coastal plain deposits. Development of all the above listed structural features ceased before the Pleistocene, and none is considered significant to the site.[1]

The relationship of these structures to the geologic history of the area is discussed in paragraph 2.5.1.1.5, Regional Geologic Conditions. A regional tectonic map is presented in figure 2.5-3. The possibility of uplift, subsidence, or collapse related to tectonic structures is discussed in paragraph 2.5.1.1.6.2, Areas of Potential Instability.

Since issuance of the construction permit in December, 1972, investigations of the geology, seismology, and tectonics within 200 miles of the site have focused on 3 main areas:

- *Charleston, South Carolina.*
- *The Upper and Middle Coastal Plains in South Carolina and eastern Georgia.*
- *The Piedmont of South Carolina, Georgia, and Alabama.*

The primary methods of investigations have been remote sensing, geophysical and earthquake monitoring, supplemented by surface reconnaissance and borings. The investigations have suggested preliminary correlation between seismic events and known geologic structures, and have inferred the existence of previously unrecognized structures in the Coastal Plain and Piedmont provinces. Data from the investigations have also been synthesized into regional studies of seismology and tectonics.

A. *Charleston, South Carolina*

Recent studies in the Charleston area have attempted to define the local crustal structure and basement configuration. Seismic refraction was used by Ackerman[83] *to detect three subsurface marker horizons. Two horizons, at depths of 100 m and 850 m, are depressed in the area of maximum destruction from the 1886 earthquake. A velocity range of 4.5 to 5.6 km/s in the shallow basement Cretaceous volcanics below the 850 m horizon is interpreted to indicate a zone of extensive fracturing or a major lithology change. A third horizon, marking the base of the Cretaceous volcanics, dips steeply to the southeast at 70 m/km. The shallow basement layer is thus interpreted to be wedge-shaped, thickening southeastward.*[83]

Depth analyses of aeromagnetic profiles in the Charleston area also suggest the presence of at least two magnetic basement surfaces.[84] *The uppermost surface, at a depth of about 3000 ft (900 m), may correspond with Cretaceous basalt found in a core hole northwest of Charleston by Gohn, et al.,*[85] *at a depth of 750 m, and with the shallow basement of Ackerman*[83] *at 850 m. At depths > 3000 ft (900 m), the aeromagnetic profiles indicate that a second magnetic horizon, possibly corresponding to crystalline basement, may be broken by east-trending faults. The magnitude of vertical displacement is thought to be on the order of 1000 ft (300 m). A third magnetic source at depths greater than 10,000 ft (3050 m) may correspond with deep-seated intrusives.*[84] *Circular aeromagnetic and gravity anomalies in the same area have also been attributed to deep mafic intrusives.*[86]

Resistivity and magnetotelluric (AMT) soundings conducted by Campbell[87] *in the Charleston region failed to detect the Cretaceous basalt found by Gohn, et al.,*[85] *at 750 m. The basalt is consequently estimated to be < 75 m thick. Electric basement in the vicinity of the core hole, which bottomed at 792 m,*[85] *is ~ 1200 m below ground surface. The*

AMT data outline a northeast-trending, 11-km wide, higher resistivity zone roughly corresponding to the region on highest damage from the 1886 earthquake. The zone is believed to be bordered on the northwest by a gravity lineament possibly representing a steeply dipping fault, with the southeast side downthrown.[87] However, resistivity soundings within the zone indicate shallower electric basement (900 m depth) than in areas outside the zone (1200 m depth). According to Campbell,[87] "If this basement represents a thicker interval of the same Cretaceous basalt encountered in the core hole, some 150 m of post-Cretaceous vertical displacement would be indicated along this (postulated) fault."

There appears to be a lack of agreement regarding details of subsurface structure determined from the above-described geophysical studies. Ackerman[83] suggests extensive fracturing within the Cretaceous volcanics, while Phillips[84] proposes displacement of the deeper crystalline basement. In the zone of maximum damage from the 1886 earthquake, Campbell[87] indicates that the Cretaceous basalt layer has been displaced upward. The same layer in the same area has been found to be depressed by Ackerman.[83] The geophysical data confirm the crustal complexity in the Charleston region, but do not provide adequate information to describe the subsurface structure. The following recent studies have also contributed significant information regarding the geology of the Charleston area:

1. *Popenoe[86] and Popenoe, et al.,[88] describe preliminary information and conclusions obtained from gravity and aeromagnetic data, ERTS imagery, and borings. (See figures 2.5-17 and 2.5-18.) At least four major structural gains exist near Charleston, one of which (circular magnetic and gravity highs) is coincident with the epicenter of a 1974 earthquake (November 22, $M_b = 4.7$). Two other grains are similar to Piedmont northwest and northeast structural trends. The dominant grain is east to west, reflected by gravity and aeromagnetic anomalies.[86] A preliminary contour map of the basement surface has been constructed based on these data.[88]*

2. *Higgins, et al.,[89] further define the Beaufort high southwest of Charleston, and speculate on the structural control of the Orangeburg scarp west of Charleston. Based on biostratigraphic interpretations, the Beaufort high is ~ 96 km long and 48 km wide, trending parallel to the coast to within 24 km of Charleston. Vertical closure is ~ 84 m on the Eocene-Oligocene contact. Faults have been proposed on the north and southeast sides,[89] although no age or sense of displacement is reported. Structural control of the Orangeburg scarp is suggested on the basis of magnetic anomalies, facies changes, ERTS lineaments, and drainage irregularities.[89]*

3. *Bollinger[90] confirms an intensity X (MM) for the epicentral region and IX for Charleston resulting from the 1886 earthquake. He estimates the body-wave magnitude from the event as 6.8, based on central United States data, and 7.1 based on western United States data.[90] An epicentral intensity X (MM) was assumed for the Hatch site in determining the safe shutdown earthquake. (See paragraph 2.5.2.9.)*

4. *Tarr and Carver[80] report that current seismic activity near Charleston is centered near Middleton Gardens, about 22 km northwest of Charleston. The*

November 22, 1974, earthquake (M_b = 4.7) had its epicenter near Middleton Gardens at a computed depth of 12 km. They report: "The focal mechanism of the November 22 earthquake is consistent with either reverse faulting on a plane dipping 78° SW or thrust faulting on a plane dipping 12° NE; both planes strike N 42° W."

5. *Long[91] has proposed a model for earthquake generation in the Charleston-Summerville area. His model requires that two crustal blocks are in contact, that the boundary between the blocks is irregular, and that one of the blocks is characterized by a linear crustal velocity structure which contains rocks having a higher rigidity than surrounding crustal rocks. He contends that his configuration could amplify regional stresses to the extent that earthquakes could occur. Gravity and seismic data indicate that such a linear structure exists in the Charleston area and intersects a zone of gravity anomalies.[91] The intersection is interpreted as a boundary between crustal blocks. The zone of gravity anomalies has been attributed to deep mafic intrusions possibly comprising a body of higher rigidity. Long's model remains speculative, however, since the existence of the required features and configuration is still unproven.*

B. *Upper and Middle Coastal Plains*

The following recent investigations have been conducted in the Upper and Middle Coastal Plain areas of eastern Georgia and South Carolina.

1. *Early Tertiary to Pleistocene or later movement has been proposed along the Belair Fault Zone near Augusta, Georgia.[62][92][93] Weathered phyllites of Precambrian(?) age have been thrust over sediments of possible early Tertiary age. The fault zone consists of a 12-mile long series of echelon breaks which individually trend ~ N 25° E. The control points indicate vertical separation of 100 ft at the northern end of the zone. About 15 ft of separation at the southern end has occurred along fault planes dipping ~ 50° SE. Deformed carbon-bearing material within the fault zone yields a radiometric age of < 2500 years before present. However, the lack of an adequate sample size and other factors may have biased the age determination.[92] The rate and characteristics of movement along the faults remain uncertain. Movement may have been by creep and not necessarily accompanied by earthquake activity.[92] No historical seismic activity has occurred along the fault zone. There is no surface indication of the fault trace visible on aerial photographs.[a] The known southern extent of the fault zone is more than 100 miles north of the site.*

2. *Several deformation features in the central part of the Upper Coastal Plain of South Carolina have been attributed to tectonic activity.[94]*

   Deformation in sediments of the upper Coastal Plain of South Carolina was examined by USGS personnel. The deformation features were reported by Owens, Prowell, and Higgins of the USGS at a Geological Society of America (GSA) meeting held in Reston, Virginia, in 1976. No written report has been prepared, so only the abstract for that GSA presentation is currently available.

The Upper Coastal Plain of South Carolina, as used in the abstract cited, refers to the area of the South Carolina Coastal Plain between McBean, Georgia, and Sumter, South Carolina. The deformations occur in sediments of the Cretaceous Middendorf Formation and in sediments of the Eocene McBean Formation and were seen where these formations are at the ground surface near the Fall Line. The features are briefly described in the abstract for the GSA presentation as follows: "Four categories of deformation were recognized: (1) intricately contorted and faulted interbedded clays and sands; (2) open folds in sandy clays; (3) disconnected slabs of clayey sand, underlain and separated by cleaner sands that have disrupted bedding; (4) domes and detached teardrop-shaped bodies of opal claystone that have sediments draped over and around them." (Owens, et al.). These workers hypothesize that at least some of the features were earthquake caused and may represent quake sheet deformation at a time when the sediments were still soft.

Some of the structures examined by Owens, et al., were seen to be overlain by undisturbed flat-lying sedimentary deposits of similar sedimentary character to the deformed strata (Prowell, personal communication). This condition implies that the time of deformation is contemporaneous with deposition, that is, Eocene time or 35 million years before the present. Dr. Prowell also pointed out that the relationship of undeformed sediments overlying the deformation features was not found at all locations.

The tectonic mechanisms proposed are folding or faulting, and earthquake-induced liquefaction, slumping, and diapirism. The location of possibly causative faults has not been reported. The features are within sediments of Upper Cretaceous through Eocene ages. No age of deformation has been assigned to the features. Although liquefaction is a postulated mechanism, laboratory determinations of grain size distribution were not conducted on the sands thought to be susceptible to liquefaction (B. B. Higgins, personal communication). Deformation in several Upper Coastal Plain localities had been previously attributed to slumping associated with solution of underlying material.[94] The geologic evidence could support either a tectonic or nontectonic mechanism for generation of the features.

3. *Marine[95] has further defined the buried Dunbarton Triassic basin. The basin is ~ 20 miles southeast of the Fall Line on the South Carolina-Georgia border, buried beneath 1150 ft of Coastal Plain sediments. Although seismic reflection, gravity, and magnetic surveys have indicated fault displacement on both the top and bottom of the basin, drilling evidence indicates no such displacement at the top of the Triassic sediments. No fault movement has occurred since development of the erosion surface on the top of the Triassic, about 100 million years ago.[95]*

C. *Piedmont*

Geophysical surveys have recently been conducted in the Piedmont province of Alabama, Georgia, and South Carolina. Aeromagnetic and aeroradioactivity maps of Alabama

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a. *O'Conner, oral presentation, March 1976.*

delineate the known major lithologies and fault zones. (See references 70, 96, 97, and 98.) The geophysical maps have supplemented extensive surface mapping of the Alabama Piedmont.[99] The Goat Rock, Bartletts Ferry, Towaliga, and Brevard Fault Zones are marked by strong contrasts in magnetic lineaments. A major subsurface fault zone based on strong, N 75°E-to-N 80°E-trending lineaments has been proposed to exist beneath Coastal Plain sediments south of the Goat Rock and Towaliga faults.[46] Age, sense, and magnitude of possible displacement have not been reported for this proposed fault zone.

Aeromagnetic maps of the Georgia Piedmont show at least five postulated fault zones.[45] The proposed Indian Springs Fault Zone is a northeast trending splay off the Towaliga Fault Zone coincident with a thick zone of cataclastic rocks near Indian Springs, Georgia. A similar cataclastic zone marks the Flat Rock Fault Zone, a splay off the Goat Rock Fault Zone. Two other geophysically-inferred fault zones, the Macon and Key Creek, may be continuous with known fault zones in South Carolina. The Morton Fault Zone is proposed as a splay off the Goat Rock Zone between the Macon and Flat Rock Zones. Structural discontinuities or cataclastic rocks coincident with these zones have been verified by field investigations.[45] The characteristics of postulated fault movement have not been reported. New fault zones and extensions of existing zones in the Georgia Piedmont are also indicated by aeroradioactivity mapping,[100] although fault characteristics are unreported.

Aeromagnetic and gravity surveys indicate possible extensions of the Towaliga and Goat Rock Fault Zones into South Carolina which is further than previously suspected. The Towaliga Fault Zone is coincident with a zone of cataclastic rocks marking the contact between the Kings Mountain and Charlotte belts. Historic and recent seismic activity has been attributed to this zone.[68] The Towaliga is proposed to extend into North Carolina near Gastonia.[100] The Goat Rock Fault Zone, previously discussed, is proposed to extend northwestward through South Carolina to the vicinity of Laurinburg, North Carolina. The fault zone is marked by a zone of cataclastic rock between the Kiokee and Carolina Slate belts.[56] Earthquake activity was cited to propose an extension of the Goat Rock Zone under Coastal Plain sediments north of Columbia, South Carolina.[65][68] Extension of the Goat Rock Fault Zone is generally preliminary and speculative, and in some cases contradicts known features of the mapped portion of the fault zone in Alabama and Georgia. Paragraph 2.5.1.1.6 provides a detailed discussion of the Great Rock Fault Zone.

Aftershock monitoring near the Clark Hill Reservoir north of Augusta, Georgia, indicates seismic activity is confined to the upper 5 km of the crust.[101] The majority of the fault plane solutions suggest left lateral strike-slip motion along a strike of N 40° E and a dip of 82° SE; remaining data indicate overthrusting, with the fault plane striking N 34° E and dipping 70° SE. The earthquakes have occurred in the immediate vicinity of Clark Hill Reservoir, and have been attributed by Talwani to fluctuations in the reservoir water level.[65][66]

D. Regional Studies

Bollinger[102] has further investigated the spatial and temporal distribution of earthquakes in the southeastern United States. The data suggest that "strain development induced by

crustal uplifting but concentrated by old Appalachian structures may be the proximate cause of the recent seismicity in the southeastern United States."[102]

Aeromagnetic surveys in the Coastal Plains of North Carolina, South Carolina, Georgia, and Alabama, show the contrast between magnetic basement beneath the Coastal Plain and the exposed crystalline Piedmont. Gravity and magnetic highs of the Coastal Plain basement have been attributed to gabbroic-basaltic terrain containing mafic plutons or volcanic centers; gravity highs and magnetic lows of the Piedmont are coincident with metavolcanic terrains. Major crustal differences between Piedmont and Coastal Plains basements are further shown by nonmatching magnetic trends.[103] The Triassic basins underlying the Coastal Plain have been extended to form a continuous series based on geophysical data;[68] the continuous basins have not been delineated by drilling.

2.5.1.1.6.1 Description of Coastal Plain Structures (HNP-1 and HNP-2)

A. Peninsular Arch

The Peninsular arch is a buried arch-like fold trending northwesterly from east-central Florida into south-central Georgia. Early Paleozoic strata make up the core of the arch. They are flanked by Lower Cretaceous and possibly Jurassic deposits. Lower Cretaceous strata are absent on the apex of the arch, indicating that the feature was positive and possibly forming during the Jurassic and Early Cretaceous. The arch was apparently inactive and covered by seas during and after the Late Cretaceous, since Gulfian and younger deposits are found over the arch.[1] The northern end of the buried Peninsular arch is ~ 85 miles southwest of the site. Material at the site is not affected by the arch.

B. Ocala Uplift

The Ocala uplift is a broad, northwest-trending anticline with its axis west of the older Peninsular arch. The two folds are apparently unrelated, since they affect material of different ages. The uplift is reflected at the surface by a broad outcrop of Ocala limestone.[1]

Undeformed late Miocene beds overlie upwarped beds of earlier Miocene and Eocene ages in the southern part of the uplift, indicating that development of the uplift may have begun in Eocene time and ceased before the end of the Miocene.[13] The Ocala uplift is over 150 miles southwest of the site. The uplift does not affect site material.

C. Apalachicola Embayment

The Apalachicola embayment is located in southwestern Georgia and northern Florida.[11] It is a relatively shallow basin or syncline representing a change in strike of coastal plain strata from predominantly east-west in eastern Alabama to ~ north-south in southwestern Georgia and northern Florida. The embayment narrows in the northeast; its axis is generally aligned northeast-southwest.[1]

Magnitude of the basin increases with depth, thereby indicating a long and continued development. Near-surface late Tertiary rocks are scarcely downwarped while Cretaceous and earlier Mesozoic strata are downwarped to a progressively greater extent. Correspondingly, the older strata are generally thicker.[(1)]

The basin area contains early Paleozoic flat-lying unmetamorphosed sediments that apparently were not involved in the severe folding of the Appalachian orogenic belt. These sediments are overlain by early and middle Mesozoic red beds, which are in turn covered by later Cretaceous marine deposits. The presence of shallow marine sediments throughout the Tertiary and the lack of faulting in the basin indicate that the area is relatively stable at present.[(1)] *The northeastern edge of the basin is 125 miles southwest of the site. Material at the site is not affected by the basin.*

D. Southeast Georgia Embayment

The Southeast Georgia embayment is a depositional basin recessed into the Atlantic Coast between Savannah, Georgia and Jacksonville, Florida. It is primarily a tectonically passive feature between the uplifted Cape Fear arch to the north and the Peninsular arch to the south.[(14)] *The basin received relatively thick sequences of Cretaceous through Miocene material from the adjacent positive areas.*[(1)] *Post-Miocene material is undeformed in the embayment and has a uniform thickness with areas to the south and north. The base of Miocene material in northwest-southeast cross-section through the site (figure 2.5-5) is nearly flat,*[(14)] *indicating that relative subsidence of the basin (or uplift of the adjacent arches) had ceased by Miocene time. The uniform elevations of Pleistocene marine terrace features in the area of the embayment also indicate a long, continued tectonic stability.*[(15)] *The site is about 11 miles west of the landward edge of the Southeast Georgia embayment.*

Other major and minor structural features have been proposed or identified on the coastal plain area within 200 miles of the site and are shown in figure 2.5-3. These features are the Chattahoochee anticline, Gordon anticline, Andersonville fault, Gulf trough, Ochlockonee fault, and Barwick arch. None of these features is of any significance to the site.

According to Patterson and Herrick (1971),[(11)] *the Chattahoochee anticline was first postulated by Veatch in 1911. It reportedly extends from the Fall Line to the Florida state line and straddles the Chattahoochee River. Veatch's proposition was based on the north-south alignment of the Chattahoochee River along the axial part of the postulated anticline, and the entrenchment of that river.*[(11)] *Other authors (Stephenson, 1928, and Toulmin, 1955)*[(9)(16)] *show a similar position for the anticline. Sever (1964)*[(17)] *shows the anticline trending northeast for about 225 miles from Panama City, Florida, into central Georgia. This position was based on mapped outcrops of Eocene rock flanked on the northwest and southeast by Oligocene rock in the Georgia coastal plain.*

An evaluation of the evidence supporting the existence of the Chattahoochee anticline (Patterson and Herrick, 1971)[(11)] *indicates that the existence of the anticline is little more than speculation. The evaluation found that:*

> *...most published reports in which structural features are proposed in the area of concern fail to spell out supporting evidence in a convincing manner. Many articles simply*

> *illustrate the axis of an anticline on a small-scale map and mention the feature by formal name in the text. Most of the questionable evidence in support of the Chattahoochee anticline was outlined by Veatch (1911) in his original proposal, and by Sever (1964) in his redefinition. The results of several investigations, both published and unpublished, are in opposition to the ideas advanced by Veatch and Sever....His (Veatch's) ideas regarding this anticline are suspect for the following reasons: 1. The course of the Chattahoochee River is nowhere diverted as it should be, if it were influenced by an uplift, and the proposed axial position of this river is an unlikely one; 2. The entrenchment of the river is not sound evidence for an anticline along it, because similar entrenchment has been noted further west in Alabama where it is attributed to regional uplift in Pliocene time.....(Patterson and Herrick, 1971, p. 3-5).*[11]

Sever's proposition was disrupted because there is no evidence for the reversal of regional dip necessary for an anticline to occur and because the outcrop pattern (Eocene material surrounded by Oligocene material) is the result of topographic differences, with Oligocene material exposed at higher elevations than Eocene material.[11] *Furthermore, regional geologic profiles (Maher, 1965, Plate 7; Maher and Applin, 1968, Plate 5)*[14][18] *show no reversal of regional dip in the vicinity of the Chattahoochee River. Accordingly, interpretations of the Chattahoochee anticline, without sufficient evidence, should be considered as no more than hypothetical.*[11]

The Gordon anticline was initially defined by Hager in 1918 to be near Gordon, in southeast Alabama.[19] *He described it as having a closure of 40 ft and an area of 10 sq mi about an east-west axis. Adams (1929) noted some irregularities of dip in outcrops along the Chattahoochee River near Gordon, but no well-defined structure.*[20] *Toulmin and La Moreaux (1963, Figure 4)*[21] *show a reversal of dip in the vicinity of Gordon on their geologic section. It appears that the Gordon anticline actually exists, but its influence is minor, and it does not affect the uppermost (Upper Eocene) beds. The anticline is over 165 miles southwest of the site and shows no influence on strata underlying the site.*

An east-trending fault having a maximum vertical displacement of 100 ft and a length of ~ 5 miles was named the Andersonville fault by Zapp (1965).[22] *The fault is located near Andersonville, Georgia, ~ 95 miles west of the site, and displaces middle Eocene Claiborne deposits but not the overlying upper Eocene Jackson deposits. It is not known whether the fault is normal or reverse, as the fault plane was not observed. Other minor structural irregularities in the same area have been attributed to underground solution of limestone in the near-surface Paleocene Midway Group and consequent irregular slumping of the overlying sediments. Although the fault passes westward into a monocline that dies out further west, the Andersonville fault may also be a solution feature.*[22]

The name Gulf Trough of Georgia was proposed by Herrick and Vorhis (1963)[8] *for a major linear structural feature of the subsurface in southwest Georgia. As first recognized by Applin and Applin in 1944,*[23] *this feature extends northeastward from the Gulf of Mexico, through the Tallahassee, Florida, area and into south-central Georgia. It has subsequently been recognized as a sediment-filled depression and termed Gulf Trough.*[13] *Various authors have described the Gulf trough as a graben, downfaulted embayment, syncline, faulted syncline, structural basin or depression, trough or channel, submarine valley or strait, and a solution valley.*[11]

Arguments favoring faulting or graben faulting as the origin for the trough do not present convincing evidence of the existence of faulting and a downthrown central block.[11] *Sever's (1966)*[24] *proposed Ochlockonee fault would be on the southeast side of such a central block. After reexamination of Sever's*

data, however, Patterson and Herrick (1971)[11] conclude that there is no evidence to suggest that movement along a fracture had occurred. A thick elongate belt of Miocene sediments fills the trough, indicating the feature is a depression of major dimensions. It apparently merges with the Apalachicola embayment to the southwest and may have been an extension of the embayment during part of its history. The shape of the trough is indicative of a sediment-filled strait or marine valley formed by erosion.[11] The nearest approach of the postulated feature is 85 miles west of the site.

A related feature, the Barwick arch, was proposed by Sever (1966)[24] to lie ~ 9 miles southeast of his proposed Ochlockonee fault. The arch was based on contours drawn on the top of the subsurface Suwanee limestone of Oligocene age that show ~ 100 ft of closure. No oil or water wells in the vicinity of the arch penetrate through Oligocene rock, so little information is available to prove or disprove the existence of such a feature.

Sever's (1966) assertion, based on well data, that the Suwanee has a uniform thickness in the region, is therefore invalid.[11] One possibility is that the apparent reversal of regional dip from the arch northwestward into the adjacent Gulf trough is an initial dip resulting from deposition on the southeast side of a strait or marine valley. Structure contour maps of the top of the Oligocene in areas south of the arch show a buried karst topography having high areas of the same magnitude as Sever's Barwick arch. This indicates that carbonate solution also may have significantly modified the apparent dips in the vicinity of the arch. The Barwick arch can be explained as an erosional or solution feature rather than a tectonic structure.[11]

2.5.1.1.6.2 ***Areas of Potential Instability (HNP-1 and HNP-2)***. *The East Gulf and Atlantic Coastal Plains within 200 miles of the site appear to be relatively stable. As seen on the crustal movement map (figure 2.5-7), the greatest uplift within the coastal plains is less than 5 mm/year. The map is based on measurements made over the past 100 years by the National Geodetic Survey. Much of it is based on interpolation between widely spaced lines of elevation that have been measured by geodetic field parties. The elevations are relative to each other and are referred to the 1929 Sea Level Datum. The site is located just inside the southern Appalachian uplift area. The uplift is regional in character and not associated with a specific tectonic feature.[25]*

The major tectonic depressions within 200 miles of the site are the Apalachicola embayment. Both of these structures are discussed in paragraph 2.5.1.1.6.1, Description of Tectonic Features. No differential movement is shown in the area of these structures on the crustal movement map (figure 2.5-7) except for slight uplift in the northern part of the Southeast Georgia embayment. In the Apalachicola embayment, near-surface late Tertiary deposits are scarcely downwarped.[1] Material in the Southeast Georgia embayment shows no evidence of deformation since the Miocene.[14] The Apalachicola embayment and Southeast Georgia embayment are, therefore, considered stable at present.

Buried and surficial karst terrains exist in the areas of regional consideration. A buried karst surface has been developed on the Suwanee limestone of Oligocene age in south-central Georgia, southwest of the site.[24] Underlying the site, deeply buried Oligocene limestone is recalcitized, and does not contain cavities. The top of the limestone conforms with the regional dip of coastal plain strata without irregularities due to solution.[26] The Dougherty Plain northwest of the site displays a karst terrain. However, the Tifton Upland, adjacent to the site, shows no evidence of solution of underlying materials.[2] Solution of limestone and development of karst features are not significant to the safety of the site.

No petroleum producing areas are located within 200 miles of the site in South Carolina and Georgia.[27] Production of petroleum in Alabama is limited to areas in the central and western parts of the state underlain by Jurassic and Lower Cretaceous producing formations.[28] Production in Florida is similarly limited. These formations do not exist within 200 miles of the site. No other mineral extraction or subsurface mining occurs or has occurred in the site area.[29] Withdrawal of ground water from the area, discussed in paragraphs 2.4.13.2 and 2.4.13.2.5, will not cause subsidence. Future subsidence does not appear to be of concern at the site.

2.5.1.1.7 Regional Ground Water Conditions (HNP-1 and HNP-2)

The regional ground water conditions are discussed in detail in paragraph 2.4.13.1.1. In general, the major source of ground water in southeastern Georgia is known as the principal artesian aquifer. The formations composing this aquifer are the Ocala limestone of Eocene age, the Suwanee limestone of Oligocene age, and the Tampa limestone of Miocene age. These three limestones and their stratigraphic equivalents generally act as a single hydrologic unit. They are confined between low permeability beds of the overlying Hawthorn Formation of Miocene age and the underlying Claiborne Group strata of middle Eocene age. The principal artesian aquifer provides adequate amounts of potable water to individual rural users as well as municipal systems. No significant cones of depression exist in the region.

2.5.1.2 Site Geology

2.5.1.2.1 Site Physiography (HNP-1 and HNP-2)

The site is within the Coastal Terraces subprovince of the Atlantic Coastal Plain physiographic province. The Coastal Terraces subprovince consists of at least seven terraces arranged in belts parallel to the Atlantic coast and extending from the Fall Line to the ocean. The nearly flat terrace surfaces slope gently seaward, although over 100 ft of relief may be developed near major stream valleys traversing the terraces. The terrace surfaces near the site are the Brandywine and Coharie Terraces, which are underlain by sandy clay and clayey sand of Pliocene(?) to early Pleistocene ages. Over most of the site, the terrace surfaces have been destroyed by fluvial processes of the Altamaha River and its local tributaries. As a result, the southern part of the site occupies a gentle, dissected slope between the terraces to the south and the Altamaha river valley to the north, while the northern and eastern parts of the site are within the nearly flat Altamaha River flood plain.

2.5.1.2.2 Site Geologic Conditions (HNP-1 and HNP-2)

During the initial geologic reconnaissance prior to preparation of the HNP-2 Preliminary Safety Analysis Report (PSAR), two areas of possible faulting were postulated to exist within 3 miles of the plant site. These areas, shown in figure 2.5-19, are located ~ 2 miles south of the site near Bay Creek in Appling County and ~ 2.5 miles northeast of the site in southern Toombs County. Subsequent investigations have concluded that no faulting exists in these areas. Observations cited as evidence for this faulting are more representative of deltaic and fluvial processes than of tectonic or structural origin.

The initial reconnaissance consisted of mapping the outcropping sediments at road cuts along all primary and many secondary roads within 20 miles of the site. Exposures of the contact between the Brandywine and Hawthorn Formations were most prevalent. The contact between lithologic units 6 and 5 of the Hawthorn Formation is exposed in the deeper road cuts and near stream valleys. The lithologies of these three sedimentary units are similar, thereby making picks of the contacts somewhat subjective at the widely spaced outcrops.

No displacement of the contacts was found at any of the individual outcrops. However, the contact between lithologic units 6 and 5 of the Hawthorn Formation north of Bay Creek was estimated to be 6 ft higher than the same contact in an exposure south of Bay Creek. Distance between the exposures is ~ 1300 ft. An aerial photograph of the site vicinity shows a possible short lineament along the trace of Bay Creek near the two above-mentioned outcrops. On the basis of these two observations, a fault was proposed to displace the surface material parallel to the trend of Bay Creek.

To verify the existence of this proposed fault, two boreholes were drilled north and south of Bay Creek. These boreholes, labeled D and F are shown in figure 2.5-20, and the drill logs are shown in figure 2.5-21. They are ~ 1700 ft apart and on opposite sides of the postulated fault trace. The boreholes penetrated through the fluival and deltaic facies of the Hawthorn Formation into the Tampa Formation. The erosional surface on top of the shallow marine silty clay of the Tampa Formation provided a well defined local marker bed along which any displacement could be detected and measured. Additional correlation was sought by gamma, resistivity, and self-potential logging of the boreholes. Both the lithologic and geophysical logs revealed that the top of the Tampa Formation was at el -130 msl in boreholes D and -129 ft msl in borehole F. This difference in elevation is negligible and compares favorably with the 15 ft of erosional relief found on the same surface in foundation borings in the plant area. Thus, based on subsurface information along the trace of the proposed fault, no offset of stratigraphic units is indicated.

Surface evidence of faulting along Bay Creek is likewise lacking. The 6 ft of relief on the unit 6 - unit 5 contact between exposures north and south of Bay Creek, mentioned above as evidence for faulting along Bay Creek, represents a southeastward dip of ~ 24 ft/mile. The regional dip of coastal plain strata in Georgia ranges between 5 and 60 ft/mile.[8] The relief at Bay Creek is therefore not anomalous when placed in the context of the regional coastal plain dip. Fluvial erosion of lithologic unit 5 prior to or contemporaneous with deposition of unit 6 explains some relief along the exposed contact. It is concluded that no surface or subsurface evidence exists to support faulting along Bay Creek.

Differences in the elevation of the Brandywine-Hawthorn contact between widely spaced exposures in southeastern Toombs County were suggested to be caused by faulting. Exposures of the contact are located along County Road 107 (figure 2.5-19) and discontinuously extend up to 6 miles northeast of the site. The examined exposures are more prevalent near numerous stream valleys, although a few road cuts not covered by vegetation also exist. Distances between the contact exposures investigated during the initial reconnaissance ranged from 0.2 to 1.4 miles. The maximum relief of the contact was found to be 17 ft between exposures ~ 0.45 miles apart. No displacements of the contact were found in the individual exposures. However, it was postulated that faulting between the exposures could have caused at least some of the relief on the contact.

A subsequent field investigation was performed to determine the location of the fault or faults thought responsible for the relief. The Brandywine-Hawthorn contact was traced continuously along the slopes of the divides separating the streams crossed by County Road 107. This traverse provided an unbroken

profile across the area for which faulting has been postulated. No displacements of the Brandywine-Hawthorn contact were found during the investigation. It is concluded that there is no evidence to support faulting in the area and that fluvial erosion was responsible for any local relief encountered along the Brandywine-Hawthorn contact.

The geologic conditions at the site are typical of the geology of the Atlantic Coastal Plain province. Deep borings 5 miles west of the site in Appling County and 10 miles north of the site in Toombs County indicate that pre-Cretaceous basement rock underlying the site consists of arkosic sandstone, and basalt or diabase. These lithologies are similar to Triassic age rocks found elsewhere along the Atlantic seaboard.(7) Overlying the basement rocks are relatively unconsolidated sedimentary units ranging in age from Early Cretaceous to Holocene. These units dip southward and southeastward at 5 to 50 ft/mile and thicken downdip. Sea level fluctuations resulted in erosion of some of the units after their deposition. Moderate relief was developed during low sea level stands and before deposition of the next stratigraphic sequence. The only structural feature near the site is the Southeast Georgia embayment, the inland edge of which is 11 miles east of the site. (See paragraph 2.5.1.1.6.1.) The embayment has had little or no influence on geologic formations underlying the site.

Materials from the following geologic units, listed from oldest to youngest, were found in geologic and foundation borings drilled at the site: Tampa and Hawthorn Formations of Miocene to Pliocene(?) ages; Brandywine terrace deposits of Pliocene(?) to Pleistocene ages; and alluvium of late Pleistocene and Holocene ages. The closest oil test well to the site (Appling County, Georgia Geological Survey 148), drilled about 5 miles west of the site, penetrated below the Tampa Formation into the following units, listed from oldest to youngest:(26)

- *Undifferentiated deposits of Early Cretaceous age.*
- *Tuscaloosa Formation and post-Tuscaloosa undifferentiated deposits of Late Cretaceous age.*
- *Clayton Formation of Paleocene age.*
- *Early Eocene (Wilcox) deposits.*
- *Tallahatta and Lisbon Formations of middle Eocene (Claiborne) age.*
- *Ocala Formation of late Eocene (Jackson) age.*
- *Undifferentiated carbonates (probably Suwanee Formation) of Oligocene age.*

The oil test well is along the strike of the coastal plain strata from the site.

The Cretaceous Formations represent a transgressive sequence, characterized by continental and deltaic deposition of the Lower Cretaceous undifferentiated and the Tuscaloosa Formation and by lagoonal to marginal marine deposition of the post-Tuscaloosa deposits. The Lower Cretaceous strata, which are probably correlative with the Comanche series found westward in the Coastal Plain province,(8) consist of sandy, micaceous clays interbedded with arkosic sands. They rest unconformably on basement rock of possible Triassic age(7) and are ~ 115 ft thick. The top of the Lower Cretaceous strata in the site vicinity

is at el -3731 ft msl. (Elevations below the Tampa Formation refer to Well Georgia Geological Survey 148.) The overlying Tuscaloosa Formation consists of ~ 910 ft of fine-grained to arkosic, carbonaceous, fossiliferous, and micaceous sand and clay. The top of the unit is at el -2821 ft msl. Post-Tuscaloosa deposits are ~ 995 ft thick[26] and are equivalent to Eutaw and Selma group deposits found in Alabama.[8] They consist of coquinoid and phosphatic sand and carbonaceous, fossiliferous, glauconitic marl. The top of these strata is at el -1866 ft msl.[26] Contacts between the preceding formations are generally conformable and somewhat indistinct, owing to the similar fossils (where present) and lithologies.[8]

The widespread, post-Cretaceous sea level regression noted elsewhere in the Gulf and Atlantic Coastal Plains[9] is represented at the site by an erosion surface developed on top of the post-Tuscaloosa deposits. The Tertiary Formations underlying the site are typical of shallow marine deposition during high sea levels and deltaic to marginal marine deposition during lower sea levels.

The basal Tertiary Formation in the site area is the Clayton Formation of Paleocene age.[8] It consists of massive crystalline limestone and carbonaceous, marly sand. Throughout the Coastal Plain provinces, the Clayton rests unconformably on the underlying Cretaceous age deposits. The unit is ~ 315 ft thick, with the top at el -1551 ft msl.[26]

Undifferentiated Wilcox Group deposits of early Eocene age represent deposition in a shallow marine environment. The Wilcox deposits underlying the site consist of ~ 90 ft of carbonaceous, micaceous, silty fossiliferous marl. The top of the unit is at el -1461 ft msl.[26] This material is correlative with the upper part of the Hatchetigbee Formation found westward in the Coastal Plain province.[8] In the site area, the early Eocene sea was restricted in extent, as indicated by the absence of extensive lower Eocene strata that are found elsewhere in the southeast.[1]

The Tallahatta and Lisbon Formations comprise the middle Eocene Claiborne Group underlying the site. The Tallahatta is ~ 160 ft thick and consists of glauconitic sand and thin stringers of fossiliferous marl. The top of the formation is at el -1301 ft msl. The Lisbon Formation consists of 610 ft of dolomitic to sandy, phosphatic limestone, with abundant glauconite and fossils. The top of the Lisbon is at el -691 ft msl.[26] Both of these formations were deposited in a shallow marine environment that was typical of the middle Eocene in the Coastal Plain provinces.[1]

The late Eocene is represented by the Ocala Formation of the Jackson Group. The Ocala is below el -411 ft msl and is 280 ft thick. It consists of crystalline, massive, extremely fossiliferous limestone that was deposited in a shallow marine environment.[26]

Material of Oligocene age underlying the site is equivalent to the upper part of the Vicksburg Group found in the Gulf Coastal Plain and to the Suwannee limestone found in Florida.[8] The Oligocene undifferentiated in Appling County consists of massive, calcitized, fossiliferous limestone deposited in a shallow sea. The unit is 120 ft thick, and the top is at el -291 ft msl.[26]

Miocene to Pliocene(?) age material at the site, penetrated by site borings, represents a regressive sequence, characterized by shallow marine limestone of the Tampa Formation overlain by marginal marine to fluvial deposits of the Hawthorn Formation. The Tampa Formation of Miocene age is ~ 160 ft thick, with a top at approximate el -130 ft msl. It consists of sandy to clayey, phosphatic, fossiliferous, and somewhat dolomitized limestone. The Hawthorn Formation of Miocene to Pliocene(?) age consists of six distinct lithologic units. The basal unit is probably lagoonal in origin, ~ 115 ft thick, and contains phosphatic, carbonaceous, well-sorted fine sand and sandy clay. The second unit is about 20 ft thick and

consists of phosphatic, fine to coarse sand, and silty clay that is calcareous in the bottom 10 ft, with abundant shell fragments. The overlying third unit, ~ 25 to 35 ft thick, contains phosphatic, clayey, fine to medium sand with some pyrite. The fourth unit is similar, but contains less phosphate and no pyrite. It is ~ 40 to 50 ft thick. The fifth unit is 20 to 40 ft thick and consists of micaceous and feldspathic fine to coarse sand and sandy clay that is locally cemented. These last three units were deposited in progressively shallower environments, ranging from estuarine to deltaic. The top unit of the Hawthorn Formation is fluvial in origin. It consists of over 50 ft of arkosic clayey sand, sandy clay, and gravel, with abundant cross-bedding. Total thickness of the Hawthorn Formation is over 300 ft.[(26)] The Hawthorn Formation has recently been reclassified as Neogene Undifferentiated by the Georgia Geological Survey; the interval includes Miocene to Pliocene ages.

In the southwestern part of the site, the Brandywine Formation of Pliocene(?) to Pleistocene age is locally exposed above approximate el 165 ft msl. The Brandywine is a fluvial to deltaic deposit consisting of poorly sorted, feldspathic, cross-bedded sand and gravel with abundant hematite concretions. The northern and eastern parts of the site are covered with alluvium associated with the Altamaha River. The flood plain deposits may be Pleistocene age depth, but the surface material is Holocene in age.[(30)] The alluvium consists of poorly sorted sand and gravel and carbonaceous silty clay.

A site geologic column showing the relationship between formations underlying the site is presented in figure 2.5-8. The regional geologic column is shown in figure 2.5-4. Logs of all site borings are presented in supplement 2B.

2.5.1.2.3 Site Structural Geology (HNP-1 and HNP-2)

The Southeast Georgia embayment is the only known structural feature in the vicinity of the site. The inner edge of the embayment is ~ 11 miles east of the site. Material underlying the site was not affected by subsidence of the basin. Details of the Southeast Georgia embayment are included in paragraph 2.5.1.1.6.1, Description of Tectonic Structures.

A site structural geology map showing contours on top of unit 5 of the Hawthorn Formation is presented in figure 2.5-9. This unit is the foundation for the plant reactor. The bedrock contours indicate that although erosion of the unit occurred before deposition of the overlying strata, there has been no structural deformation of the bearing stratum.

2.5.1.2.4 Site Geologic Map (HNP-1 and HNP-2)

A geologic map of the site is presented in figure 2.5-10. This map shows the locations of Category I structures and the known and inferred contacts between materials exposed at the site. An areal geologic map is shown in figure 2.5-11.

2.5.1.2.5 Site Geologic History (HNP-1 and HNP-2)

The geologic history of the site is closely allied with the geologic history of the Atlantic Coastal Plain province. Underlying the site are rocks of Triassic to Holocene ages (except Jurassic) that extend to or

have equivalents in other areas of the Atlantic and Gulf Coastal Plains. Structural features found elsewhere in the Coastal Plain provinces have not influenced the geology of the site.

Following erosion of the Appalachian belt Paleozoic rocks, Triassic continental deposits and igneous rocks were laid down in inland basins or graben-faulted areas. During the Jurassic, the Triassic rocks were eroded. Deposition of Jurassic strata did not occur in the area of the site.[(1)]

Initial deposition of marine and fluvial coastal plain sediments in the vicinity of the site occurred in the Cretaceous. Undifferentiated Lower Cretaceous strata, probably equivalent to the Comanche series, were deposited on the eroded Triassic deposits. As the Late Cretaceous sea transgressed inland, a sequence of fluvial to shallow marine material, represented by the Tuscaloosa Formation and post-Tuscaloosa undifferentiated, was deposited. A period of erosion marked the end of the Cretaceous.[(1)]

Tertiary depositional patterns were determined by cyclic advances and retreats of the sea and relative subsidence of areas marginal to the present coast. As the sea spread inland, near short to moderately deep marine deposits (predominantly carbonates) were laid down on the eroded surface of older units. The greatest thickness of material was deposited in slowly subsiding basins, such as the Southeast Georgia embayment. As the sea retreated, newly deposited material was eroded. This pattern of transgression and deposition followed by regression and erosion was repeated throughout the Tertiary and continued into the Quaternary. During the Pleistocene, the sea encroached progressively less on the Coastal Plain province, with earlier deposited material, such as the Brandywine Terrace deposits, extending further inland and higher than younger material. In the vicinity of the site, the terrace deposits were partially eroded by the Altamaha River and its local tributaries. The Altamaha River flood plain has been developed during the late Pleistocene and Holocene.

The characteristics of each formation found at the site are included in paragraph 2.5.1.2.2, Site Geologic Conditions. A site geologic column is shown in figure 2.5-8.

2.5.1.2.6 Plot Plan (HNP-1 and HNP-2)

Information concerning the locations of major structures of the plant, including all Category I structures, and borings made at the site are presented in figures 2A-1 and 2A-2. The graphic logs of the borings are shown in figures 2B-1 through 2B-112. These figures, along with a discussion of findings, are presented in supplements 2A and 2B.

2.5.1.2.7 Geologic Profiles and Plant Foundations (HNP-1 and HNP-2)

The relationship of the major foundations to subsurface materials is presented in the form of generalized subsurface profiles shown in figure 2A-3. The ground water conditions are discussed in subsection 2.4.13. The significant engineering characteristics of the subsurface materials are discussed in supplement 2A. All Category I buildings are founded on cemented sand or silty sand of the Hawthorn Formation.

2.5.1.2.8 Excavations and Backfill

The methods of excavation and compaction of fills are discussed in section 2A-8 of supplement 2A. The plant area excavation plan and sections are shown in figure 2A-37. The compaction criterion is 95% (minimum) of the maximum dry density as determined by American Society of Testing Materials D 1557 (Modified Proctor).

2.5.1.2.9 Evaluation of Local Engineering Geology (HNP-1 and HNP-2)

2.5.1.2.9.1 Prior Earthquake Effects*. There is no evidence to suggest that surficial or subsurface materials at the site have been affected by prior earthquake activity. No fault planes were penetrated by the numerous site borings or exposed in any of the excavations. The tops of formations and beds within formations have not been offset by faulting or slumping. The steep slopes between the plant area and the Altamaha River flood plain are not marked by slumps. Streams courses are not offset along any lineations associated with structural features. No topographic features can be attributed to seismic activity. Earthquake activity apparently has had no effect on the materials at the site.*

2.5.1.2.9.2 Deformational Zones*. Inspection of outcrops, excavations, and subsurface samples of the Hawthorn Formation, which is the foundation material for the major plant structures, has revealed that there are no deformational zones within Hawthorn material. There are no reversals of dip of the Hawthorn Formation in the vicinity of the site. Exposures of the Hawthorn do not contain joints or fractures. Core samples of Hawthorn material at the site do not exhibit shear zones or fractures.*

2.5.1.2.9.3 Zones of Alteration or Weakness*. The foundation material (Hawthorn Formation) has not been altered by chemical weathering. By contrast, local cementation of the upper part of the Hawthorn has occurred. The top of the Hawthorn was eroded prior to deposition of the Brandywine terrace deposits, effectively stripping any weathering profile that had developed. Calcareous material underlying the Hawthorn is well crystallized and shows no evidence of solution. There are no surface sinkholes indicating solution of underlying material. There are no zones of structural weakness composed of crushed or disturbed materials underlying the site.*

2.5.1.2.9.4 Bedrock Stress*. Over 4000 ft of relatively unconsolidated coastal plain deposits overlie bedrock of possible Triassic age beneath the site. Therefore, bedrock stresses are not applicable in considering the design and operation of the plant.*

2.5.1.2.9.5 Potentially Unstable Soils*. Numerous field and laboratory tests indicate that there are no potentially unstable soils at the site under any of the plant structures. The characteristics of the soils underlying the site are discussed in detail in supplement 2A.*

There are no soils or rocks under any plant structures that have potentially undesirable characteristics which would cause them to respond adversely to expected seismic events. Sandy overburned soils were

analyzed for liquefaction potential and found not susceptible to liquefaction. (See subsection 2A.5.2 of supplement 2A.)

2.5.1.2.9.6 ***Effects of Man's Activities****. The effects of man's activities on geologic conditions at the site are discussed in paragraph 2.5.1.1.6.2, Areas of Potential Instability. There are no mining or mineral extraction activities occurring near the site, and ground water extraction is nominal in this area of low population. Therefore, there are no human activities that will affect site geologic conditions.*

2.5.1.2.10 Site Ground Water Conditions (HNP-1 and HNP-2)

Site ground water conditions are described in detail in paragraphs 2.4.13.1.2 and 2.4.13.2.2. In general, the site is underlain by a shallow unconfined aquifer and a deeper lying, minor confined aquifer. These aquifers are separated by a layer of silty cemented sand ~ 40 to 50 ft thick that forms an effective aquiclude, preventing migration of water from one aquifer to the other. The unconfined aquifer lies above el 100 to 120 ft msl in the plant area, with the unconfined water table generally reflecting the site topography. The minor confined aquifer consists of silty sands of the Hawthorn Formation between approximate el 65 and 0 ft msl. Piezometric levels generally are below el 80 ft msl, and the potentiometric surface slopes northeastward toward the Altamaha River. The river is hydraulically connected to the two aquifers.

Present and projected usage of ground water in the vicinity of the site, discussed in paragraphs 2.4.13.2.1 and 2.4.13.2.5 will not affect the present ground water conditions. Ground water for plant usage is withdrawn from a deep, confined aquifer (at a maximum rate of 327 gal/min) that is not hydraulically connected with aquifers utilized by most offsite domestic wells.

2.5.1.2.11 Geophysical Survey Results (HNP-1 and HNP-2)

Results of geophysical surveys conducted at the site are shown in figures 2A-5 and 2A-6. A discussion of the geophysical exploration and the results are included in subsection 2A.1.4.

2.5.1.2.12 Static and Dynamic Properties (HNP-1 and HNP-2)

The static and dynamic properties of the site materials are discussed in section 2A.3 (Laboratory Testing) of supplement 2A. The results of the laboratory testing are presented in tables 2A-1 and 2A-5.

Laboratory testing, procedures, and classification procedures are discussed in section 2A.3. Grain-size classification is presented in table 2A-1 and figure 2A-8 and discussed in subsection 2A.3.1 of supplement 2A.

Consolidation characteristics are presented in figures 2A-9 and 2A-10 and table 2A-4 and discussed in subsection 2A.3.3 of supplement 2A.

In situ moisture content is shown in table 2A-2. Atterberg limits are shown in table 2A-1. Triaxial shear test data are presented in table 2A-5 and the test results are presented in figure 2A-11. Cyclic triaxial testing is discussed in subsection 2A.3.5 with the results of testing presented in figures 2A-12 and 2A-13.

The geophysical explorations performed are discussed in subsection 2A.1.4 of supplement 2A.

2.5.1.2.13 Safety Criteria and Analysis Techniques

The foundation conditions provide the safe support of all structures. The safety criteria and methods of analysis are discussed in detail in supplement 2A, section 2A.5.

2.5.2 VIBRATORY GROUND MOTION

2.5.2.1 Site Geologic Conditions

The lithologic, stratigraphic, and structural geologic conditions at the site, including the geologic history, are discussed in paragraph 2.5.1.1.5, Regional Geologic Conditions; paragraph 2.5.1.2.2, Site Geologic Conditions; paragraph 2.5.1.2.3, Site Structural Geology; and paragraph 2.5.1.2.5, Site Geologic History.

Over 4000 ft of coastal plain sediments, ranging in age from Cretaceous to Holocene, underlie the site. These sediments consist of sand, clay, marl, sandstone, shale, and limestone. No structural features affect materials underlying the site.

The Hawthorn Formation of Miocene to Pliocene(?) age is the bearing stratum for the plant Category I structures. It consists of cemented sand, silty clay, and silty sand with clay layers. Engineering properties of the Hawthorn Formation are discussed in supplement 2A.

2.5.2.2 Underlying Tectonic Structures

Tectonic structures underlying the region surrounding the site are discussed in paragraph 2.5.1.1.6.1, Description of Tectonic Structures, and are shown in figure 2.5-3. There are no known or suspected structural features within 11 miles of the site. Structures in the coastal plain within 200 miles of the site have not been active since the end of Miocene time. Therefore, no structural features are of significance to the site.

2.5.2.3 Behavior During Prior Earthquakes

The effects of prior earthquakes on materials at the site are discussed in paragraph 2.5.1.2.9.1, Prior Earthquake Effects. Earthquake activity has apparently had no effect on site materials.

2.5.2.4 Engineering Properties of Site Materials

The properties of the materials underlying the site are discussed in detail in supplement 2A. The locations of the seismic traverse lines are shown in figure 2A-4. The compressional, shear, and Rayleigh waves are shown in figure 2A-5 and 2A-6. Geophysical exploration results are discussed in subsection 2A.1.4. Boring logs are presented in supplement 2B.

In situ moisture content is shown in table 2A-2. Triaxial shear test data are presented in table 2A-5 and the test results are presented in figure 2A-11. Cyclic loading is discussed in subsection 2A.3.5, with the result of testing presented in figures 2A-12 and 2A-13.

2.5.2.5 Earthquake History

The site is within a broad region of infrequent seismic activity encompassing southern Alabama, southern Georgia, and northern Florida. Figure 2.5-12, Seismic Risk Map of the United States, shows that the site is in zone 1, an inactive seismic region characterized by a few low-magnitude and low-intensity shocks. The nearest zone 2 area is about 55 miles away, and the nearest zone 3 is about 90 miles distant. Zone 1 is described as follows: "Minor damage; distant earthquakes may cause damage to structures with fundamental periods greater than 1.0 seconds; corresponds to intensities V and VI of the MM Scale" (Modified Mercalli Intensity Scale of 1931, table 2.5-1).

Table 2.5-2 is a list of historically reported earthquakes having epicenters within 200 miles of the site and epicentral intensities of IV (MM) or greater. The locations of these events are shown in figure 2.5-13, Tectonic and Epicenter Map. Of the 76 earthquakes listed, 53 had epicenters within the Coastal Plain province, including one in the Atlantic Ocean. Thirty of the coastal plain events were confined to the vicinity of Charleston, South Carolina. No earthquakes occurring in the coastal plain outside of the Charleston area had an epicentral intensity greater than VI (MM). (See references 31, 32, 33, and 34.)

A review of the literature indicates that none of the 76 events had damaging effects within the plant vicinity or in Baxley, Georgia, ~ 10 miles to the south. Isoseismal and felt area reports indicate that the site area sustained its largest intensity during the 1886 Charleston event that had an epicentral intensity of X. The site intensity during that event was probably VI (MM).[35][36] Estimates of ground acceleration and duration of shaking for the site during the earthquake are included in paragraph 2.5.2.9, Maximum Earthquake.

2.5.2.6 Correlation of Epicenters with Geologic Structures

Although a few epicenters are near major faults shown in figure 2.5-13, the many thrust and normal faults that have been mapped in the Appalachian provinces do not have a record of surface breakage during historic times.[36] Ten events have occurred on the coastal margin of the Southeast Georgia embayment in the Coastal Plain province. Regional cross sections through the embayment show no evidence of faulting.[14] Since the epicenters listed in table 2.5-2 cannot be reasonably associated with geologic structures, they are assigned to tectonic provinces. Boundaries of the tectonic provinces are discussed in paragraph 2.5.1.1, Regional Geology. It should be noted that the high seismicity in the vicinity of Charleston, South Carolina, is not typical of the generally quiescent Coastal Plain province. (See table 2.5-2 and figure 2.5-13.)

2.5.2.7 Identification of Active Faults

No faults in the Coastal Plain province within 200 miles of the site are considered active. A discussion of coastal plain faults is included in paragraph 2.5.1.1.6.1, Description of Tectonic Structures.

Faults within the Appalachian Mountain System are over 90 miles from the site and are in a separate tectonic province. These faults (figure 2.5-13) do not exhibit evidence of surface displacement during historic time.[36] If movement along these faults at depth has caused earthquakes in historic times, none has had an epicentral intensity greater than VII-VIII (MM) (table 2.5-2); and the intensity at the site would be much lower. They are therefore not significant in establishing the SSE.

2.5.2.8 Description of Active Faults

No active faults exist within 200 miles of the site. (See paragraph 2.5.2.7, Identification of Active Faults).

2.5.2.9 Maximum Earthquake

The highest intensity sustained in the vicinity of the site during historic times resulted from the August 31, 1886, Charleston, South Carolina event. The earthquake was centered about 160 miles northeast of the site and had an epicentral intensity of X (MM). The shock lasted 35 to 40 s in the epicentral area.[31] The intensity in the area of the site, as shown in Dutton's report, was VII (Rossi-Forel scale).[35] This corresponds to middle VI on the Modified Mercalli scale.[37] The reports upon which Dutton based his isoseismal map, shown in figure 2.5-14, indicate that the middle intensity VI (MM) in the vicinity of the site is a maximum value. The nearest reported damage from the 1886 event occurred in the Savannah, Georgia, area about 70 miles east of the site, where the intensity was VI to VII (MM).[35] Savannah is about 90 miles from the epicenter, or about 70 miles closer than the site. The site intensity of middle VI (MM) corresponds to a horizontal surface acceleration of 0.064 g on Neumann's (1954) curve and 0.047 g on Hershberger's (1956) curve.

2.5.2.10 Design Basis Earthquake

For the DBE an intensity of VII (MM) is selected. This intensity corresponds with the highest damage sustained at Savannah, Georgia, during the 1886 Charleston event. Outside of the anomolous Charleston seismic area, no events within 200 miles of the site in the coastal plain have had epicentral intensities higher than VI (MM). It is unlikely that an intensity VII (MM) event has ever been felt at the site. An intensity of VII (MM) is equivalent to 0.12 g on both the Neumann (1954) and Hershberger (1956) curves. However, a horizontal surface acceleration of 0.15 g is conservatively selected for the DBE. The selected maximum vertical acceleration is two-thirds the maximum horizontal acceleration.

A detailed discussion of response spectra, damping factors, and time history accelerogram is presented in section 3.7.

2.5.2.11 Operating Basis Earthquake (OBE)

The highest intensity felt at the site was VI (MM). This corresponds to an OBE with a horizontal surface acceleration of 0.064 g on Neumann's (1954) curve and 0.047 g on Hershberger's (1956) curve.

For conservatism, a value of 0.08 g is selected for the OBE. The selected maximum vertical surface acceleration is two-thirds the maximum horizontal acceleration. Section 3.7 provides a discussion of the response spectra for the OBE.

2.5.3 SURFACE FAULTING

There are no active faults within 200 miles of the site. (See paragraph 2.5.1.1.6.1.) The nearest occurrence of known surface faulting is the Andersonville fault, 95 miles west of the site, which has been inactive since middle Eocene time.[22] There is no surface faulting in the vicinity of the site; therefore, it is not necessary to design the plant for surface faulting.

2.5.3.1 Geologic Conditions of the Site

The lithologic, stratigraphic, and structural geologic conditions of the site and vicinity, including geologic history, are discussed in paragraph 2.5.1.1.5, Regional Geologic Conditions; paragraph 2.5.1.2.2, Site Geologic Conditions; paragraph 2.5.1.2.3, Site Structural Geology; and paragraph 2.5.1.2.5, Site Geologic History.

2.5.3.2 Evidence of Fault Offset

Pertinent publications, geologic investigations in the site vicinity, and investigations of construction excavations indicate that there is no fault offset at or near the ground surface in the vicinity of the site.

2.5.3.3 Identification of Active Faults

No faults in the Coastal Plain provinces within 200 miles of the site are considered active. A discussion of coastal plain faults is included in paragraph 2.5.1.1.6.1, Description of Tectonic Structures.

2.5.3.4 Earthquakes Associated With Active Faults

None of the earthquakes that have had epicenters within 200 miles of the site can be reasonably associated with active faults. Earthquakes in this area are listed in table 2.5-2, and none is associated with faults within 5 miles of the site.

2.5.3.5 Correlation of Epicenters With Active Faults

No epicenters of historically reported earthquakes can be correlated with active faults within 5 miles of the site. There are no active faults in the site vicinity.

2.5.3.6 Description of Active Faults

There are no active faults within 5 miles of the site. A discussion of coastal plain faults is included in paragraph 2.5.1.1.6.1, Description of Tectonic Structures.

2.5.3.7 Faulting Investigation Zone

Published reports of the site area and geologic investigations at and near the site indicate that the area contains no faults. The coastal plain strata dip southeastward, with no reversals of dip or offset in the beds. A detailed faulting investigation was not required.

2.5.3.8 Justification for Nonexistence of Surface Faulting

Data presented in paragraph 2.5.1.1.6.1, Description of Tectonic Structures; paragraph 2.5.1.2.3, Site Structural Geology; and in figure 2.5-11, Areal Geologic Map, indicate that no faulting exists in the vicinity of the site. No surface offsets were found in geologic investigations at the site or in the area surrounding the site. Coastal plain strata in southeastern Georgia dip southeastward at 5 to 50 ft/mile, with the dip increasing with depth. No fault planes were penetrated by site geologic or foundation borings, nor were structural offsets indicated in the area between borings. Published reports on the geology of the area do not present any information on surface faults. It is concluded that surface faulting is not present in the site area and does not require further consideration.

2.5.4 STABILITY OF SUBSURFACE MATERIALS

Information presented in this section concerns the stability of soils and rock beneath the plant foundations during the vibratory ground motion associated with the DBE. In general, this information is included in section 2.5 and supplement 2A and is cross-referenced to appropriate subsections.

2.5.4.1 Geologic Features

2.5.4.1.1 Areas of Potential Instability

A discussion of areas of actual or potential surface or subsurface subsidence, uplift, or collapse is included in paragraph 2.5.1.1.6.2, Areas of Potential Instability. The plant foundations will not be affected by movement in the areas discussed. No areas of potential surface or subsurface subsidence exist at the plant site.

2.5.4.1.2 Deformational Zones

The site foundation material does not contain deformational zones of any kind. A discussion of the foundation material with respect to zones of deformation is included in paragraph 2.5.1.2.9.2, Deformational Zones.

2.5.4.1.3 Zones of Alteration or Weakness

Paragraph 2.5.1.2.9.3, Zones of Alteration or Weakness, contains a discussion of these aspects of materials underlying the site. There are no altered or weak zones in the site foundation materials.

2.5.4.1.4 Bedrock Stress

Over 4000 ft of coastal plain deposits overlie bedrock of possible Triassic age beneath the site.[7] *The Triassic rock was eroded before deposition of the Cretaceous and Cenozoic sedimentary deposits. It is unlikely that unrelieved residual stresses exist in the bedrock. Major deformation has not occurred since Triassic time.*[1]

2.5.4.1.5 Potentially Unstable Soils

The characteristics of soils underlying the site are discussed in supplement 2A. There are no soils or rocks under any plant structures that have potentially undesirable responses to seismic events.

2.5.4.2 Properties of Underlying Materials

(HNP-2) subsurface conditions were investigated by the following borings:

- *Reactor and radwaste buildings by 8 borings numbered 587 through 594 and one numbered 401 drilled during initial investigations.*

- *The turbine building by 6 borings numbered 596 through 600.*

The conditions were verified by borings RFI-1 through RFI-10 (figures 2B-113 through 2B-122) which were performed as part of the foundation inspection of HNP-2.

The borings were made with a rotary wash boring process which utilized a heavy viscous drilling fluid to stabilize the sides and bottom of the drill holes. Standard penetration tests were made in accordance with ASTM Specification D1586-67 at 5-ft intervals throughout the borings. The standard penetration test samples were inspected, classified, and used as the basis for selecting depths to obtain 3-in.-diameter undisturbed Shelby-Tube-type samples.

The area of the powerblock initially varied in elevation from about el 135 to 145 ft. General site grading lowered this area to ~ el 130 ft. Excavation for HNP-1 extended to ~ el 75 ft and extended southward to ~ the centerline of the HNP-2 reactor. The area excavated was covered with a reinforced concrete working slab which varied in thickness from 8 in. to 1 ft. Borings for HNP-2 were made from both the level of the excavation for the HNP-1 powerblock and the general elevation of the yard area.

The soil conditions determined by the borings in the HNP-2 powerhouse area are generally consistent with the soil conditions determined by the borings in the HNP-1 powerblock area.

The borings made from the general yard level initially encountered firm to dense, multi-colored, clayey, fine to medium sands and very stiff to hard, fine, sandy clays extending downward to ~ el 120 ft.

These sands and clays are underlain by very dense, gray, clayey, fine to medium sand which in most locations, is partially cemented. Within this generally cemented sand zone are scattered layers and inclusions of very hard clay and very dense uncemented sands. These sands extend to ~ el 75 ft.

The cemented sands are underlain by firm to very dense, gray-green fine sands and clayey fine sands which extend to ~ el 30 ft. Within this zone, thin layers of lenses of gray-green plastic clay, which vary in thickness from 3 to 6 ft, were encountered from el 60 to 70 ft. Below el 30 ft, dense to very dense, gray, slightly clayey, fine sands with thin, hard, clay layers were encountered. The dense sands extend to ~ el 0 ft. Below el 0 ft, very hard, gray-green, silty clays were encountered.

The properties of the underlying materials are discussed in detail in supplement 2A. Subsurface profiles in the HNP-2 powerblock are shown in figure 2A-3.

Laboratory testing procedures and classification procedures are discussed in section 2A.3. Grain size classification is presented in table 2A-1 and figure 2A-8 and discussed in subsection 2A.3.1 of supplement 2A. Atterberg limits test results are shown in table 2A-1.

Consolidation characteristics are presented in figures 2A-9 and 2A-10 and table 2A-4 and discussed in subsection 2A.3.3 of supplement 2A.

In situ moisture contents are shown in table 2A-2. Triaxial shear test data are presented in table 2A-5, and the test results are presented in figure 2A-11. Triaxial testing is discussed in subsection 2A.3.5, with the results of testing presented in figures 2A-12 and 2A-13.

The geophysical exploration performed is discussed in subsection 2A.1.4 of supplement 2A.

2.5.4.3 Plot Plan

Information concerning the locations of major structures of the plant, including all Category I structures, and borings made at the site are presented in figures 2A-1 and 2A-2. These figures, along with discussion of findings, are presented in detail in section 2A.2 of supplement 2A. The logs of the borings are included in supplement 2B.

The locations of seismic traverse lines are shown in figure 2A-4; the subsurface profiles in the powerblock area are shown in figure 2A-3; and a surface distribution of geologic formations is shown in figure 2A-7 of supplement 2A. Regional geologic profiles are shown in figures 2.5-5 and 2.5-6. A plot of the piezometer locations is presented in figure 2.4-37.

2.5.4.4 Soil and Rock Characteristics

The site subsurface conditions and the soil characteristics are discussed in detail in supplement 2A. The locations of the seismic traverse lines are shown in figures 2A-4. The compressional, shear, and Rayleigh wave velocities are shown in figures 2A-5 and 2A-6. Geophysical exploration results are discussed in subsection 2A.1.4. Boring logs are presented in supplement 2B.

2.5.4.5 Excavations and Backfill

The plant area excavation plan and sections are presented in figure 2A-37. A discussion of excavation and backfill is presented in section 2A.8.

2.5.4.6 Ground Water Conditions

Hydrology and ground water conditions are described and discussed in detail in section 2.4. Where plant excavation occurs below the water table, dry and stable foundation conditions were maintained during construction by conventional dewatering methods.

Permanently sealed piezometers have been installed around the exterior of the HNP-1 powerblock area. After the installation of these piezometers, continuous dewatering of the foundation soils was accomplished by eductor well points. The piezometer locations are shown in figure 2.4-37. This dewatering has resulted in water levels within the permanent piezometers which vary from ~ el 50 to 80 ft. During 1974, the water levels within the different permanent piezometers ranged from el 70 to

78 ft. It is anticipated that the stabilized ground water level in the (HNP-1) and (HNP-2) powerblock areas will coincide with approximate el 70 to 75 ft.

2.5.4.7 Dynamic Loading Response

Cyclic triaxial test results and the dynamic response of site soils are discussed in subsection 2A.3.5 of supplement 2A. The responses of specimens of representative site soils indicate that there should be no adverse effects on the site soils under dynamic loading. The result of the cyclic triaxial tests are presented in figures 2A-12 and 2A-13.

2.5.4.8 Liquefaction Potential

Within the area of the principal structures, there are no soils susceptible to liquefaction when subjected to the stress condition imposed by the DBE. For verification, laboratory dynamic triaxial tests were performed on samples from the plant area. The results of these tests were used to determine safety factors against liquefaction for several piezometric levels at various locations in the powerblock area. (See subsection 2A.5.2 of supplement 2A.) The shear stresses produced by the DBE were found to be far less than the dynamic strength of the soil indicated by the dynamic triaxial tests.

In addition, the penetration resistances of the sand zones considered are much higher (the soil is much denser) than sands that have been liquefied in other parts of the world where this phenomenon has been observed. (Penetration resistance histograms are presented in figures 2A-14 and 2A-15.) Generally, 15 to 25% of the sands at the site pass through the No. 200 sieve. This shows that the soils are not truly cohesionless and are not susceptible to liquefaction. Also, the foundation soils are at least 13 million years old (Miocene) and are highly preconsolidated; whereas, where liquefaction has occurred, the soils have been recent alluvium, glacial outwash, or loose manmade fills.

2.5.4.9 Earthquake Design Basis

Basis of earthquake design is the DBE with a maximum horizontal acceleration of 15% of the acceleration of gravity. Background for selection of the DBE is provided in subsection 2.5.2.10, Design Basis Earthquake.

2.5.4.10 Static Analyses

Foundation investigations and evaluations for HNP-2 principal structures (the reactor, radwaste, and turbine buildings) and auxiliary structures have been carried out as part of investigations encompassing a large portion of the site. The extensive information and knowledge of the site subsurface conditions that has been accumulated for HNP-1 is used to augment and verify the evaluation of HNP-2 foundation conditions.

The stability of the soil foundations for static as well as dynamic loads and for adverse ground water level conditions was evaluated by performing bearing capacity, settlement, liquefaction, and slope

stability analyses. The results of these analyses show that the foundations are capable of safely supporting the plant structures. These analyses are discussed in sections 2A.5 and 2A.6.

2.5.4.11 Criteria and Design Methods

Mat foundations were considered flexible or rigid, depending on the stiffness of the structure relative to the foundation material. A minimum safety factor of three against a static bearing capacity or shear failure was used in the design. Settlement analyses were based on the theory of consolidation in the case of saturated primarily cohesive soil and on the theory of elasticity in the case of unsaturated or noncohesive soils.

Liquefaction analyses were made using laboratory cyclic triaxial test results of relatively undisturbed soil samples and the results of standard penetrations tests. These are discussed in subsection 2A.5.2.

Slope stability during static and dynamic loading was evaluated by means of the circular arc and slices method of analysis (section 2A.6). The minimum acceptable factors of safety against sliding or shear failure for all slopes were established as follows:

A. *Normal conditions: For normal operating conditions with the most adverse water level, the minimum factor of safety is 1.5.*

B. *Earthquake conditions: For the addition of the effect of the DBE to the normal operating conditions, the minimum factor of safety is 1.1.*

C. *Construction conditions: For temporary construction conditions, the minimum factor of safety is 1.3.*

All slopes at the site are designed to meet or exceed the above criteria. The results of the stability analyses are discussed in section 2A.6.

2.5.4.12 Techniques to Improve Subsurface Conditions

The site and foundation materials are stable and capable of safely supporting the plant loads under static as well as dynamic conditions. Therefore, the improvement of subsurface conditions was not needed.

2.5.5 SLOPE STABILITY

Slope stability analyses indicate that the permanent and construction slopes in the plant area are stable. Analyses also indicate that the natural bank between the river and upper plant level is quite stable for both static and dynamic conditions. The minimum calculated safety factors are presented in section 2A.6.

2.5.5.1 Slope Characteristics

The following slope sections were analyzed for stability under static and earthquake conditions:

- *Section through intake structure.*
- *Section of the riverbank upstream of intake structure.*

The slope cross section, soil properties, and design conditions for the riverbank upstream of the intake structure are shown in figure 2A-27.

2.5.5.2 Design Criteria and Analyses

Refer to paragraph 2.5.4.11.

2.5.5.3 Logs of Core Borings

Graphic logs of test borings made for the design of the principal structures of HNP-1 and HNP-2 are presented in supplement 2B. A discussion of the methods used and results obtained is included in sections 2A.1 and 2A.2. The boring locations are shown in figures 2A-1 and 2A-2.

2.5.5.4 Compaction Specifications

Onsite and locally available offsite sandy clays, clayey sands, and silty fine sands were used as backfill around Seismic Category I structures and in conduit trenches. Backfill was placed on inspected subgrade in thin layers and compacted by vibratory compactors to an average of 95% of the maximum dry density as determined by ASTM D-1557 (Modified Proctor). Representative compaction tests are shown in figure 2A-38.

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TABLE 2.5-1 (SHEET 1 OF 8)

MODIFIED MERCALLI INTENSITY SCALE, 1931[(a)]

I. *Not felt except rarely under especially favorable circumstances. Under certain conditions, at and outside the boundary of the area in which a great shock is felt:*

- *Sometimes birds, animals reported uneasy or disturbed.*
- *Sometimes dizziness or nausea experienced.*
- *Sometimes trees, structures, liquids, bodies of water may sway; doors may swing, very slowly.*

II. *Felt indoors by few, especially on upper floors, or by sensitive or nervous persons. Also, as in grade I, but often more noticeably:*

- *Sometimes hanging objects may swing, especially when delicately suspended.*
- *Sometimes trees, structures, liquids, bodies of water may sway; doors may swing, very slowly.*
- *Sometimes birds, animals reported uneasy or disturbed.*
- *Sometimes dizziness or nausea experienced.*

III. *Felt indoors by several; motion usually rapid vibration:*

- *Sometimes not recognized to be an earthquake at first.*
- *Duration estimated in some cases.*
- *Vibration like that due to passing of light, or lightly loaded trucks, or heavy trucks some distance away.*
- *Hanging objects may swing slightly.*
- *Movements may be appreciable on upper levels of tall structures.*
- *Rocked standing motor cars slightly.*

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a. Adapted from Sieberg's (1923) Mercalli-Cancani scale, modified and condensed. Quoted from Wood and Neumann (1931).

TABLE 2.5-1 (SHEET 2 OF 8)

IV. Felt indoors by many; outdoors by few:

- *Awakened few, especially light sleepers.*
- *Frightened no one, unless apprehensive from previous experience.*
- *Vibration like that due to passing of heavy, or heavily loaded trucks.*
- *Sensation like heavy body striking building, or falling of heavy objects inside.*
- *Rattling of dishes, windows, doors; glassware and crockery clink and clash.*
- *Creaking of walls, frame, especially in the upper range of this grade.*
- *Hanging objects swung, in numerous instances.*
- *Disturbed liquids in open vessels slightly.*
- *Rocked standing motor cars noticeably.*

V. Felt indoors by practically all; outdoors by many or most; outdoors direction estimated:

- *Awakened many, or most.*
- *Frightened few; slight excitement; a few ran outdoors.*
- *Buildings trembled throughout.*
- *Broke dishes, glassware, to some extent.*
- *Cracked windows in some cases, but not generally.*
- *Overturned vases, small or unstable objects, in many instances, with occasional fall.*
- *Hanging objects, doors, swung generally or considerably.*
- *Knocked pictures against walls, or swung them out of place.*
- *Opened, or closed, doors, shutters, abruptly.*
- *Pendulum clocks stopped, started, or ran fast or slow.*
- *Moved small objects, furnishings, the latter to slight extent.*

TABLE 2.5-1 (SHEET 3 OF 8)

- *Spilled liquids in small amounts from well-filled open containers.*
- *Trees, bushes, shaken slightly.*

VI. *Felt by all, indoors and outdoors:*

- *Frightened many; excitement general; some alarm, many ran outdoors.*
- *Awakened all.*
- *Persons made to move unsteadily.*
- *Trees and bushes shaken slightly to moderately.*
- *Liquid set in strong motion.*
- *Small bells rang (church, chapel, school, etc.).*
- *Damage slight in poorly built buildings.*
- *Fall of plaster in small amount.*
- *Cracked plaster somewhat, especially fine cracks, chimneys in some instances.*
- *Broke dishes, glassware, in considerable quantity, also some windows.*
- *Fall of knick-knacks, books, pictures.*
- *Overturned furniture in many instances.*
- *Moved furnishings of moderately heavy kind.*

VII. *Frightened all; general alarm; all ran outdoors:*

- *Some, or many, found it difficult to stand.*
- *Noticed by persons driving motor cars.*
- *Trees and bushes shaken moderately to strongly.*
- *Waves on ponds, lakes, and running water.*

TABLE 2.5-1 (SHEET 4 OF 8)

- *Water turbid from mud stirred up.*
- *Incaving to some extent of sand or gravel stream banks.*
- *Rang large church bells, etc.*
- *Suspended objects made to quiver.*
- *Damage negligible in buildings of good design and construction, slight to moderate in well-built ordinary buildings, considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid-up without motor), spires, etc.*
- *Cracked chimneys to considerable extent, walls to some extent.*
- *Fall of plaster in considerable to large amount, also some stucco.*
- *Broke numerous windows, furniture to some extent.*
- *Shook down loosened brickwork and tiles.*
- *Broke weak chimneys at the roofline (sometimes damaging roofs).*
- *Fall of cornices from towers and high buildings.*
- *Dislodged bricks and stones.*
- *Overturned heavy furniture, with damage from breaking.*
- *Damage considerable to concrete irrigation ditches.*

VIII. *Fright general; alarm approaches panic:*

- *Disturbed persons driving motor cars.*
- *Trees shaken strongly, branches, trunks, broken off, especially palm trees.*
- *Ejected sand and mud in small amounts.*
- *Changes: temporary, permanent; in-flow of springs and wells; dry wells renewed flow; in temperature of spring and well waters.*
- *Damage slight in structures (brick) built especially to withstand earthquakes.*

TABLE 2.5-1 (SHEET 5 OF 8)

- *Considerable damage in ordinary substantial buildings, partial collapse; racked, tumbled down, wooden houses in some cases; threw out panel walls in frame structures, broke off decayed piling.*
- *Fall of walls.*
- *Cracked, broke, solid stone walls seriously.*
- *Wet ground to some extent, also ground on steep slopes.*
- *Twisting, fall, of chimneys, columns, monuments, also factory stacks, towers.*
- *Moved conspicuously; overturned very heavy furniture.*

IX. Panic general:

- *Cracked ground conspicuously.*
- *Damage considerable in (masonry) structures built especially to withstand earthquakes:*
  - *Threw out of plumb some wood frame houses built especially to withstand earthquakes.*
  - *Great in substantial (masonry) buildings, some collapse in large part; or wholly shifted frame buildings off foundations, racked frames.*
  - *Serious to reservoirs; underground pipes sometimes broken.*

X. Cracked ground, especially when loose and wet, up to widths of several inches; fissures up to a yard in width ran parallel to canal and stream banks:

- *Landslides considerable from riverbanks and steep coasts.*
- *Shifted sand and mud horizontally on beaches and flat land.*
- *Changed level of water in wells.*
- *Threw water on banks of canals, lakes, rivers, etc.*
- *Damage serious to dams, dikes, embankments.*
- *Damage severe to well-built wooden structures and bridges; some destroyed.*
- *Developed dangerous cracks in excellent brick walls.*

TABLE 2.5-1 (SHEET 6 OF 8)

- *Destroyed most masonry and frame structures, also their foundations.*
- *Bent railroad rails slightly.*
- *Tore apart, or crushed endwise, pipe lines buried in earth.*
- *Open cracks and broad wave folds in cement pavements and asphalt road surfaces.*

XI. *Disturbances in ground many and widespread, varying with ground material:*

- *Broad fissures, earth slumps, and land slips in soft, wet ground.*
- *Ejected water in large amount charged with sand and mud.*
- *Caused sea waves (tidal waves) of significant magnitude.*
- *Damage severe to wood frame structures, especially near shock centers.*
- *Great to dams, dikes, embankments, often for long distances.*
- *Few, if any (masonry), structures remained standing.*
- *Destroyed large well-built bridges by the wrecking of supporting piers or pillars.*
- *Affected yielding wooden bridges less.*
- *Bent railroad rails greatly, and thrust them endwise.*
- *Put pipe lines buried in earth completely out of service.*

XII. *Damage total; practically all works of construction damaged greatly or destroyed:*

- *Disturbances in ground great and varied, numerous shearing cracks.*
- *Landslides, falls of rock of significant character, slumping of riverbanks, etc., numerous and extensive.*
- *Wrenched loose, tore off, large rock masses.*
- *Fault slips in firm rock, with notable horizontal and vertical offset displacements.*
- *Water channels, surface and underground, disturbed and modified greatly.*

TABLE 2.5-1 (SHEET 7 OF 8)

- *Dammed lakes, produced waterfalls, deflected rivers, etc.*
- *Waves seen on ground surfaces (actually seen, probably in some cases).*
- *Distorted lines of sight and level.*
- *Threw objects upward into the air.*

MODIFIED MERCALLI INTENSITY SCALE OF 1931 (Abridged)

I. *Not felt except by a very few under especially favorable circumstances.*

II. *Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.*

III. *Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.*

IV. *During the day, felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls made cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.*

V. *Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles and other tall objects sometimes noticed. Pendulum clocks may stop.*

VI. *Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.*

VII. *Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.*

VIII. *Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Disturbed persons driving motor cars.*

IX. *Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.*

TABLE 2.5-1 (SHEET 8 OF 8)

X. *Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from riverbanks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.*

XI. *Few, if any (masonry), structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.*

XII. *Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.*

TABLE 2.5-2 (SHEET 1 OF 3)

CHRONOLOGICAL LISTING OF EARTHQUAKES WITHIN 200 MILES OF THE EDWIN I. HATCH NUCLEAR PLANT

Date		Latitude N	Longitude W	Locality	Epicentral Intensity (MM)	Magnitude	Felt Area (mi²)	Seismic Area	Distance From Site (mi)	Reference
1872,	6-17	(22.1	83.3)[(a)]	Milledgeville, Ga.	V			Appalachian	100	Eppley
1875,	11-01	(33.8	82.5)	Northern Georgia	VI		25,000	Appalachian	135	Eppley
1879,	1-12	(29.5	82.0)	Northern Florida-2 events	VI		25,000	Coastal Plain	170	Eppley
1885,	10-17	(33.0	82.8)	Sandersville, Ga.	IV			Coastal Plain	85	Oak Ridge
1886,	8-27	(33.1	80.2)	Near Charleston, S.C.	V			Charleston	152	Oak Ridge
1886,	8-31	(32.9	80.0)	Charleston, S.C.-2 events	X		2,000,000	Charleston	155	Eppley
1886,	9-03	(32.9	80.0)	Charleston, S.C.	VI			Charleston	155	Oak Ridge
1886,	9-05	(32.9	80.0)	Charleston, S.C.	VI			Charleston	155	Oak Ridge
1886,	9-21	(32.9	80.0)	Charleston, S.C.	V-VI			Charleston	155	Oak Ridge
1886,	9-27	(32.9	80.0)	Charleston, S.C.	VI			Charleston	155	Oak Ridge
1886,	10-22	(32.9	80.0)	Charleston, S.C.	VI		30,000	Charleston	155	Eppley
1886,	10-22	(32.9	80.0)	Charleston, S.C.	VII		30,000	Charleston	155	Eppley
1886,	11-05	(32.9	80.0)	Charleston, S.C.	VI		30,000	Charleston	155	Eppley
1887,	1-04	(32.9	80.0)	Charleston, S.C.	VI			Charleston	155	Eppley
1887,	6-03	(32.9	80.0)	Charleston, S.C.	IV			Charleston	155	Oak Ridge
1888,	1-12	(32.9	80.0)	Charleston, S.C.	VII			Charleston	155	Oak Ridge
1893,	6-21	(30.4	81.7)	Jacksonville, Fla.	IV			Coastal Plain	112	Oak Ridge
1900,	10-12	(30.4	81.7)	Jacksonville, Fla.-8 events	V		Local	Coastal Plain	112	Oak Ridge
1901,	12-01	(32.9	80.0)	Charleston, S.C.	IV-V			Charleston	158	Oak Ridge
1903,	1-23	(32.8	80.0)	Charleston, S.C.	IV			Charleston	152	Oak Ridge

TABLE 2.5-2 (SHEET 2 OF 3)

Date		Latitude N	Longitude W	Locality	Epicentral Intensity (MM)	Magnitude	Felt Area (mi^2)	Seismic Area	Distance From Site (mi)	Reference
1903,	1-23	(32.1	81.1)	Savannah, Ga.	VI		10,000	Coastal Plain	76	Oak Ridge
1907,	4-19	(32.9	80.0)	Charleston, S.C.	V		10,000	Charleston	155	Eppley
1908,	1-15	(33.0	80.2)	Summerville, S.C.	III-IV			Charleston	148	Oak Ridge
1908,	3-03	(33.1	80.2)	Summerville, S.C.	III-IV			Charleston	152	Oak Ridge
1908,	3-07	(33.1	80.2)	Summerville, S.C.	III-IV			Charleston	152	Oak Ridge
1908,	10-28	(33.1	80.2)	Summerville, S.C.	III-IV			Charleston	152	Oak Ridge
1912,	6-12	(32.9	80.0)	Summerville, S.C.	VII		35,000	Charleston	155	Eppley
1912,	6-20	(32.0	81.0)	Savannah, Ga.	V			Coastal Plain	83	Oak Ridge
1912,	10-22	(32.7	83.5)	Macon, Ga.	IV		1,500	Coastal Plain	88	Oak Ridge
1912,	12-07	(34.7	81.7)	Union County, S.C.	III-IV			Appalachian	196	Oak Ridge
1913,	1-01	(34.7	81.7)	Union County, S.C.	VII-VIII		43,000	Appalachian	196	Oak Ridge
1914,	3-05	(33.5	83.5)	Morgan County, S.C.	VI		50,000	Appalachian	130	Eppley
1914,	9-22	(33.0	80.3)	Near Summerville, S.C.	V		30,000	Charleston	140	Oak Ridge
1916,	3-02	(34.5	82.7)	Anderson, S.C.-6 events	IV-V			Appalachian	182	Oak Ridge
1923,	12-31	(34.8	82.5)	Greenville, S.C.	IV			Appalachian	199	Oak Ridge
1933,	12-19	(33.0	80.2)	Summerville, S.C.	IV-V		Local	Charleston	148	Oak Ridge
1934,	12-09	(33.0	80.2)	Summerville, S.C.	IV			Charleston	148	Oak Ridge
1935,	11-13	(29.9	81.3)	St. Augustine, Fla.-2 events	IV			Coastal Plain	151	Oak Ridge
1943,	12-28	(33.0	80.2)	Summerville, S.C.	IV			Charleston	148	Oak Ridge
1945,	7-26	34.3	81.4	North of Lake Murray, S.C.	VI		25,000	Appalachian	175	Eppley
1952,	11-18	(30.6	84.6)	Quincy, Fla.	IV			Coastal Plain	160	Oak Ridge
1952,	11-19	(32.8	80.0)	Charleston, S.C.	V			Charleston	150	Eppley

TABLE 2.5-2 (SHEET 3 OF 3)

Date		Latitude N	Longitude W	Locality	Epicentral Intensity (MM)	Magnitude	Felt Area (mi^2)	Seismic Area	Distance From Site (mi)	Reference
1956,	1-05	(34.3	82.4)	Due West, S.C.-2 events	IV			Appalachian	168	Oak Ridge
1956,	5-19	(34.3	82.4)	Due West, S.C.	IV			Appalachian	168	Oak Ridge
1956,	5-27	(34.3	82.4)	Due West, S.C.	IV			Appalachian	168	Oak Ridge
1958,	10-20	(34.5	82.8)	Anderson, S.C.	V			Appalachian	183	Oak Ridge
1959,	8-03	(33.0	79.5)	East of Charleston, S.C.	VI		25,000	Coastal Plain	186	Eppley
1960,	7-23	(33.0	80.0)	Charleston, S.C.	V		Local	Charleston	155	Eppley
1963,	4-11	(34.8	82.4)	Greenville, S.C.	IV			Appalachian	199	Oak Ridge
1963,	5-04	(32.2	79.7)	Southeast of Charleston, S.C.	IV			Coastal Plain	158	USC & GS
1964,	3-12	(33.2	83.4)	Macon, Ga.	(V)	4.4	400	Appalachian	110	NOAA
1964,	4-20	(34.0	81.0)	Near Columbia, S.C.	V			Coastal Plain	165	Oak Ridge
1967,	10-23	33.4	80.7	Southeast of Orangeburg, S.C.	(V)	3.8		Coastal Plain	143	NOAA
1968,	9-22	34.0	81.5	South of Lake Murray, S.C.	(IV)	3.7	400	Appalachian	155	Oak Ridge
1971,	5-19	(33.4	80.6)	East of Orangeburg, S.C.	IV	3.4		Coastal Plain	148	NOAA
1971,	7-13	(34.7	82.9)	Seneca, S.C.	IV			Appalachian	195	Oak Ridge
1972,	2-03	33.5	80.4	Lake Marion, S.C.	V	4.5		Coastal Plain	158	NOAA
1974,	8-02	33.9	82.5	Lincoln County, Ga.	V	4.5-4.9		Appalachian	140	NOAA
1974,	11-22	32.9	80.0	Charleston, S.C.	V-VI	4.5		Charleston	155	NOAA
1976,	12-27	32.2	82.5	Reidsville, Ga.	IV-V	3.7		Coastal Plain	15	NOAA

a. Parentheses around coordinates indicate an approximate epicentral location.

TABLE 2.5-3

DATA FOR EPICENTERS SHOWN IN FIGURE 2.5-15

Date	*Latitude (N)*	*Longitude (W)*	*Magnitude*
October 26, 1879 (F)	*34.5*	*81.1*	*-*
September 22, 1968 (I)	*34.0*	*81.5*	*3.7*
January 4, 1974 (I)	*33.66*	*82.40*	*<1*
February 14, 1974 (I)	*33.62*	*82.48*	*2.7*
October 28, 1974 (S)	*33.79*	*81.92*	*3.0*
November 5, 1974 (F)	*33.73*	*82.22*	*3.7*
December 8, 1974 (I)	*34.17*	*80.81*	*1.0*
November 16, 1975 (I)	*34.28*	*80.55*	*3.0*

TABLE 2.5-4 (SHEET 1 OF 3)

CHRONOLOGICAL LISTING OF EARTHQUAKES (INTENSITY III AND GREATER) SHOWN IN FIGURE 2.5-16 (EXCLUSIVE OF CHARLESTON-SUMMERVILLE AREA)

Date	Locality	°N	°W	Felt Area (mi^2)	Epicentral Intensity/ Magnitude
1799, April 11	Camden, S.C.	-	-	-	IV[(a)]
1826, Oct. 15	Savannah, Ga.	32.0	81.0	-	-[(b)]
1872, June 17	Milledgeville, Ga.	33.1	83.3	-	V
1875, July 28	Milledgeville, Ga.	33.1	83.3	-	-[(b)]
1875, Nov. 1	N. Georgia	33.8	82.5	25,000	VI
1879, Oct. 26	Winnsboro, S.C.	34.5	81.1	-	III
1884, Mar. 31	Milledgeville, Ga.	33.1	83.3	-	III
1885, Oct. 17	Sandersville, Ga.	33.0	82.8	-	IV
1903, Jan. 23	Ga.-S.C. region	32.1	81.1	10,000	VI
1911, Apr. 20	N.C.-S.C. area	35.2	82.7	600	V
1912, June 20	Savannah, Ga.	32.0	81.0	-	V
1912, Oct. 23	Dublin, Macon, and Perry, Ga.	32.7	83.5	1,500	IV
1912, Dec. 7	Union County, S.C.	34.7	81.7	-	III-IV
1913, Jan. 1	Union County, S.C.	34.7	81.7	43,000	VI-VII
1914, Mar. 5	Near Atlanta, Ga.	33.5	83.5	50,000	VI
1914, Mar. 7	Darlington and Florence, S.C.	34.3	79.8	-	III-IV
1916, Mar. 2	Anderson, S.C.	34.5	82.7	-	IV-V
1924, Jan. 1	Greenville, S.C.	34.8	82.5	-	IV
1924, Oct. 20	Pickens County, S.C.	35.0	82.6	56,000	V

TABLE 2.5-4 (SHEET 2 OF 3)

Date	*Locality*	*°N*	*°W*	*Felt Area (mi^2)*	*Epicentral Intensity/ Magnitude*
1929, Jan. 3	*Sumter, S.C.*	*33.9*	*80.3*	*-*	*IV[(a)]*
1929, Oct. 27	*Due West, S.C.*	*34.3*	*82.4*	*-*	*IV[(a)]*
1930, Dec. 9	*Due West, S.C.*	*34.3*	*82.4*	*-*	*III[(a)]*
1930, Dec. 25	*Chesterfield County, S.C.*	*34.5*	*80.3*	*-*	*III[(a)]*
1931, May 6	*Due West, S.C.*	*34.3*	*82.4*	*-*	*IV[(a)]*
1933, June 29	*Eatonton, Ga.*	*-*	*-*	*-*	*IV-V[(a)]*
1935, Jan. 1	*N.C.-Ga. border*	*35.1*	*83.6*	*7,000*	*V*
1945, July 26	*Murray Lake, S.C.*	*34.3*	*81.4*	*25,000*	*IV-V*
1956, Jan. 5	*Due West, S.C.*	*34.3*	*82.4*	*-*	*IV*
1956, May 19	*Due West, S.C.*	*34.3*	*82.4*	*-*	*IV*
1956, May 27	*Due West, S.C.*	*34.3*	*82.4*	*-*	*IV*
1957, Nov. 24	*N.C.-Tenn. border*	*35.0*	*83.5*	*4,100*	*VI*
1958, Oct. 20	*Anderson, S.C.*	*34-1/2*	*82-3/4*	*-*	*V*
1959, Oct. 26	*Northeast S. Carolina*	*34-1/2*	*80-1/4*	*4,800*	*VI*
1960, Mar. 12	*-*	*33.0*	*79.0*	*3,500*	*V*
1963, April 11	*Greenville, S.C.*	*34.8*	*83.4*	*-*	*IV*
1963, May 4	*-*	*32.2*	*79.7*	*-*	*IV*
1964, Mar. 12	*Central Georgia*	*33.2*	*83.4*	*400*	*V*
1964, Apr. 20	*Columbia, S.C.*	*34.0*	*81.0*	*-*	*V*
1967, Oct. 23	*-*	*33.4*	*80.7*	*-*	*V*
1968, Sept. 22	*Richland and Lexington County, S.C.*	*34.0*	*81.5*	*400*	*IV/3.7*

TABLE 2.5-4 (SHEET 3 OF 3)

Date	Locality	°N	°W	Felt Area (mi^2)	Epicentral Intensity/ Magnitude
1971, May 19	Near Orangeburg, S.C.	33.34	80.56	-	V
1971, July 13	Near Newry, S.C.	-	-	2,000	VI
1971, July 31	Near Orangeburg, S.C.	33.4	80.7	-	III
1972, Feb. 3	-	33.5	80.4	26,000	V/4.5
1972, Feb. 6	St. George, S.C.	-	-	-	III-IV[(a)]
1972, Aug. 14	Southern, S.C.	-	-	2,500	3.0
1974, Aug. 2	-	33.95	82.50	-	VI/4.3
1974, Oct. 28	-	33.79	81.92	-	3.0
1974, Nov. 5	-	33.73	82.22	-	3.7
1974, Dec. 3	-	33.95	82.50	-	3.6
1975, Mar. 7	-	34.92	81.33	-	3.4
1975, Mar. 7	-	34.92	81.33	-	3.8
1975, Nov. 4	Hartsville, S.C.	-	-	-	3
1975, Nov. 16	-	34.28	80.55	-	3.0
1975, Nov. 25	-	34.95	82.91	-	3.5

a. Bechtel estimate of maximum intensity.
b. Inadequate data to assign an intensity.
c. It seems possible that this date should be May 5, corresponding to that of a shock centered in Alabama.

TABLE 2.5-5 (SHEET 1 OF 3)

CHRONOLOGICAL LISTING OF EARTHQUAKES CENTERED IN CHARLESTON-SUMMERVILLE AREA (INTENSITY III AND GREATER)

Date	Locality	°N	°W	Felt Area (mi^2)	Epicentral Intensity/ Magnitude
1857, Dec. 19	Charleston, S.C.	32.9	80.0	-	IV-V[(a)]
1886, Aug. 31	Charleston, S.C.	32.9	80.0	2,000,000	IX-X
1886, Oct. 22	Charleston, S.C.	32.9	80.0	30,000	VI
1886, Oct. 22	Charleston, S.C.	32.9	80.0	30,000	VII
1886, Nov. 5	Charleston, S.C.	32.9	80.0	30,000	VI
1903, Jan. 24	Charleston, S.C.	32.8	80.0	-	IV
1907, Apr. 19	Charleston, S.C.	32.9	80.0	10,000	V
1908, Jan. 15	Summerville, S.C.	33.0	80.2	-	III-IV
1908, Mar. 3	Summerville, S.C.	33.1	80.2	-	III-IV
1908, Mar. 7	Summerville, S.C.	33.1	80.2	-	III-IV
1908, Oct. 26	Summerville, S.C.	33.1	80.2	-	III
1908, Oct. 28	Summerville, S.C. aftershock	33.1	80.2	-	III-IV
1912, June 12	Summerville, S.C.	33.0	80.2	35,000	VII
1914, June 1	Waterboro, S.C.	33.0	80.3	-	III
1914, July 14	Summerville, S.C.	33.1	80.2	-	III
1914, Sept. 22	Near Summerville, S.C.	33.0	80.2	30,000	V
1915, Dec. 20	Charleston, S.C.	32.8	79.9	-	III-IV
1916, June 25	Summerville, S.C.	33.1	80.2	-	III
1920, Aug. 1	Summerville, S.C.	33.1	80.2	-	IV-VII

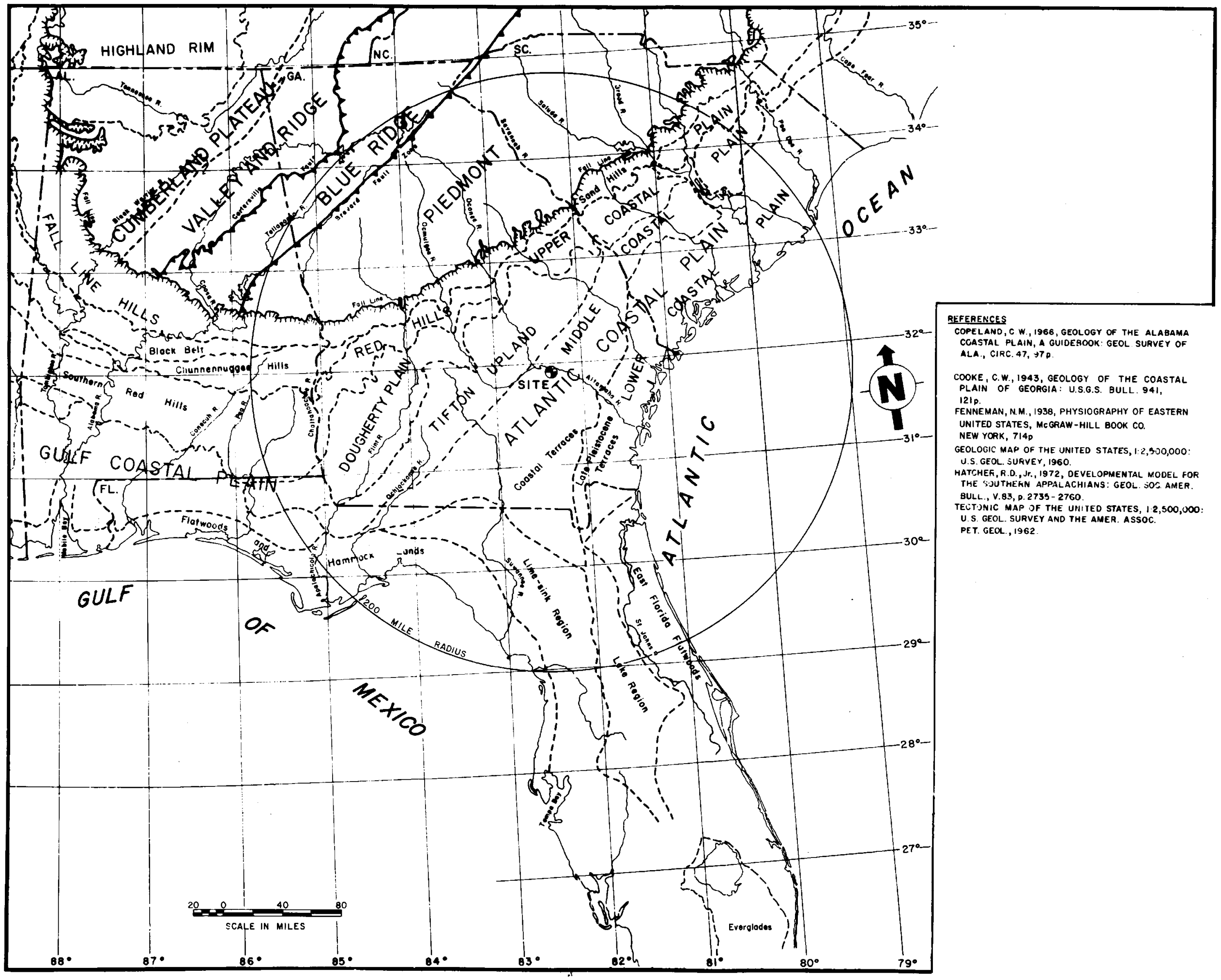
TABLE 2.5-5 (SHEET 2 OF 3)

Date	Locality	°N	°W	Felt Area (mi^2)	Epicentral Intensity/ Magnitude
1921, Apr. 19	Summerville, S.C.	33.1	80.2	-	III
1921, Apr. 23	Summerville, S.C.	33.1	80.2	-	III
1923, Mar. 23	Charleston and Summerville, S.C.	32.9	80.0	-	III
1923, May 4	West of Charleston	33.1	80.2	-	III
1924, Feb. 14	Summerville, S.C.	33.1	80.2	-	III
1924, June 3	Summerville, S.C.	33.1	80.2	-	III
1933, July 25	Summerville, S.C.	33.1	80.2	-	III
1933, Dec. 19	Summerville, S.C.	33.0	80.2	-	IV-V
1933, Dec. 23	Summerville, S.C.	33.0	80.2	-	IV[(a)]
1934, Dec. 9	Summerville, S.C.	33.0	80.2	-	IV
1935, Feb. 6	Summerville, S.C.	33.0	80.2	-	IV[(a)]
1935, Oct. 20	Summerville, S.C.	33.0	80.2	-	IV[(a)]
1940, Jan. 5	Summerville, S.C.	33.0	80.2	-	IV
1943, Dec. 28	Summerville, S.C.	33.0	80.2	-	IV
1944, Jan. 28	Summerville, S.C.	33.0	80.2	-	IV[(a)]
1945, Jan. 30	Summerville, S.C.	33.0	80.2	-	IV[(a)]
1945, May 18	3 miles southwest of Charleston, S.C.	-	-	-	III[(a)]
1945, June 5	Near Charleston	32.8	80.0	-	III[(a)]
1946, Feb. 8	Summerville, S.C.	33.0	80.2	-	III[(a)]
1947, Nov. 1	Summerville, S.C.	33.0	80.2	-	IV[(a)]
1949, Feb. 2	Summerville, S.C.	33.0	80.2	-	IV[(a)]

TABLE 2.5-5 (SHEET 3 OF 3)

Date	Locality	°N	°W	Felt Area (mi^2)	Epicentral Intensity/ Magnitude
1949, June 27	Summerville, S.C.	33.0	80.2	-	III[a]
1951, March 3	Summerville, S.C.	33.0	80.2	-	IV[a]
1951, Dec. 30	Summerville, S.C.	33.0	80.2	-	IV[a]
1952, Nov. 19	Charleston, S.C.	32.9	80.0	-	V
1959, Aug. 3	Near Charleston, S.C.	33.0	79.5	25,000	VI
1960, July 23	Charleston, S.C.	33.0	80.0	-	V
1961, May 20	Summerville, S.C.	33.0	80.2	-	III
1961, Oct. 17	Summerville, S.C.	33.0	80.2	-	III
1968, July 9	Near Charleston, S.C.	-	-	-	VI
1974, Nov. 22	Charleston-Summerville, S.C.	32.9	80.1	-	VI/4.7

a. Bechtel estimate of maximum intensity.



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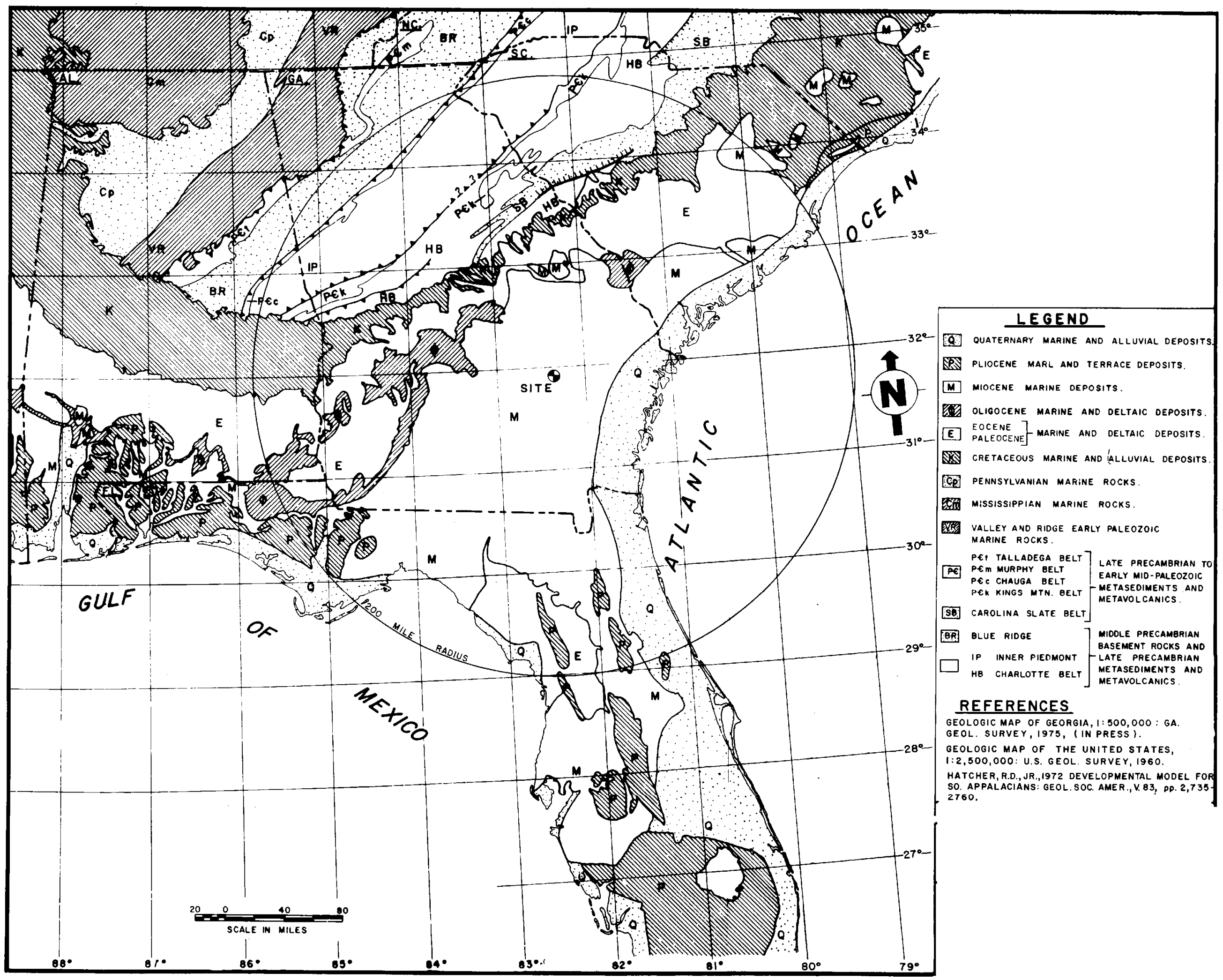
REGIONAL PHYSIOGRAPHIC MAP

FIGURE 2.5-1



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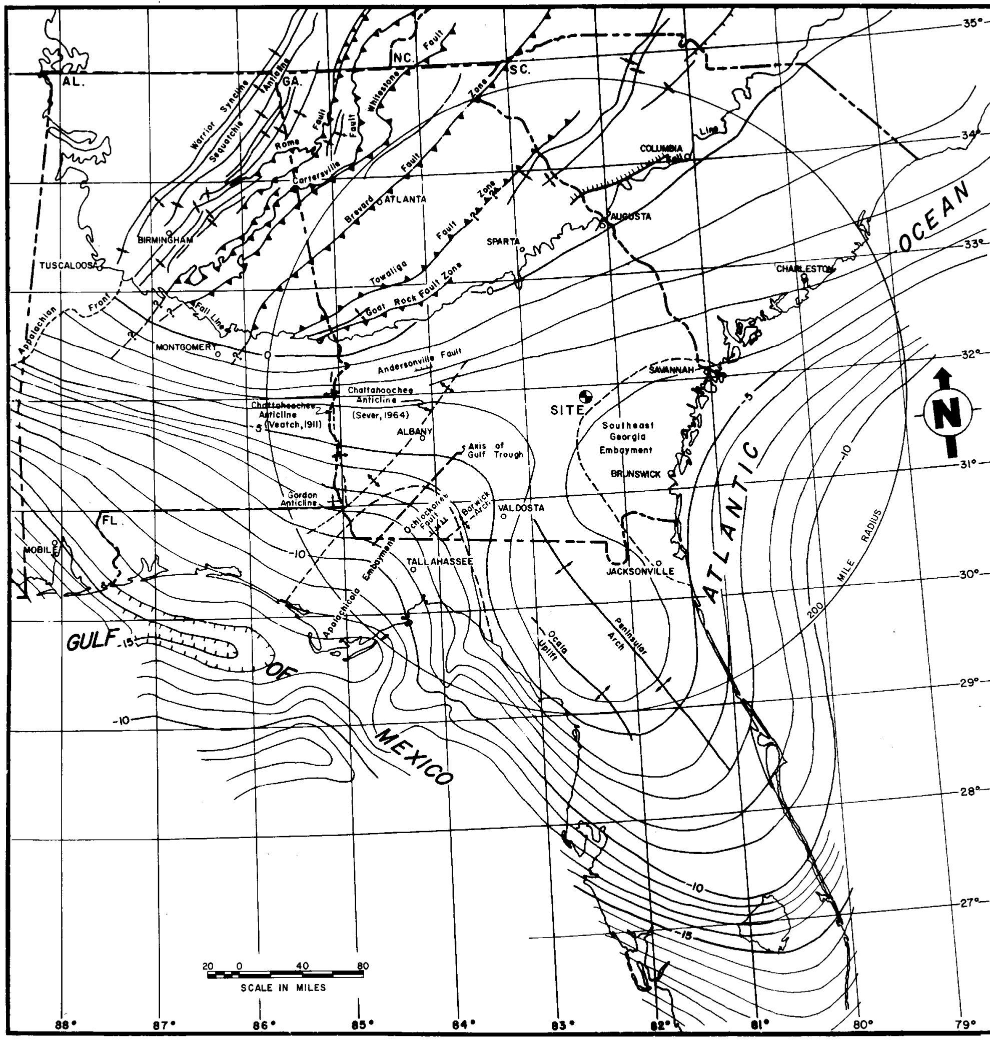
REGIONAL GEOLOGIC MAP

FIGURE 2.5-2



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LEGEND

THRUST FAULT

NORMAL FAULT

ANTICLINAL AXIS

SYNCLINAL AXIS

STRUCTURE CONTOURS ON PRE-MESOZOIC SURFACE (IN THOUSANDS OF FEET)

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REGIONAL TECTONIC MAP

FIGURE 2.5-3

ERA	PERIOD	EPOCH	GEOLOGIC UNIT	APPROX. MAX. THICKNESS (FT)	LITHOLOGIC DESCRIPTION
CENOZOIC	QUATERNARY	HOLOCENE	ALLUVIAL & MARINE DEPOSITS		SAND, GRAVEL, AND CLAY IN FLOODPLAINS; CALCAREOUS SILT AND CLAY NEAR COAST.
		PLEISTOCENE	TERRACE DEPOSITS	100+	NON-MARINE SAND, GRAVEL, AND CLAY ADJACENT TO RIVERS; MARINE, SOMEWHAT CALCAREOUS SAND AND MARL IN COASTAL TERRACES PARALLEL TO SHORELINE.
	TERTIARY	PLIOCENE	TERRACE DEPOSITS	20	SAND AND GRAVEL; MARL IN DOWNDIP AREAS NEAR COAST.
		PLIOCENE (?)	HAWTHORN FM	634	SANDY, MICACEOUS CLAY, AND ARKOSIC SAND.
		MIOCENE	TAMPA FM		DOLOMITIC LIMESTONE; INTERBEDDED CLAY AND CALCAREOUS SAND UPDIP.
		OLIGOCENE	VICKSBURG GP	287+	LIMESTONE AND MARL, WITH OCCASIONAL CALCAREOUS CLAY AND DOLOMITE.
		EOCENE	OCALA FM (JACKSON GP)	725	FOSSILIFEROUS, CALCAREOUS CLAY AND LIMESTONE; WEATHERED SANDY CLAY UPDIP.
			CLAIBORNE GP: LISBON FM	1190+	FOSSILIFEROUS CLAY, LIMESTONE, AND MARL; CALCAREOUS SAND.
			CLAIBORNE GP: TALLAHATTA FM		SILTY SAND; CALCAREOUS, MICACEOUS, SILTY CLAY AND LIMESTONE WITH SOME LIGNITE.
			WILCOX GP	400+	CALCAREOUS, SILTY FINE SAND AND MARL.
		PALEOCENE	CLAYTON FM (MIDWAY GP)	615+	LIMESTONE AND SANDY MARL; OOLITIC BEDS AND GYPSUM DOWNDIP; CLAY AND SAND UPDIP.
MESOZOIC	CRETACEOUS	GULFIAN	PROVIDENCE RIPLEY CUSSETA BLUFFTOWN EUTAW	1755	SAND, FOSSILIFEROUS AND PHOSPHATIC: INTERBEDDED FOSSILIFEROUS, MICACEOUS, PYRITIFEROUS, SHALY MARL; LOWER UNITS GENERALLY MORE CALCAREOUS, WITH OCCASIONAL COQUINA HORIZONS.
			TUSCALOOSA FM	910	TERRIGENOUS SAND, GRAVEL, AND KAOLINITIC CLAY; MARINE SILT AND CLAY, WITH SOME CARBONATES DOWNDIP.
		COMANCHEAN	UNDIFFERENTIATED	2630	ARKOSIC SAND AND MICACEOUS CLAY; RED BEDS UPDIP; GYPSUM, SHALE, AND CARBONATES DOWNDIP.
	TRIASSIC (?)		UNDIFFERENTIATED		MICACEOUS CLAY, ARKOSIC SAND, AND SANDSTONE; INTERBEDDED DIABASE AND BASALT.
PALEOZOIC			UNDIFFERENTIATED		GRANITE, CRYSTALLINE SCHIST, BIOTITE GNEISS; SHALE.

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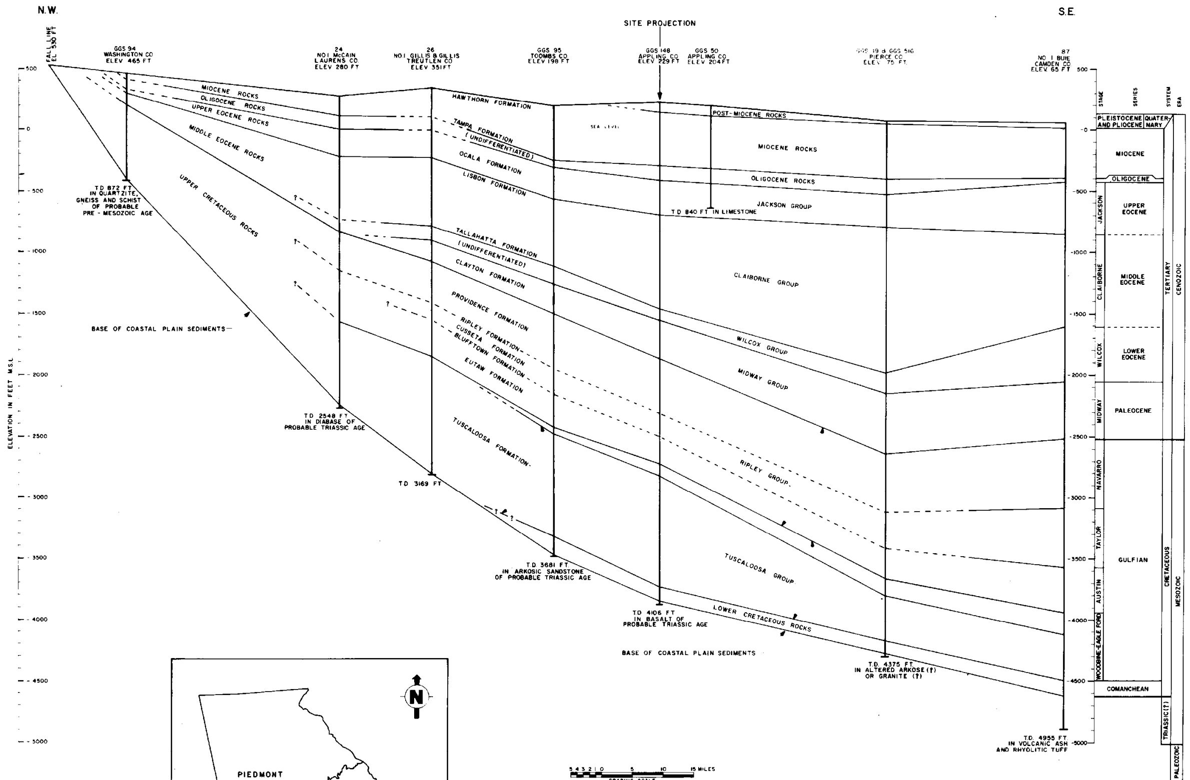
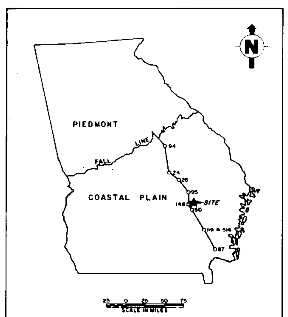
FIGURE 2.5-4





INDEX MAP OF GEORGIA SHOWING LINE OF CROSS SECTION

5 4 3 2 1 0 5 10 15 MILES
GRAPHIC SCALE

NOTES

1. MARINE EQUIVALENT OF TUSCALOOSA FORMATION REFERRED TO AS ATKINSON FORMATION IN EXTREME SOUTH GEORGIA.
2. HAWTHORN FORMATION INCLUDED IN PLIOCENE ("Neogene Undifferentiated") BY GEORGIA GEOLOGICAL SURVEY (APRIL, 1975).

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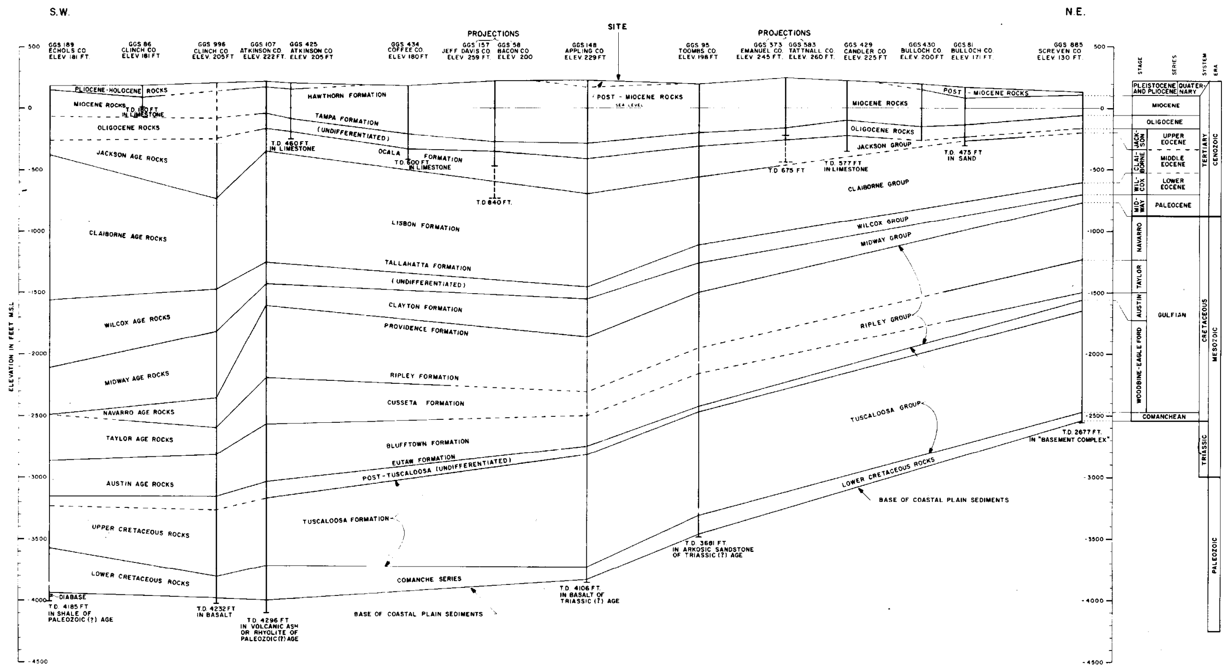
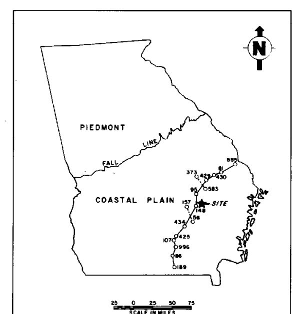
REGIONAL GEOLOGIC PROFILE NW-SE

FIGURE 2.5-5





INDEX MAP OF GEORGIA SHOWING LINE OF CROSS SECTION

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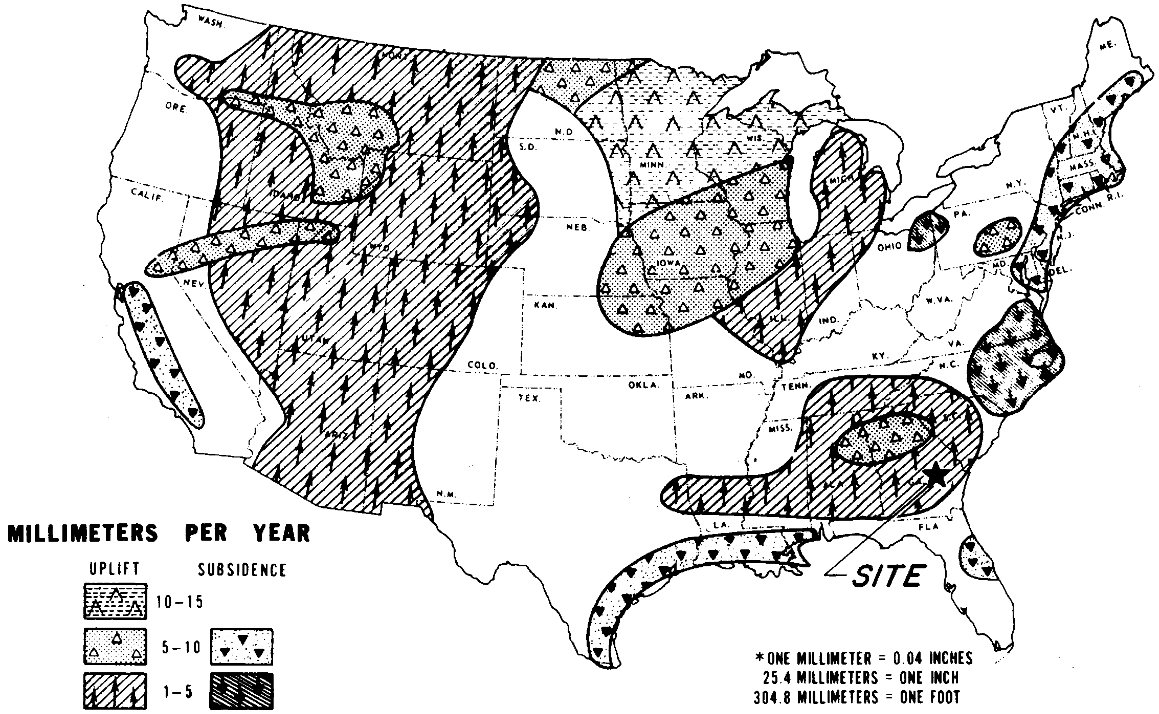
REGIONAL GEOLOGIC PROFILE NE-SW

FIGURE 2.5-6



PROBABLE VERTICAL MOVEMENTS OF THE EARTH'S SURFACE
AUGUST 18, 1972

REFERENCE

U.S. DEPARTMENT OF COMMERCE NEWS, NOAA 72-122: SEPTEMBER 22, 1972.

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CRUSTAL MOVEMENT MAP

FIGURE 2.5-7

ERA	PERIOD	EPOCH	GEOLOGIC UNIT	APPROX. THICKNESS (FT)	LITHOLOGIC DESCRIPTION
CENOZOIC	QUATERNARY	HOLOCENE	ALLUVIUM	55+	SAND AND GRAVEL; CARBONACEOUS SILTY CLAY.
		PLEISTOCENE – PLIOCENE (?)	BRANDYWINE FM	10+	CROSS-BEDDED SAND AND GRAVEL WITH HEMATITE CONCRETIONS.
	TERTIARY		HAWTHORN FM	300+	PHOSPHATIC, FINE TO COARSE SAND; SANDY, CALCAREOUS CLAY; ARKOSIC, CROSS-BEDDED SAND AND GRAVEL; OCCASIONAL PYRITE.
		MIOCENE	TAMPA FM	160	SANDY TO CLAYEY, PHOSPHATIC, FOSSILIFEROUS LIMESTONE; PARTLY DOLOMITIZED.
		OLIGOCENE	UNDIFFEREN-TIATED	120	MASSIVE, CALCITIZED, FOSSILIFEROUS LIMESTONE.
		EOCENE	OCALA FM (JACKSON GP)	280	MASSIVE, CRYSTALLINE, FOSSILIFEROUS LIMESTONE.
			CLAIBORNE GP: LISBON FM	610	SANDY, PHOSPHATIC, DOLOMITIC LIMESTONE; ABUNDANT GLAUCONITE AND FOSSILS.
			CLAIBORNE GP: TALLA-HATTA FM	160	GLAUCONITIC, CALCAREOUS SAND AND THIN, FOSSILIFEROUS MARL LAYERS.
			WILCOX GP	90	CARBONACEOUS, MICACEOUS, SILTY, FOSSILIFEROUS MARL; OCCASIONAL GLAUCONITIC SAND LAYERS.
		PALEOCENE	CLAYTON FM	315	MASSIVE, CRYSTALLINE LIMESTONE; INTERBEDDED WITH CARBONACEOUS, MICACEOUS, GLAUCONITIC MARLY SAND.
MESOZOIC	CRETACEOUS	GULFIAN	POST-TUSCALOOSA DEPOSITS	955	COQUINOID, PHOSPHATIC SAND; CARBONACEOUS, FOSSILIFEROUS, GLAUCONITIC MARL; OCCASIONAL PYRITE.
			TUSCALOOSA FM	910	FINE-GRAINED TO ARKOSIC, CARBONACEOUS, FOSSILIFEROUS, MICACEOUS SAND AND CLAY.
		COMANCHEAN	UNDIFFEREN-TIATED	115	SANDY, MICACEOUS CLAY; ARKOSIC SAND.
	TRIASSIC (?)		UNDIFFEREN-TIATED		ARKOSIC SANDSTONE; BASALT OR DIABASE.

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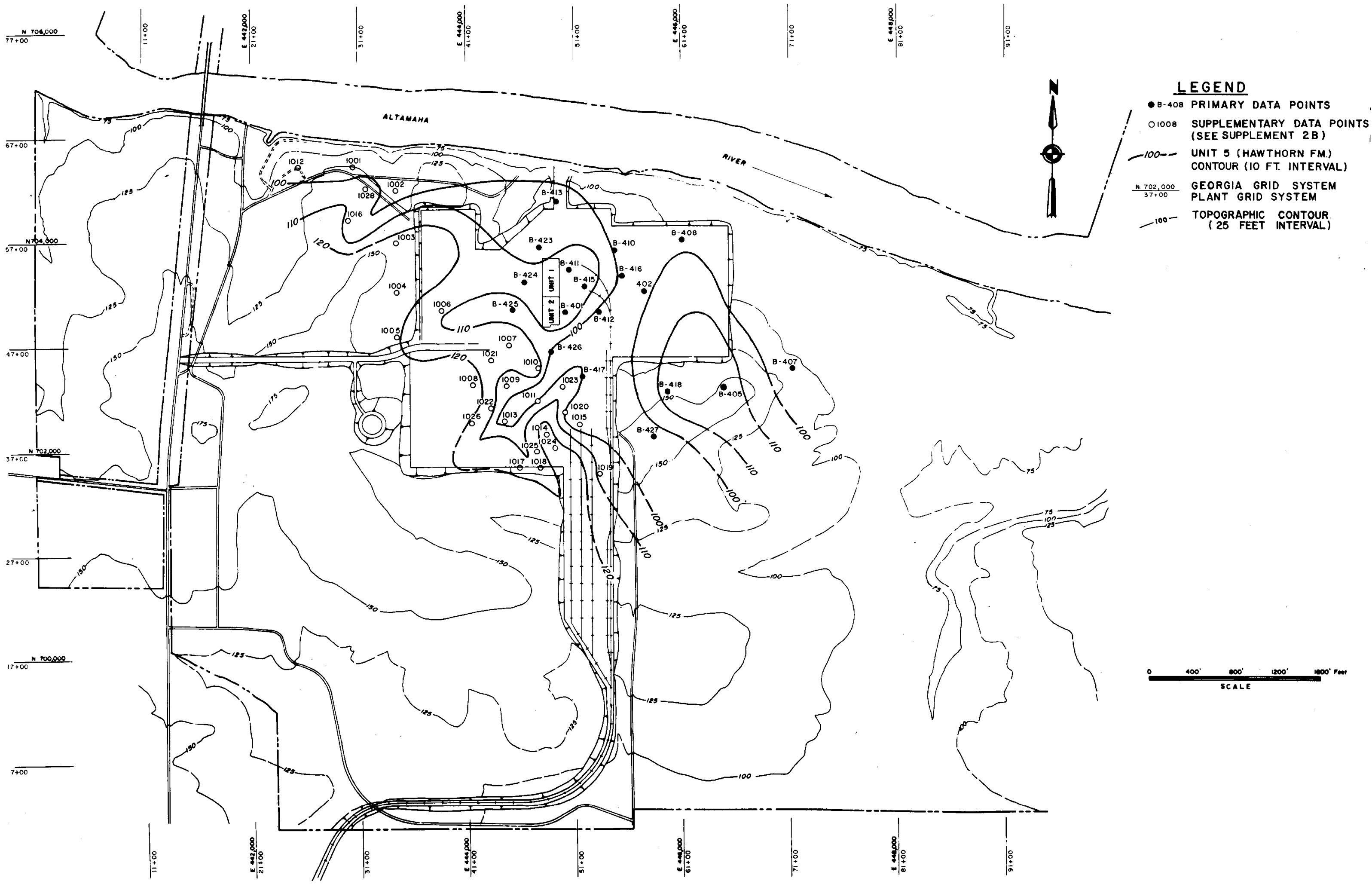
SITE GEOLOGIC COLUMN

FIGURE 2.5-8



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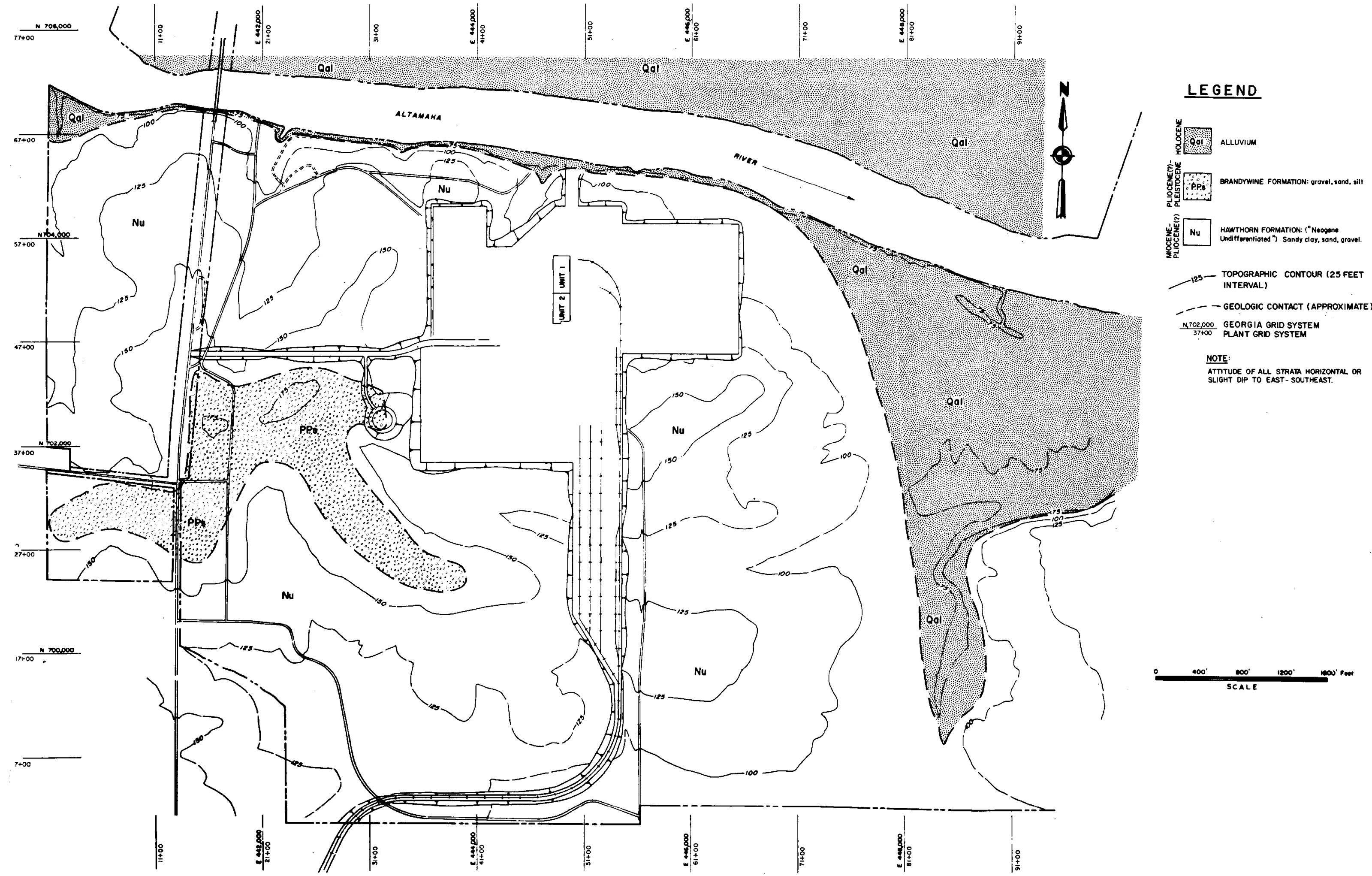
CONTOURS ON TOP OF UNIT 5 HAWTHORN FORMATION

FIGURE 2.5-9



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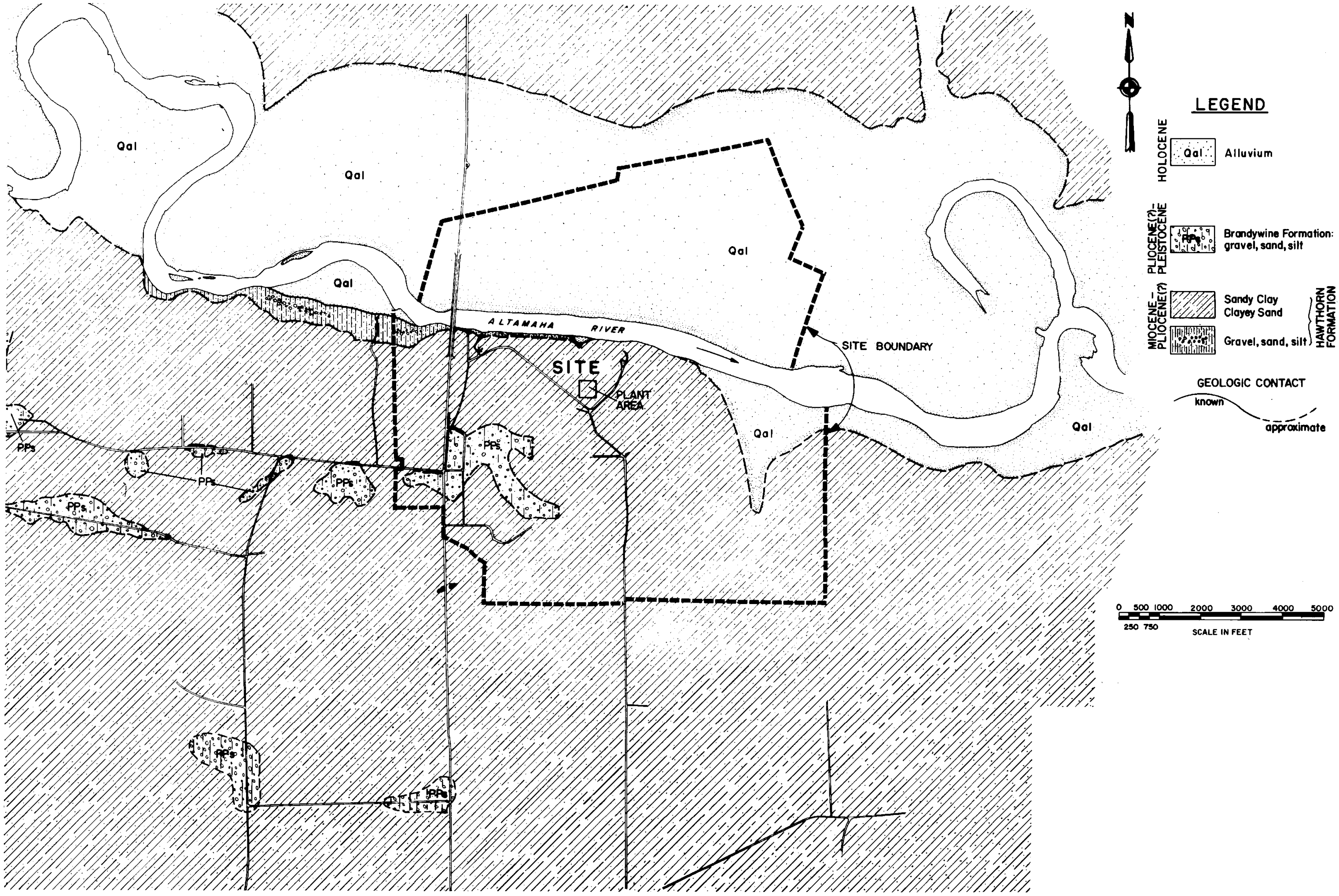
SITE GEOLOGIC MAP

FIGURE 2.5-10



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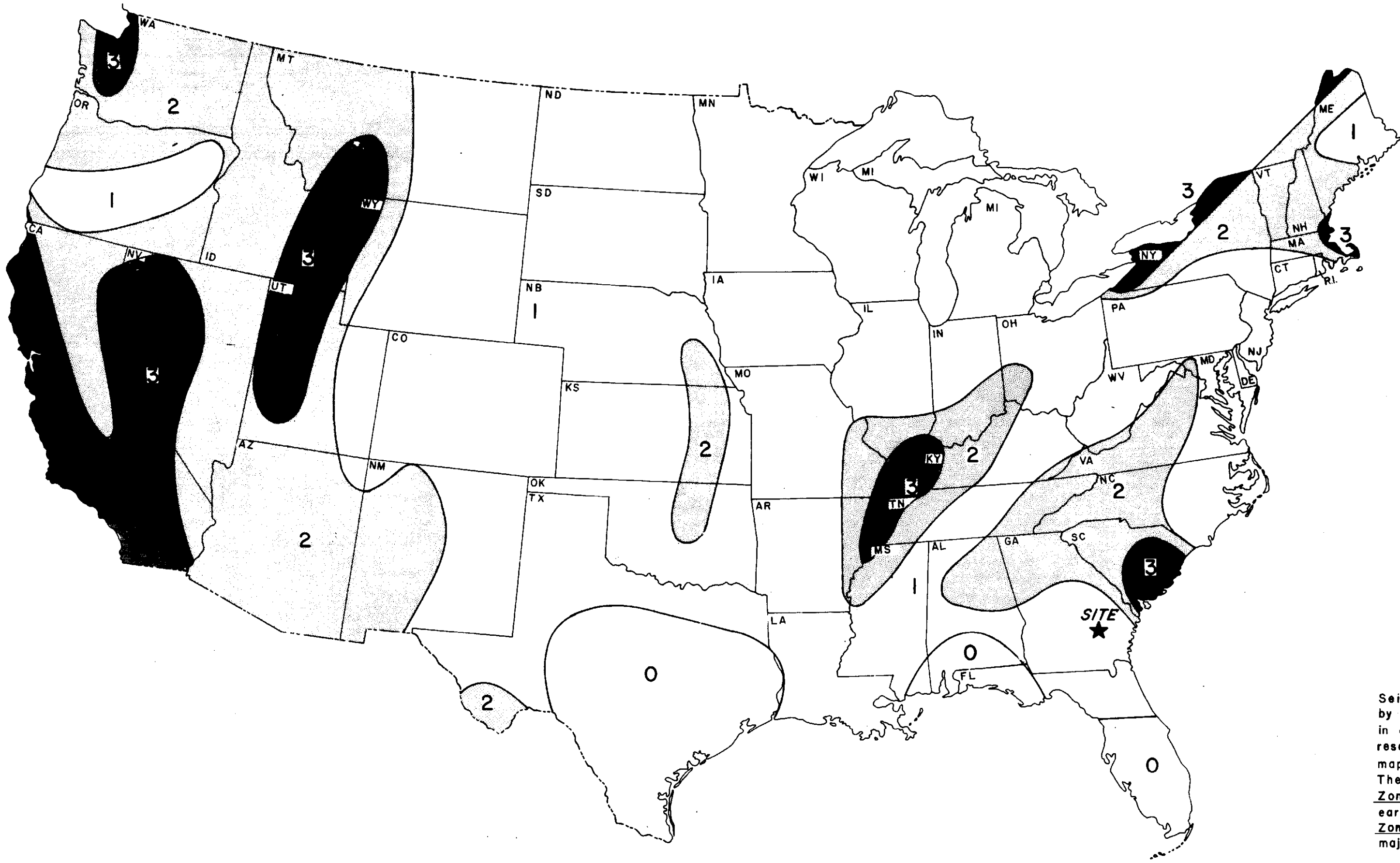
AREA GEOLOGIC MAP

FIGURE 2.5-11



LEGEND

DAMAGE

0 - NONE

1 - MINOR

2 - MODERATE

3 - MAJOR

Seismic risk map for conterminous U.S., developed by ESSA/Coast and Geodetic Survey and issued in January 1969. Subject to revision as continuing research warrants, it is an updated edition of the map first published in 1948 and revised in 1951. The map divides the U.S. into four zones: Zone 0, areas with no reasonable expectancy of earthquake damage; Zone 1, expected minor damage; Zone 2, expected moderate damage; Zone 3, where major destructive earthquakes may occur.

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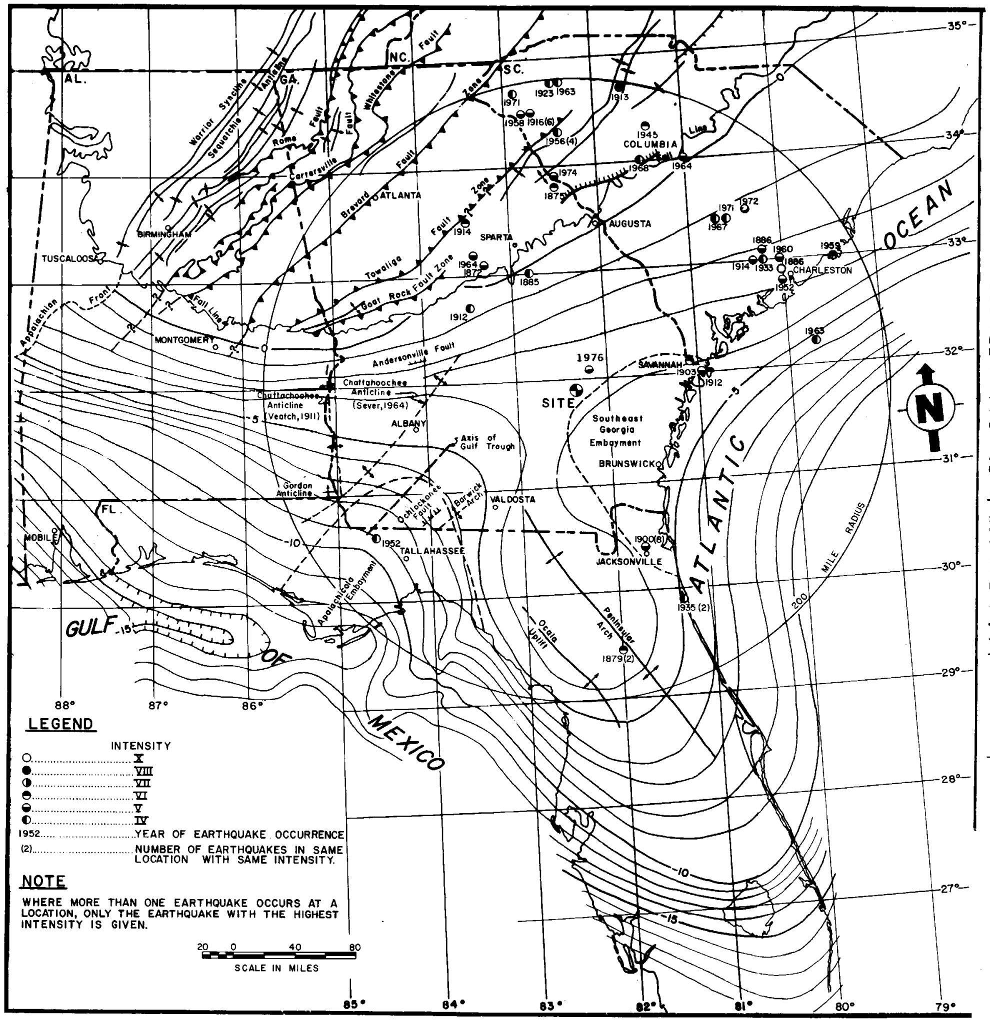
SEISMIC RISK MAP

FIGURE 2.5-12



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LEGEND

THRUST FAULT

NORMAL FAULT

ANTICLINAL AXIS

SYNCLINAL AXIS

STRUCTURE CONTOURS ON PRE-MESOZOIC SURFACE (IN THOUSANDS OF FEET)

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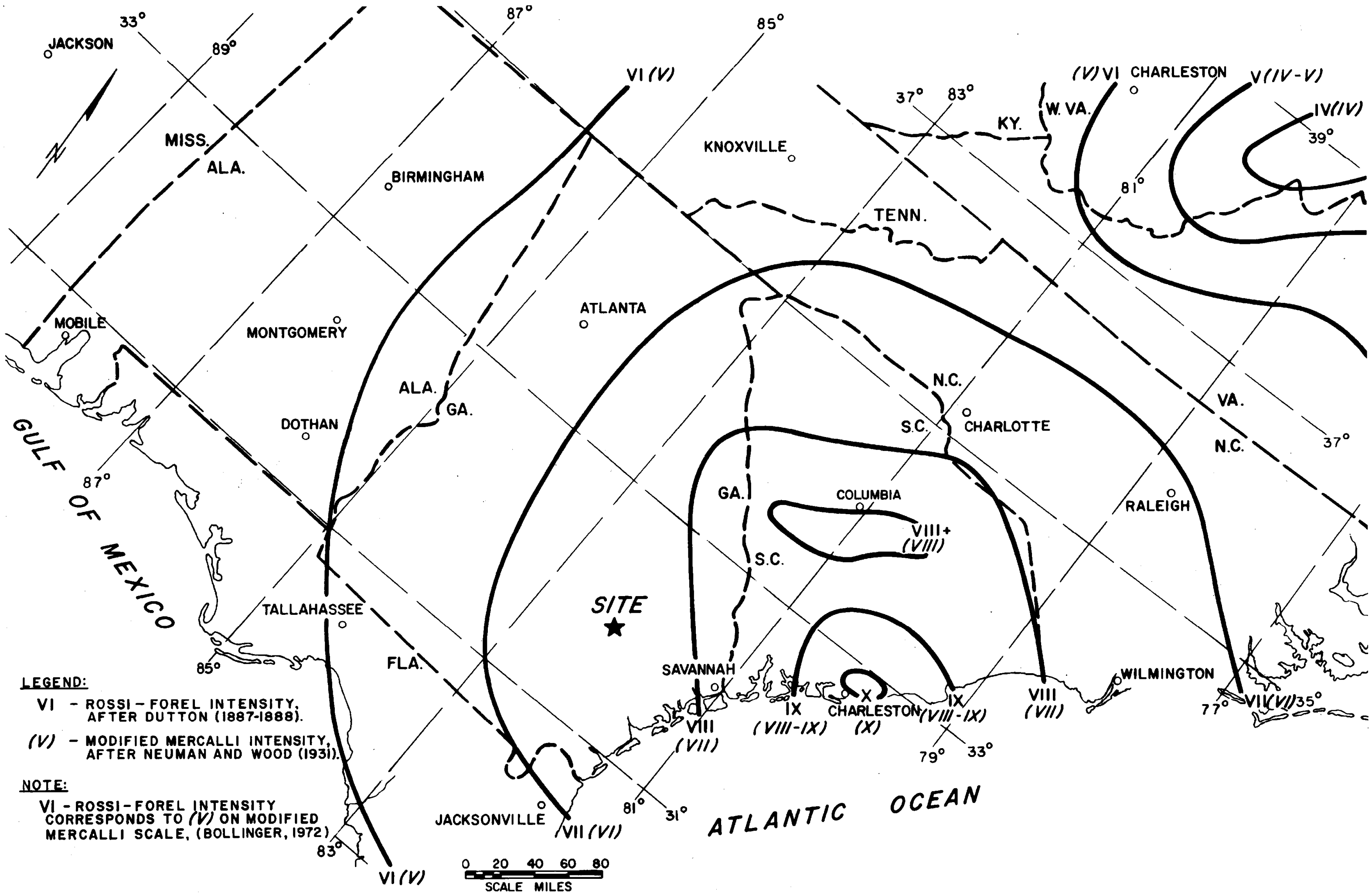
TECTONIC AND EPICENTER MAP

FIGURE 2.5-13



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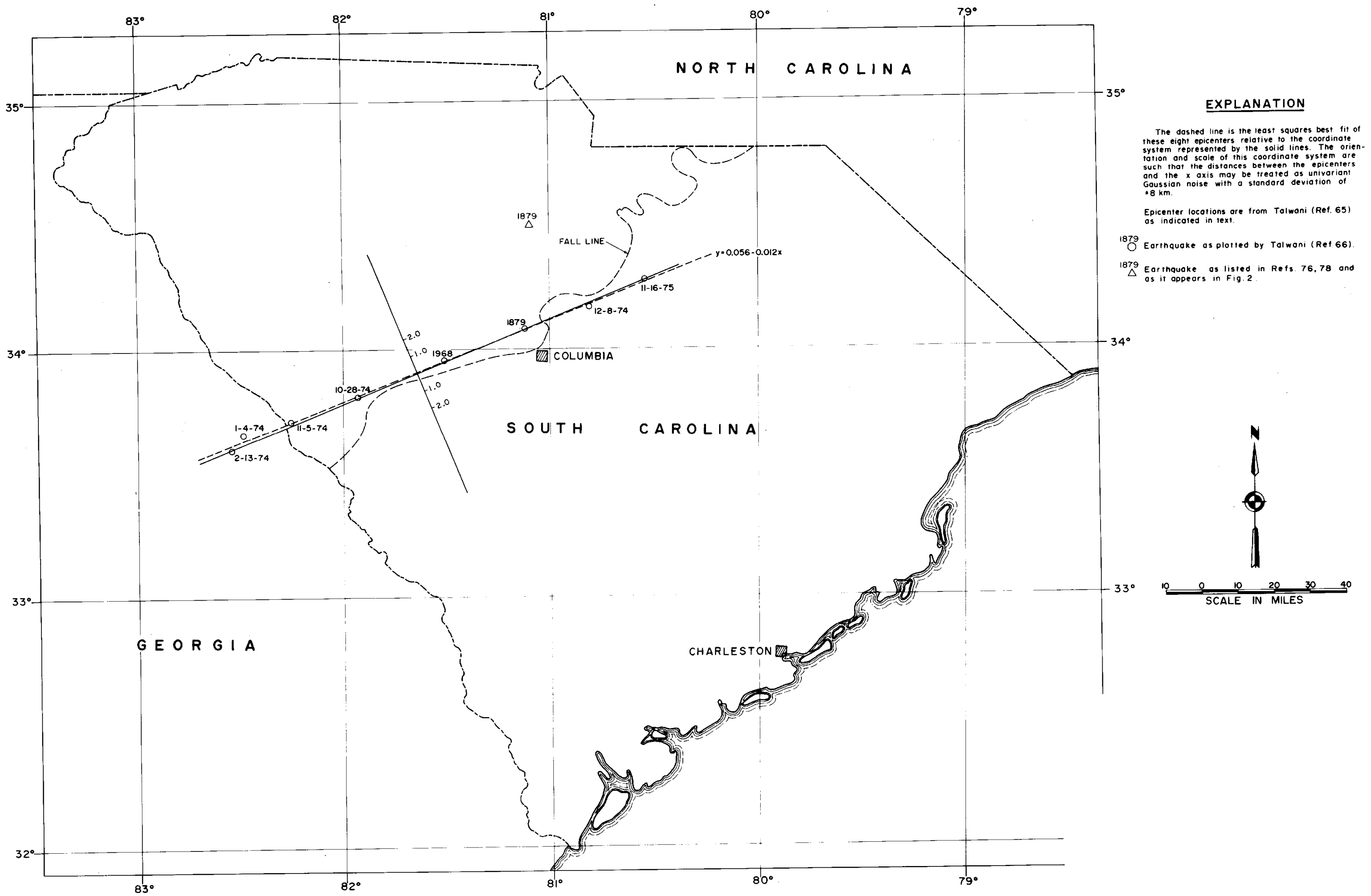
ISOSEISMAL MAP OF 1886 CHARLESTON, S.C., EARTHQUAKE

FIGURE 2.5-14



EXPLANATION

The dashed line is the least squares best fit of these eight epicenters relative to the coordinate system represented by the solid lines. The orientation and scale of this coordinate system are such that the distances between the epicenters and the x axis may be treated as univariant Gaussian noise with a standard deviation of ±8 km.

Epicenter locations are from Talwani (Ref. 65) as indicated in text.

1879 ○ Earthquake as plotted by Talwani (Ref 66).

1879 △ Earthquake as listed in Refs. 76, 78 and as it appears in Fig. 2.

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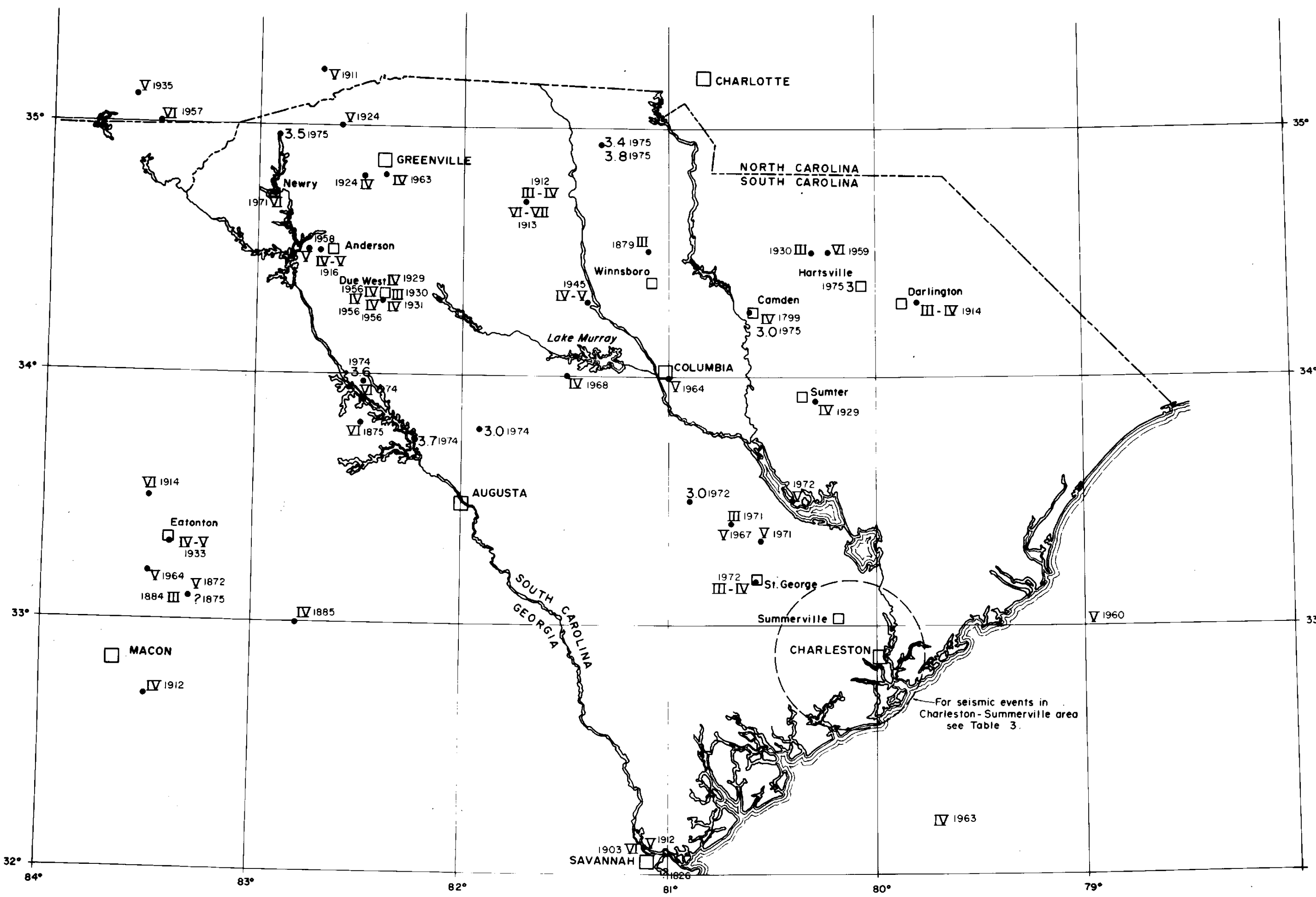
EPICENTRAL LOCATIONS NEAR THE GOAT ROCK FAULT

FIGURE 2.5-15



EXPLANATION

V — Roman Numeral indicates the epicenter and maximum intensity (Modified Mercalli) of an earthquake.

Intensity III and greater are shown.

3.5 — Arabic Numeral indicates the epicenter and magnitude of an earthquake.

Magnitudes 3.0 and greater are shown.

V 1903 — Dates of earthquakes are indicated.

Table 2 lists earthquakes shown on map.

Table 3 lists earthquakes centered in the Charleston - Summerville area.

? — See Table 2.

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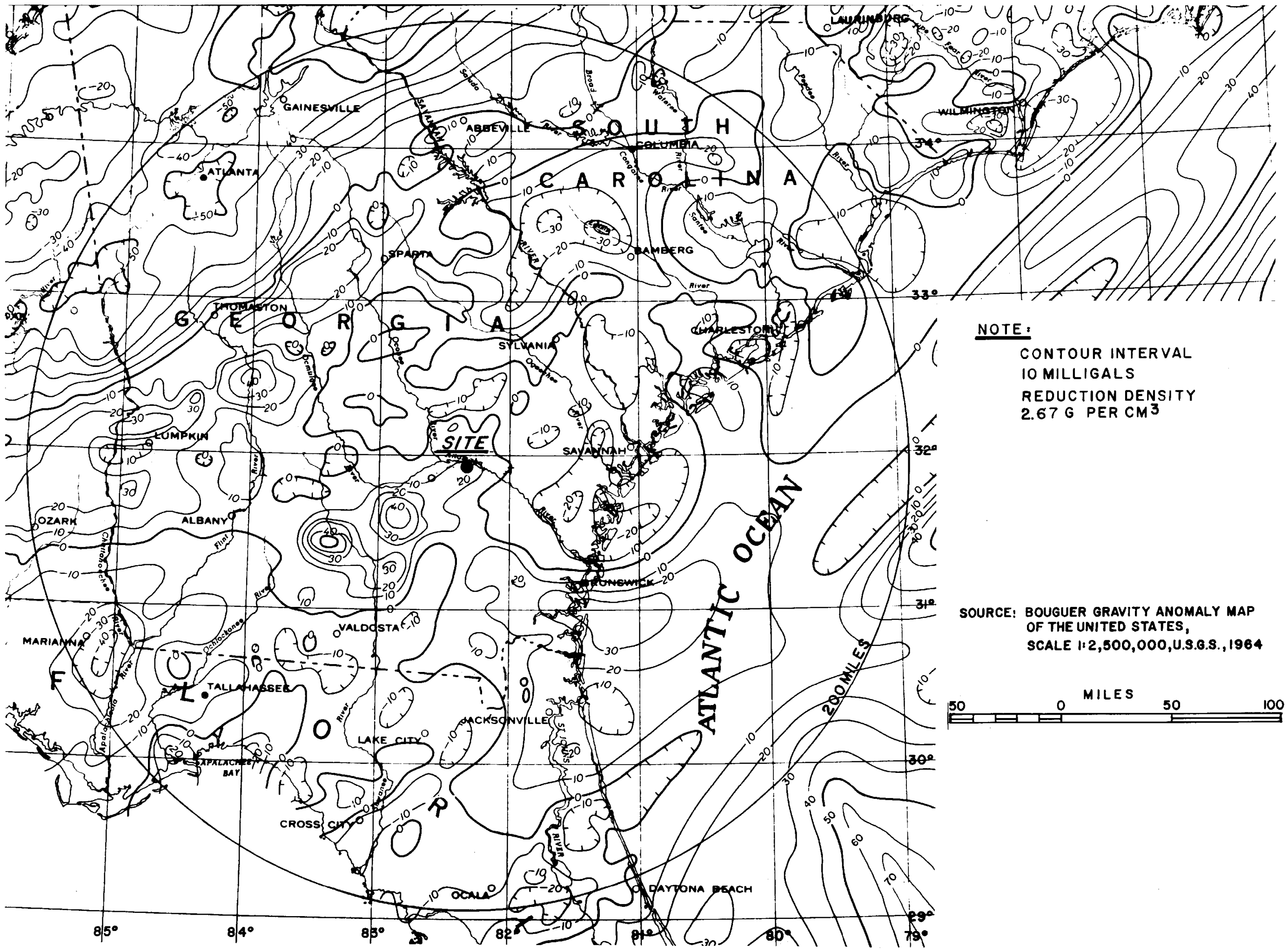
SOUTH CAROLINA EARTHQUAKE EPICENTERS 1799 TO 1975

FIGURE 2.5-16



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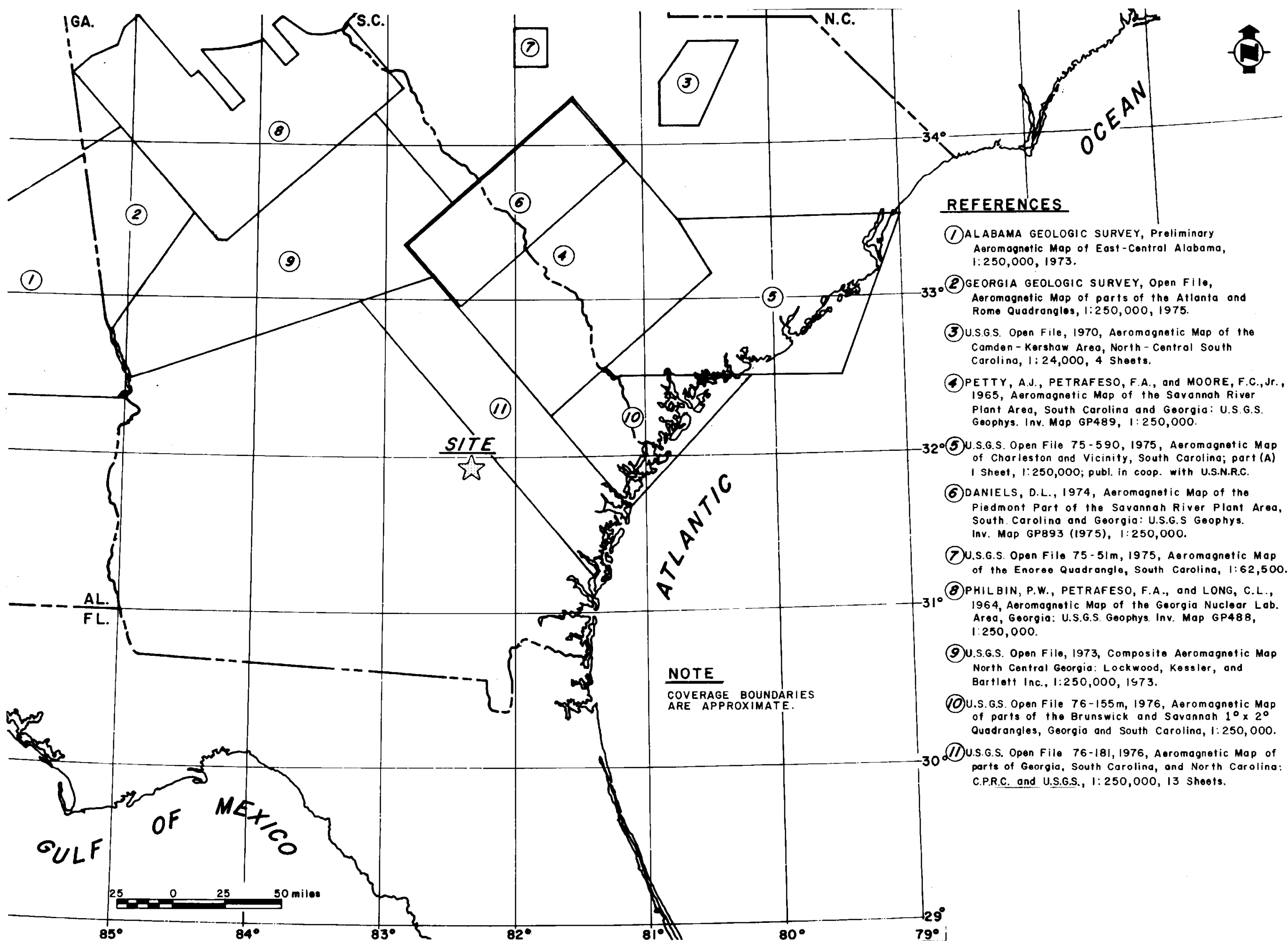
BOUGHER GRAVITY ANOMALY MAP
SOUTHEASTERN UNITED STATES

FIGURE 2.5-17



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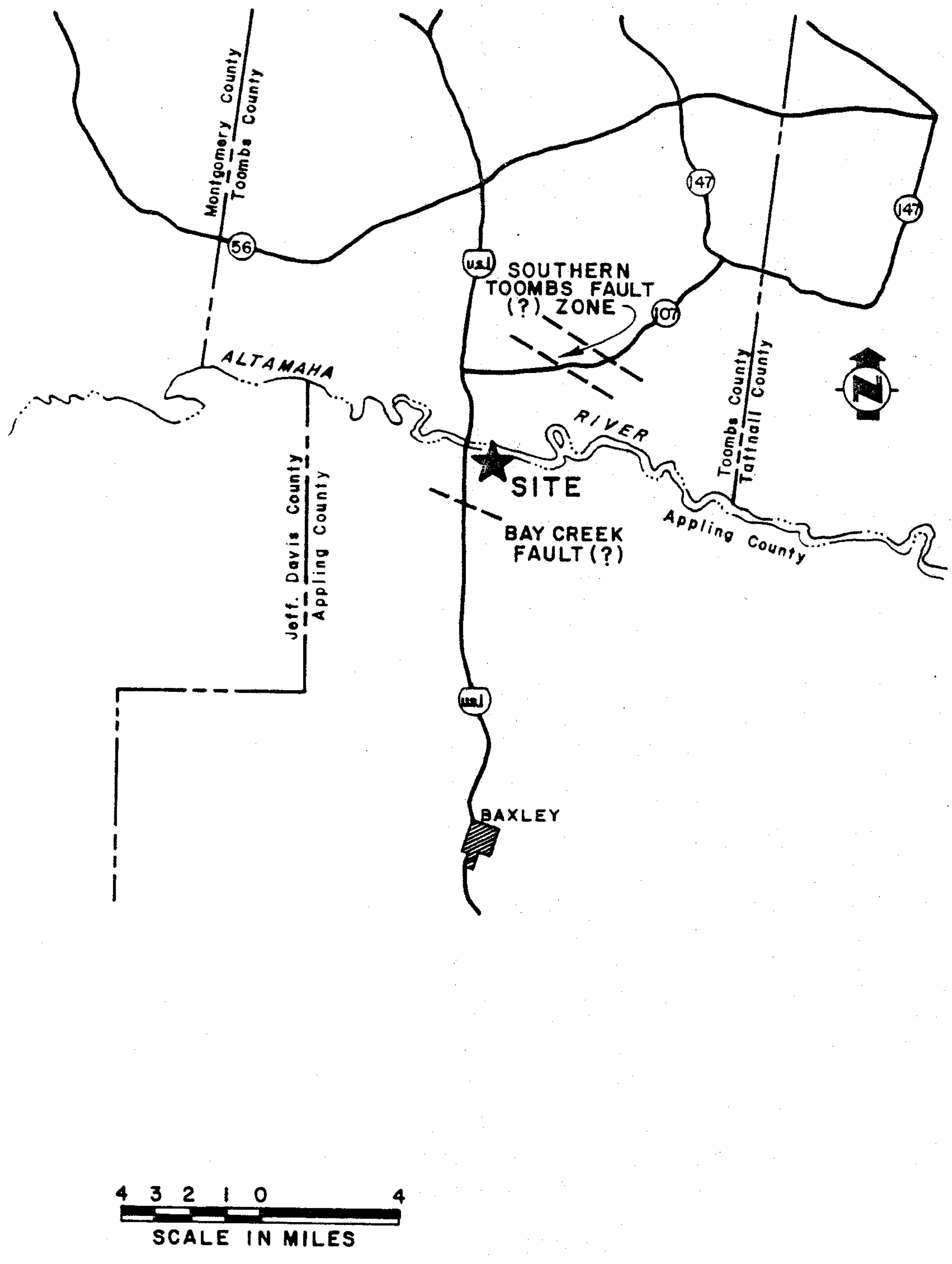
INDEX OF AEROMAGNETIC COVERAGE ALABAMA, GEORGIA, AND SOUTH CAROLINA

FIGURE 2.5-18



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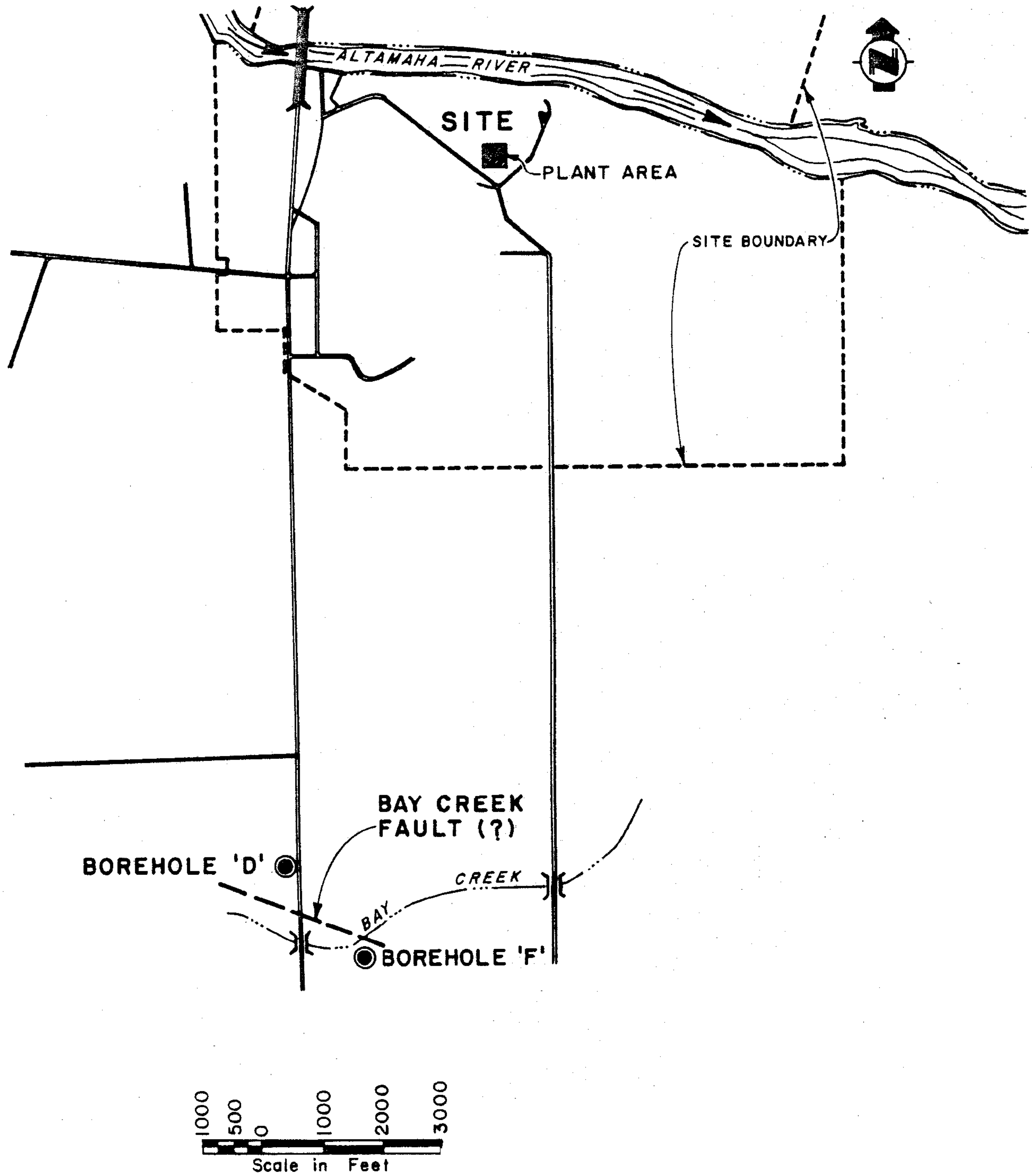
POSTULATED FAULTS IN SITE VICINITY

FIGURE 2.5-19



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SOUTHERN COMPANY *Energy to Serve Your World®* **SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 1 AND UNIT 2**	BOREHOLES AT BAY CREEK
	FIGURE 2.5-20

GEOLOGIC LOG — BORING NO. D — LOCATION: NORTH SIDE BAY CREEK

DEPTH	ELEV	DESCRIPTION	GEOLOGIC UNIT
0	135	CLAYEY SAND AND SAND, MEDIUM TO VERY COARSE, POORLY SORTED, PEBBLES COMMON.	HAWTHORN VI
	121	SANDY CLAY AND SAND, FINE TO COARSE, POORLY TO FAIR SORTED: CLAY CONTENT INCREASES WITH DEPTH: LOCALLY CEMENTED.	HAWTHORN V
	73	SAND AND CLAYEY SAND, FINE TO COARSE, POORLY TO WELL SORTED: LOCALLY CEMENTED: CLAY STRINGERS COMMON NEAR BASE: TAN PHOSPHATE SPARSE TO ABUNDANT.	HAWTHORN IV
	19	INTERBEDDED CLAYEY SAND AND SAND, MEDIUM TO COARSE, WELL SORTED: BROWN AND BLACK PHOSPHATE COMMON.	HAWTHORN III
	-3	SAND AND SANDSTONE, MEDIUM TO VERY COARSE, POORLY SORTED: BLACK AND BROWN PHOSPHATE ABUNDANT.	HAWTHORN II
	-37	SANDSTONE AND CLAYEY SAND, VERY FINE TO COARSE, POORLY TO WELL SORTED: BROWN AND BLACK PHOSPHATE COMMON TO ABUNDANT. CLAY CONTENT INCREASES WITH DEPTH: PHOSPHATE CONTENT DECREASES WITH DEPTH.	HAWTHORN I
	-130	CLAYEY SAND AND SANDY MARL, FINE TO COARSE, FAIR TO WELL SORTED: CALCAREOUS, FOSSILIFEROUS: PHOSPHATE SPARSE TO ABUNDANT.	TAMPA FORMATION
350		BORING TERMINATED AT 347 FEET	

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150, 160, 170, 180, 190, 200, 210, 220, 230, 240, 250, 260, 270, 280, 290, 300, 310, 320, 330, 340, 350

GEOLOGIC LOG — BORING NO. F — LOCATION: SOUTH SIDE BAY CREEK

DEPTH	ELEV	DESCRIPTION	GEOLOGIC UNIT
0	130	SANDY CLAY AND CLAYEY SAND, FINE TO VERY COARSE, POORLY SORTED, GRAVEL NEAR BASE	HAWTHORN VI
	115	SANDY CLAY AND CLAYEY SAND, FINE TO VERY COARSE, FAIR SORTED; CLAY CONTENT INCREASES WITH DEPTH; LOCALLY CEMENTED	HAWTHORN V
	67	SAND AND CLAYEY SAND, FINE TO COARSE, FAIR TO WELL SORTED; CLAY STRINGERS COMMON NEAR BASE; TAN PHOSPHATE RARE TO ABUNDANT, INCREASING WITH DEPTH.	HAWTHORN IV
	24	SAND, MEDIUM TO COARSE, WELL SORTED; TAN PHOSPHATE ABUNDANT; CLAY STRINGERS COMMON TO ABUNDANT.	HAWTHORN III
	-17	CLAYEY SAND AND SANDSTONE, VERY FINE TO VERY COARSE, POORLY TO WELL SORTED; BROWN AND BLACK PHOSPHATE COMMON TO ABUNDANT; OCCASIONAL CLAY LAYER. CALCAREOUS CEMENT NEAR BASE.	HAWTHORN II
	-48	CLAYEY SAND AND CLAY, VERY FINE TO COARSE, FAIR TO WELL SORTED; PHOSPHATE SPARSE TO ABUNDANT; CLAY CONTENT INCREASES WITH DEPTH, WITH CLAY LAYER 235 FT. TO 258 FT. OCCASIONAL SLICKENSIDES *	HAWTHORN I
	-128	CLAYEY SAND AND SANDY MARL, FINE TO VERY COARSE, POORLY TO WELL SORTED; CALCAREOUS, OCCASIONAL LIMESTONE; PHOSPHATE COMMON TO ABUNDANT FOSSILIFEROUS.	TAMPA FORMATION
350		BORING TERMINATED AT 345	

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150, 160, 170, 180, 190, 200, 210, 220, 230, 240, 250, 260, 270, 280, 290, 300, 310, 320, 330, 340, 350

* IN THIS CASE, THE SLICKENSIDES ARE SLIPPAGE PLANES CONSIDERED CONTEMPORANEOUS WITH CONSOLIDATION OF THE CLAY LAYER, CAUSED BY STRESSES INITIATED BY WEIGHT OF OVERLYING SEDIMENTS

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TEST BORING RECORD
BORING NUMBERS D AND F

FIGURE 2.5-21

SUPPLEMENT 2A

SUBSURFACE INVESTIGATION AND FOUNDATIONS

INTRODUCTION

This supplement presents the findings and conclusions of the subsurface soil investigation and foundation analyses for the Edwin I. Hatch Nuclear Plant (HNP) in Appling County, Georgia, performed by Law Engineering Testing Company.

The HNP-2 site is situated within the Coastal Plain Geologic Province which is underlain entirely by sedimentary deposits. The sediments at the site include Holocene alluvium near the river and Miocene to Pliocene(?) deltaic and marine deposits elsewhere. Beneath the Miocene deposits are older, soft rock-like deposits ranging in age from Oligocene to Cretaceous with an estimated thickness of over 4000 ft.

The HNP-2 site was evaluated and selected after detailed and extensive geologic field mapping, geophysical explorations, air photo reconnaissance and interpretation, aerial inspection, piezometer observations, subsurface exploration and sampling, and materials identification, classification, and testing. Site geology and seismology are discussed in section 2.5.

2A.1 FIELD EXPLORATION

Foundation investigations and evaluations were performed on a continuing basis as required. These investigations included test borings made for the design of principal structures of HNP-1 and HNP-2. Borings 587 through 600 have verified that previous soils data are applicable to HNP-2. The borings locations and the locations of Category 1 structures are shown on figures 2A-1 and 2A-2.

In addition to the above, sampling and testing of soils for backfill design were accomplished. This included the location of borrow areas for backfill material sources. Soils evaluations for location and design of the condenser cooling water intake structure and offgas stack have been accomplished. Details of sampling, testing, and evaluations are recorded in the following subsections.

2A.1.1 PLANT LOCATION

The plant site is in Appling County, Georgia, on the south bank of the Altamaha River about 3500 ft downstream from the river's intersection with U.S. Highway No. 1, 10 miles north of Baxley, Georgia, and 17 miles southwest of Reidsville, Georgia.

2A.1.2 RECONNAISSANCE

During field exploration, intensive studies of available geologic literature pertinent to the area were made. An extensive air photo study was made covering an area 90 miles east-west by 70 miles north-south (6300 square miles). From these photos, detailed studies of stream patterns, densities, and anomalies were made. These features, as well as topographic features reflecting geologic structure, were studied.

2A.1.3 BORING, SAMPLING, AND GROUNDWATER INVESTIGATION

Site investigations included soil test borings made at ~ 136 locations. Of the total, 64 borings were completed for the principal purpose of soil classification, analysis, and testing to establish foundation design criteria for the principal structures. Twenty-eight soil test borings were located within or in areas immediately adjacent to the reactor, radwaste, and turbine building foundations. These are presented in figures 2A-1 and 2A-2. Borings 562 through 575 and RFI-1 through RFI-10 were performed during construction as part of the foundation inspection of HNP-1 and HNP-2, respectively.

The subsurface investigation in the stack area consisted of 5 soil test borings, B-477 through B-481, which were made on 50-ft centers along the tentative east-west centerline of the stack. Two additional borings, B-584 and B-585, were made at the selected location of the stack along the north-south centerline.

Borings B-576 through B-578 were made in the Altamaha River, drilling from a barge, at the location of the temporary cellular cofferdams. Borings B-579, B-580, B-586, and B-603 were made within or immediately adjacent to the intake structure. Boring B-581 was made near the top of the bluff, south of the intake structure.

Soil sampling and penetration testing were performed in accordance with American Society of Testing Materials (ASTM) Specification D 1586-67.[1] Representative portions of the soil samples thus obtained were placed in glass jars and transported to the soils laboratory. In the laboratory, the samples were examined to verify the driller's field classifications. Test boring records are included in supplement 2B; they graphically show soil descriptions and penetration resistances. Based on the site borings, subsurface profiles for HNP-2 are shown as figure 2A-3.

Split tube samples were used for visual examination and classification tests. Undisturbed samples were obtained by forcing sections of 3-in. OD tubing into the soil at the desired sampling levels. This sampling procedure is described by ASTM Specification D 1587. Each tube, together with the encased soil, was carefully removed from the ground, made airtight, and then transported to the laboratory. Locations and depths of undisturbed samples are shown on the test boring records (supplement 2B).

In order to study the ground water levels at this site, piezometers were installed both in the plant area and to distances of 1 mile outside the plant area. Cased open-hole piezometers were installed to measure surficial groundwater levels. Deep piezometers, sealed in the less pervious strata above, were installed at various elevations to measure piezometric levels of deeper pervious strata. The locations and levels of piezometers are presented in subsection 2.4.13.

Field variable head permeability tests were performed in selected piezometers in accordance with the procedures outlined by the Department of the Navy, Bureau of Yards and Docks Design manual.[2] Field well permeameter tests were performed near the surface in accordance with the procedures outlined by the Bureau of Reclamation.[3]

2A.1.4 GEOPHYSICAL EXPLORATION

Seismic traverses were conducted to determine compressional and shear wave velocities through the overburden soils above el 0 ft. Two traverse lines ~ 600 ft long were run at approximate right angles to each other (figure 2A-4). One traverse was run approximately along the proposed centerline of the HNP-1 turbine building and the other was run through the HNP-1 reactor and turbine buildings. Seismic velocities were determined by refraction methods using a Dresser RS-4 12-channel seismograph. Dynamite blasts placed in hand augered holes ~ 3 ft below the surface were used as the energy source. Geophone spacings of 25 and 50 ft were utilized.

The data obtained from the refraction surveys are shown on figures 2A-5 and 2A-6. The average velocities within the materials to a maximum depth of 100 ft below the ground surface are:

- *Compressional wave velocity = 6600 ± 300 ft/s.*

- *Shear wave velocity = 2450 ± 200 ft/s.*

These data are refraction measurements and, therefore, horizontal velocities. The velocity of vertically propagated waves is ~ 80 to 85% of these values. These velocities represent a single refracting layer which extends to an undetermined depth. They are representative average velocities to a depth of at least 50 ft. No significant velocity increase appears to exist to a depth of l00 ft below the ground surface.

In addition to measurement of compressional and shear velocities, Rayleigh wave arrivals were observed on the seismograph records. The Rayleigh wave velocity observed is about 2200 ± 200 ft/s. This is in good agreement with the measured shear wave velocities.

2A.2 <u>SITE CONDITIONS</u>

2A.2.1 TOPOGRAPHY AND DRAINAGE

The topography of the site is a gently rolling surface sloping toward the river. The elevation of the site along the river is ~ 75 ft,[a] and it rises to ~ el 172 ft in the southern portion of the site. Thus, there is about 100 ft of relief.

In the northwestern corner of the site, the south bank of the Altamaha River is bordered by a narrow floodplain. However, in the area of the site, the floodplain broadens rapidly at an approximate

---

a. All elevations refer to USC and GS mean sea level (msl) in feet.

35-degree angle with the course of the river. This floodplain enlargement is the result of an old stream channel, as shown by the meander scar on figure 2A-7.

The drainage of the site is mainly accomplished by a wet weather branch or a drainage swale that nearly bisects the site in a northeast-southwest direction. This drainage feature branches where it intercepts the floodplain. One of the branches flows northwest into the river while the other branch generally flows east paralleling the river for about a mile before intercepting it. Both follow the old channel.

2A.2.2 SOIL CONDITIONS

The site is heavily wooded and in the floodplain area is covered with dense underbrush. In the majority of locations where test borings were made, the site is covered with a surface veneer of topsoil and organic debris. In some locations, borings were placed along existing roads where this organic layer is thin or absent.

The general soil conditions near the site can be characterized in four different areas. The different soil characteristics peculiar in each area occur in the upper soils above ~ el 50 ft. The deeper indurated soils, below el 50 ft, underlying the entire area are quite similar in character and composition. The conditions are portrayed in the cross sections of the plant site which show the significant stratifications and the soil penetration resistances (figure 2A-3). Detailed soil descriptions and penetration resistance data are shown on the boring logs in supplement 2B.

2A.2.2.1 Central Area

Borings B-401, B-411, B-425, and B-426 were made in the central portion of the proposed plant location area. These borings were made well south of the rocky bluff along the river. The ground surface generally varies between el 125 to 130 ft, with boring B-411 on a small ridge with the ground surface elevation of 144 ft.

The uppermost soil stratum in this area consists of firm to dense multicolored silty sands and clayey sands containing scattered clay lenses. The lower boundary of this stratum is between el 100 to 110 ft.

Underlying these surficial sands is a zone of very dense sand to ~ el 80 ft. This dense zone is cemented to varying degrees, creating lens-like to massive rock strata within the zone. The penetration resistances in this zone are generally greater than 100 blows per ft. Within this zone are scattered, very hard, partially cemented clay lenses. The bottom of the zone is indistinct, but approximately at el 80 ft.

Underlying this dense, partially cemented zone and extending approximately to el 0 ft are irregular strata of fine sands and silty sands. These fine sands are generally dense to very dense with penetration resistances from 30 to 50 blows per ft. However, there are erratic zones that are somewhat looser. These zones generally occur between el 80 and 40 ft. Within this same elevation range, the fine sands are somewhat clayey or silty. Below el 40 ft, the fine sands are generally clean; however, silty zones are scattered throughout this zone. Also within this fine sand zone are irregular lenses and nodules of hard clays, which, in many instances, are partially cemented.

Below el 0 ft are generally very dense, gray green fine sands with clay seams. Occurring within this zone are lenses or layers of cream-colored calcareous fine to medium sand, some of which are partially cemented.

2A.2.2.2 <u>Eastern Area</u>

The second area of relatively consistent soil conditions is defined by borings B-402, B-404, B-405, B-406, B-407, B-408, B-410, B-416, B-418, B-427, B-430, B-431, and B-432. In this area there is a surface zone of firm-to-dense, multicolored clayey and silty sands with erratic clay lenses between el 100 to 120 ft, similar to that in the central zone.

Underlying these multicolored sands are stiff, light gray, fine sandy clays and firm, clayey fine sands. The base of this unit is between el 80 to 90 ft.

These sands and clays are underlain by a widespread, thick zone of firm to stiff, light gray green and tan plastic clay which extends as deep as els 60 to 70 ft. These clays display the potential to undergo volume change (swell) with moisture fluctuation or stress relief, as verified by X-ray diffraction identification and swell tests. The base of this plastic clay is sometimes marked by a very hard, partially cemented clay or partially cemented sand layer of limited thickness (2 to 5 ft).

Underlying these clays are firm to very dense fine sands with clay seams. These are similar to the fine sands encountered below el 80 ft in the central portion of the site. These fine sands, in turn, are underlain by the same very dense gray green fine sands with clay seams previously described in the central area.

2A.2.2.3 <u>Intermediate Area</u>

Between these two areas of relatively different soil conditions is a transition zone marked by borings B-412, B-413, B-417, B-423, B-424, B-428, B-429, and B-435. Again, these borings encountered a zone of firm-to-dense, multicolored clayey and silty sands with clay zones that extend to between els 100 and 110 ft.

Underlying these surficial sands is a hard layer which extends to ~ el 75 ft. This hard layer displays partial cementation and is similar to the cemented layer encountered between el 110 and 80 ft by the five borings in the central portion of the site, initially discussed in this section. The hard layer encountered by these transition borings is poorly defined. However, throughout this hard layer there are marked higher penetration resistances.

The partially cemented or hard zone is underlain by interlayered firm sands and plastic clays. The base of this zone is quite irregular and lies between el 45 to 60 ft. Except at B-423 and B-424, these interlayered sands are looser and the clays are softer than their equivalents that underlie the cemented sands and clays in the central portion of the site. The clay seams are similar in plasticity and mineralogy to the plastic clays encountered in the remainder of the site.

Underlying these interlayered sands and clays are dense to very dense fine sands with clay lenses and seams. These sands are similar to the dense sands generally encountered below el 50 ft over the entire site.

2A.2.2.4 Floodplain

The fourth area of generally consistent soil conditions is along the floodplain of the river where the upper zones of sands and clays, described in the previous paragraphs, have been eroded by the river and replaced by Altamaha River alluvium. Borings B-403, B-409, B-419, B-420, B-421, and B-422 were made in the floodplain area of the river. All of these borings encountered Holocene alluvial soil underlying the organic topsoil. In B-4O3 and B-4O9, the alluvium extends to depths of 12 and 13 ft and consists of firm multicolored clayey sands. In borings B-419 through B-422, the alluvial soils initially are stiff to firm mottled fine sandy clays. These soils become coarser with depth and grade to loose and firm sands at depths varying from 4 to 13 ft. These alluvial soils extend to depths varying from 18 to 24 ft. Corresponding elevations are from 58 to 48 ft.

Underlying the alluvium are firm and dense silty fine sands which contain some hard and partially cemented clay seams or layers. These sands and clays are similar to the dense sands encountered throughout the site below els 45 to 80 ft. At borings B-42O and B-421, interlayered fine sand and plastic clay, similar to that described in the intermediate area borings, was found above el 33 to 34 ft, respectively.

Borings B-576 through B-578 were made in the Altamaha River, drilling from a barge, at the location of the temporary cellular cofferdams. Borings B-579, B-580, B-586, and B-603 were made within or immediately adjacent to the intake structure. Boring B-581 was made near the top of the bluff, south of the intake structure. Boring B-581 encountered the intact geologic sequence as described previously.

Moving northward from boring B-581, the upper portions of the geologic sequence have been removed by past activity of the Altamaha River. In some areas, erosion has been followed by deposition of recent river alluvium at the ground surface. Borings B-576 through B-580 and B-586 encountered alluvium at the ground surface. In these borings, the thickness of the alluvial blanket varied from 4 ft in B-577 to 19.5 ft in B-579. Minimum elevations of the base of the alluvium ranged from 49 ft in boring B-457 to 68 ft in boring B-580.

2A.2.2.5 Plant Location Soil Conditions

The soil conditions determined by borings in the power block area are generally consistent with the soil conditions described in subsection 2A.2.2.1. Geologic sections are shown on figure 2A-3. The locations of the borings representing the power block are shown on figure 2A-2.

Detailed soil descriptions and penetration resistances are shown on the boring logs in supplement 2B. The uppermost soils in the plant area consist of firm to very dense purple, brown, and gray clayey fine to medium sand with some clay layers. These soils were encountered from the surface to depths of 18 to 29 ft or to between el 110 and 120 ft.

The next unit encountered beneath the firm to very dense clayey sand zone is a stratum of very dense gray clayey fine to medium sand which extends to ~ el 75 ft. These dense sands are generally described as partially cemented sands. Within this zone are scattered layers and inclusions of very hard cemented clay and dense sands.

The cemented sands are underlain by firm to very dense gray green sand and clayey sands to ~ el 0 ft. Thin layers of lenses, between 3 and 6 ft thick, of very stiff to hard plastic clay were found between el 60 and 70 ft by borings B-411, B-440, B-445, R-449, T-463, and T-464. These clays are harder than their equivalents found at other areas of the site. Thicker layers of firm gray green slightly clayey fine sands, between 5 and 15 ft in thickness, were found between el 50 and 65 ft by borings B-440 through B-443, R-444, T-445, B-446, B-448, B-449, and R-466. Penetration resistances within these zones generally ranged between 9 and 18 blows per ft.

Very hard gray green silty clays were encountered below ~ el 0 ft.

2A.3 LABORATORY TESTING (HNP-1 AND HNP-2)

Each undisturbed sample and split-spoon sample obtained during the field operations was carefully inspected by a soils engineer and representative samples were selected for detailed testing. Each selected sample, still in its steel tube, was cut into sections with a high-speed abrasive saw. Portions of each sample were removed for moisture and specific gravity determinations. From these data, the sample's void ratio, unit weight, and saturation were calculated.

2A.3.1 ROUTINE CLASSIFICATION

Detailed classification tests were made of selected samples. These tests included Atterberg Limits on the clayey samples and grain size determinations on the sandier samples. The liquid limit is the moisture content at which the soil will flow as a heavy viscous fluid and is determined in accordance with ASTM D 423-66. The plastic limit is the moisture content at which the soil begins to lose its plasticity and is determined in accordance with ASTM D 424-59. The soil's plasticity characteristic is represented by its plasticity index (PI) which is the difference between the liquid limit (LL) and the plastic limit (PL). The results of Atterberg Limits testing are included in table 2A-1. Also, in situ moisture contents are shown on table 2A-2.

The grain size tests were performed on the sandier samples. The grain size distribution of particles coarser than the No. 200 sieve was determined by passing the samples through a standard set of nested sieves. In most samples, the materials were sufficiently fine that it was necessary to wash the fraction finer than the No. 200 sieve through the sieve. In those samples where the material passing the No. 200 sieve was a major fraction of the total sample, the fine materials were suspended in water and the grain size distribution measured by their rate of settlement. These tests were conducted in accordance with ASTM Specifications D 421-58 and D 422-63. The results of all grain size tests are given in table 2A-1. The grain size curves for borings from the power block area, HNP-1 and HNP-2, and the intake structure are shown on figure 2A-8.

2A.3.2 X-RAY DIFFRACTION TESTS

The minerals comprising the sediments were identified by X-ray diffraction and petrographic techniques. The clay fraction was separated from the sand fraction and made into a slurry of known clay concentration. The clay slurry was placed on petrographic slides and air dried. One of the sedimentation slides, thus prepared on each sample, was further treated with ethylene glycol at 60°C for

1 h. Both the air-dried sedimentation slide and glycol-treated slides were then irradiated on a Phillips X-ray diffraction unit and the X-ray diffractogram analyzed for characteristic clay minerals. The results are presented as the tabulation of X-ray diffraction tests, in table 2A-3.

2A.3.3 CONSOLIDATION AND SWELL TESTS

Consolidation and swell tests were performed on representative samples obtained from each soil type or stratum encountered by the borings. Standard testing procedures in accordance with ASTM D 2435-65T were utilized in performing the tests. In all tests, unidimensional consolidometers were used. All consolidation samples were carefully trimmed to discs 2 in. in diameter and 1-in. thick. These discs were sandwiched between porous stones and placed in a stainless steel ring. Samples were loaded to their in situ effective overburden pressure in three increments of load. The samples were then unloaded to simulate the condition of general excavation during construction. The minimum unloaded pressure utilized was 300 lb/ft^2. At the unloaded condition, the samples were given free access to water and allowed to swell or consolidate fully. After all deformation on the micrometer dial had terminated at this inundated-unloaded condition, the samples were loaded in increments to axial pressures of 16,000 lb/ft^2.

The results of consolidation tests are given in table 2A-4. The void ratio vs pressure curves for borings from the power block area are shown on figure 2A-9. Consolidation test results for HNP-1 and HNP-2 are compared in figure 2A-10.

2A.3.4 TRIAXIAL SHEAR TESTS

Quick triaxial compression tests were performed on selected undisturbed samples. These samples were extruded from their sampling tubes and trimmed into cylinders 1.4 in. in diameter. Sample lengths were between 2 and 2.5 times their diameter. Some samples contained numerous inclusions of hard clay which made sample trimming difficult. In these cases, sample diameters of 2.8 in. were utilized. The trimmed samples were encased in rubber membranes and placed in a compression chamber. Each test consisted of a series of three samples and each sample was confined at different air pressures. Axial load was applied to each sample until it failed in shear. The results of all triaxial shear tests are given in table 2A-5. The Mohr envelopes and stress-strain curves for borings from the power block area are shown on figure 2A-11.

2A.3.5 CYCLIC TRIAXIAL COMPRESSION TESTS

Dynamic triaxial sheer tests were performed on undisturbed samples to evaluate the potential for liquefaction due to repetitive loading. The samples tested were selected from the most sandy portions of the proposed foundation materials between approximate el 50 and 80 ft, where penetration resistances below 20 blows per ft were encountered. Samples representing only the loosest conditions were tested.

Samples of 1.4-in. diameter were utilized for these tests. These samples were encased in rubber membranes and placed within the triaxial compression chamber. Saturation of the samples was obtained by inducing a back pressure within the sample. The samples were allowed to remain under this back pressure for periods of between 12 and 48 h. After saturation was essentially complete, the back pressure and chamber pressure were simultaneously increased. The samples were then allowed to remain under

this back pressure condition until equilibrium was reached, but not < 1 h. In all cases, the equilibrium time was under 1 h.

Confining pressures of from 4000 to 6000 lb/ft^2 were utilized for the testing. These are comparable to the average effective confining stresses at the site after construction grading has been completed. A wide range of axial deviator stresses was employed, from ± l500 lb/ft^2 to ± 3100 lb/ft^2. The largest deviator stresses were greater than the shear stresses generated by potential earthquakes. The range was designed to establish the relation between cyclical shear stress and the number of cycles required for liquefaction of the loosest sandy soils at the site.

Initial liquefaction was defined as the point where the pore water pressure equals the confining pressure. The corresponding strains were generally between 1 and 5%, although a few were as much as 10%. The condition, therefore, is not the complete liquefaction defined by continuing large strains. Instead, the definition used herein lies between the initial and partial liquefaction described by Seed and Lee[(4)] and is referred to as momentary liquefaction. Large strains, denoting complete liquefaction, generally required many additional cycles. However, the condition is more difficult to define accurately from the test data.

The results are summarized in figure 2A-12 in which the deviator stress is plotted as a function of the number of cycles required to produce liquefaction as herein defined. The results have been normalized by expressing the deviator stress as a fraction of the confining stress. A second curve, figure 2A-13, shows the deviator stress required to produce complete liquefaction.

2A.4 <u>STRUCTURAL DATA</u>

HNP-2 structures are of similar size and produce similar loadings as the structures for HNP-1; also, the elevations of these structures coincide with the elevations of the structures of HNP-1. The following information applies to HNP-2.

2A.4.1 SITE GRADING

The original site level varied from el 75 ft near the river to ~ el 145 ft at the power block area, el 125 ft at the switchyard, and el 118 ft at the cooling towers. Excavations ranging between el 107 and 75 ft were made for the reactor building, turbine building, and radwaste building.

2A.4.2 REACTOR BUILDING

The reactor building is 149 ft by 149 ft in plan area. The static foundation pressures range between 6.2 and 10.1 ksf with a 14.77 ksf maximum edge pressure under maximum transient load conditions. The level for the bottom of the foundation mat is at el 75 ft.

2A.4.3 TURBINE AND CONTROL BUILDING

The combined turbine and control building is 355 ft by 160 ft in plan area. Within the turbine building, the turbine pedestal is 175 ft by 40 ft. The average static foundation pressures for the building and

pedestal is 6 ksf. The bottom of the mat foundation is at el 105 ft. Intake and discharge water passages extend beneath the turbine pedestal to about el 87 ft.

2A.4.4 RADWASTE BUILDING

The radwaste building is an L-shaped structure in plan with dimensions 142 ft 3 in. x 93 ft 9 in. in the north-south sides and 121 ft 0 in. x 86 ft 6 in. in the east-west sides. The maximum static foundation pressure is 8.48 ksf. The bottom of the mat is at el 96 ft.

2A.4.5 INTAKE STRUCTURE

In plan view, the major portion of the structure is rectangular with dimensions ~ 66 ft by 53 ft. A smaller portion of the structure, projecting north, is ~ 45 ft by 27 ft. The gross weight of the structure is 19,400 kips, producing a contact pressure of 4100 lb/ft^2 at el 52 ft over the total foundation area.

Surrounding the intake structure is a system of cellular cofferdams which have top elevations ranging from 85 to 105 ft. The fill surface is graded to slope from el 110 ft at the intake structure to the cofferdams.

The intake structure foundation was inspected by means of soil test borings, laboratory grain size distribution tests, and visual inspection, and conditions were found to be commensurate with those determined by the predesign investigations and studies.

Six soil test borings (figures 2B-133 through 2B-138) were drilled within the intake excavation. The soils' consistency was determined by standard penetration tests performed in accordance with ASTM D 1586. Grain size tests were performed to determine the particle size and distribution of six samples from boring IFI-1. The grain size distribution for soils coarser than a No. 200 sieve was determined by passing the samples through a standard set of nested sieves. Materials passing the No. 200 sieve were suspended in water, and the grain size distribution of the finer fraction was measured by the rate of settlement for 3 of the 6 samples tested. These tests are similar to those described by ASTM D 421-65 and D 422-23. The results are presented on figure 2A-8 (sheets 24-29).

The foundation subgrade was carefully inspected and found to be firm, dry, undisturbed soils typical of those encountered in the preconstruction investigation. The average penetration resistance measured within the fine sands, which constitute the foundations materials for the intake structure, was slightly lower than the average penetration resistance previously measured during the initial investigation for the intake. However, the lowest penetration resistances measured during the inspection are greater than the lowest values measured during the predesign investigation. To evaluate the distribution of standard penetration resistance values within the sands between el 15 and 55 ft, histograms were plotted for both the inspection and the predesign borings. The resulting histogram (figure 2A-14) shows that 80% of the soils had penetration resistances of 25 blows or greater at the time of the original foundation investigation.

The inspection borings, which were made from near the base slab level, indicate that 80% of the soils have penetration resistances of 20 blows or greater. Foundation soils have been unloaded ~ 2000 lb/ft^2 because of excavation to the base slab level. The slight decrease in penetration resistance representing

the lowest quintile of standard penetration resistances is attributed to the decrease in confining stresses within the sand caused by unloading.

In addition to studying the penetration resistances representing the fine sand portion of the bearing stratum, the penetration resistances representing equal elevations were studied. From this analysis a penetration resistance was obtained which represents the bearing stratum both before construction and after excavation. A summary of these data is presented below.

Before Construction			*After Excavation*		
el (ft)	*SPT*	*rd (%)*	*el (ft)*	*SPT*	*rd (%)*
45	27	87	45	25	90
35	30	85	35	20	25
25	27	77	25	20	75

The relative densities tabulated were obtained from the relationship of Standard Penetration Test (SPT) and effective overburden pressure published by Gibbs and Holtz. As can be seen from this tabulation, although the SPT has decreased, the relative density of this stratum is essentially unchanged.

Dewatering was accomplished within the intake excavation by a deep-ejector-well point system. At the time of final excavation of the intake, water levels within the excavation appeared to vary between el 46 and 39 ft. Data obtained from 5 piezometers located within the excavation indicated that the water level varied between approximate el 44 and 39 ft. Water levels encountered within the six soil test borings drilled near the foundation level indicated that water levels varied between el 46 and 42 ft. This data substantiates that ground water levels were maintained at least 5 ft below the lowest portion of the intake excavation.

The intake excavation was graded to ~ el 52 ft on September 10, 1971. A final 1 ft of excavation was performed on September 13, 1971. A bulldozer and a front-end loader were used to perform the excavation. Upon completion of excavation, the exposed surface was methodically probed and any loose or disturbed areas were undercut.

The mud slab was placed on September 13, 1971, the same day that final excavation was performed. The mud slab was ~ 1- to 1 1/2-ft thick with its bottom between el 50.5 and 51 ft.

2A.4.6 MAIN STACK

The height of the reinforced concrete stack is 120 m (394 ft) above yard grade (el 119.5 ft). The foundation details and loads are as follows:

- *Plan dimensions - octagon with 36 ft inscribed radius, top of cap el 108 ft 6 in.*
- *Bottom of cap el 97 ft 6 in.*
- *Pile cut off el 98 ft 3 in.*

- *164-14BP73 100-ton piles at 4- to 6-ft spacing in 5 rings with radii of 6 ft, 16 ft, 20 ft, 30 ft, and 34 ft, piles driven to el 20 ft.*

- *Loads on pile foundation of 114,000 kip-ft moment, 21-500-kips vertical load at pile cap.*

A shear of 800 kips is supported by the piles and pile cap.

Subsequent to the preproduction pile load tests, it was decided to modify the field installation methods as follows for production driving:[16]

A. *Substitute a Vulcan OR hammer with a rated energy of 30,225 ft-lb for the smaller McKiernan-Terry S68 hammer with a rated energy of 26,000 ft-lb.*

B. *Predrill to el 62 using a 20-in.-diameter auger, and then predrill to el 45 using a 14-in.-diameter auger.*

C. *Remove the soil from the 20-in.-diameter hole.*

D. *Drive the piles and then place concrete to fill the 20-in.-diameter predrill hole.*

These procedures were used to install the production piles. Pile tips ranged from el 25 to el 10, and final resistances of the piles almost always exceeded 50 blows for the last foot of penetration.

A typical production pile, R4-30, was selected for load testing. This pile was driven to 48 blows for the last foot, with the pipe tip at el 12. The load test was carried to 240 tons with no indication of failure.

The significant results of the load test are as follows:

Load (tons)	*Deflection Gross*	*(in.) Net*
100	*0.19*	*Not available*
200	*0.49*	*0.05*
240	*0.58*	*0.07*

This load test of a typical production pile confirmed that the piles in the stack foundation are capable of supporting more than 200 tons.

The stack was completed in 1973, and surveys of the stack since that time indicate that the maximum settlement recorded is 0.3 in., which is considerably less than the estimated 2 in. Most of the settlement is elastic shortening of the pile.

2A.4.7 DIESEL GENERATOR BUILDING

Plan dimensions of the building are ~ 196 ft by 103.5 ft; static foundation pressure is < 3 ksf; bottom of mat foundation is at el 125 ft.

2A.4.8 SERVICE WATER PIPING AND ELECTRICAL DUCTS

The service water piping and electrical duct banks follow the routes indicated on figure 2A-1, sheet 2. The subsurface profile of these routes is shown in figure 2A-3, sheet 7. The soils underlying these buried structures are stable and capable of carrying the small net additional loads imposed on them by the pipes and ducts. The granular soils between el 50 and 80 ft underlying the routes are similar in character and consistency to the materials in the power block area and have factors of safety against liquefaction comparable to the valves given for the yard area shown on figure 2A-24.

In addition, the following provisions were employed to ensure that allowable piping stresses are not exceeded due to differential settlement between the various foundation and fill materials along the service water piping alignment:

A. *The site subsurface investigation by drilling and sampling establishes the types of foundation materials beneath the pipelines. Thus, the pipeline routing is selected to provide a satisfactory support for normal operation and extreme conditions such as earthquakes.*

B. *Buried pipelines are installed in ditches dug into natural ground and fill and backfilled with either compacted soil or controlled-density fill (see section 2A.9). The net additional loading on the foundation soils is generally very small since the pipe and the volume of water in it replace the volume of soil. Thus, generally, bearing capacity and settlement are not a problem unless the pipes are laid in or above very soft soils. As can be seen from figure 2A-3, sheet 7, the pipeline is founded on soils competent to support the pipeline loads.*

C. *Soils under the pipelines were evaluated for their liquefaction potential, which precludes any large displacements due to seismic settlement of the sands.*

D. *Between the intake structure and the plant area (figures 2A-1, sheet 2, and 2A-3, sheet 7) the pipe alignment passes through either compacted soil fill or controlled-density fill (see section 2A.9). The compacted soil fill was placed 9 months prior to installation of the pipe, and, therefore, any significant settlement of the fill or the underlying materials generally occurred prior to placement of the pipe. However, a portion of the pipe alignment compacted soil fill was determined to be improperly compacted and subsequently replaced with controlled-density fill as described in section 2A.9.*

E. *The pipes are placed on sand bedding or controlled-density fill (see section 2A.9) to provide a base that will accommodate adjustments of the pipe length over varying local conditions of foundation soil stiffness.*

F. After construction, the pipes were hydraulically tested before they were placed in service.

2A.5 FOUNDATION EVALUATION

The soil conditions delineated by the soil test borings in the HNP-2 power block area compare very favorably with the soil conditions delineated by the subsurface investigations in the HNP-1 area. The soils which constitute the foundation materials for the HNP-2 structures have similar characteristics to the soils which support the HNP-1 structures.

It is noted that the penetration resistances within the fine sands which constitute the foundation materials for the HNP-2 reactor building are somewhat lower than the penetration resistances measured during the initial investigation of the HNP-1 structures. The decrease in penetration resistance at the HNP-2 area is attributed to the unloading which has been accomplished by grading and excavation for HNP-1.

The original borings in the HNP-1 area were drilled from elevations varying from 135 to 146 ft. The frequency distribution of all penetration resistances made in these borings within the fine sands below el 75 ft was plotted. The resulting histogram showed that 80% of the soils had a penetration resistance of 31 blows or greater. Subsequently, additional borings were made as a part of the foundation inspection in the HNP-1 area. These borings were made from the base slab elevation of ~ el 75 ft. The frequency distribution of the penetration resistances of these borings was also plotted and indicates that 80% of the penetration resistances were in excess of l3 blows per ft. The difference in the penetration resistance representing 80% of the soils is attributed to unloading effects and is a generally recognized occurrence in sand. Figure 2A-15 is the histogram for HNP-1 before and after excavation.

Borings for the HNP-2 investigation were made from both the elevation of the excavation for HNP-1 and the general yard level at el 130 ft. As was done for the HNP-1 data, the frequency distribution of all borings within the fine sands below el 75 ft was plotted. A penetration resistance of 24 blows per ft or greater was determined to represent 80% of the soils in the HNP-2 area. Six of the borings studied were made at elevations below el 105 ft, whereas the remaining nine borings were made at elevations near el 130 ft. Ten additional borings were made as a part of the foundation inspection in the HNP-2 area. These borings were made from the base slab elevation of ~ el 75 ft. The frequency distribution of the penetration resistances of these borings was also plotted and indicates that 80% of the soils have penetration resistances of 15 blows or greater. As with HNP-1, the decrease in penetration resistance is attributed to the decrease in confining stress within the sand caused by unloading. Figure 2A-16 is the histogram for HNP-2 before and after excavation. From a comparison of this data with HNP-1 (figure 2A-15), it is concluded that the consistency of the soils as measured by penetration tests for HNP-2 closely agrees with the consistency of the soils previously determined in the HNP-1 area.

In addition to the above comparison, the grain size distribution for the foundation soils of HNP-1 and HNP-2 compares very favorably. Figure 2A-8, sheet 30, compares the results of grain size tests with depth. Figure 2A-8, sheet 31 through sheet 34, compares the shapes of curves at specific elevations. Also, the settlement properties of soils under the two units (as compared in figure 2A-10) are similar. The results of these laboratory tests further support the similarity of the soil conditions of HNP-1 and HNP-2.

The soil at this site is quite satisfactory for the nuclear power plant with its imposed static and transient loads. The site excavation to el 130 ft and the pit excavation for the reactor building amount to an

unloading equal to ~ 50% of the imposed static load. In the turbine building and radwaste building areas the excavation unloading is approximately equal to the imposed static load.

The strength characteristics of the dense foundation soils complemented by the weight of the soil removed by site grading and pit excavation provide an excellent foundation for the plant structures. The allowable static-bearing capacity is in excess of 15,000 lb/ft^2.

Settlement predictions have been made for the HNP-2 reactor building, turbine building, turbine-generator pedestal, and radwaste building. These predictions utilized stresses calculated in accordance with the Westergaard theory(5) and considered stress overlap from all nearby foundations. For stress calculation purposes, the loads imparted to the foundations were considered to be the structural dead loads plus live loads. The soils compressibility characteristics were determined by one-dimensional laboratory consolidation tests.

The total settlement of the HNP-1 reactor building was calculated to be 3 in. The total settlement of the HNP-1 turbine building, including the turbine pedestal, and the radwaste building was calculated to be 1.5 in. The majority of the total settlements occurred during construction. For HNP-2, the total settlement for the reactor building is estimated to be 4.5 in. The estimated post-construction settlement is predicted to be 0.5 in. for the reactor building and < 0.5 in. for the turbine and radwaste buildings. The maximum post-construction differential settlement of the structures should be < 0.5 in. A summary of settlement estimates for both units is shown on table 2A-6.

To confirm the predicted settlements and check piping stress due to differential settlement, a program of monitoring the base slab settlements of the major plant structures was established.

It should be noted that the observed settlement beneath the HNP-2 reactor building reported in figure 2A-17, sheet 2, is not a complete record of the settlement that the soil has experienced. These data reflect only the settlement that has taken place since construction of the mat was begun. It does not include the settlement which occurred during the construction of HNP-1. The HNP-1 reactor building was completed at approximately the same time the HNP-2 mat was started.

The record in figure 2A-16, sheet 2, provides information on the consolidation after construction. Comparison of these data with figure 2A-18, sheet 1, indicates that the 2 reactor buildings have experienced most of the settlement which can be expected, and the settlement now occurring is the expected consolidation after construction. (Refer to table 2A-6.)

The settlement of HNP-2 due to the construction of HNP-1 can be estimated by examining the HNP-1 record (figure 2A-18, sheet 1) for the effect of the construction of HNP-2 and then extrapolating that settlement to HNP-2. This increases the total settlement to ~ 2 in. (slightly less than the observed values for HNP-1 total settlement).

Although this adjusted value is still less than the predicted settlement (table 2A-6), it is reasonable. In soils engineering practice, a settlement estimate is considered good if the observed settlement is within ± 50% of the predicted value. In this case, the settlement predicted for the HNP-2 reactor building after construction is 4 in. (an additional 0.5 in. is predicted for long-term consolidation after construction; see table 2A-6). The 2 in. of settlement which appears to have taken place is within 50% of the predicted value.

The settlement records for the HNP-1 radwaste building are shown in figure 2A-18, sheet 2. A settlement-monitoring program for the HNP-2 radwaste building has been started. Figure 2A-17, sheet 5, provides the settlement record for the HNP-2 radwaste building.

Static undrained triaxial sheet tests were performed utilizing both consolidated and unconsolidated conditions. Elastic moduli were calculated using the elastic portion of the stress strain curves from these tests. The graph of average elastic moduli (figure 2A-19) represents weighted averages at confining pressures commensurate with in situ overburden stresses. Individual moduli ranges per geologic formation are also shown on figures 2A-20 through 2A-23.

The HNP-2 foundation subgrade was inspected to confirm that the subsurface conditions encountered during construction were similar to those previously determined by the predesign investigations and studies. This was done by means of soil test borings, hand auger borings with cone penetrometer tests, laboratory grain size distribution, triaxial and consolidation tests, and visual inspection.

The inspections confirmed that subsurface conditions closely coincide with the conditions predicted by the predesign soil investigations which are reported in section 2.5 and supplement 2A.

Borings RFI-1 through RFI-10 were performed as part of this inspection. The logs of these borings are shown in supplement 2B, figures 2B-113 through 2B-122, and the locations are indicated in figure 2A-2.

Fifteen hand auger borings with cone penetrometer tests were performed within the HNP-2 reactor area. These borings were advanced by manually twisting a sharpened steel auger into the ground. The soils encountered were identified from the cuttings brought to the surface.

At regular intervals, the auger was removed and the soil consistency measured with a cone penetrometer. The conical point was first seated to penetrate any loose cuttings and then driven an additional 1 3/4 in. with blows from a 15-lb hammer falling 20 in. The number of hammer blows to achieve this penetration was recorded and is an index to the soil strength and density.

Cone penetrometer data were used to detect soft surface areas and to confirm suitable surface conditions. Boring records showing soil stratigraphy and consistency determined by the hand auger and cone penetrometer methods are given in table 2A-7.

Grain size tests were performed to determine the particle size and distribution of the samples tested. The grain size distribution of soil particles coarser than a No. 200 sieve was determined by passing the samples through a standard set of nested sieves. Materials passing the No. 200 sieve were suspended in water and the grain size distribution measured by the rate of settlement in water. These tests are similar to those described by ASTM D 421-58 and D 422-54T. The results are presented on table 2A-1 and figure 2A-8.

Consolidated-undrained triaxial sheer tests were performed to determine the shear strength of two representative samples of foundation soils. The test results are presented in table 2A-5. The stress-strain curves and Mohr diagrams are shown on figure 2A-11.

Two undisturbed samples were selected for consolidation testing to determine the settlement characteristics of the foundation soils. These test results are presented in table 2A-4 and figure 2A-9.

The average penetration resistance measured within the fine sands, which constitute the foundation materials for the HNP-2 reactor building, was slightly lower than the penetration resistances measured during the initial investigation for this structure. This was also observed during the HNP-1 inspection.

The relative density of the loosest sand sample was also determined from the relationship of standard penetration resistance and effective overburden pressure published by Gibbs and Holtz. This analysis indicates that the loosest sand zone encountered during the HNP-2 foundation inspection has a relative density of 50% and that generally the relative density is much higher. This relative density of 50% for the loosest sands below foundation level is the same relative density value determined during predesign liquefaction studies for the loosest sand zones. Therefore, the safety factors against liquefaction, as presented in the predesign foundation evaluation, are valid.

Data from triaxial sheer tests performed on firm clayey sand samples obtained from immediately below HNP-2 area confirm that the soil strengths, which have previously been reported and used in bearing capacity analyses of the powerhouse structures, are conservative.

Consolidation tests performed on representative samples of the loosest sands and softest clay below HNP-2 indicate that the settlement characteristics are similar to soils previously reported in the predesign and settlement analyses. Therefore, the settlement estimates needed no revision.

The predicted and actual observed settlement of the major Category I structures are discussed in subsection 2A.5.3. The settlement records are given for each monument on figures 2A-17 and 2A-18, sheets 1 through 4.

2A.5.1 PLANT FOUNDATIONS

In the plant area, very dense partially cemented sands with clay seams are present down to ~ el 80 ft. This partially cemented layer is generally underlain by sands containing hard clay seams or lenses which extend to ~ el 0 ft. In this area of the site, the soft to firm plastic clay was not encountered.

2A.5.1.1 Reactor Building

The HNP-2 reactor building, with its foundation at el 75 ft, bears on firm-to-dense sands and clayey sands with layers of plastic clay. Using soil strength parameters based on triaxial test data, the computed safety factor against bearing capacity failure for this foundation is in excess of three.

The sands which support the reactor building are, in general, dense and incompressible. Settlement occurring from these sands is calculated to be on the order of 4.5 in. This settlement is relatively uniform and will occur primarily during construction. Maximum post-construction settlement beyond a period of 6 months is in the range of 0.5 in. The foundation soils will not be adversely affected by earthquake shock or vibratory loading.

2A.5.1.2 Turbine and Control Buildings

The HNP-2 turbine and control buildings with the bottom of the foundation slabs at el 105 ft, bear on a relatively thick zone of cemented sands underlain by firm-to-dense clayey sands with lenses or layers of plastic clays. These soils are capable of safely supporting the design loads with a bearing capacity safety factor in excess of 3.

The total settlement of the foundation soils is calculated to be 2 in. The zone of cemented sands beneath the turbine slab is incompressible. However, stress increases within the underlying zone of firm sands due to the weight of the building account for the expected settlement. Most of this settlement occurred during construction. The long-term post-construction settlement will be negligible. The foundation soils will not be adversely affected by shock or vibratory loading.

2A.5.1.3 Radwaste Building

The radwaste building, with its base slab at el 100 ft, bears on soils comparable to those described for the reactor building. These soils are capable of safely supporting the design loads for the radwaste building. The total calculated settlement for this structure is 3.5 in. The settlement occurred largely during construction with negligible post-construction settlement.

2A.5.1.4 Intake Structure

Support of the intake structure is on a mat foundation at el 52 ft. Based on 4100 lb/ft^2 bearing pressure, the safety factor against bearing capacity failure is in excess of 5. Both the intake structure alone and the influence of the adjacent cofferdam were considered in this evaluation.

The settlement of the structure prior to the placement of the backfill was expected to be ~ 1.5 in. The additional settlement due to the zone of backfill around the structure is in the order of 0.5 in. As of April 1976, the total measured settlement of the intake structure was 1 1/4 in.

2A.5.1.5 Main Stack

The stack is supported on pile foundations. The 11-ft-thick base pad is octagonal with a 72-ft-diameter inscribed circle. The foundation system consists of 165 100-ton piles in 5 concentric rings. The bearing strata for piles are the dense sands below el 50 ft. Static analyses indicate that the l4-in.-H sections develop 100-ton capacity when driven to ~ el 20 ft. In order to obtain the required embedment, predrilling to ~ el 45 ft was required to penetrate the dense and hard soils above that elevation.

Predrilling was limited to 14-in. or 20-in.-diameter holes. In order to ensure lateral support throughout the pile length, the 20-in.-diameter holes were backfilled with concrete after driving the piles. Analysis based on the previously given loading data and laboratory consolidation tests indicate that settlement of the pile group is in the order of 2 in.

2A.5.1.6 Diesel Generator Building

The diesel generator building, with its spread mat foundation at el 125 ft, bears on very dense clayey fine to medium sand with some clay layers extending to ~ el 120 ft.

Between el 120 and 70 ft are very dense medium to fine clayey sands with scattered layers and inclusions of very hard cemented clay and dense sands. The foundation pressure is < 3 ksf.

2A.5.2 SOIL LIQUEFACTION POTENTIAL

Irregular localized seams of sandy soils having penetration resistances < 20 blows per ft were found between el 50 and 80 ft. These soils were considered possibly susceptible to liquefaction. Cyclical shear tests were run on samples of the loosest sandy materials encountered, as described in subsection 2A.3.5.

The results of these tests were analyzed assuming that the site may undergo an earthquake where maximum horizontal acceleration at the ground surface is 0.15 g corresponding to the design basis earthquake for this site. The approach advanced by Seed and Idriss[6] was utilized to evaluate safety factors with respect to liquefaction.

If the soil between the ground surface and a depth h below the ground surface responded as a rigid body to the motions induced by the earthquake, the maximum shear stress in the soil at depth h would be equal to $0.15\gamma h$*, when* γ *is the unit weight of the soil. Since the soil is, in fact, a deformable material, the actual maximum shear stress will be somewhat <* $0.15\gamma h$*, the reduction depending on the distance h and the characteristics of the soil profile. Studies by H. B. Seed show that the maximum shear stress at a depth of 70 ft (or el 58 ft) would be ~ 70% of the value corresponding to a rigid body behavior, giving a maximum shear stress at this depth of ~* $0.105\gamma h$*.*

For any given earthquake, the maximum acceleration (and the corresponding maximum shear stress) occurs only once. A typical strong motion earthquake, such as the El Centro, may include about 10 large cycles of motion as well as many smaller ones. The average amplitude of shear stress or acceleration for these cycles is substantially less than the maximum. A conservative value for the average is ~ 75% of the maximum. On this basis, it was concluded that at el 58 ft, the effective average shear stress developed for 10 cycles during the maximum hypothetical earthquake would be ~ $0.75 \times 0.105\gamma h = 0.08\gamma h$*.*

The safety factor against liquefaction is defined as the shear stress required to cause liquefaction of soil samples in 10 cycles divided by the average shear stress induced by the 10 largest shocks of the earthquake. Safety factors have been calculated for both momentary and complete liquefaction at various points beneath the proposed structures and outside the proposed structures.

The cyclical shear stress required to produce liquefaction at 10 cycles is a function of the soil's relative density as well as the confining pressure.

$$\tau_{L10} = \frac{\sigma' \, x \, R_D}{constant}$$

where:

τ_{L10} = *shear stress to cause liquefaction in 10 cycles.*

σ' = *effective overburden pressure.*

R_D = *relative density.*

The constant of proportionality determined by Seed and Idriss[6] for Sacramento river sand is 200. Assuming that this relationship applies to the soil at the HNP site, the data for momentary liquefaction indicates a constant of 172. This indicates that the sands tested are ~ 15% stronger under cyclical loading conditions than the sands tested by Seed and Lee.[4] This additional strength is attributed to the following factors:

A. *The samples tested are undisturbed samples which have been subjected to preconsolidation pressures and the development of some permanent microstructure.*

B. *The samples tested contain up to 14% clay. Therefore, they are not truly cohesionless materials.*

Applying the same equation of proportionality to the point defined as complete or continuing liquefaction, it was determined that constant of proportionality is 151.

The cyclical triaxial shear test requires that samples be isotropically consolidated for testing. The in situ state of stress, however, varies from this isotropic condition by the value of K_o. It has been found that the cyclical triaxial shear test results are somewhat more optimistic than those of the simple shear but that they are proportional. According to Professor Seed,[7] the triaxial test results realistically simulate the soil behavior during actual earthquakes if the dynamic strengths are multiplied by 0.55. The dynamic strengths discussed herein have been reduced in this fashion.

In making the analyses, two possible relative density conditions were evaluated. The average relative density of the soils tested was 50%. (The use of the relative density concept in silty clayey sands is questionable, but the determination was made as a guide.)

The relative density corresponding to the poorest 20% of the sandy soils in question is 60%, contrasting with the average of 50% of the samples tested. Therefore, one set of safety factors was computed utilizing the test strengths corrected for a relative density of 60%. A second set was computed directly from the test results.

The safety factors were also computed for four different water levels - el 77, 85, 93, and 105 ft. These levels represent the highest observed level in the plant area, a 10-year flood peak, a 1000-year flood peak, and a maximum theoretical flood, respectively.

The safety factor against liquefaction is defined as the shear stress required to cause liquefaction of soil samples in 10 cycles divided by the average shear stress induced by the 10 largest shocks of the earthquake. Safety factors have been calculated for both momentary and complete liquefaction at various points beneath and outside the proposed structures as shown on figure 2A-24. The lowest computed safety factor was 1.4.

In view of severe stresses selected for laboratory testing, even the loosest sands (those encountered between el 80 and 50 ft at this site) is not adversely affected by shock or vibratory loadings of the magnitudes and durations that occur due to earthquake forces.

Other characteristics which also support the fact that there is no liquefaction hazard at the site are:

A. *The penetration resistances of the sands beneath the bearing stratum have been compared with penetration resistance data of sands which have and have not experienced liquefaction during a 1964 earthquake in Niigata, Japan.*[8][9] *Those sands that experienced liquefaction display a preponderence of penetration resistances between 5 and 15 blows per ft. Most sands at the site have penetration resistance values between 25 and 55 blows per ft. The sands from the site are denser than the sands that did not experience liquefaction at Niigata, and are far denser than the sands that experienced partial liquefaction.*

B. *The grain size distributions of the sands at the site have also been compared with the Niigata data. Although some of the sands fall partially within the range of the sands at Niigata, in most cases the site materials contain more fines (15 to 25% of the material passes the No. 200 sieve) than potentially liquefiable materials are considered to have.*

C. *In areas where liquefaction has occurred, the soils have been alluvium, Quaternary glacial outwash, or loose uncompacted fill. The soils at this site are at least l3 million years old (Miocene) and have been heavily preconsolidated.*

It is concluded that the soils at this site display a very large margin of safety against liquefaction failure if subjected to earthquake shocks of the magnitude postulated for this site.

2A.5.3 BUILDING SETTLEMENT MONITORING

A comprehensive building settlement monitoring program was begun for selected buildings in HNP Units 1 and 2 in 1980. The plan tracked total building settlement and differential settlement across structures from 1980 to 2006. In January 2007, it was determined that time deflection/settlement curves for virtually all points of possible settlement were essentially flat and had been for 25 years. Southern Company issued correspondence (Log # NL-07-0175) providing justifications for terminating the building settlement monitoring program. This section of the FSAR is included for historic reference only.

Building settlement is monitored for the following buildings:

- *HNP-2 reactor building.*
- *HNP-2 turbine building.*
- *HNP-2 radwaste building.*
- *Diesel generator building.*

- *Intake structure.*
- *Main stack.*
- *Control building.*
- *HNP-1 reactor building.*
- *HNP-2 turbine building.*

The HNP-1 buildings are monitored due to their proximity to the HNP-2 buildings.

Building settlement monitoring is divided into the following major categories:

A. *Total Settlement*

   The total settlement of each structure is measured and compared with the predicted settlement to assess the accuracy of the settlement predictions and to obtain an indication of settlement trends.

B. *Differential Settlement Across Structures*

   The differential settlement across each building is measured to assess the tilt of the building and to compare this value with the allowable tilt which is based on preventing building structural and equipment damage.

C. *Penetration Differential Settlement*

   Two types of differential settlement are considered under this category. The differential movement at a penetration can be caused by differential settlement between adjacent buildings or between building and soil. The allowable displacements at the penetrations are calculated using pipe stress analysis methods and are compared with settlements measured at nearby benchmarks.

2A.5.3.1 <u>Total Settlement</u>

2A.5.3.1.1 Measurement

Settlement values are determined by measurements which are compared with known elevations at established reference benchmarks. The locations of the reference benchmarks are shown on drawing no. H-12523. Elevations of the benchmarks were originally established from a United States Geological Survey (USGS) benchmark in Toombs County. The benchmarks are situated in the yard in such a way as to avoid accidental displacement and facilitate the settlement surveys of the Category I buildings. They are far enough away to avoid settling with the buildings and are placed in areas isolated from traffic which might disturb the marker. Precautions were also taken to provide proper soil and anchorage

conditions to ensure the stability of the benchmarks. There is no procedure for periodically checking the elevations of the reference benchmarks.

A plant operating procedure established a detailed method for monitoring settlement of Category I structures for HNP-1 and HNP-2. A series of special drawings were also drawn to clearly locate the benchmarks and establish a fixed survey route. These drawings are referenced in and supplement the procedure. The procedure establishes the order in which specific survey routes are followed and requires closure of each survey route for a specific building or structure before continuing. Acceptance criterion for closure error is 0.005 ft. The procedure establishes a specific format for recording the final elevation data. This procedure establishes as much consistency as possible from one survey to the next in order to make any change or abnormality immediately apparent.

The benchmarks established inside all buildings except the reactor building are 1/2- to 3/4-in. self-drilling "red head" expansion anchor bolts set in the floor or walls of the structure. Benchmarks on the exterior walls of structures are similar. Benchmarks in the reactor building are 3/4 in. x 3/4 in. x 6 in. to 12 in. brass embedded in the concrete floor. This leaves ~ 1/4 in. of the bar exposed above the floor resulting in a 3/4 in. to 3/4 in. x 1/4 in. exposed benchmark. Outside benchmarks are poured in place concrete posts ~ 1 ft x 1 ft square by 2 ft 6 in. long with a maximum of 1 ft exposed above ground level. This leaves a minimum of 1 ft 6 in. embedded below ground. A 3/4-in. galvanized bolt is embedded in the center of the top of the post, and the top is sloped away from the center for drainage.

Measurements of structure movements were obtained by periodically reading the elevations of benchmarks established generally at the beginning of construction. Settlement versus time curves for each structure, except the intake structure, were developed. (See figures 2A-17 and 2A-18.) Only one benchmark was originally set on the intake structure; four new benchmarks were set in July 1978. The total measured settlement of each structure was obtained by averaging the settlements at each benchmark. These average measured settlements in most cases represent the total settlements since the beginning of construction, although the settlement records are not always clear on precisely at which stage of foundation construction the monitoring started.

2A.5.3.1.2 Comparison of Predicted Versus Measured

The ratios are highest at the control and intake structures; these were constructed earlier than the HNP-2 buildings and have had longer to settle. No significant settlement of either of these structures has occurred since October 1978. It can be observed that the settlement curves have flattened out. As predicted, the large majority of settlement appears to have taken place during construction due to the mainly granular nature of the foundation soils. The ratios of the measured settlements to the predicted settlements are shown in table 2A-8.

2A.5.3.1.3 Allowable Total Settlements

The total allowable settlements are 4.5 in. for the reactor building and 2.5 in. each for the control building, diesel generator building, main stack, and intake structure.

2A.5.3.2 Differential Settlement Across Structures

2A.5.3.2.1 Allowable Differential Settlements

To establish allowable differential settlements across the Category 1 buildings, the foundations are assumed to be completely rigid. As the building settles, the entire structure moves vertically and/or rotates as a plane rigid body. The allowable differential settlement values place a limit on the amount or rotation of each building as settlement occurs. Two criteria were developed to cover the buildings under consideration; the choice of criterion is based primarily on distance to adjacent buildings. The criteria are summarized in table 2A-9.

The first criterion covers structures which are not in close proximity to other buildings, i.e., the main stack, the intake structure, and the diesel generator building. The criterion developed limits the tilt of the building to ensure the appearance and proper functioning of all operating systems and equipment. In order to satisfy this criterion, a limiting settlement profile slope of 0.002 radians was used to calculate allowable differential settlements between the established benchmarks in the corners of each building. The 0.002 slope value is for structures with rigid foundations and is tabulated in the Navy Design Manual.[10]

The second criterion applied to structures concentrated in the powerblock and separated by a gap of 3 in. from surrounding structures. Included in this group are the control building, turbine building HNP-1 and HNP-2, reactor building HNP-1 and HNP-2, and the radwaste building HNP-2. The criterion developed for these structures limits the tilt of each building to ensure that two adjacent buildings do not touch during a possible operating basis earthquake (OBE). A summary of the allowable slope calculation procedure is shown in figure 2A-26.

Based on the allowable slopes derived for the buildings, the allowable differential settlement values were calculated between the established benchmarks in each of the Category I buildings. The allowable settlements represent the worst case. In order to touch during the earthquake, the buildings must lean towards each other and both must reach or exceed the allowable tilt simultaneously. The fact that a building has reached the maximum allowable tilt value does not necessarily mean that touching would occur during the OBE.

2A.5.3.2.2 Measured Differential Settlements

To determine the actual differential settlements, reference elevations have been established for the benchmarks in each building. A reference elevation is defined here as an elevation which can be compared with current survey elevation readings to indicate the existing degree of differential settlement. These reference elevations are based on the survey readings taken at the approximate structure completion date of each building. This date corresponds to the time when the structure is assumed to be properly aligned, both with respect to itself and to any adjacent building. Existing differential settlement will be measured from this reference date and compared with the allowables.

For those cases where a benchmark location has been altered in the field since the completion date of the building, an adjustment must be made to the reference elevation. This adjustment ensures that the

reference elevation can be compared directly with the current readings to establish differential settlement.

Using the reference elevations and the latest survey values, the settlement of each benchmark from the reference date to the present can be determined. A comparison of settlement values of any two benchmarks within a building provides the differential settlement between the benchmarks. The reference dates and elevations are summarized in table 2A-10.

2A.5.3.2.3 Comparison of Allowable and Measured Settlements

Comparison of the existing and allowable differential settlements of the Category I structures of HNP-2 indicates that there has been little differential settlement to date, and that the existing settlement is well below the allowable differential settlement values for each of the buildings examined. The comparisons are provided in table 2A-11.

2A.5.3.3 Penetration Differential Settlement

2A.5.3.3.1 Allowable Differential Settlements

The amount of differential movement each penetration can withstand before the pipe or pipe anchor (or support) becomes overstressed was computed for penetrations entering the building directly from the soil and for penetrations passing between adjacent buildings. Either the pipe or the pipe anchor can become overstressed due to penetration settlement.

For pipes, the allowable stress criterion is:

$$\frac{iM_D}{Z} \leq 3.0\ S_C \text{ (reference 11)}$$

where:

i	=	*stress intensification factor.*
Z	=	*pipe section modulus.*
M_D	=	*moment due to building settlement.*
S_C	=	*allowable stress in cold condition.*

For anchors, the allowable stress criterion is:

$$M_D \leq M_{\text{anchor design}}$$

or

$\sigma \leq \sigma_{allowable}$

where:

$M_{anchor\ design}$ = *moment from pipe stress analysis (seismic, thermal).*

$\sigma_{allowable}$ = *particular allowable stress in anchor parts (bearing, bending, bolt shear, etc.)*

For penetrations leading from the structure into the soil, the moments in the pipes and anchors produced by building settlement were computed by one of three methods. The first method is more conservative by assuming the pipe anchor to be rigid; with this assumption, small settlements tend to produce large stresses in the pipe and anchor. The second method assumes a degree of flexibility in the anchor; moments are obtained from a computer calculation using a pipe stress program. The third method assumes changes to have been made in the pipe anchors to allow more flexibility, and also requires computer solution.

For penetrations passing between adjacent structures, the moments in the pipes and anchors produced by the differential movements of the structures were computed by one of the two methods outlined in table 2A-12. Again, the first method is more conservative by assuming rigid anchors and double-acting hangers. The second method assumes a degree of flexibility in the anchors and considers single-acting hangers, where applicable.

In all of the penetrations analyzed except at the intake structure, methods 1 or 2 indicated allowable settlements large enough to present no major measurement problems in the future life of the plant. At the intake structure, allowable settlements calculated by methods 1 or 2 were unacceptably low. The intake structure penetrations were reanalyzed by assuming that anchors and supports were modified to allow more flexibility in the pipe; fixity was assumed to be ~ 10-pipe diameters outside the walls (method 3). This analysis, assuming the modifications, produced acceptably high allowable penetration settlements.

The conservative assumptions used in determining the allowable settlements involve mainly soil behavior. No account is taken of the fact that some movement of the soil adjacent to the building takes place as building movement occurs. Movement of the soil with the building reduces the amount of differential settlement between building and soil. In addition, time and relaxation effects are not taken into account. Settlement of the building is slow enough to ensure that stresses built up in the soil due to penetration movement is redistributed with time, reducing the level of stress in the pipes and anchors.

2A.5.3.3.2 Measured Differential Settlements

Differential settlements of the penetrations since installation were measured by assuming the settlement of the penetration to be the same as the settlement of the nearest benchmark. For each penetration leading from the structure into the soil, reference was made to the appropriate settlement curve to obtain the maximum settlement which had occurred since the date of penetration completion. It should be noted that the maximum settlement is not necessarily the settlement between the penetration completion date and the present. The settlement pattern of most of the benchmarks is presently nearly level, with dips and peaks; maximum settlement frequently occurs in one of the dips established prior to the present.

For penetrations passing between adjacent structures, the settlement of the benchmarks closest to the penetration on both structures must be considered. The settlement curves of the two benchmarks from date of penetration installation to present are compared. Review of the curves indicates that maximum differential settlement does not necessarily occur on the most recent date.

2A.5.3.3.3 Comparison of Allowable and Measured Settlements

The ratios of the maximum measured settlement to the allowable settlement for the penetration pipes and anchors have been calculated and are indicated in tables 2A-13 through 2A-20. It is evident that, at present, the measured settlements exceed ~ 30% of the allowable in only isolated cases, and the majority are < 20% of the allowable. These low ratios reflect two related factors: first, the majority of the penetrations have been installed since late 1976; and second, settlement values since late 1976 have been very small. In general, the small ratios are more a function of small measured settlements than large allowable settlements.

2A.6 SLOPE STABILITY

Slope stability analyses indicate that the permanent slopes in the plant area as designed are stable and conservative. The stability of the intake structure and the river bluff immediately adjacent thereto have been analyzed. The calculated minimum safety factors for various conditions are as follows:

	Failure Mode	*Minimum Safety Factor*
A.	*Circular arc-river banks ~ 100 ft upstream of the intake structure, pseudostatic,[a] a = 0.15 g*	*1.7*
B.	*Circular arc through intake structure-static*	*3.4*
C.	*Circular arc through intake structure, pseudostatic,[a] a = 0.15 g*	*2.3*
D.	*Sliding through intake structure-static*	*2.8*
E.	*Sliding through intake structure, pseudostatic,[a] a = 0.15 g*	*2.1*

The section analyzed for condition A above is shown on figure 2A-27.

Sloped excavations to the base slab levels of the power block structures were maintained. The firm to dense clayey sands which extend from the surface to ~ el 120 ft, were excavated no steeper than 45 degrees with the horizontal. The very dense sands which underlie the dense clayey sands to ~ el 75 ft are stable for excavation slopes flatter than one-half horizontal to 1 vertical. These requirements necessitated the use of a composite slope for the power block excavation (figure 2A-27). The factor of safety against massive failure was 1.3. No significant breakouts occurred within the power block excavation.

The seismic stability of the riverbank in the site vicinity was later reevaluated. The following information is presented to demonstrate the stability of the riverbank and the margin of safety against its failure during or after the design basis earthquake

Three additional standard penetration test borings, B-2001, B-2002, and B-2003 were performed in the vicinity of the river bluff upstream of the intake structure. These borings were used to define better the extent of the low-blow-count zone encountered in boring B-458. The locations of these borings are shown in figure 2A-1, and the logs are presented in supplement 2B. A fourth boring, B-2001A, was drilled next to boring B-2001 to obtain undisturbed samples for laboratory testing.

Index properties tests, i.e., grain size, water content, unit weight, and Atterberg limits, were performed on all samples from boring B-2001A. Consolidated, undrained, static triaxial shear tests were performed on undisturbed samples. Stress-controlled cyclic triaxial tests were performed on undisturbed samples of the low-blow-count sands. Results of the index-properties tests are given in table 2A-1 and figure 2A-8. The static triaxial test results are shown in table 2A-5 and figure 2A-11. A summary of the cyclic strength results are presented in table 2A-21 and figure 2A-28.

The profile developed for analysis of the river bluff is shown in figure 2A-30. A pocket of loose sands was encountered in borings B-2001, B-2002, B-2003, and B-458. No such loose material was found in boring B-459.

The individual cyclic triaxial test data plots are presented in figure 2A-29, sheets 1 through 13. Note that the plots present peak-to-peak ranges of measured quantities.

2A.6.1 LIQUEFACTION POTENTIAL OF LOOSE ZONE

As a first step in analyzing the stability of the bluff, the liquefaction potential of the pocket of loose sands has been evaluated by using the method developed by Seed and Idriss.[10] The cyclic strength data shown in figure 2A-28 are used for this evaluation. The factor of safety against liquefaction is defined as:

a. Earthquake forces considered to be equivalent static forces.

$$Factor\ of\ safety = \frac{\tau_f}{\tau_d}$$

where:

τ_f = *the cylic shear stress required to cause 10% double-amplitude strain in five uniform stress cycles.*

τ_d = *the equivalent average uniform shear stress induced by the design earthquake.*

Five uniform cycles are recommended by Seed et.al.[11] *as an adequately conservative representation of earthquakes, such as the design basis earthquake (DBE), with magnitudes ranging from 5 to 6.3. The equivalent average uniform shear stress, τ_d, is taken as:*

$$\tau_d = 0.65\, r_d\, \tau_{max}$$

in which τ_{max} is the peak seismically induced shear stress. τ_{max} can be determined by conservatively assuming that the soil column responds to the earthquake as a rigid beam. For the HNP DBE with a maximum acceleration of 0.15g, the peak shear stress at any point in the profile is:

τ_{max} = *0.15 x (total weight of soil above the point being considered).*

This approach is conservative because soil is flexible. The maximum shear stress is there fore somewhat less than the rigid beam value. The factor r_d is a stress-reduction coefficient with a value < 1 (r_d = 1 for a rigid shear beam). Seed and Idriss[10] *have developed and published a range of r_d values for soils for depths up to 100 ft. The complete range of these values, as well as the rigid-beam assumption, has been used in the liquefaction evaluation presented herein.*

The cyclic shear stress required to cause 10% double-amplitude strain in five cyles is defined as:

$$\tau_f = C_r\,(SR)\,\bar{\sigma}_V$$

C_r is a correction factor to relate laboratory to insitu conditions.

The value C_r ranges from 0.57 to 0.9, De Alba et al.[12] *A value of C_r = 0.57 issued for this study. (SR) is the stress ratio for five cycles of uniform stress from figure 2A-28. $\bar{\sigma}_V$ is the effective normal pressure on the assumed horizontal failure plane.*

The results of the analysis are presented in figure 2A-31 for a column of soil corresponding to that in boring 2001. In addition to the material in the loose pocket (els 38 to 73), the top 10 ft of the silty sand below el 32 has been checked. It may be seen that the factors of safety against a shear strain of ± 5% and, therefore, against the development of high-pore water pressures are on the order of 2 to 3, indicating that the actual pore pressure developed by the induced stresses are very low. In fact, following the procedure described by Seed et al.[13] *for conditions where the factor of safety against cyclic liquefaction is 2, the ratio of the number of cycles developed to the number of cycles required to cause liquefaction for the induced cyclic shear stress is on the order of 10^{-4}, and the corresponding value of the*

excess pore pressure ratio developed is negligibly small. Clearly, such pore pressures would have negligible effects on the stability of the slope; furthermore, this method of estimating pore pressures is conservative since conditions underlying a sloping surface are likely to lead to even lower pore pressures than those under a level ground surface, as was assumed in the above analysis.

2A.6.2 SLOPE STABILITY ANALYSIS

Since the pore pressures developed in the loosest sand zone by the earthquake shaking are not likely to be significant, the possibility of sliding in the slope because of the maximum inertia forces generated by the earthquake can be computed by using a conventional slope stability analysis, including a lateral force equal to the DBE multiplied by the mass of the potential sliding mass. The results of this type of computation are shown in figure 2A-27.

The slope was analyzed by using the simplified Bishop method of circular arc stability analysis (McDonnell Douglas Corporation, 1974 [14]) including a horizontal inertia force represented by a seismic coefficient of 0.15. The computed factor of safety for the most critical surface under these conditions was found to be 1.7. The critical circle is shown in figure 2A-27. The shear-strength parameters are based on the data obtained from the triaxial tests on samples from boring 2001A.

Since the cyclic strength characteristics shown in figure 2A-28 appear to be unduly high with regard to the standard penetration-resistance values of these soils, a result which may be due to some degree of sample disturbance, it was considered appropriate to check further the seismic stability of the slope by using an analysis based on the assumption that the earthquake shaking might possibly induce liquefaction of the loose zone of sand (soil 4, figure 2A-27) toward the end of the earthquake shaking and that, following the earthquake, the soil in this zone would make no contribution to the continued stability of the slope. This is believed to be a conservative approach since the liquefaction analysis shows that no such liquefaction could occur and, also, if it should occur for a magnitude 6 earthquake, it would have to be near the end of the period of strong shaking which would last only a few seconds; thereafter, the stability of the slope would be determined by the static stress conditions with no inertia forces included but with zero shear strength assigned to the loose zone of soil.

The results of this conservative analysis are shown in figure 2A-32. It may be seen that, even assuming zero strength for soil 4, the computed minimum factor of safety is 1.6.

Accordingly, it is concluded that in spite of the presence of a loose zone of soil in parts of the riverbank profile, the section nevertheless provides an ample margin of safety against sliding during or following the DBE event.

In addition the following analysis, observations, and conclusions regarding the stability of the river bank slope were made. An analysis of the river bank slope was made with the loose zone extending to the outer river bank. The simplified Bishop method of circular arc stability analysis (McDonnell Douglas Corporation, 1974 [14]) including a horizontal inertia force represented by a seismic coefficient of 0.15, was used. The computed factor of safety for the most critical surface under this condition was 1.6. The original and extended zones of soil 4, along with the critical circle, are shown in figure 2A-33. The shear-strength parameters are based on the data obtained from the triaxial tests on samples from boring 2001A.

Although the slope is safe for the assumed condition of an extended loose zone, such an assumption is unreasonable when the descriptions of the low-blow-count zones in borings 459 and 2002 are considered. The material in boring 459 is brown and gray slightly clayey fine sand. It is an alluvial deposit. The material in boring 2002 is a light gray green clayey silty fine sand. The loose zones in borings 459 and 2002 are 2 different soil deposits.

The landward extension of the loose zone postulated from borings 485, 486, 487, 488, 494, 511, 516, 521, and 522 does not have any impact on the service water piping and safety-related structures. Borings 413, 423, 435, 456, 495, 512, 519, and 522 have N values greater than or equal to 10. The existence of these N values precludes the possibility of such an extension. The subsurface profiles, figure 2A-3, sheets 1 through 7, also demonstrate this. The granular soils underlying the service water piping (figure 2A-3, sheet 7) are similar in character and consistency to the materials in the power block area and have factors of safety against liquefaction comparable to the values given for the yard area shown in figure 2A-24.

No uncemented layers were encountered in the cemented clayey silty sand zone during the river bank test borings. The shear strength of this zone (soil 2), represented by sampled UD-3 and UD-4 from boring 2001A (figure 2A-11, sheet 5), has been reevaluated, using triaxial test results from samples taken from the same soil layer elsewhere on the site (B-446, UD at 53 to 56 ft and B-456, UD at 23 to 25 ft, figure 2A-11, sheets 2 and 3). The shear strength from these tests was evaluated at a strain level of 2%.

A pseudostatic analysis of the slope was then made for a seismic coefficient of 0.15, assuming this reduced strength for the cemented zone (soil 2) and the extension of the loose zone (soil 4) to the outer bank. The computed factor of safety for the most critical surface under this condition was 1.4. The critical circle is shown in figure 2A-34.

Consolidated drained triaxial strength tests have not been performed on site soil specimens. The specimens in the completed consolidated undrained triaxial tests performed on samples from boring B-2001A were not saturated by the back-pressure method before shearing.

The unconfined water surface in the upper stratum of the Hawthorn formation has very little effect on the stability of the river bank slope. The water is confined to the upper layer by the clayey silty coarse-to-fine cemented sand zone (soil 2) and is not connected to the lower strata. The presence of the unconfined water surface will slightly reduce the resistance to sliding contributed by the friction angle of the upper soil zone (soil 1), but this is insignificant because the soil has a relatively high value of cohesion, 1000 lb/ft^2.

A pseudostatic analysis of the slope was then made for a seismic coefficient of 0.15 and a water surface at el 122 in the slope; this assumes that the water is not confined to the upper layer by the cemented sand zone, (soil 2). The computed factor of safety for the most critical surface under this condition was 1.2. The original and higher water levels, and the critical circle are shown in figure 2A-35.

2A.7 <u>LATERAL EARTH PRESSURES</u>

The earth pressure developed as a result of backfilling. The structural excavations depend on the type of material utilized and the elevation of groundwater. Normally, the at-rest condition is developed when relatively unyielding walls are backfilled after construction. However, the operation of heavy compaction

equipment adjacent to the walls may result in the development of lateral earth pressures in excess of those calculated on the basis of the at-rest condition. Therefore, temporary bracing of the walls was provided in order to prevent excessive deflection of the walls during backfilling operations. The at-rest earth pressures were calculated using a coefficient (K_o) of 0.5. The onsite low plasticity sandy clays or clayey sands and silty fine sands were utilized as backfill.

Exterior walls for the turbine building are designed about 3-ft thick at el 130 ft, increasing uniformly with depth to 6-ft thick at el 100 ft. Reactor building walls are 3 1/2-ft thick. These walls thicknesses, together with stiffness afforded by the internal floor wall and wall system are considered to present essentially a rigid, nonyielding wall. Therefore, lateral pressures were based on the at-rest earth pressure condition.

Figure 2A-36 is an earth pressure diagram for a wall height of 54 ft between el 129 and 75 ft for the reactor building. Pressure diagrams for any shorter wall were obtained by using that portion of the diagram between the appropriate elevations. This pressure diagram is based on backfill comprised of the aforementioned sands and clayey sands available onsite. Maximum ground water was assumed at el 122 ft based on the surficial water level perched above the lower impervious layer.

For design purposes, the earth pressures developed during an earthquake were assumed to be equivalent to 1.3 times the static earth pressure.

2A.8 EXCAVATION AND BACKFILL

The plant area excavation plan and sections are shown on figure 2A-37. The firm to dense clayey sands extending from the surface to between el 120 to 110 ft, were excavated no steeper than 1 (H): 1 (V) slopes.

The very dense sands or cemented sands underlying the firm to dense clayey sands to ~ el 70 to 80 ft were excavated no steeper than 1/2 (H): 1 (V). The safety of the slope geometry was verified by stability analysis (section 2A.6).

A concrete working mat was employed to prevent disturbance to the exposed soils. The working mat thickness was dependent of the presence of sand seams within the cemented zone, the firm clayey sands and plastic clays between el 50 and 60 ft, and the thickness of the cemented zone remaining in place below the foundation level.

The purpose of the working mat was to prevent damage to the foundation soils from heavy equipment moving over the surface and to prevent moisture changes of underlying plastic clay seams. The working mat thickness was 6 in. for the turbine and radwaste buildings and 12 in. for the reactor structure.

Proper and efficient dewatering was used to maintain a dry excavation and prevent damage to foundation soils from hydrostatic uplift. The contractor was required to maintain the water table at least 5 ft below the excavation level and provide a dry excavation.

Dewatering was accomplished within the reactor building excavation by a deep ejector well-point system. The ejector header lines were installed on a narrow berm at el 87 ft on the east, south, and west sides of the reactor building excavation. On the north side of the HNP-2 reactor excavation, the ejector header lines were supported at el 87 ft by brackets tied to the existing HNP-1 reactor building wall. The well

points on the east, south, and west sides of the excavation were operational starting September 13, 1971. Installation of the northern line of well points was delayed until completion of a retaining wall at the southeast corner of the existing HNP-1 reactor building. Pumping from the northern well-point line was started on October 20, 1971.

Groundwater levels were monitored by ten piezometers located within the HNP-2 excavation and by observations of several of the drill holes. Piezometers within the HNP-2 excavation were sealed and had tip elevations varying from el 45 to 62 ft. Water levels surrounding the powerhouse area were also measured by the system of permanent piezometers which was installed to monitor ground water between el 50 and 60 ft.

The data from the piezometers and measurements of water levels within open holes indicated that two water levels exist. A perched water table exists above the clay and sand with clay seams stratum immediately below the foundation level. Dewatering lowered the general water table within the sandy portions of the foundation material to levels much below foundation elevation. Water levels in the permanent piezometers surrounding the HNP-2 excavation indicated that the well-point system lowered the ground water table within the deeper sand zone to below el 65 ft over a broad area surrounding HNP-2. Within the excavation, piezometers installed within the deeper sand strata indicated water levels as deep as el 45 to 50 ft. Piezometers installed above ~ el 60 ft and water levels in shallow drill holes indicated that ground water occurred as high as ~ el 71 ft. This water formed in sand zones above the strata of clays and sands, with thin clay layers located at, and immediately below, the foundation level.

The foundation subgrade was carefully inspected and found to be firm, dry, undisturbed soils typical of those encountered in the preconstruction investigation. At no time was any upward flow of water noted. The relatively high perched water table in some portions of the excavation in no way adversely affected the foundation soil. The exposed foundation surface was methodically probed and all loose or disturbed areas were undercut.

Hydraulic communication between the minor confined aquifer and the unconfined groundwater did not occur. The excavations at the site did not expose this lower aquifer. As shown on figure 2A-3, sections A-A through F-F, and figure 2A-37, HNP-2 excavation plan, the deepest excavations were for the HNP-1 and HNP-2 reactor buildings. Material in these areas was excavated to el 73.2 ft msl. In addition, two small sumps were excavated to el 66.6 ft for the HNP-2 reactor building. The subsurface profiles (figure 2A-3) also show materials comprising the base of the confining layer. The basal portion of the confining layer in the plant area consists of sandy plastic clay with fine sand layers (stratum G) and clayey fine and very fine sand with plastic clay layers (stratum H). In some areas stratum G is not present but is replaced by a thick layer of stratum H. The irregular base of the confining layer is generally below el 65 ft msl, and in many areas, below el 60 ft msl. The highest occurrence of the base of the confining layer below the HNP-2 reactor building was in B-592 (section E-E, figure 2A-3), where the base was encountered at el 67.5 ft msl. At this location, a 5.7-ft section of the basal portion of the confining layer separates the excavation from the lower aquifer.

Further assurance that the HNP-2 reactor building excavation had not exposed the lower aquifer was provided by the ten inspection borings (RFI-1 through RFI-10). The RFI-series borings were drilled as part of a comprehensive foundation inspection program. The logs of these borings are shown in supplement 2B figures 2B-113 through 2B-122, and their locations are shown on figure 2A-2. These borings encountered 2.5 to 10 ft of stratum G material and 5 to 11.5 ft of stratum H material. The minimum thickness of the basal portion of the confining layer encountered in these borings was 11.8 ft in

RFI-1. A similar inspection program was conducted for HNP-1 structures. Borings B-562 through B-575, shown on figures 2B-89 through 2B-91, encountered 0 to 5.5 ft of stratum G material, and 6 to 20 ft of stratum H material. The minimum thickness of the basal portion of the confining layer encountered in these borings was 8 ft in B-571.

Two borings near the sumps for the HNP-2 reactor building encountered a minimum of 12.6 ft of the confining layer below the excavation for the reactor building. Boring RFI-2 was located 10 ft south of the south sump (figure 2A-37). Stratum G was encountered between el 68.7 ft and 65.7 ft msl. Stratum H was encountered between el 72.1 and 68.7 ft msl, and again between el 65.7 and 59.5 ft msl. Since the base of the south sump was at el 66.6 ft msl, ~ 7.1 ft of the basal portion of the confining layer exists below the base of the sump. Boring B-455 was drilled prior to excavation ~ 24 ft north of the north sump. Stratum G was not encountered. Stratum H was penetrated between el 73.4 and 60.4 ft msl. Since the base of the north sump was at el 66.6 ft msl, ~ 5.2 ft of the basal portion of the confining layer exists below the base of the sump. The presence of at least 5 ft of intact material comprising the confining layer below the deepest plant excavations precludes hydraulic communication between the upper and lower aquifer.

Onsite and locally available offsite sandy clays, clayey sands, and silty fine sands were used as backfill around Seismic Category I structures and in conduit trenches. The offsite soils were brought to the site from a borrow pit in Toombs County, 5 miles north of the plant site on U.S. Highway 1. Backfill was placed on inspected subgrade in thin layers and compacted to an average of 95% of the maximum dry density as determined by ASTM D 1557 (Modified Proctor). Representative moisture-density relationships for the backfill materials are shown in figure 2A-38.

2A.9 EXCAVATION AND REPLACEMENT OF BACKFILL FOR THE INTAKE STRUCTURE BURIED PIPING AND CONCRETE DUCTS

2A.9.1 INTRODUCTION

During routine maintenance operations at the intake structure in the summer of 1979, a localized failure of the asphalt pavement occurred. The 20-in.^{2} outrigger pad of a 90-ton truck crane rested on the asphalt pavement with a heavy load being lifted by the crane. The location of the failure is shown on figure 2A-39.

As a result of the incident, a preliminary investigation (phase I investigation) was undertaken by Georgia Power Company. The investigation consisted of the following:

- *A visual inspection of the area surrounding the intake structure.*
- *A test pit with density tests.*
- *Soil test borings.*
- *Installation of piezometers.*

2A.9.2 ASSESSMENT OF IN SITU BACKFILL

2A.9.2.1 Phase I Findings

The results of the initial investigation around the south side of the intake structure indicated the following:

- *Medium dense soil near the surface and loose soil around the service water pipes immediately adjacent to the intake structure.*
- *A void beneath and around the western-most 30-in.-diameter service water pipe.*
- *Relatively weak soil backfill just south of the intake structure and around the pipe for a distance of ~ 50 ft.*
- *A 6-in.-diameter pipe column used for temporary support of the pipes during construction at the south end of the test pit, ~ 20-ft south of the intake.*

The temporary support was cut and removed. Dial gauges were used to determine deflections of the pipe after removing the support. Measured deflections were ~ 0.09 in.

2A.9.2.2 Phase II Findings

In order to supplement the phase I investigation program and to better define the nature and extent of the loose soils, a phase II investigation was performed by Law Engineering Testing Company (LETCO). All field and laboratory testing was made in accordance with the appropriate ASTM specifications. The investigation consisted of the following:

- *15 SPT soil borings, including 35 thin-walled tube samples for density and strength tests.*
- *10 Dutch Cone Penetrometer soundings.*
- *Four ground water observation wells.*
- *Four test pits, including five horizontal plate load tests, sand cone in-place density tests, moisture-density relationship tests, and drive tube samples for in-place soil densities.*

Existing conditions determined as a result of the phase I and II field investigations are summarized on figure 2A-40. Detailed subsurface profiles are given in figure 2A-41.

2A.9.2.3 Chronology of Intake Structure Construction

Preconstruction subsurface conditions are shown on figure 2A-42. The original construction sequence of the intake structure backfill and adjacent pipelines is shown on figure 2A-43.

Difficulty may have been experienced in placing the backfill soils on the south side of the intake structure due to:

- *Overhanging portion of intake.*

- *Pipelines installed prior to backfilling.*

This construction sequence may have resulted in inadequate backfill compaction in this area.

2A.9.2.4 Evaluations

The results of the phase II soil borings, Dutch Core Penetrometer tests, and various field tests were evaluated to provide a detailed assessment of subsurface conditions.

The SPT results for the backfill on the east and west sides of the intake structure are summarized on figures 2A-44 and 2A-45, respectively. The south side results are summarized on figure 2A-46. It should be noted that the SPT results (figure 2A-46) indicate a distinct change in the density of the backfill at ~ el 80 ft. Figures 2A-45 and 2A-46 also indicate a loose layer at ~ el 65 ft, which is near the existing ground water level.

The ground water table in the backfill adjacent to the intake structure corresponds closely to the river water level. At the time of the field investigations, the river water level ranged between el 65 ft and el 68 ft.

Undisturbed thin-walled tube samples obtained from borings were measured for density in the field. Table 2A-22 summarizes dry density and water content of these samples.

Visual inspections of the test pits were documented by LETCO using field mapping techniques and photographs.[15] Test pits excavated near the intake structure (T-1 and TP-2 on figure 2A-41) revealed voids under the exposed pipes ranging from ~ 1/2 in. seen in TP-2 to ~ 3 in. seen in T-1. The results of the density tests made as the test trench (T-1) was excavated indicated an upper layer of strong, well-compacted fill material. The fill material became weaker with depth. These results are summarized in table 2A-23.

The results of the horizontal plate load tests performed on the pipe backfill sand to determine modules of subgrade reaction (K) are summarized on figures 2A-47 through 2A-51. For plate sizes ranging from 12.0-in. square to 30-in. round, K ranges from 300 to 800 lb/in^3.

2A.9.2.5 Conclusions

Based on the evaluation of conditions, the following conclusions were reached:

A. *The existing soil backfill around the service water pipes and other utilities located immediately south, east, and west of the intake structure was loose and needed to be removed to natural soil levels.*

B. The soil backfill in the original pipe trench excavation was loose and needed to be removed to natural soil levels for a distance of ~ 120-ft south of the intake structure. Test pits, borings, and Dutch Cones substantiated this lateral extent.

C. It was possible that other pipe supports existed under the service water pipes and they needed to be removed. The same procedure used for removal of the first support, including measurements of deflections, was recommended.

D. Replacement of the loose backfill soil with well compacted structural backfill in the congested south side of the intake structure would be difficult and time consuming.

2A.9.2.6 Structural Properties of K-Krete

It was recommended that, after securing the existing pipes and ducts, the soil backfill within the affected area should be removed down to natural soils and replaced with a controlled-density fill that did not require compaction. It was further recommended that the controlled-density fill be K-Krete.

A testing program for K-Krete was conducted by LETCO.[(15)] Laboratory tests on K-Krete indicated the following:

- *Unit weight of ~ 128 lb/ft^3.*
- *7-day unconfined compressive strength of about 3-4 kip/ft^2.*
- *7-day triaxial shear strength parameters of ϕ = 34 degrees, C=1 kip/ft^2. The results of the unconfined compression and triaxial shear tests are shown in tables 2A-24 and 2A-25.*

Field tests on K-Krete indicated:

- *Unit weight of ~ 132 lb/ft^3.*
- *7-day modules of subgrade reaction of about 1800 lb/in.^{3} for both 18-in. and 30-in.-diameter plates, as summarized on figures 2A-52 and 2A-53, respectively.*
- *A coated and wrapped 10-in.-diameter pipe as having 30 lb/in.^{2} of adhesion with K-Krete.*
- *Negligible shrinkage movements.*

2A.9.3 DESIGN, CONSTRUCTION, AND BACKFILLING

2A.9.3.1 Excavation and Backfilling Construction

The buried utilities in the shallow excavation portion south of coordinate N63 + 10 were first uncovered, and temporary structural supports were provided at intervals of design spans specified on the construction drawings. Prior to backfilling with K-Krete, all pipes were coated and wrapped with

Tapecoat TC Cold Prime and Tapecoat CT Tape Coat, respectively. Backfilling was performed at all times in such a manner as to avoid abrasion or other damage to tape protection on pipes.

The support system for the deep excavation portion at the east, west, and south ends of the intake structure consisted of soldier piles and timber lagging secured in place by means of bracing members. The bracing members, consisting of wales and struts, were preloaded to minimize ground displacements. The construction methods followed for erecting the excavation support system ensured speed and safety of erection and provided the required horizontal and vertical stability of the ground behind the wall and in front of the wall at the bottom of the excavation. The component members of the support system were designed to safely support the foundation loads, earth pressures, surcharge loads, utility loads, and dead loads of the structural systems. The surcharge loading for the vertical excavation support system was based on a maximum construction loading of 600 lb/ft^2. Dead weight, internal pressure, and seismic computer piping stress analyses performed for the piping systems, spans, and existing conditions at various stages of the construction showed that the uncovered Seismic Class 1 piping satisfied the operability criteria (i.e., pressure + weight + DBE seismic stresses were less than the pipe yield stress) as well as equations 8 and 9 of NC 3600 of the American Society of Mechanical Engineers (ASME) Code. The results of the analyses were submitted to Nuclear Regulatory Commission (NRC) Region II with the January and February 1981 progress reports.

Corrugated Metal Pipe (CMP) sleeves extending for distances of 6 1/2 to 24 ft and butting against the south end wall of the intake structure were installed around all Seismic Class 1 pipes. This was done in order to provide flexibility for 1/2 in. differential settlement between the intake structure and the adjoining K-Krete backfill.

The extent of the excavation down to the natural subgrades is shown in figure 2A-54. All loose soil was removed before the excavations were backfilled with K-Krete.

2A.9.3.2 __Dewatering System__

A dewatering system was required for excavating around the intake structure. Dewatering was accomplished within the deep excavation portion by a deepwell system. At the time of the final excavation to subgrade el between 52.5 to 57 ft, ground water levels below the excavation were around el 50 ft according to the data obtained from four piezometers located around the intake structure. Wellpoint Dewatering Corporation drawing D-81-15 shows details of the dewatering system. Continuous operation and complete effectiveness of the dewatering system installation was maintained until sufficient K-Krete backfill was poured to offset the hydrostatic uplift pressures.

2A.9.3.3 __Limits of Excavation in the South End__

An engineering inspection of the bedding condition of the service water pipes was performed to determine the southern limit of the excavation. A test pit was excavated manually at coordinates east 4980 and north 6209 downward ~ 6 to 8 in. below the invert (el 102.54) of the 18-in.-diameter residual heat removal (RHR) line (2E-11-RHR). A second test pit was excavated immediately west of the western-most 30-in. diameter service water line (P-41) at coordinates east 4938 and north 6206. The excavation was extended manually downward ~ 6 in. below the invert (el 102.31) of the pipe. The pipes were determined to be bedded directly on the backfill sand.

Based on the design requirements for bedding and support of the service water lines at HNP and the observations made at the site, it was concluded that the bedding for the service water lines is continuous at the southern limit of the backfill modifications of the piping backfill at the HNP intake structure. The pipes are continuously supported on the bottom as well as the sides and tops by well-compacted sand or clayey sand. The conditions which were observed in the field met the intent of the design and no further K-Kreting was necessary south of coordinate north 6200.

2A.9.3.4 Final Grade Preparation

The excavated portions were backfilled with K-Krete up to ~ el 110. The last 2 ft to the original site grade (~ el 112) were achieved by gravel and soil backfill and then asphalt paving as shown in figure 2A-55. The site grounds, fencing, and security systems were restored to their original configurations.

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TABLE 2A-1 (SHEET 1 OF 6)

CLASSIFICATION DATA

							Particle Distribution (%)		
Boring No.	Sample No.	Depth (ft)	Elevation	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	+4.76 mm (gravel)	4.76 to 0.074 mm (sand)	-0.074 mm (fines)
B-401	UD-5	60.25	75				0	66	34
B-401	UD-7	100.25	35				0	76	24
B-402	2	10	110	63	18	45			
B-402	UD	15.5 17.5	103	25	12	13	0	71	29
B-402	UD	29 31	90	38	13	25	0	29	71
B-402	UD	45 47	74	137	28	109	0	14	86
B-402	13	47.5	72.5	123	24	99			
B-402	UD	54 56	65	188	39	149	0	20	80
B-402	UD	64-66	55				0	81	19
B-402	UD	74 76	45				0	86	14
B-402	S-31	120	0	122	73	49	0	21	79
B-402	S-33	130	-10				0	17	83
B-403	UD-1	7 9	92				0	57	43
B-403		26	74	87	21	66			
B-403	UD	53	47				0	89	11
B-404		12 14	104	80	22	58	0	25	85
B-404	UD-2	33.2	84.3	52	24	28			
B-404	UD-3	44	73.5	116	26	90			
B-404	UD-4	82 84	34				0	85	15
B-404	UD-4	82 84	34				0	77	23
B-404		107 109	7 9				0	75	25
B-404B		33	84.5	60	31	29			
B-405	UD	42	110				0	51	49
B-405		72 74	80				3	52	45

TABLE 2A-1 (SHEET 2 OF 6)

Boring No.	Sample No.	Depth (ft)	Elevation	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Particle Distribution (%) +4.76 mm (gravel)	4.76 to 0.074 mm (sand)	-0.074 mm (fines)
B-405		113	40				0	12	88
B-406		23 25	97				0	90	10
B-406	11	45	78	146	50	96			
B-406		67 69	53				0	88	12
B-406	UD	77 78.5	42				0	83	17
B-407	8	40	70	110	32	78			
B-407	UD	64 66	45				0	84	16
B-407	UD	89	21				0	84	16
B-408		37 39	73				0	46	54
B-408		42	68	163	62	101	0	19	81
B-408	UD-7	62	48				0	80	20
B-408	UD-8	77 79	33				0	84	16
B-409		17 19	88				0	47	53
B-409	3	24	81.6	116	28				
B-409		28	77				0	12	88
B-409	6	41	64.6	47	21	26			
B-409	UD	64	42				0	88	12
B-410	-	52	64	122	42	80			
B-410	-	67	49	38	20	18			
B-410	-	71	45	35	22	13			
B-410	-	106.5	9.5	138	84	54			
B-410D	UD	60 62	55				0	57	43
B-410D	UD	69	47				0	67	33
B-410D		74.5	42				0	89	11
B-410D	60	86 86.5	30				0	91	9
B-410D	UD	97	19				0	90	10
B-410D		110	6				0	91	9
B-410D	UD	115	1				0	55	45

TABLE 2A-1 (SHEET 3 OF 6)

							Particle Distribution (%)		
Boring No.	Sample No.	Depth (ft)	Elevation	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	+4.76 mm (gravel)	4.76 to 0.074 mm (sand)	-0.074 mm (fines)
B-411	S-1	4.5	139.5-144				0	63	37
B-411	S-3	16-21	123-128				0	69	31
B-411	S-4	21-30	144-123					62	38
B-411		29-30		39	12	17			
B-411	UD	72	72					79	21
B-411	UD	92	52				0	85	15
B-411	UD	112	32				0	78	22
B-412	UD	52	70				0	81	19
B-412	UD	52	70				0	70	30
B-412		70	52				0	75	25
B-412	UD	77	45				0	71	29
B-412		78	38	66	26	40			
B-412		92	29				0	89	11
B-413	UD-2	57	52				0	84	16
		59							
B-413	UD-3	67	42				0	81	19
		69							
B-413	UD-4	88	22				0	76	24
B-413	UD	102	8				0	82	18
B-413	UD	102	7				0	88	12
		104							
B-415A		97	32				0	89	11
		98							
B-416	UD	82	32				0	80	20
B-418	UD-4	72	70				0	79	21
		74							
B-418	UD-5	102	40				0	54	46
		104							
B-418		112	30				1	83	16
		114							
B-425	UD	62	66				0	27	73
		64							
B-425	UD	73	56				0	90	10
B-427	UD	87	79				0	52	48
		88							
B-451		10	127.9				0	81	19
B-451		20	117.9				1	58	41

TABLE 2A-1 (SHEET 4 OF 6)

Boring No.	Sample No.	Depth (ft)	Elevation	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Particle Distribution (%) +4.76 mm (gravel)	4.76 to 0.074 mm (sand)	-0.074 mm (fines)
B-451		35	102.9				0	66	34
B-453	UD	67.33	70.0				0	68	32
B-453	UD	67.33	70.0				0	80	20
B-453	UD	67.75	69.6				0	79	21
B-453	UD	67.75	69.6				0	77	23
R-444		10	135.6				0	66	34
R-444		25	120.6				0	29	71
R-444		25	120.6				0	59	41
R-444		55	90.6				0	28	72
R-444		75	70.6				0	77	23
R-444		85	60.6				0	78	22
R-449		65	75.4				0	62	38
R-449		76.5	93.9				0	80	20
R-449		95	45.4				0	78	22
R-449-B	UD	81.75	58.75				0	80	20
R-449-B	UD-2	82.25	58.25				0	85	15
R-452		10	127.2				0	57	43
R-452		20	117.2				0	77	23
R-452		25	112.2				0	76	24
R-452		30	107.2				3	70	27
R-452		65	72.2				0	63	37
R-452		77	60.2				7	62	31
R-452		95	42.2				0	73	27
R-452		115	22.2				0	87	13
R-455		60	73.0				0	78	22
R-455		70	63.0				0	85	15
R-455		80	53.0				0	76	24
R-455		95	38.0				0	81	19
R-465		8.5	130.2				0	73	27
R-465		16	122.7				0	41	59
R-465		35	103.7				0	79	21
R-465		55	83.7				0	80	20
R-465		70	68.7				0	47	53
R-465		80	58.7				0	84	16
R-465		95	43.7				0	75	25
R-588	1	4.5-6.0	123.0				0	72	28
R-588	4	19.0-20.5	108.5				0	60	40

TABLE 2A-1 (SHEET 5 OF 6)

Boring No.	Sample No.	Depth (ft)	Elevation	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Particle Distribution (%) +4.76 mm (gravel)	4.76 to 0.074 mm (sand)	-0.074 mm (fines)
R-588	11	54.0-55.5	73.5				0	54	46
R-588	13	64.0-65.0	63.5				0	83	17
R-588	15	74.0-75.5	53.5				0	84	16
R-588	16	79.0-80.5	48.5				0	79	21
R-588	23	114.0-115.5	13.5				0	84	16
R-593	1	4.0-5.5	124.1				0	31	69
R-593	3	19.0-20.5	109.1				0	48	52
R-593	5	29.0-30.5	99.1				0	69	31
R-593	13	79.0-80.5	49.1				0	74	26
R-593	19	94.0-95.5	34.1				0	86	14
R-593	19	109.0-110.5	19.1				0	86	14
R-593	21	119.0-120.5	9.1				0	78	22
T-460		8.5	134.0				0	66	34
T-460		11	131.5				0	68	32
T-460		15	127.5				1	57	42
T-460		20	122.5				0	24	76
T-460		70	72.5				0	77	23
T-460		80	62.5				0	41	59
T-460		90	52.5				0	68	32
T-460	UD	72.5	70.0				0	83	17
T-460	UD	73.75	68.75				0	80	20
T-464		6	135.7				0	42	58
T-464		15	126.7				0	32	68
T-464		50	91.7				0	40	60
T-464		80	61.7				0	79	21
T-464		90	51.7				0	77	23
T-600	1	4.5-6.0	122.6				0	33	67
T-600	3	14.0-15.5	113.1				2	76	22
T-600	7	34.0-35.5	93.1				0	81	19
T-600	13	64.0-65.5	63.1				0	78	22
T-600	16	79.0-80.5	48.1				0	84	16
T-600	22	109.0-110.5	18.1				0	86	14
T-600	24	119.0-120.5	8.1				0	81	19
B-578			16.0				0	87	13
B-578			27.0				0	89	11
B-579			21.0				0	93	7
B-579			46.0				0	88	12

TABLE 2A-1 (SHEET 6 OF 6)

Boring No.	Sample No.	Depth (ft)	Elevation	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Particle Distribution (%) +4.76 mm (gravel)	4.76 to 0.074 mm (sand)	-0.074 mm (fines)
B-580			19.0				0	84	16
B-580			35.0				0	87	13
B-580			49.0				0	75	25
B-580			78.0	126	47	79	0	12	88
B-581			101.0				0	60	40
B-603			43.0				0	85	15
B-603			53.0				1	71	28
RFI-1	1		70.2 - 68.7				0	75	25
RFI-1	2		65.2 - 63.7				0	23	77
RFI-1	3		60.2 - 58.7				2	75	23
RFI-1	4		55.2 - 53.7				0	78	22
RFI-1	5		50.2 - 48.7				0	79	21
RFI-1	8		35.2 - 33.7				3	75	22
RFI-4	1		71.5 - 70.0				0	85	15
RFI-4	4		61.5 - 60.0				0	80	20
RFI-6	6		51.5 - 50.0				0	7	28
RFI-4	7		41.5 - 40.0				0	8	20
2001A	1	10 - 12		57	14	43	0	33	67
2001A	2	20 - 22		37	10	27	0	73	27
2001A	3	30 - 32.5		34	18	16	0	48	52
2001A	4	40 - 42.5		33	18	15	0	57	43
2001A	5	50 - 52.5					0	87	13
2001A	6	55 - 57		30	19	11	0	80	20
2001A	7	60 - 62		33	21	12	0	78	22
2001A	8	72 - 74.5		34	19	15	0	76	24
2001A	9	75 - 77.5		36	16	20	0	77	23
2001A	10	80 - 82.5					0	82	18
2001A	11	85 - 87.5		75	21	54	0	45	55
2001A	12	90 - 92.5		28	19	9	0	81	19
2001A	13	100 - 102.5		38	21	17	0	83	17

TABLE 2A-2 (SHEET 1 OF 2)

***IN SITU* MOISTURE CONTENTS**

Boring No.	Sample No.	Depth (ft)	Elevation	Moisture Content (%)
B-402		18	102	18.6
B-402		30	90	19.4
B-403	UD-1	8	92	20.8
B-404	UD-1	2	106	11.9
B-404	UD-1	13	105	21.8
B-404	UD	33	85	51.8
B-405		43	110	23.6
B-405		73	80	83.1
B-406		24	97	22.6
B-408		26	85	22.3
B-409	UD-2	17	89	23.6
B-409	UD-2	18	88	31.0
B-409	UD-3	23	83	90.9
B-4l2A	UD-3	56	66	36.4
B-442	1	6	140	13.4
B-442	5	20.5	125.5	20.8
B-442	8	40	106	18.5
B-442	9	45	101	15.0
R-444	1	5	141	11.9
R-444		10	136	14.8
R-444	2	15	131	14.0
R-444		25	121	26.8
R-444		25	121	15.3
R-444	7	30	116	20.1
R-444	8	35	111	13.1
R-444	10	45	101	22.3
R.444		55	91	15.2
R-444	12	60	86	21.3
B-446	1	5	138	18.5
B-446	3	10	133	13.5
R-448	1	5	137	17.6
R-448	3	10	132	10.7
R-448	4	15	127	18.4
R-448	5	20	122	15.5
R-448	9	40	102	23.9
R-448	11	50	92	17.5
B-451		10	128	24.6

TABLE 2A-2 (SHEET 2 OF 2)

Boring No.	Sample No.	Depth (ft)	Elevation	Moisture Content (%)
B-451		20	118	19.0
B-451		35	103	24.2
R-452	1	5	132	10.8
R-452	2	7.5	129.5	13.3
R-452A	1	8.5	128.5	14.3
R-452		10	127	16.6
R-452B	UD-1	10	127	16.6
R-452A	2	11	126	19.0
R-452	4	15	122	16.7
R-452		20	117	28.4
R-452		25	112	22.6
R-452		30	107	20.7
R-452	8	35	102	29.4
T-460		8.5	133.5	12.8
T-460		11	131	16.1
T-460		15	127	17.3
T-460A	1	15.5	126.5	22.9
T-460		20	122	23.3
T-463	1	5	139	13.4
T-463	3	10	134	11.8
T-463	4	15	129	25.2
T-463	6	30	114	15.3
T-463	8	40	104	21.2
T-464		6	136	17.6
T-464		15	127	22.4
T-464A	1	17.5	124.5	25.4
T-464	UD-1	19	123	16.4
T-464		50	92	18.1
R-465		8.5	130.5	12.4
R-465		16	123	22.9
R-465		35	104	14.5
R-465		55	84	8.4
R-466	1	6	130	3.6
R-466	2	8.5	127.5	16.9

TABLE 2A-3

X-RAY DIFFRACTION TESTS
MINERAL COMPOSITION (%)

Boring No.	Depth (ft)	Quartz	Montmorillonite	Illite	Attapulgite-Sepiolite	Kaolinite	Phosphorite and Miscellaneous
403	33.5	10	60	5	20	5	
404	60.0	10	45		30		15
405	75.0	20	45	5	20	5	5
406	60.0	20	30		25		25
408	55.0	20	35		35		10
409	45.0	20[a]	20		40		20
410	25.0	40	18	3		39	
412	30.0	25	26	4	20		
412	65.0	15[a]	60	5		45	
412	70.0	20	60		20		
420	20.0	20[a]	40		35		5
422	20.0	20[a]	40	5	20		20
426	40.0	40	45	3		12	
426	55.0	30	45	4		21	
426	90.0	20	78	2			
427	125.0	8	80	2	10		

a. Includes some cristobalite (opal).

TABLE 2A-4 (SHEET 1 OF 3)

CONSOLIDATION AND SWELL TEST DATA

Boring No.	Sample No.	(ft)	Elevation	Initial Void-Ratio	Unit Weight (lb/ft^3)	Moisture Content (%)	Overburden Stress (ksf)	Preconsolidation (ksf)	Compression Index	Swell (in./in.)
401	UD-5	60	73		117	15.2		6.5	0.04	0
401	UD-7	100	35	0.93	106	35.4	4.8	1.4	0.23	0
401	UD-7	100	35	1.07	105	40.6	4.5	3.0	0.25	0
401	UD-9	140	-5	0.64	116	25.8	7.2	5.0		0
402	UD-12	46	74	2.39	91.5	91.6	1.0	6.2	2.16	0
402	UD-15	55	65	3.99	77.8	140.1	1.5	11.0	2.67	0
402	UD-18	65.5	54.5	1.005	104.0	31.2	3.1	3.5	0.14	0
402	UD-21	75.0	45.0	1.195	96.6	34.3	3.6	4.2	0.25	0
403	UD	26	74	1.697	95.9	63.6	1.6	3.8	0.78	0.0148
403	UD	31	69	2.905	85.4	110.9	3.0	5.9	0.40	0.0287
403	UD	53	47	1.114	101.4	30.7	3.5	3.0	0.09	0
404	UD-1	12	106	3.85	134.9	11.9	0.5	2.8	0.07	0
404	UD	33	85	1.28	103.3	50.0	2.6	2.3	0.34	0.00351
404	UD	43	75	1.74	94.9	76.4	3.3	2.1	0.85	0.007
404	UD-4	83	35	0.959	110.0	33.3	5.0	1.4	0.04	0
404	UD-5	93	25	0.968	107.0	32.4	5.0	3.0	0.07	0
404	UD-6	108	10	0.982	108.3	31.4	6.0	3.6	0.13	0
405	UD-1	73	80	2.342	89.4	83.1	6.5	2.0	0.16	0
405	UD	78	75	2.99	82.0	110.2	6.5	3.4	0.23	0.0404
405	UD-5	87	66	1.887	92.6	64.2	7.1	1.5	0.10	0
405	UD	88	65	1.024	107.4	38.5	7.1	2.9	0.12	0.00074
405	UD	112	41	1.339	94.8	44.1	9.0	2.8	0.091	0.00085
406	UD-3	43	76.5	2.58	89.3	99.9	3.3	2.9	1.16	0.0014
406	UD-17	68	53	1.186	103.8	39.0	4.6	5.0	0.165	0.00069
407	UD-2	65	45	1.145	108.3	41.5	4.5	2.5	0.087	0.00092
407	UD	89	21	0.93	108.0	30.8	5.3	2.2	0.11	0
408	UD-3	33	78	0.825	116.1	29.2	2.6	5.8	0.19	0
408	UD-3	34	77	1.93	96.7	74.3	2.5	5.9	0.56	0.017
408	UD-4	37	74	1.425	99.0	63.4	3.0	1.0	0.58	0
408	UD-4	38	73	3.185	83.6	123.4	2.9	5.5	0.99	0.015
408	UD-5	42	69	2.33	87.0	111.3	3.1	1.2	0.96	0
408	UD-5	43	68	3.155	83.4	122.0	2.8	6.2	1.01	0.0095

TABLE 2A-4 (SHEET 2 OF 3)

Boring No.	Sample No.	(ft)	Elevation	Initial Void-Ratio	Unit Weight (lb/ft^3)	Moisture Content (%)	Overburden Stress (ksf)	Preconsolidation (ksf)	Compression Index	Swell (in./in.)
408	UD	53	58	1.488	94.9	46.1	3.8	4.5	0.2	0.0012
408	UD-5	63	48	1.015	110.0	37.9	4.2	1.1	0.09	0.0034
409	UD-2	17	89		125.0	23.6		4.9	0.16	0
409	UD-3	23	83	2.51	86.6	90.9	2.4	3.5	1.20	0
409	UD-4	28	78	3.16	84.8	124.0	2.5	3.8	2.09	0
409	UD-5	33	73	2.90	86.1	115.2	2.9	3.4	1.83	0
409	UD-6	41	63	0.853	113.4	32.6	3.5	4.0	0.127	0.0008
409	UD-8	63	43	1.316	102.3	46.1	3.6	3.3	0.15	0.00043
410D	UD	52	64	2.17	90.9	83.4	3.6	3.5	0.48	0.00155
410D	UD	59	57	1.061	106.9	35.5	4.0	4.9	0.12	0.00048
410D	UD	61	55	0.941	110.4	36.8	4.0	2.4	0.14	0.00051
410D	UD	67	49	0.824	114.9	32.4	4.5	2.4	0.14	0.00164
410D	UD	71	45	0.773	118.4	27.8	4.9	3.2	0.06	0.00028
410D	UD	74.5	41.5	0.8575	115.6	30.4	5.0	0.63	0.03	0.00027
410D	UD	79	37	1.71	92.4	58.8	5.3	6.3	0.22	0
401D	UD	106.5	9.5	1.204	105.7	45.4	7.6	0.8	0.15	0
410D	UD	110	6	0.812	115.4	28.4	7.0	1.2	0.07	0
410D	UD	115	1	1.537	99.3	57.0	7.9	1.2	0.08	0.00039
411	UD	92	52	1.076	107.5	39.5	5.2	3.0	0.19	0
411	UD-20	93	51	1.036	109.8	38.7	5.1	2.4	0.16	0.00048
411	UD	112	32	0.72	117.6	24.8	6.1	1.3	0.08	0.00058
412	UD	78	44	1.194	104.2	43.7	4.5	1.2	0.20	0.0018
413	UD	28	81	1.03	104.0	30.1	1.7	9.0	0.23	0
413	UD-5	58	52	0.706	118.2	25.3	4.0	2.6	0.06	0.00029
413	UD-3	68	42	1.248	101.8	44.3	4.6	3.2	0.11	0.00076
413	UD-4	88	22	1.069	108.0	41.0	5.7	4.1	0.10	0.0014
413	UD	102	8	0.672	118.8	23.1	5.8	2.1	0.12	0
415A	UD-2	97	32.5	0.898	112.8	33.0	7.0	4.0	0.03	0.00079
416	UD-4	82	32	0.891	107.0	26.1	4.9	1.5	0.07	0
417	UD	97	30.5	3.96	79.7	121.7	8.1	2.8	0.28	0
418	UD-4	73	70	0.827	109.7	28.7	6.1	0.38	0.088	0.0011
418	UD-5	103	40	1.975	93.3	72.2	8.2	5.8	0.23	0.00007
418	UD-6	112	31	0.901	111.7	32.9	6.0	3.0	0.10	0
425	UD	63	66	1.446	100.5	52.1	6.5	1.9	0.12	0.0092
427	UD	87	70.5	1.907	92.4	67.3	9.3	3.0	0.09	0.0038

TABLE 2A-4 (SHEET 3 OF 3)

Boring No.	*Sample No.*	*(ft)*	*Elevation*	*Initial Void-Ratio*	*Unit Weight (lb/ft^3)*	*Moisture Content (%)*	*Overburden Stress (ksf)*	*Preconsolidation (ksf)*	*Compression Index*	*Swell (in./in.)*
R-444	UD	78	68	0.974	112	33.7	5.2	4.8	0.15	
R-448	UD-15	72	70	0.696	119	21.1	5.2	5.0	0.05	
R-449	UD	85	55	0.902	115.5	33.3	5.5	7.8	0.39	
R-455	UD	63	70	1.001	110	35.5	3.5	9.0	0.22	
T-460	UD	83	69	0.991	111	37.3	5.7	8.0	0.39	
B-411	Bag	0-4.5	144-139.5	0.414	133.5	12.9			0.07	
B-411	Bag	16	123-128	0.372	134.2	10.5			0.09	
B-411	Bag	21	123-116	0.495	127.4	15.2			0.10	
R-587	UD-1	17.75	67.25	1.95	95.5	70.5			0.84	
R-594	UD-1	61.5	66.7	0.985	110.5	33.8			0.23	
T-597	UD-1	62.25	63.85	1.89	95.2	67.0			0.61	
RFI-2	UD	20-22	53.7-51.7	0.90	115.0	29.8			0.118	
RFI-9	UD	6-8	69-67	1.65	97.4	61.2			0.185	

TABLE 2A-5 (SHEET 1 OF 2)

TRIAXIAL SHEAR DATA

Boring No.	Sample No.	Depth (ft)	Elevation	Type Test	Saturation (%)	Unit Weight (lb/ft^3)	Moisture Content (%)	Void-Ratio	Cohesion (ksf)	Friction Angle (degrees)
402	UD	17-19	102	quick	94.0	129.0	18.6	0.53	0.72	16.5
402	UD	29-31	90	quick	100.0	130.0	19.4	0.51	1.18	10.5
402	UD	45-47	74	quick	92.0	93.9	69.8	2.05	0.43	11.0
403	UD-1	7-9	92	quick	99.0	129.0	20.8	0.57	0.67	19.5
404	UD-1	13	105	quick	87.0	122.0	21.8	0.67	1.44	8.5
404	UD	32-34	85	quick	100.0	105.0	51.8	1.35	0.63	3.8
404	UD-4	82	36	quick	100.0	108.0	47.8	1.27	0.90	18.5
405	UD-1	42-44	110	quick	100.0	125.0	23.6	0.60	1.11	10.0
405		78	75	quick	100.0	89.2	107.6	2.74	1.2	15.5
406		24	97	quick	99.0	125.0	22.6	0.60	1.0	31.5
406	19	78	43	quick	98.0	108.0	44.3	1.20	3.8	19.5
408	UD	25-27	85	quick	100.0	129.0	22.3	0.59	2.7	3.5
408	UD	34	75	quick	100.0	112.0	41.3	1.10	1.4	0.5
408		37-39	73	quick	95.0	92.1	78.8	2.19	0.3	31.5
409		17-19	88	quick	100.0	122.0	31.0	0.74	1.0	2.5
409		27-29	78	quick	95.0	92.0	77.8	2.11	0.5	8.5
413		102-104	6	quick	91.0	118.9	24.6	0.70	0.4	39.0
441	UD	87-89	55	quick	96.0	117.6	27.8	0.70	1.1	3.0
444	UD	77-79	67	quick	98.0	116.2	33.8	0.91	1.75	11.0
444	UD	95-97	47	quick	95.0	107.2	43.8	1.21	1.6	0.0
446		53-56	89	quick		98.3			0.0	20.5
446	UD	87-89	55	quick	90.0	107.8	37.9	1.10	1.9	4.5
448	UD-15	70-74	70	unconsolidated-undrained	89.0	117.6	26.2	0.79	1.8	9.5
449	UD	79-81	60	unconsolidated-undrained	97.0	116.5	31.5	0.86	0.9	12.5
449	UD-4	98-100	41	unconsolidated-undrained	95.0	104.4	44.8	1.20	3.7	16.5
451	UD	72-74		quick	96.0	116.6	32.0	0.91	2.75	11.0
452	UD-1	9-11	127	unconsolidated-undrained	87.0	129.6	16.6	0.52	1.2	16.0
452	UD	85-87	51	quick	94.0	105.8	44.7	1.26	1.85	9.5

TABLE 2A-5 (SHEET 2 OF 2)

Boring No.	Sample No.	Depth (ft)	Elevation	Type Test	Saturation (%)	Unit Weight (lb/ft^3)	Moisture Content (%)	Void-Ratio	Cohesion (ksf)	Friction Angle (degrees)
456		23-25	80	quick		138.9			0.2	59.0
456	UD	42-44	63	quick	100.0	95.2	77.7	1.44	3.3	3.0
456	UD	44-46	61	quick	98.0	110.2	39.4	1.07	2.0	0.0
458	UD-1	56-58	62	quick	96.0	118.8	29.2	0.81	1.6	11.5
458	UD	77-79	41	quick	97.0	102.0	54.4	1.48	1.2	6.0
460	UD	72-74	70	quick	93.0	121.0	24.8	0.71	2.8	10.0
464	UD-1	18-20	123	consolidated-undrained	87.0	129.3	16.4	0.50	1.6	5.5
411	101% std proctor		139.5	quick	89.0	134.9	12.8	0.40	2.4	7.5
411	100% std proctor		123	quick	76.0	135.0	10.0	0.37	1.5	19.0
411	100% std proctor		116	quick	81.0	127.2	15.2	0.50	1.3	4.0
RFI-3	UD	6.0-7.5	68.5-67	consolidated-undrained	87.0	103.0	43.5	1.31	2.0	22.0
RFI-6	UD	23.0-27	52-50	consolidated-undrained	80.0	105.0	32.9	1.08	1.7	19.0
2001A	UD-1	10-12		consolidated-undrained		127.1	23.4	0.589	1.0	24.0
2001A	UD-2	20-22		consolidated-undrained		130.6	15.7	0.488	1.0	28.5
2001A	UD-3 and UD-4	30-42.5		consolidated-undrained		127.9	14.5	0.497	5.0	40.0
2001A	UD-5	50-52.5		consolidated-undrained		128.6	19.4	0.566	1.5	36.0
2001A	UD-9	75-77.5		consolidated-undrained		100.2	48.2	1.456	0.3	20.0
2001A	UD-10	80-82.5		consolidated-undrained		103.6	42.5	1.315	0.9	22.0
2001A	UD-13	100-102.5		consolidated-undrained		107.3	30.2	1.008	2.5	26.0

TABLE 2A-6

SUMMARY OF SETTLEMENT ESTIMATE

	Total (in)	Immediate (in)	Total Consolidation (in)	Consolidation After Construction (in)	Percent Tilt
Unit 1					
Reactor	3.1	2.2	0.9	0.8	0
Turbine	1.5	1.0	0.5		.05[a]
Radwaste	1.5	1.0	0.5		.10[a]
Unit 2					
Reactor	4.5	3.2	1.3	0.5	0
Turbine	2.0	1.4	0.6		.10[a]
Radwaste	3.5	2.4	1.1		.10[a]
Control	2.0	1.4	0.6		.10[a]

NOTES:

1. Total settlement - Settlement resulting from structural loads of building and influence of adjacent structural loads.

2. Immediate settlement - Part of the total settlement which occurs immediately upon loading.

3. Total consolidation - Part of the total settlement which occurs after the load is applied and immediate settlement has occurred.

4. Consolidation after construction - Part of consolidation settlement which occurs after completion of construction.

5. Tilt - Tilt obtained by dividing estimated maximum edge-to-edge differential settlement by distance between edges and multiplied by 100.

a. Toward reactor.

TABLE 2A-7 (SHEET 1 OF 2)

HNP-2 INSPECTION AUGER BORINGS

Boring No.	*Depth (ft) From - To*	*Soil Description*	*Penetration Resistance (ft)*
C-1	*N 51 + 12, E 49 + 67 el 73 ft*		
	0 - 1.0	*Loose-to-firm, green-gray fine sand and clayey fine sand with thin green clay layers.*	*0.2 - 9 1.0 - 12*
C-2	*N 50 + 81, E 49 + 67 el 72.5 ft*		
	0 - 1.0	*Loose-to-firm, light-gray and green-gray fine sand and clayey fine sand with thin green clay layers.*	*0.2 - 9 1.0 - 12*
C-3	*N 50 + 55, E 49 + 67 el 72.7 ft*		
	0 - 1.0	*Loose-to-firm, light-gray and green-gray fine sand and clayey fine sand.*	*0.3 - 8 1.0 - 11*
C-4	*N 51 + 12, E 50 + 28 el 73.17 ft*		
	0 - 1.0	*Loose, light-gray and green-gray fine sand and clayey fine sand.*	*0.2 - 7 1.0 - 9*
C-5	*N 50 + 87, E 50 + 28 el 73.17 ft*		
	0 - 1.0	*Loose-to-firm, light-and green-gray fine fine sand and clayey fine sand.*	*0.2 - 7 1.0 - 9*
C-6	*N 50 + 55, E 50 + 28 el 73.17 ft*		
	0 - 1.0	*Loose-to-firm, light-gray and green-gray fine sand and clayey fine sand.*	*0.2 - 9 1.0 - 15*
C-7	*N 51 + 12, E 50 + 90 el 73.17 ft*		*0.2 - 5*
	0 - 1.0	*Loose, light-gray and green-gray fine sand and clayey fine sand.*	*1.0 - 9*
C-8	*N 50 + 87, E 50 + 90 el 73.17 ft*		
	0 - 1.0	*Loose-to-firm, light-gray and green-gray fine sand and clayey fine sand.*	*0.2 - 8 1.0 - 11*
	Depth (ft)		*Penetration*

TABLE 2A-7 (SHEET 2 OF 2)

Boring No.	*From - To*	*Soil Description*	*Resistance (ft)*
C-9	*N 50 + 55, E 50 + 90* *el 73.17 ft* *0 - 1.0*	*Loose, light-gray and green-gray fine sand and clayey fine sand.*	*0.2 - 6* *1.0 - 8*
C-10	*N 50 + 20, E 50 + 90* *el 73.17 ft* *0 - 1.0*	*Firm, gray, very clayey fine sand.*	*0.2 - 10* *1.0 - 12*
C-11	*N 51 + 12, E 50 + 50* *el 73.0 ft* *0 - 1.0*	*Loose, green-gray and light-gray fine sand and clayey fine sand with thin, green clay layers.*	*0.2 - 7* *1.0 - 8*
C-12	*N 50 + 87, E 50 + 50* *el 73.0 ft*	*Loose-to-firm, light-gray and green-gray fine sand and clayey fine sand with thin, clay layers.*	*0.2 - 7* *1.0 - 11*
C-13	*N 50 + 55, E 50 + 50* *el 73.17 ft*	*Loose-to-firm, light-gray and green-gray fine sand and clayey fine sand with thin, clay layers.*	*0.2 - 8* *1.0 - 11*
C-14	*N 51 + 12, E 50 - 70* *el 72.0 ft*	*Loose-to-firm, light-gray and green-gray fine sand and clayey fine sand with thin, clay layers.*	*0.2 - 8* *1.0 - 11*
C-15	*N 50 + 87, E 50 + 70* *el 72.5 ft*	*Loose, light-gray and green-gray fine sand and clayey fine sand with thin, clay layers.*	*0.2 - 6* *1.0 - 8*

TABLE 2A-8

TOTAL SETTLEMENT VALUES

Structure	Values of Predicted Settlement (ft) Immediate	Post-Construction[a]	Total Consolidation	Total	Measured Settlement[b]	Ratio of Measured to Predicted (%)
HNP-2 reactor building	0.27	0.04	0.11	0.38	0.14	37.3
HNP-2 radwaste building	0.20	0.00	0.09	0.29	[c]	[c]
Control building	0.12		0.05	0.17	0.13	79.8
HNP-2 turbine building	0.12	0.00	0.05	0.17	0.02	11.4
Diesel generator building				0.17	0.07	42.0
Main stack				0.17	0.03[d]	16.8
Intake structure				0.17	0.12[e]	70.8
HNP-1 reactor building	0.18		0.08	0.26	0.23	87.5
HNP-1 radwaste building	0.09		0.04	0.13	0.06	45.6
HNP-1 turbine building	0.09		0.04	0.13	0.10	80.0

a. Post-construction settlement is part of total consolidation settlement.
b. Measured settlements as of January 1997.
c. Benchmarks established after end of construction: Average settlement indicates heave.
d. Benchmarks established after end of construction.
e. Benchmarks destroyed and relocated; measure is approximate only.

TABLE 2A-9

CRITERIA FOR DETERMINING ALLOWABLE DIFFERENTIAL SETTLEMENTS ACROSS STRUCTURES

Structure	*Criteria*
Diesel generator building	*a. Appearance*
Main stack	*b. Equipment and system operation*
Intake structure	
HNP-2 reactor building	*a. Gap between buildings*
Control building	*b. Operating basis earthquake*
HNP-2 turbine building	
HNP-2 radwaste building	

TABLE 2A-10 (SHEET 1 OF 2)

SUMMARY OF REFERENCE DATES AND ELEVATIONS ACROSS STRUCTURES

Structure	Date	Benchmark	Elevation (ft)
HNP-2 reactor building	May 1976	1	129.914[a]
	May 1976	2	129.877[a]
	May 1976	3	129.918
	May 1976	4	129.864
HNP-2 radwaste building	October 1975	5	132.266
	October 1975	6	132.240
	October 1975	7	132.268
	October 1975	8	132.309
Control building	January 1975	9	111.886
	January 1975	10	111.842
	January 1975	11	111.923
	January 1975	12	111.920
HNP-2 turbine building	May 1976	13	129.926[a]
	May 1976	14	129.960[a]
	May 1976	15	129.960[a]
	January 1976	16	129.911[a]
Diesel generator building	January 1975	17	131.933[a]
	January 1975	18	131.643[a]
	January 1975	19	131.017[a]
	January 1975	20	131.328[a]
Main stack	October 1974	21	199.978
	October 1974	22	119.972
	October 1974	23	119.986
Intake structure	October 1974	24	[b]
	October 1974	25	
	October 1974	26	
	October 1974	27	
HNP-1 reactor building	May 1976	28	129.772
	May 1976	29	129.718
	May 1976	30	129.884[a]
	May 1976	31	129.744
Structure	Date	Benchmark	Elevation (ft)

TABLE 2A-10 (SHEET 2 OF 2)

HNP-1 turbine building	*January 1975*	*NE*	*111.837*
	January 1975	*SE*	*111.842*
	January 1975	*NW*	*111.913*
	January 1975	*SW*	*111.922*

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a. Reference elevations adjusted to account for benchmark alternations.
b. Elevations on intake structure not recorded at end of construction.

TABLE 2A-11

SUMMARY OF DIFFERENTIAL SETTLEMENTS ACROSS STRUCTURES

Structure	Reference Date	Direction of Tilt	Between Benchmark Nos.	Differential Settlement (in.) Allowable	Differential Settlement (in.) Measured[a]	Ratio of Measured to Allowable (%)
HNP-2 reactor building	May 1976	N-S	1 and 2	0.40	0.01	3
		N-S	3 and 4	0.41	0.02	6
		E-W	1 and 3	1.67	0.12	7
		E-W	2 and 4	1.61	0.11	7
HNP-2 radwaste building	October 1975	N-S	5 and 6	1.85	0.41	22
		N-S	7 and 8	1.92	0.34	18
		E-W	5 and 7	1.58	0.04	2
		E-W	6 and 8	0.96	0.04	4
Control building	January 1975	N-S	9 and 10	1.00	0.16	16
		N-S	11 and 12	0.95	0.22	23
		E-W	9 and 11	3.01	0.25	8
		E-W	10 and 12	3.46	0.19	6
HNP-2 turbine building	May 1976	N-S	13 and 14	2.69	0.26	10
		N-S	15 and 16	2.46	0.52	21
		E-W	13 and 15	2.96	0.30	10
		E-W	14 and 16	3.37	0.05	1
Diesel generator building	January 1975	N-S	17 and 18	5.09	0.28	5
		N-S	19 and 20	4.73	0.17	4
		E-W	17 and 19	2.47	0.49	20
		E-W	18 and 20	2.47	0.05	2
Main stack	October 1974	N-S	21 and 22	0.44	0.14	33
		E-W	21 and 23	0.55	0.08	15
		E-W	22 and 23	0.50	0.06	12
Intake structure	October 1974	N-S	24 and 25	2.50	0.12	5
		N-S	26 and 27	2.50	0.04	1
		E-W	24 and 26	0.65	0.02	4
		E-W	25 and 27	1.25	0.11	9

a. Measured settlements as of January 1997.

TABLE 2A-12

CALCULATION OF MOMENT M_D DUE TO BUILDING SETTLEMENT PENETRATIONS BETWEEN ADJACENT STRUCTURES

Method No.	Assumptions	Type of Calculation
1	*Rigid anchor* *Double-acting hangers* *Piping modeled only through 3 or 4 supports*	*Hand calculation using basic beam formulas* *Computer calculation*
2	*Some anchor flexibility* *Single-acting hangers* *Piping modeled only through 3 or 4 supports*	*Computer calculation*

TABLE 2A-13

SUMMARY OF PENETRATION DIFFERENTIAL SETTLEMENTS HNP-2 REACTOR BUILDING AND SOIL

			Maximum Differential Settlement (in.)			Ratio of Measured to Allowable (%)	
			Measured	Allowable			
Penetration	Date of Installation	Nearest Benchmark	to Date[a]	Pipe	Anchor	Pipe	Anchor
8 in. No. 1	November 1977	1	0.16	0.73	1.03	21	15
10 in. No. 2	December 1977	1	0.13	1.22	0.96	11	14
18 in. No. 3	April 1978	1	0.14	1.26	0.72	11	20
18 in. No. 4	March 1978	1	0.18	1.26	0.72	14	25
6 in. No. 8	November 1977	1	0.16	1.22	0.75	13	21
10 in. No. 10	January 1978	2	0.19	1.22	0.96	16	20
18 in. No. 11	March 1978	2	0.22	1.26	0.72	17	30
20 in. No. 12	January 1978	2	0.19	1.13	0.56	17	34
18 in. No. 13	April 1978	2	0.22	1.26	0.72	17	30
14 in. No. 24	November 1977	2	0.19	1.15	1.06	17	18
10 in. No. 41	February 1977	2	0.11	1.28	0.88	8	12
16 in. No. 42	March 1978	2	0.22	1.06	0.73	20	30
14 in. No. 134	March 1978	1	0.18	1.15	1.13	16	16
20 in. No. 161	January 1978	1	0.14	1.13	0.71	13	20

a. Measured settlements as of January 1997.

TABLE 2A-14

**SUMMARY OF PENETRATION DIFFERENTIAL SETTLEMENTS
DIESEL GENERATOR BUILDING AND SOIL**

Penetration	Date of Installation	Nearest Benchmark	Maximum Differential Settlement (in.) Measured to Date[a]	Allowable Pipe	Allowable Anchor	Ratio of Measured to Allowable (%) Pipe	Ratio of Measured to Allowable (%) Anchor
6 in.	January 1978	17	0.06	0.57	0.57	11	11
10 in.	December 1971	17	0.38	0.92	-	42	-

a. Measured settlements as of January 1997.

TABLE 2A-15

SUMMARY OF PENETRATION DIFFERENTIAL SETTLEMENTS MAIN STACK AND SOIL

Penetration	Date of Installation	Nearest Benchmark	Maximum Differential Settlement (in.) Measured to Date[a]	Maximum Differential Settlement (in.) Allowable in Pipe	Ratio of Measured to Allowable (%)
18 in.	June 1974	23	0.06	0.45	5
12 in.	May 1974	23	0.06	0.45	5
20 in.	June 1974	22	0.01	0.46	8
6 in.	May 1974	22	0.01	0.46	8

a. Measured settlements as of January 1997.

TABLE 2A-16

SUMMARY OF PENETRATION DIFFERENTIAL SETTLEMENTS INTAKE STRUCTURE AND SOIL

Penetration	Date of Installation	Nearest Benchmark	Maximum Differential Settlement (in.) Measured to Date[a]	Maximum Differential Settlement (in.) Allowable[b] Pipe	Maximum Differential Settlement (in.) Allowable[b] Support	Ratio of Measured to Allowable (%) Pipe	Ratio of Measured to Allowable (%) Support
30 in. el 97.28 ft	January 1978	25	0.22	1.38	0.66	16	33
12 in.	July 1974	25	0.22	0.99	1.19	22	18
18 in. (II)	February 1978	25	0.22	2.78	2.03	8	11
30 in. el 91.75 ft (I)	January 1978	25	0.22	1.38	0.66	16	33
30 in. el 91.75 ft (II)	January 1978	25	0.22	1.38	0.66	16	33
18 in. (I)	February 1978	27	0.11	2.47	1.27	4	9
6 in.	April 1976	26	0.07	1.50	0.94	5	8

a. Measured differential settlements are estimates only, since complete records do not exist prior to July 1978. Measured settlements are as of January 1997.
b. Allowable settlement is based on both pipe stress and support loads.

TABLE 2A-17

SUMMARY OF PENETRATION DIFFERENTIAL SETTLEMENTS
HNP-2 REACTOR BUILDING AND RADWASTE BUILDING

Penetration	Date of Installation	Nearest Benchmark	Maximum Differential Settlement (in.) Measured to Date[(a)]	Maximum Differential Settlement (in.) Allowable Pipe	Maximum Differential Settlement (in.) Allowable Anchor	Ratio of Measured to Allowable (%) Pipe	Ratio of Measured to Allowable (%) Anchor
1 in. No. 51	October 1977	2 and 5	0.19	1.54	-	12	-
6 in. No. 51	October 1977	2 and 5	0.19	1.14	-	17	-
1.5 in. No. 102	November 1977	2 and 5	0.18	1.07	-	17	-
8 in. No. 153	February 1977	4 and 5	0.12	1.48	0.88	8	14

a. Measured settlements as of January 1997.

TABLE 2A-18

SUMMARY OF PENETRATION DIFFERENTIAL SETTLEMENTS HNP-2 REACTOR BUILDING AND CONTROL BUILDING

Penetration	Date of Installation	Nearest Benchmark	Maximum Differential Settlement (in.) Measured to Date[a]	Allowable Pipe	Allowable Anchor	Ratio of Measured to Allowable (%) Pipe	Ratio of Measured to Allowable (%) Anchor
24 in. No. 59	May 1978	3 and 10	0.30	26.03	13.23	1	2
3 in. No. 60	January 1978	3 and 10	0.38	1.48	0.62	26	62
18 in. No. 61	August 1977	3 and 10	0.40	9.55	1.86	4	21
24 in. No. 61	September 1976	3 and 10	0.32	12.14	5.06	3	6
4 in. No. 68	January 1978	3 and 10	0.38	1.84	0.85	21	45
4 in. No. 69	January 1978	3 and 10	0.38	1.99	0.78	19	49

a. Measured settlements as of January 1997.

TABLE 2A-19

SUMMARY OF PENETRATION DIFFERENTIAL SETTLEMENTS
HNP-2 REACTOR BUILDING AND TURBINE BUILDING

Penetration	Date of Installation	Nearest Benchmark	Maximum Differential Settlement (in.) Measured to Date[a]	Allowable Pipe	Allowable Anchor	Ratio of Measured to Allowable (%) Pipe	Anchor
10 in. No. 43	May 1978	4 and 13	0.01	2.12	1.68	1	1
4 in. No. 44	January 1978	4 and 13	0.01	1.30		1	
3 in. No. 57	November 1977	4 and 13	0.01	4.17		0	
18 in. No. 57	July 1977	4 and 13	0.14	9.55	1.59	2	9
24 in. No. 57 (el 154.46 ft)	September 1976	4 and 13	0.26	25.13	10.59	1	2
24 in. No. 57 (el 154.55 ft)	September 1976	4 and 13	0.26	22.54	9.05	1	3
8 in. No. 84	February 1977	4 and 13	0.17	1.13	1.01	15	17
10 in. No. 90	January 1978	4 and 13	0.01	2.51	1.77	0	1
3 in. No. 92	December 1977	4 and 13	0.04	1.78	1.55	2	2

a. Measured settlements as of January 1997.

TABLE 2A-20

SUMMARY OF PENETRATION DIFFERENTIAL SETTLEMENTS
HNP-2 REACTOR BUILDING AND HNP-1 REACTOR BUILDING

Penetration	Date of Installation	Nearest Benchmark	Maximum Differential Settlement (in.) Measured to Date[a]	Maximum Differential Settlement (in.) Allowable Pipe	Maximum Differential Settlement (in.) Allowable Anchor	Ratio of Measured to Allowable (%) Pipe	Ratio of Measured to Allowable (%) Anchor
8 in. No. 183	January 1978	1 and 29	0.10	0.99	0.53	10	18
8 in. No. 184	December 1977	1 and 29	0.08	3.58	2.30	2	4

a. Measured settlements as of January 1997.

TABLE 2A-21

CYCLIC TRIAXIAL TEST DATA FROM BORING 2001A

Sample	Depth (ft)	Test	Chamber Pressure (lb ft^2)	Deviator Stress (lb ft^2)	Cycles to 5% Double-Amplitude Strain	Cycles to 10% Double-Amplitude Strain	Unit Dry Weight (lb ft^3)	Water Content (%)
6	55-57	A	6000	3604	4.3	6	95.1	27.7
		B	6000	2956	15	18	94.2	26.8
		C	6000	3205	1.9	3.5	91.3	25.0
7	60-62	D	6000	3113	18	25	94.5	28.9
		E	6000	3812	9.4	13	86.2	33.3
		F	6000	3175	35	41	90.9	31.8
8	72-74.5	G	6000		1	1	81.4	40.3
		H	6000	2902	10	13	76.0	43.1
		I	6000	2491	28	32	74.8	46.3
9	75-77.5	M	6000		1	1	70.7	49.6
		N	6000	2903	51	56	69.7	46.2
		O	6000	3116	32	48	71.0	45.4
12	90-92.5	J	6000	3699	3.2	7	101.0	23.5
		K	6000	3395	33	42	92.5	27.4
		L	6000	3515	90	100	91.3	27.3

TABLE 2A-22

SUMMARY OF DENSITY DETERMINATIONS FROM THIN-WALLED TUBE SAMPLES

Boring	Depth (ft)	Recovery (ft)	Wet	Moisture (%)	Dry
BU-1A	7-8.2	1.2	126.4		
	15-17	1.9	118.8	6.7	111.0
	25-27	2.0	137.9	9.2	126.2
	40-41.2	1.1	117.8	11.6	105.6
	52-52.9	0.8	114.1	26.3	90.3
BU-2A	4-5.6	1.6	137.1	14.0	120.4
	8-9.1	1.0	146.3	24.1	117.9
	5-6.5	1.4	143.3	16.5	123.0
	11.5-12.4	0.6	133.3	22.5	108.8
BU-4	15-17	1.8	108.5		
	20-22	0.9	107.9		
	47-49	0.8	97.2		
BU-6A	4-6	1.9	122.5	15.6	106.0
	17-19	1.9	132.5	6.4	124.5
	27-29	1.9	129.6	8.9	119.0
	37-38	1.0	122.7	10.1	111.4
BU-9A	4-5.9	1.8	135.5	17.8	115.0
	8-9.4	1.2	127.3	11.2	114.5
B7-10A	4-5.7	1.6	125.9	10.9	113.6
	11-12.7	1.6	86.2	15.6	74.6
BU-11	11-11.7	0.6	134.6	14.7	117.4
BU-12	9-10.3	1.3	135.8		
BU-13	8-10	1.9	144.4	12.5	128.4
	23-25	1.9	127.8	9.7	116.5
BU-14	5.5-7.3	1.6	139.7	12.9	123.7
	18-19.3	1.2	125.0	5.5	118.5
	28-29.9	1.9	125.6	6.1	118.4
BU-15	10-11.9	1.8	131.1	11.3	117.8
	18-20	1.9	123.8	6.8	115.9

TABLE 2A-23 (SHEET 1 OF 2)

TEST PIT AND DENSITY TEST SUMMARY - MODIFIED PROCTOR

Test Pit No.	Test No.	Depth (ft)	Wet Weight (lb/ft^3)	Moisture (%)	Dry Weight (lb/ft^3)	Proctor No.	Compaction (%)
TP-1	1	- 3.0	107.2	2.0	105.1	6	85.8
TP-1	2	- 3.0	105.2	3.7	101.4	6	82.5
TP-1	3	- 5.0	129.3	9.9	117.7	6	96.1
TP-1	4	- 5.0	128.1	9.1	117.4	6	95.8
TP-1	5	- 7.0	129.3	12.4	115.0	7	97.7
TP-1	6	- 7.0	128.1	14.4	112.0	7	95.2
TP-1	7	- 9.0	118.1	4.0	113.6	7	96.5
TP-2	8	- 2.0	138.2	11.0	124.5	10	97.3
TP-2	9	- 4.0	128.6	14.2	112.6	6	91.9
TP-2	10	- 6.0	123.5	14.7	107.7	6	87.9
TP-2	11	- 8.0	111.9	12.9	99.1	9	94.6
TP-4	12	- 3.5	122.0	12.5	108.4	10	84.7
TP-4	13	- 5.5	131.0	11.7	117.3	11	93.1
TP-4	14	- 7.5	133.1	27.7	104.2	6	85.1
TP-4	15	- 6.0	123.2	15.6	106.6	6	87.0
TP-5	16	- 4.5	130.4	9.5	119.1	7	100.0
TP-5	17	- 6.5	122.0	10.4	110.5	6	90.2
TP-5	18	- 8.5	129.5	16.6	111.1	6	90.7
TP-5	19	-10.0	136.5	13.8	119.9	7	100.0

TABLE 2A-23 (SHEET 2 OF 2)

Proctor No.	Maximum Dry Density (lb/ft^3)	Optimum Moisture Content (%)
5	127.2	8.0
6	122.5	11.2
7	117.7	9.0
9	104.8	13.5
10	128.0	9.1
11	126.0	9.0
12	126.7	9.0

TABLE 2A-24

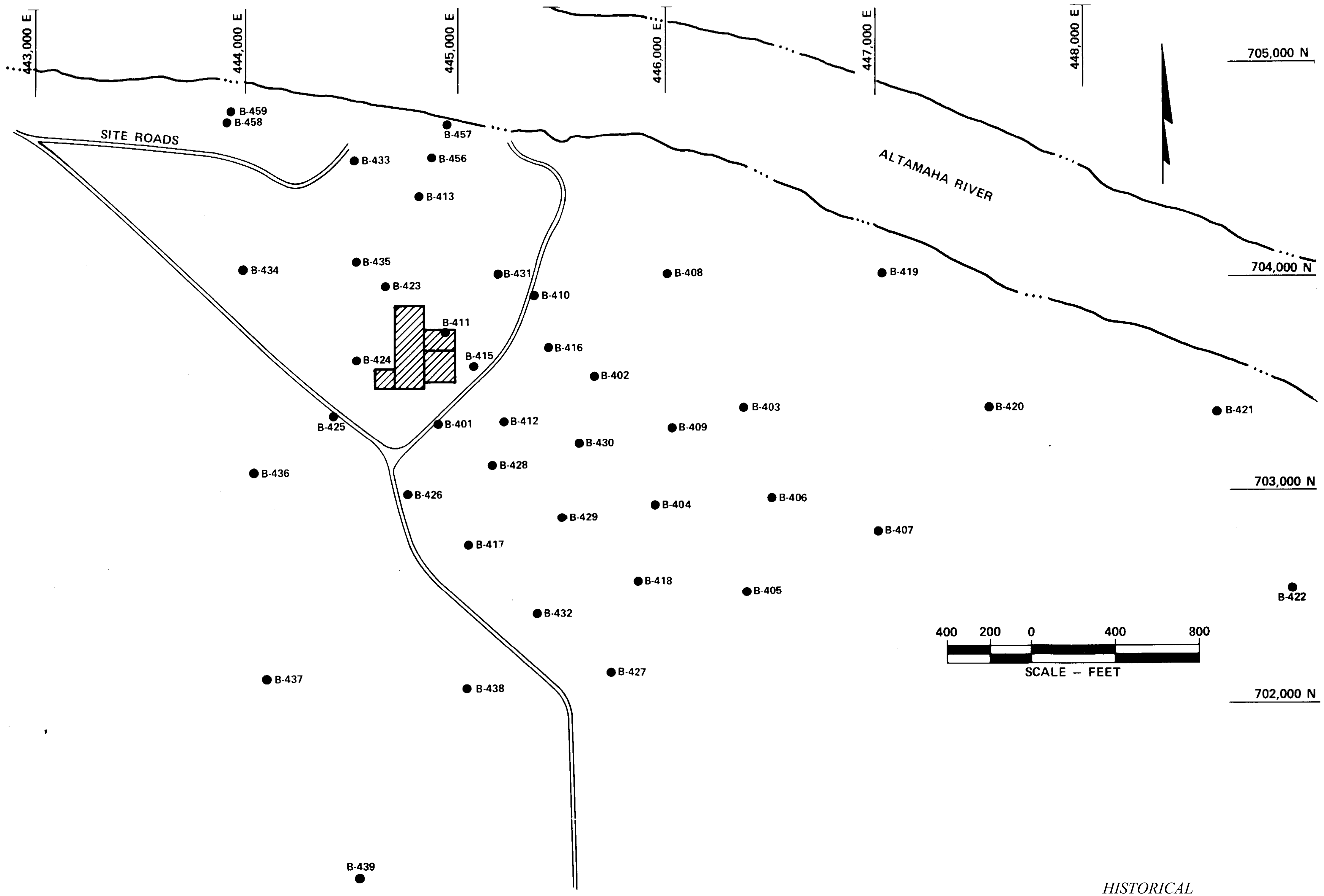
RESULTS OF UNCONFINED COMPRESSION STRENGTH TESTS ON K-KRETE CYLINDERS

Cylinder	Time of Test (days)	Stress (psi)	Strain (in./in.)	Bulk Unit Weight (lb/ft^3)	Type of Mix
1	7	22.3	0.034	127.9	Lab mix
2	7	24.7	0.089	127.9	Lab mix
3	7	26.3	0.094	127.9	Lab mix
4	7	13.4	0.040	133.1	Field mix
5	7	10.6	0.042	133.1	Field mix
6	7	13.9	0.058	131.3	Field mix
7	28	17.0	0.056	133.1	Field mix
8	28	21.7	0.088	131.3	Field mix
9	28	21.6	0.076	133.1	Field mix

TABLE 2A-25

RESULTS OF UNCONSOLIDATED TRIAXIAL SHEAR TESTS UNDRAINED ON K-KRETE

Time of Test (days)	*Cohesion (c)*	*Angle of Shear Resistance (ϕ)*
7	*1 ksf*	*34°*
14	*1.2 ksf*	*29°*
28	*1.2 ksf*	*31°*



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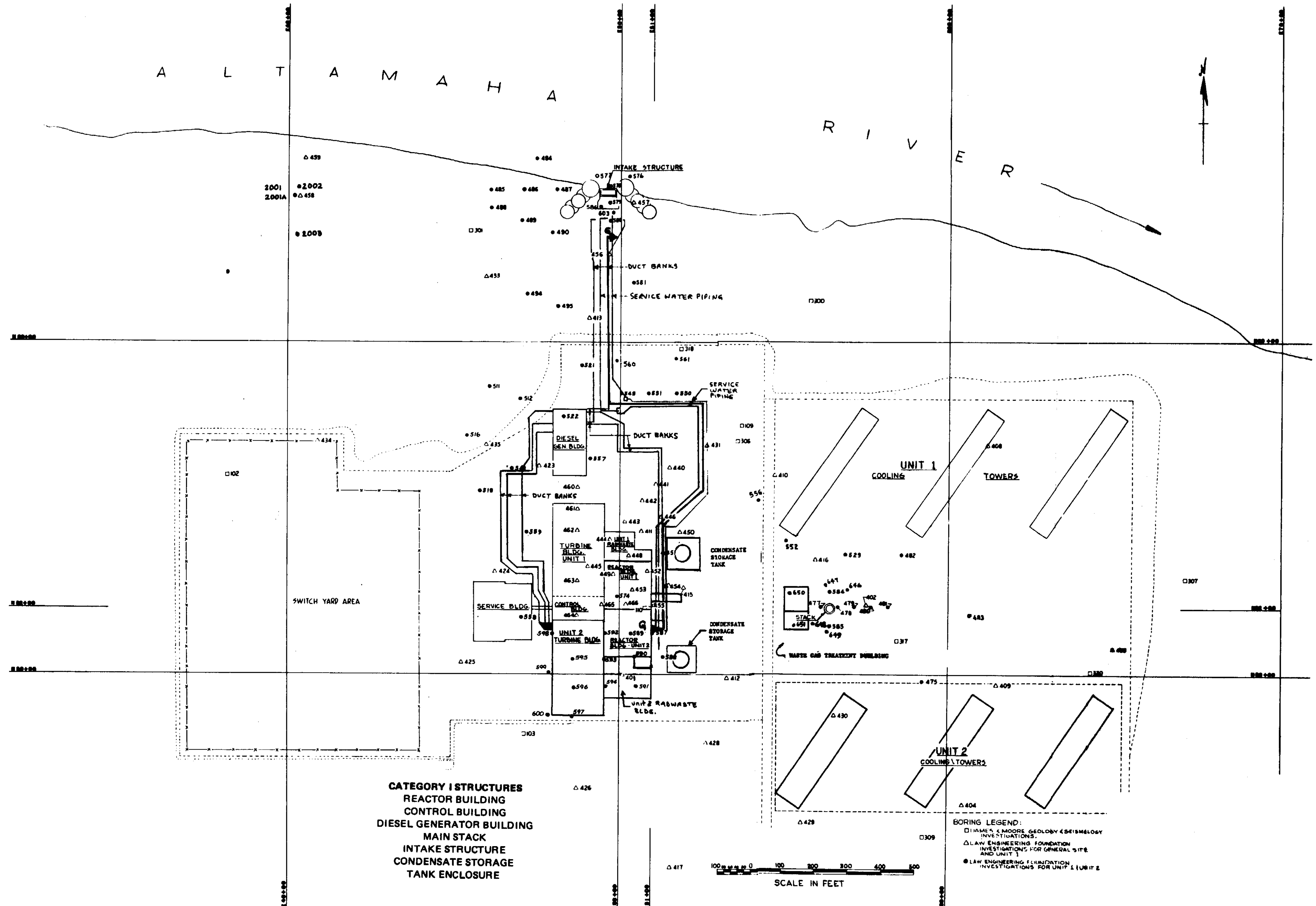
FOUNDATION BORINGS
LOCATION PLAN

FIGURE 2A-1 (SHEET 1 OF 2)



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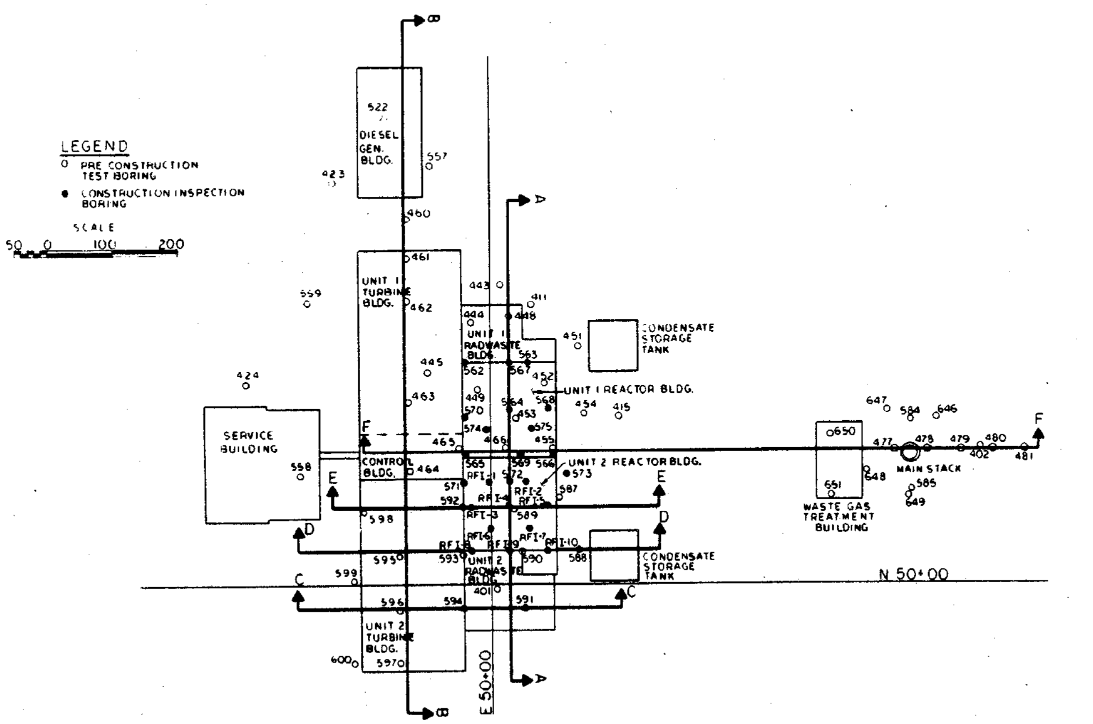
FOUNDATION BORINGS
LOCATION PLAN

FIGURE 2A-1 (SHEET 2 OF 2)



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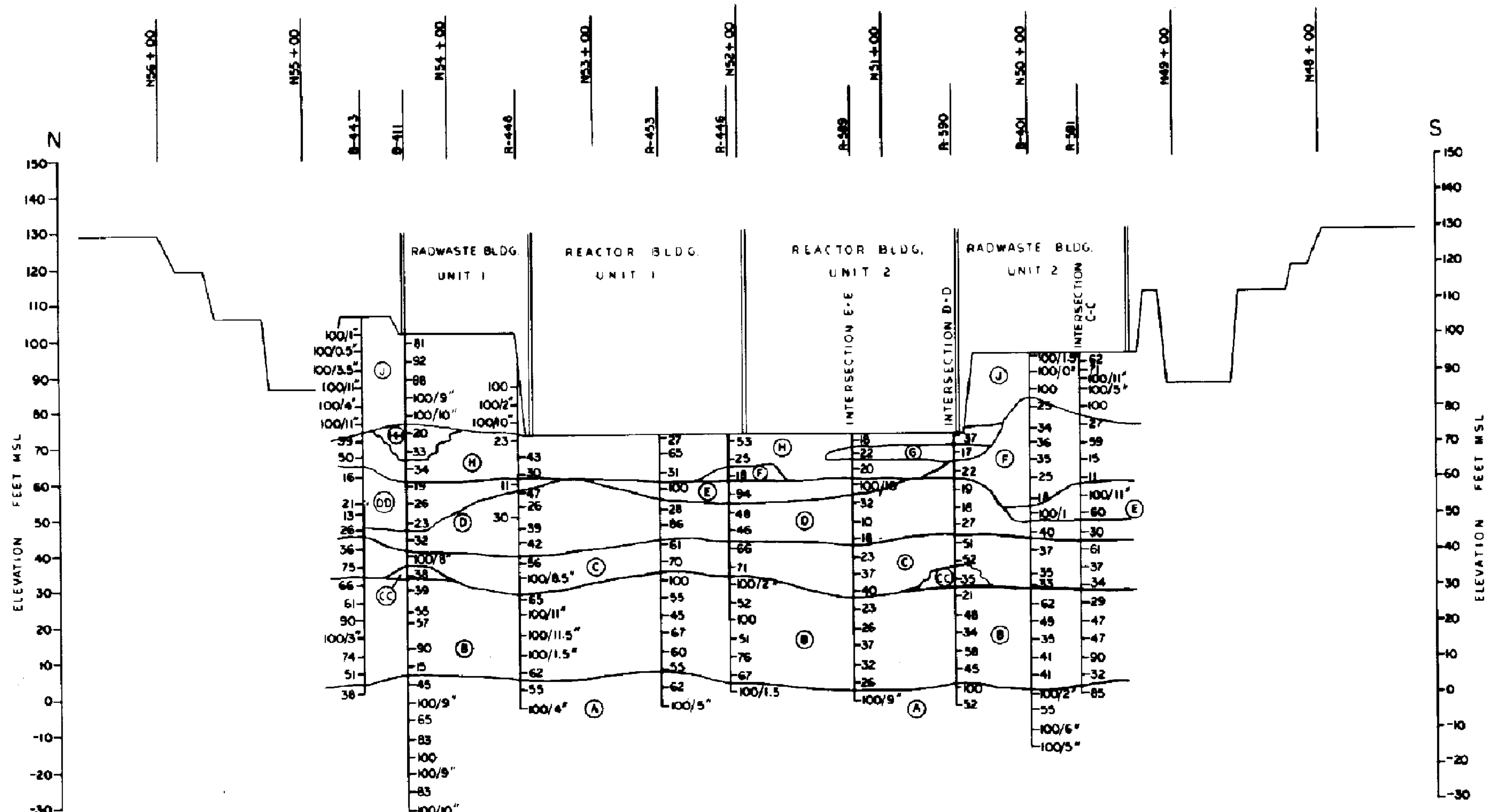
BORING AND PROFILE LOCATIONS IN PLANT AREA

FIGURE 2A-2



SECTION A-A

CORRELATION OF PENETRATION RESISTANCE
WITH
RELATIVE DENSITY AND CONSISTENCY

	NUMBER OF BLOWS, N	RELATIVE DENSITY
	0 - 4	VERY LOOSE
SANDS	4 - 10	LOOSE
	10 - 30	FIRM
	30 - 50	DENSE
	OVER 50	VERY DENSE
		CONSISTENCY
	0 - 2	VERY SOFT
	2 - 4	SOFT
SILTS AND CLAYS	4 - 8	FIRM
	8 - 15	STIFF
	15 - 30	VERY STIFF
	30 - 50	HARD
	OVER 50	VERY HARD

SOIL LITHOLOGIES

- (A) Gray green silty CLAY with fine sand seams
- (B) Gray tan and white slightly clayey fine SAND with clay inclusions and layers
- (C) Gray green slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (CC) Gray green slightly sandy plastic CLAY
- (D) Light gray slightly clayey fine and very fine SAND with partially cemented clay inclusions
- (DD) Light green gray silty slightly clayey fine and very fine SAND with few clay inclusions
- (E) Gray green clayey partially cemented SAND and CLAY
- (EE) Gray very clayey very fine SAND
- (F) Green gray clayey very fine and fine SAND with clay inclusions
- (FF) Gray slightly clayey fine SAND
- (G) Gray green slightly fine sandy plastic CLAY with thin fine sand seams
- (H) Green gray slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (HH) Green and gray clayey fine to medium SAND with clay inclusions
- (J) Gray clayey fine to medium SAND with clay inclusions, partially cemented
- (K) Light gray clayey very fine SAND with few clay inclusions
- (L) Gray clayey fine to coarse SAND
- (M) Gray partially cemented fine sandy silty CLAY
- (N) Gray and tan fine to medium sandy CLAY
- (P) Gray very silty fine SAND with partially cemented clay inclusions
- (R) Gray and brown silty fine to medium SAND with clay inclusions

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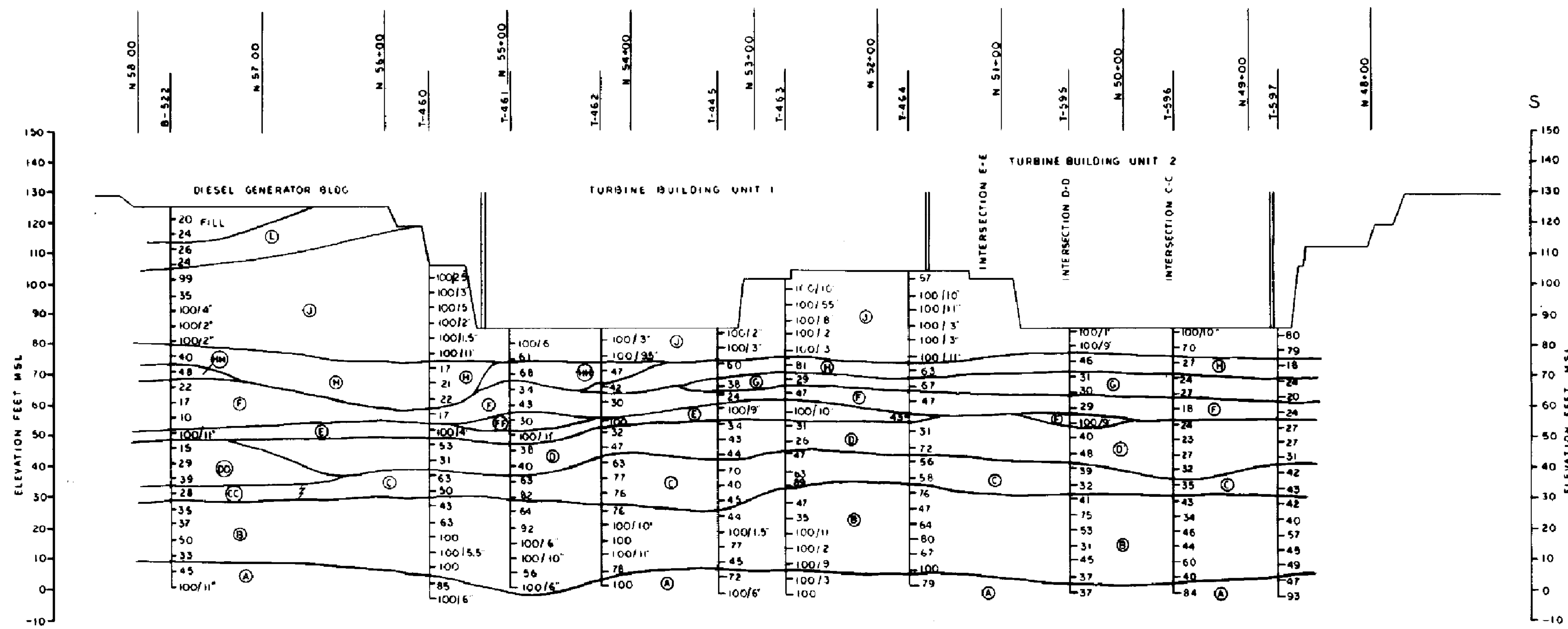
SUBSURFACE PROFILES A-A

FIGURE 2A-3 (SHEET 1 OF 7)



SECTION B-B

CORRELATION OF PENETRATION RESISTANCE
WITH
RELATIVE DENSITY AND CONSISTENCY

	NUMBER OF BLOWS, N	RELATIVE DENSITY
	0 - 4	VERY LOOSE
SANDS	4 - 10	LOOSE
	10 - 30	FIRM
	30 - 50	DENSE
	OVER 50	VERY DENSE
		CONSISTENCY
	0 - 2	VERY SOFT
	2 - 4	SOFT
SILTS AND CLAYS	4 - 8	FIRM
	8 - 15	STIFF
	15 - 30	VERY STIFF
	30 - 50	HARD
	OVER 50	VERY HARD

SOIL LITHOLOGIES

- A Gray green silty CLAY with fine sand seams
- B Gray tan and white slightly clayey fine SAND with clay inclusions and layers
- C Gray green slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- CC Gray green slightly sandy plastic CLAY
- D Light grey slightly clayey fine and very fine SAND with partially cemented clay inclusions
- DD Light green gray silty slightly clayey fine and very fine SAND with few clay inclusions
- E Gray green clayey partially cemented SAND and CLAY
- EE Gray very clayey very fine SAND
- F Green gray clayey very fine and fine SAND with clay inclusions
- FF Gray slightly clayey fine SAND
- G Gray green slightly fine sandy plastic CLAY with thin fine sand seams
- H Green gray slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- HH Green and gray clayey fine to medium SAND with clay inclusions
- J Gray clayey fine to medium SAND with clay inclusions, partially cemented
- K Light grey clayey very fine SAND with few clay inclusions
- L Grey clayey fine to coarse SAND
- M Grey partially cemented fine sandy silty CLAY
- N Grey and tan fine to medium sandy CLAY
- P Grey very silty fine SAND with partially cemented clay inclusions
- R Grey and brown silty fine to medium SAND with clay inclusions

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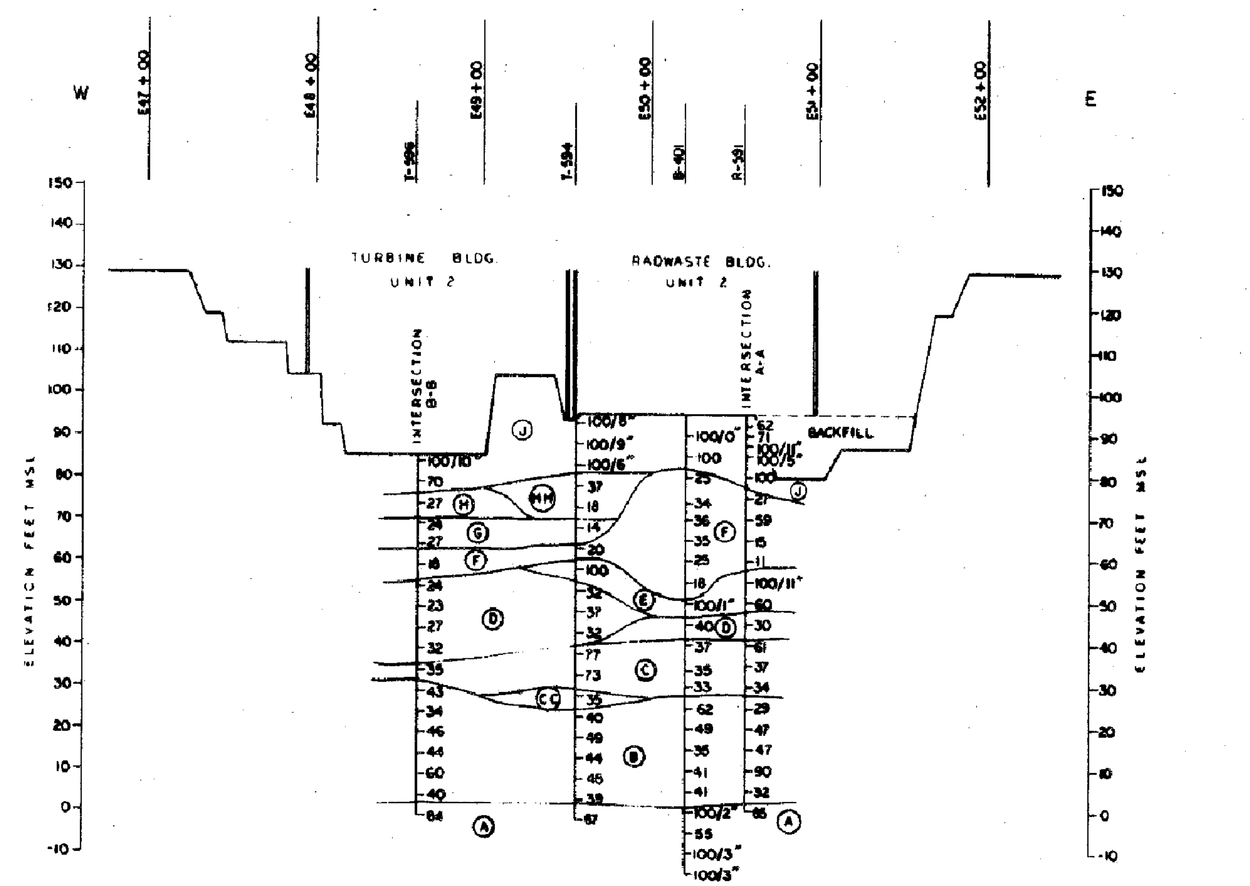
SUBSURFACE PROFILES
B-B

FIGURE 2A-3 (SHEET 2 OF 7)



SECTION C-C

CORRELATION OF PENETRATION RESISTANCE
WITH
RELATIVE DENSITY AND CONSISTENCY

	NUMBER OF BLOWS, N	RELATIVE DENSITY
	0 - 4	VERY LOOSE
SANDS	4 - 10	LOOSE
	10 - 30	FIRM
	30 - 50	DENSE
	OVER 50	VERY DENSE
		CONSISTENCY
	0 - 2	VERY SOFT
	2 - 4	SOFT
SILTS AND CLAYS	4 - 8	FIRM
	8 - 15	STIFF
	15 - 30	VERY STIFF
	30 - 50	HARD
	OVER 50	VERY HARD

SOIL LITHOLOGIES

- (A) Gray green silty CLAY with fine sand seams
- (B) Gray tan and white slightly clayey fine SAND with clay inclusions and layers
- (C) Gray green slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (CC) Gray green slightly sandy plastic CLAY
- (D) Light gray slightly clayey fine and very fine SAND with partially cemented clay inclusions
- (DD) Light green gray silty slightly clayey fine and very fine SAND with few clay inclusions
- (E) Gray green clayey partially cemented SAND and CLAY
- (EE) Gray very clayey very fine SAND
- (F) Green gray clayey very fine and fine SAND with clay inclusions
- (FF) Gray slightly clayey fine SAND
- (G) Gray green slightly fine sandy plastic CLAY with thin fine sand seams
- (H) Green gray slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (HH) Green and gray clayey fine to medium SAND with clay inclusions
- (J) Gray clayey fine to medium SAND with clay inclusions, partially cemented
- (K) Light gray clayey very fine SAND with few clay inclusions
- (L) Gray clayey fine to coarse SAND
- (M) Gray partially cemented fine sandy silty CLAY
- (N) Gray and tan fine to medium sandy CLAY
- (P) Gray very silty fine SAND with partially cemented clay inclusions
- (R) Gray and brown silty fine to medium SAND with clay inclusions

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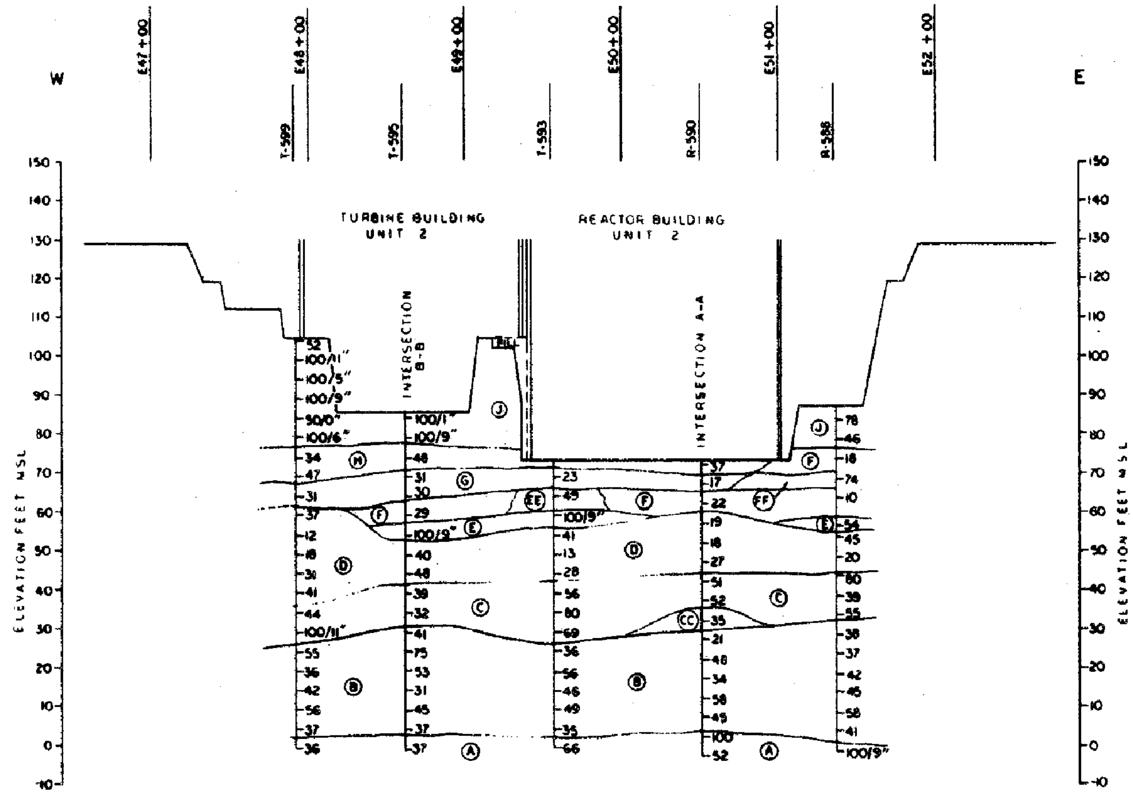
SUBSURFACE PROFILES
C-C

FIGURE 2A-3 (SHEET 3 OF 7)



SECTION D-D

CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY

	NUMBER OF BLOWS, N	RELATIVE DENSITY
	0 - 4	VERY LOOSE
SANDS	4 - 10	LOOSE
	10 - 30	FIRM
	30 - 50	DENSE
	OVER 50	VERY DENSE
		CONSISTENCY
	0 - 2	VERY SOFT
	2 - 4	SOFT
SILTS AND CLAYS	4 - 8	FIRM
	8 - 15	STIFF
	15 - 30	VERY STIFF
	30 - 50	HARD
	OVER 50	VERY HARD

SOIL LITHOLOGIES

- (A) Gray green silty CLAY with fine sand seams
- (B) Gray tan and white slightly clayey fine SAND with clay inclusions and layers
- (C) Gray green slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (CC) Gray green slightly sandy plastic CLAY
- (D) Light gray slightly clayey fine and very fine SAND with partially cemented clay inclusions
- (DD) Light green gray silty slightly clayey fine and very fine SAND with few clay inclusions
- (E) Gray green clayey partially cemented SAND and CLAY
- (EE) Gray very clayey very fine SAND
- (F) Green gray clayey very fine and fine SAND with clay inclusions
- (FF) Gray slightly clayey fine SAND
- (G) Grey green slightly fine sandy plastic CLAY with thin fine sand seams
- (H) Green gray slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (HH) Green and gray clayey fine to medium SAND with clay inclusions
- (J) Grey clayey fine to medium SAND with clay inclusions, partially cemented
- (K) Light gray clayey very fine SAND with few clay inclusions
- (L) Grey clayey fine to coarse SAND
- (M) Grey partially cemented fine sandy silty CLAY
- (N) Gray and tan fine to medium sandy CLAY
- (P) Grey very silty fine SAND with partially cemented clay inclusions
- (R) Gray and brown silty fine to medium SAND with clay inclusions

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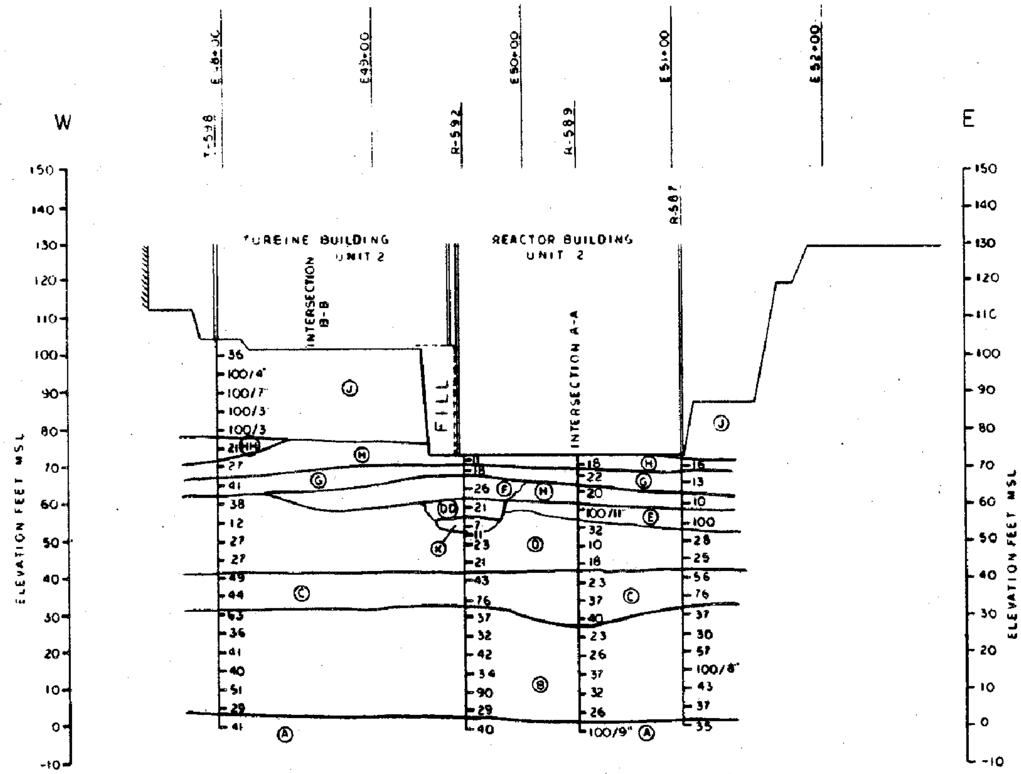
SUBSURFACE PROFILES
D-D

FIGURE 2A-3 (SHEET 4 OF 7)



SECTION E-E

CORRELATION OF PENETRATION RESISTANCE
WITH
RELATIVE DENSITY AND CONSISTENCY

	NUMBER OF BLOWS, N	RELATIVE DENSITY
	0 - 4	VERY LOOSE
SANDS	4 - 10	LOOSE
	10 - 30	FIRM
	30 - 50	DENSE
	OVER 50	VERY DENSE
		CONSISTENCY
	0 - 2	VERY SOFT
	2 - 4	SOFT
SILTS AND CLAYS	4 - 8	FIRM
	8 - 15	STIFF
	15 - 30	VERY STIFF
	30 - 50	HARD
	OVER 50	VERY HARD

SOIL LITHOLOGIES

- (A) Gray green silty CLAY with fine sand seams
- (B) Gray tan and white slightly clayey fine SAND with clay inclusions and layers
- (C) Gray green slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (CC) Gray green slightly sandy plastic CLAY
- (D) Light gray slightly clayey fine and very fine SAND with partially cemented clay inclusions
- (DD) Light green gray silty slightly clayey fine and very fine SAND with few clay inclusions
- (E) Gray green clayey partially cemented SAND and CLAY
- (EE) Gray very clayey very fine SAND
- (F) Green gray clayey very fine and fine SAND with clay inclusions
- (FF) Gray slightly clayey fine SAND
- (G) Gray green slightly fine sandy plastic CLAY with thin fine sand seams
- (H) Green gray slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (HH) Green and gray clayey fine to medium SAND with clay inclusions
- (J) Gray clayey fine to medium SAND with clay inclusions, partially cemented
- (K) Light gray clayey very fine SAND with few clay inclusions
- (L) Gray clayey fine to coarse SAND
- (M) Gray partially cemented fine sandy silty CLAY
- (N) Gray and tan fine to medium sandy CLAY
- (P) Gray very silty fine SAND with partially cemented clay inclusions
- (R) Gray and brown silty fine to medium SAND with clay inclusions

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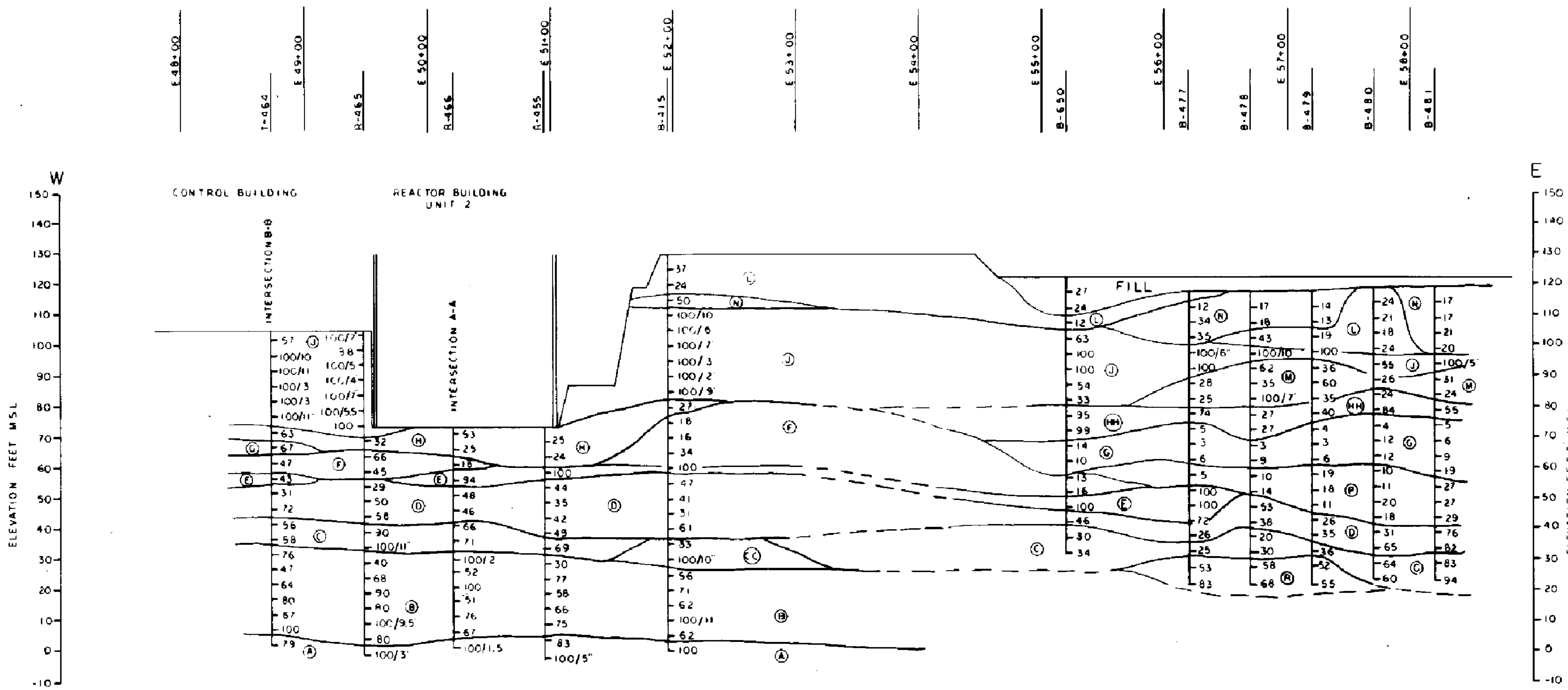
SUBSURFACE PROFILES
E-E

FIGURE 2A-3 (SHEET 5 OF 7)



SECTION F-F

CORRELATION OF PENETRATION RESISTANCE
WITH
RELATIVE DENSITY AND CONSISTENCY

	NUMBER OF BLOWS, N	RELATIVE DENSITY
	0 - 4	VERY LOOSE
SANDS	4 - 10	LOOSE
	10 - 30	FIRM
	30 - 50	DENSE
	OVER 50	VERY DENSE
		CONSISTENCY
	0 - 2	VERY SOFT
	2 - 4	SOFT
SILTS AND CLAYS	4 - 8	FIRM
	8 - 15	STIFF
	15 - 30	VERY STIFF
	30 - 50	HARD
	OVER 50	VERY HARD

SOIL LITHOLOGIES

- (A) Gray green silty CLAY with fine sand seams
- (B) Gray tan and white slightly clayey fine SAND with clay inclusions and layers
- (C) Gray green slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (CC) Gray green slightly sandy plastic CLAY
- (D) Light gray slightly clayey fine and very fine SAND with partially cemented clay inclusions
- (DD) Light green gray silty slightly clayey fine and very fine SAND with few clay inclusions
- (E) Gray green clayey partially cemented SAND and CLAY
- (EE) Gray very clayey very fine SAND
- (F) Green gray clayey very fine and fine SAND with clay inclusions
- (FF) Gray slightly clayey fine SAND
- (G) Gray green slightly fine sandy plastic CLAY with thin fine sand seams
- (H) Green gray slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- (HH) Green and gray clayey fine to medium SAND with clay inclusions
- (J) Gray clayey fine to medium SAND with clay inclusions, partially cemented
- (K) Light gray clayey very fine SAND with few clay inclusions
- (L) Gray clayey fine to coarse SAND
- (M) Gray partially cemented fine sandy silty CLAY
- (N) Gray and tan fine to medium sandy CLAY
- (P) Gray very silty fine SAND with partially cemented clay inclusions
- (R) Gray and brown silty fine to medium SAND with clay inclusions

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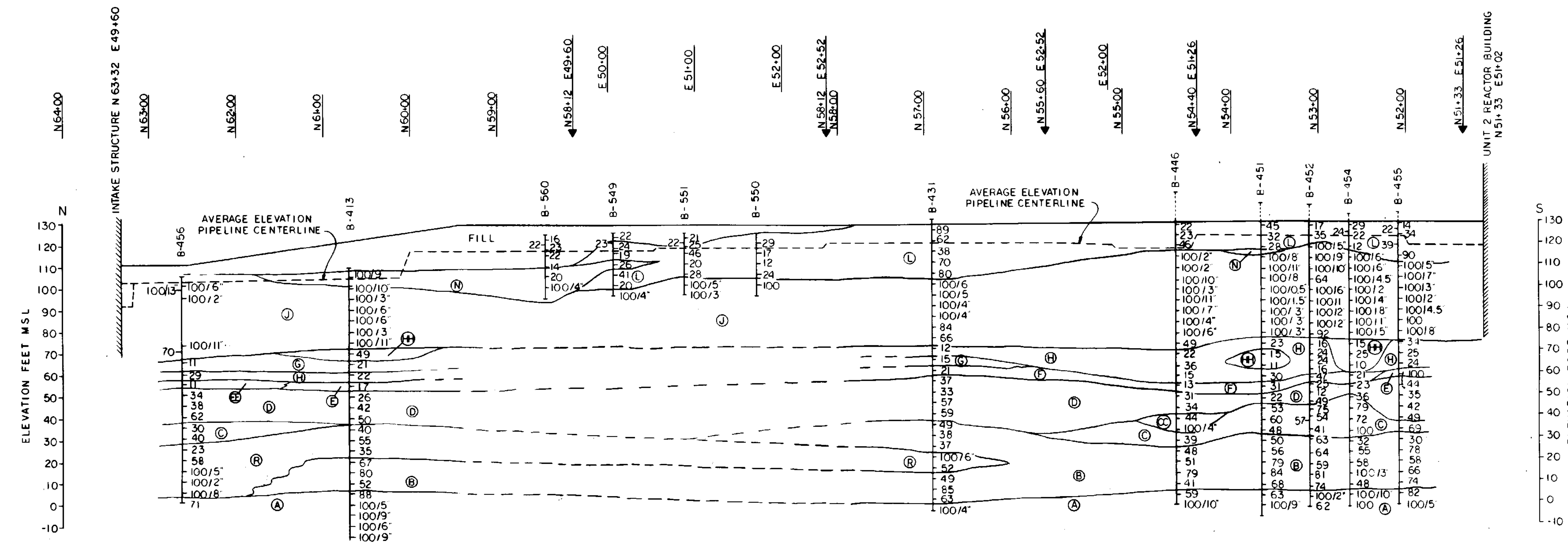
SUBSURFACE PROFILES
E-E

FIGURE 2A-3 (SHEET 6 OF 7)



SECTION G-G

CORRELATION OF PENETRATION RESISTANCE
WITH
RELATIVE DENSITY AND CONSISTENCY

	NUMBER OF BLOWS, N	RELATIVE DENSITY
	0 - 4	VERY LOOSE
SANDS	4 - 10	LOOSE
	10 - 30	FIRM
	30 - 50	DENSE
	OVER 50	VERY DENSE
		CONSISTENCY
	0 - 2	VERY SOFT
	2 - 4	SOFT
SILTS AND CLAYS	4 - 8	FIRM
	8 - 15	STIFF
	15 - 30	VERY STIFF
	30 - 50	HARD
	OVER 50	VERY HARD

SOIL LITHOLOGIES

- A Gray green silty CLAY with fine sand seams
- B Gray tan and white slightly clayey fine SAND with clay inclusions and layers
- C Gray green slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- CC Gray green slightly sandy plastic CLAY
- D Light gray slightly clayey fine and very fine SAND with partially cemented clay inclusions
- DD Light green gray silty slightly clayey fine and very fine SAND with few clay inclusions
- E Gray green clayey partially cemented SAND and CLAY
- EE Gray very clayey very fine SAND
- F Green gray clayey very fine and fine SAND with clay inclusions
- FF Gray slightly clayey fine SAND
- G Gray green slightly fine sandy plastic CLAY with thin fine sand seams
- H Green gray slightly clayey fine and very fine SAND with plastic clay layers and inclusions
- HH Green and gray clayey fine to medium SAND with clay inclusions
- J Gray clayey fine to medium SAND with clay inclusions, partially cemented
- K Light gray clayey very fine SAND with few clay inclusions
- L Gray clayey fine to coarse SAND
- M Gray partially cemented fine sandy silty CLAY
- N Gray and tan fine to medium sandy CLAY
- P Gray very silty fine SAND with partially cemented clay inclusions
- R Gray and brown silty fine to medium SAND with clay inclusions

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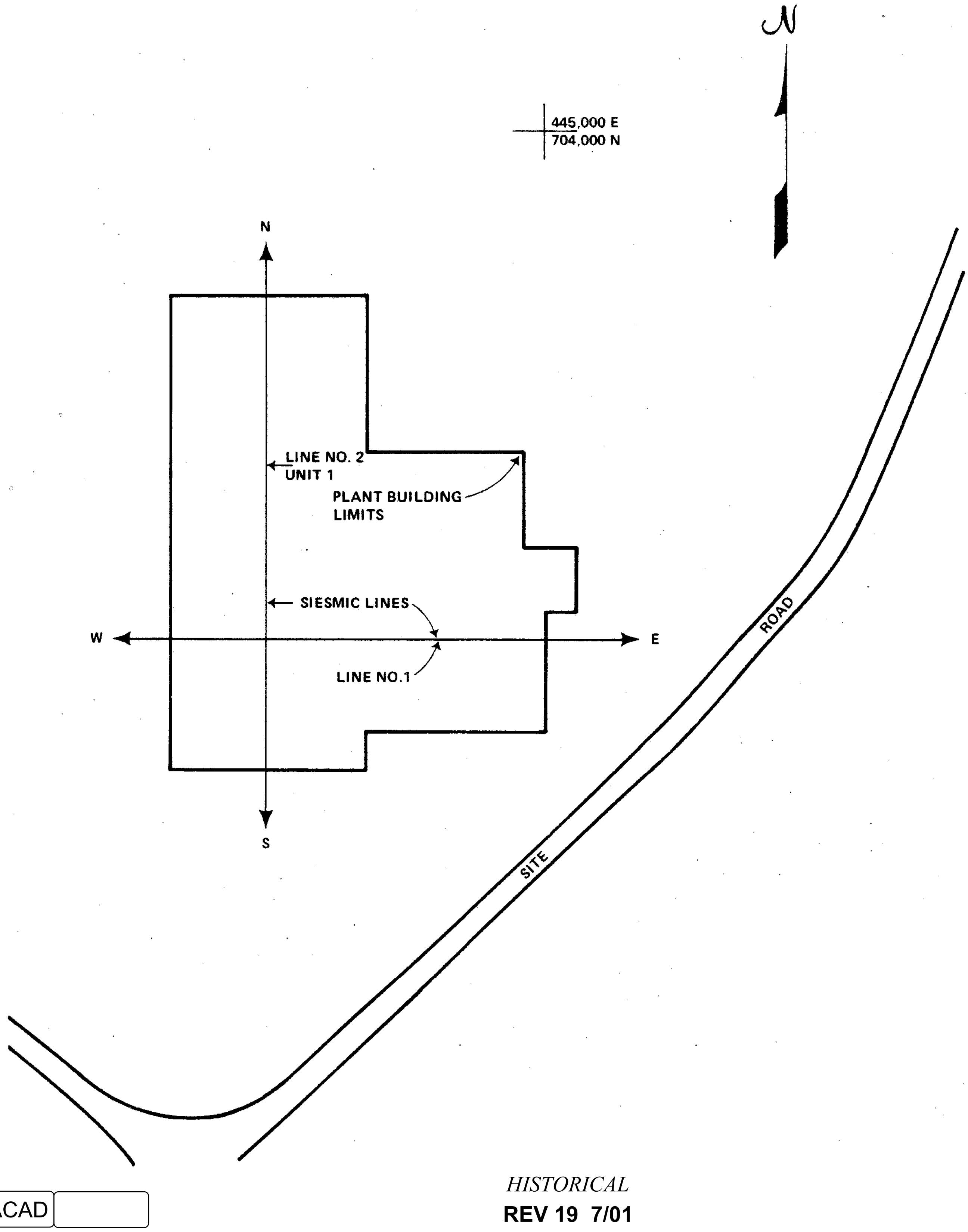
SUBSURFACE PROFILES
G-G

FIGURE 2A-3 (SHEET 7 OF 7)



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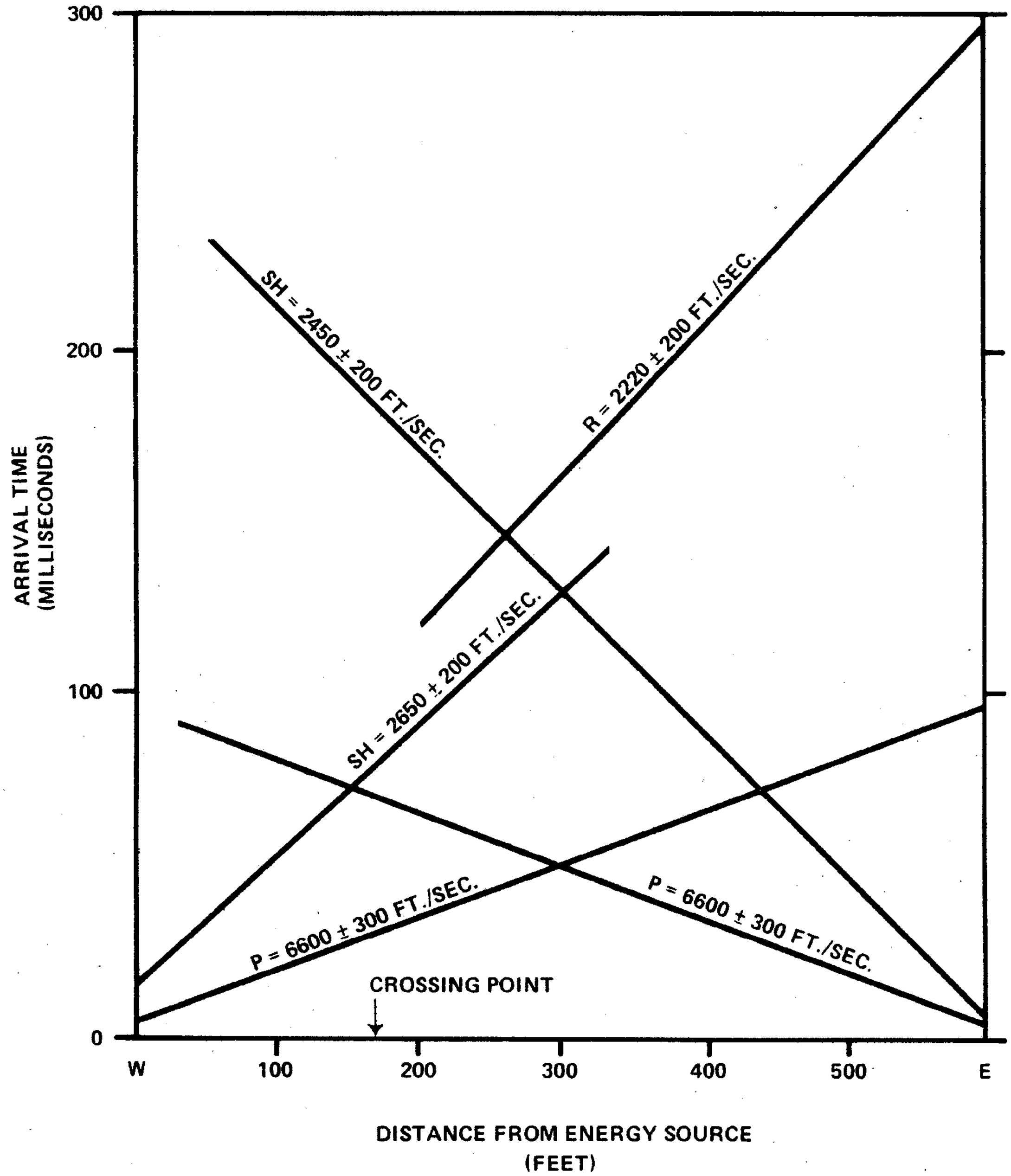
SEISMIC TRAVERSE LINES

FIGURE 2A-4



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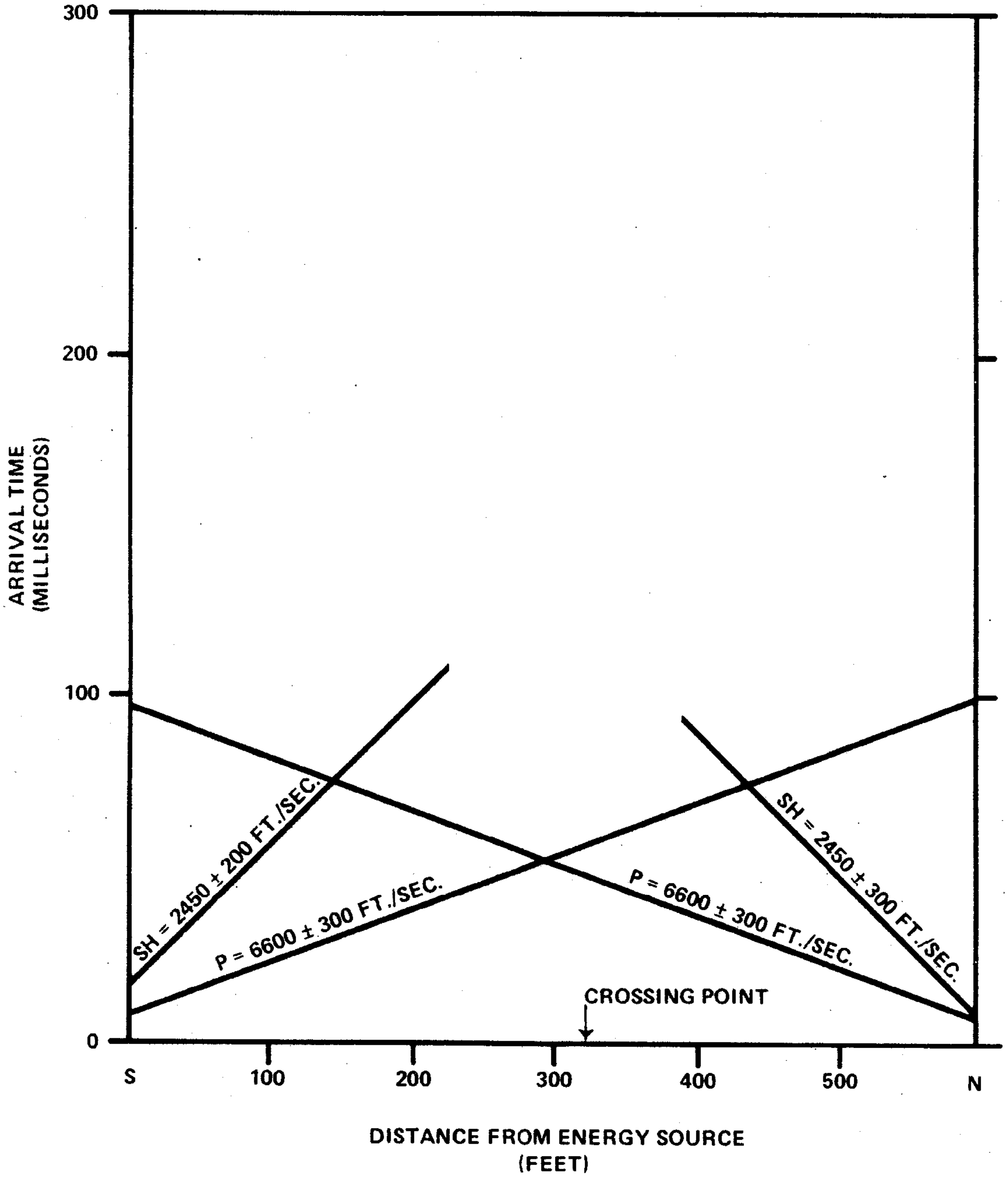
REFRACTION SURVEY
LINE 1

FIGURE 2A-5



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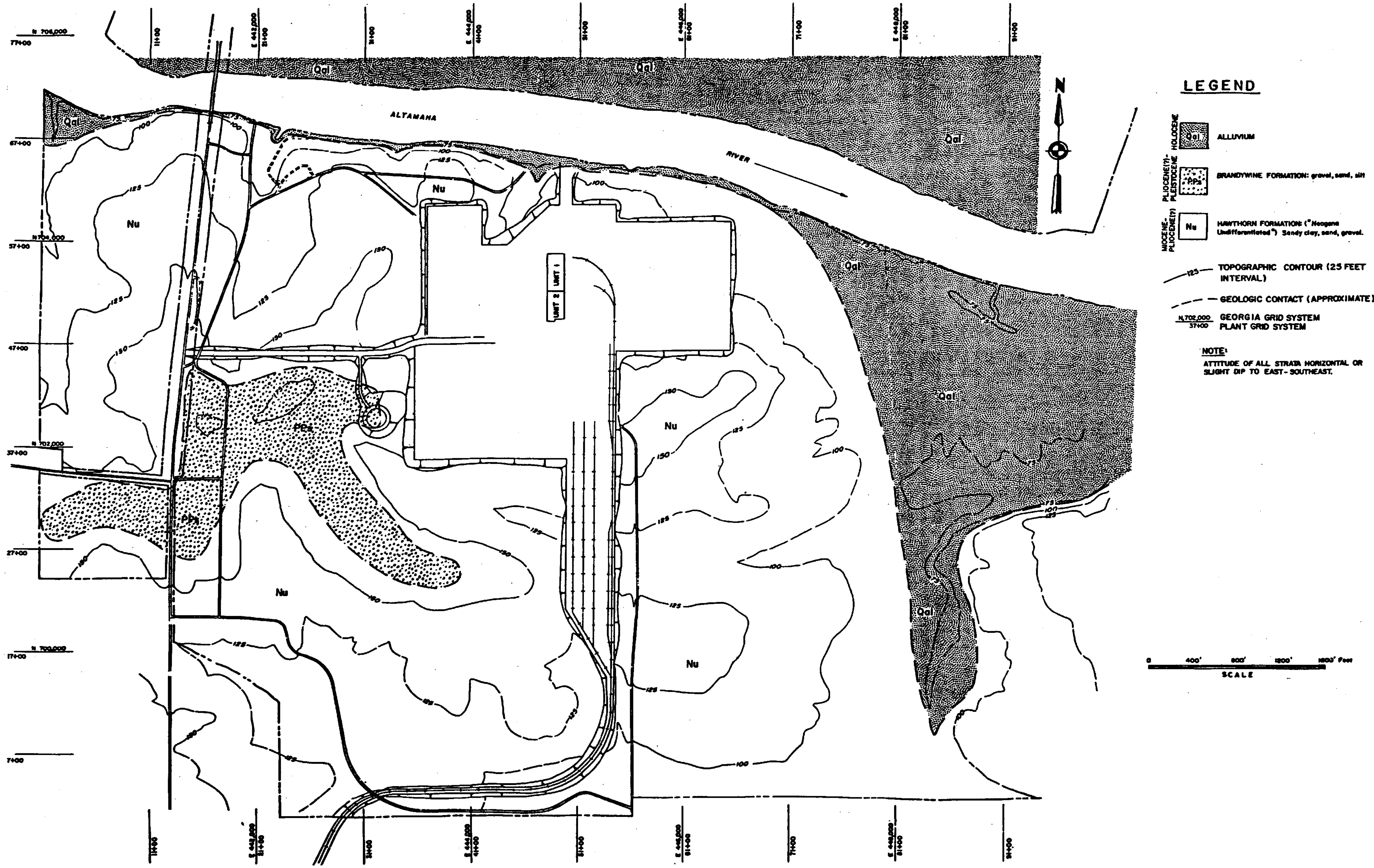
REFRACTION SURVEY
LINE 2

FIGURE 2A-6



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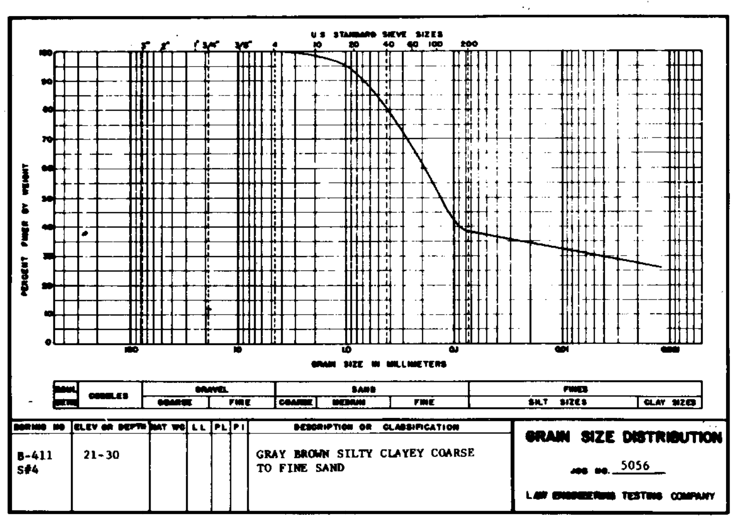
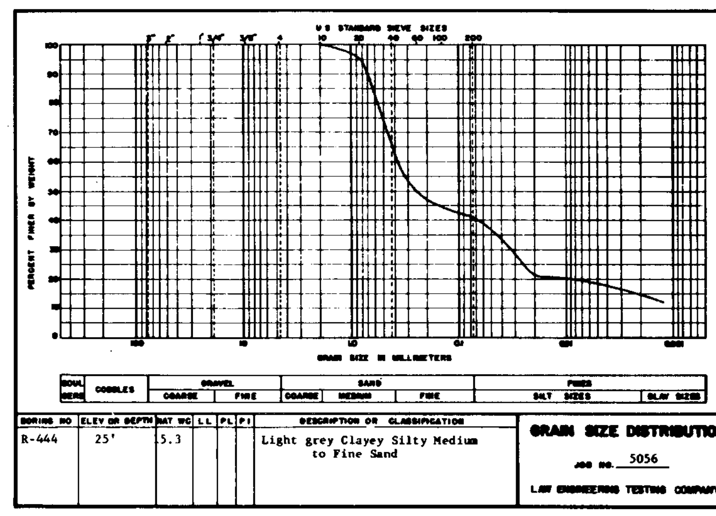
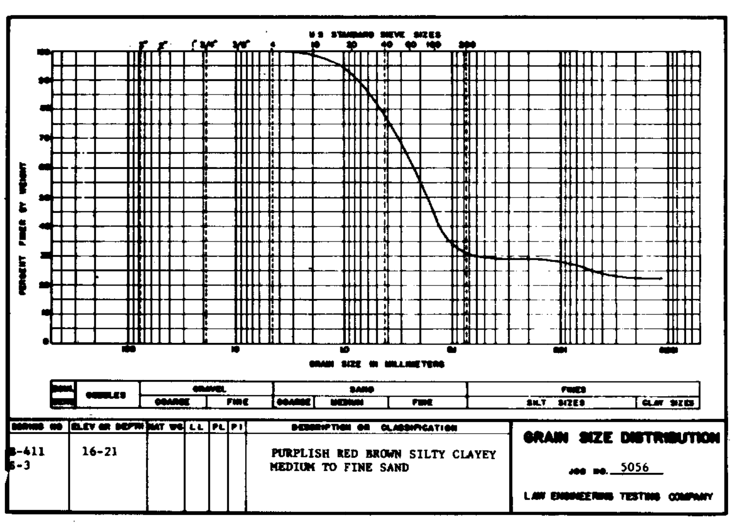
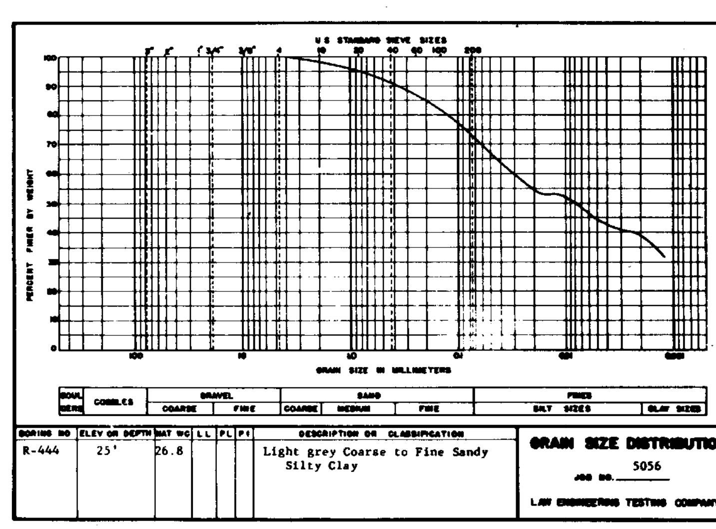
SURFACE DISTRIBUTION
OF GEOLOGIC FORMATIONS

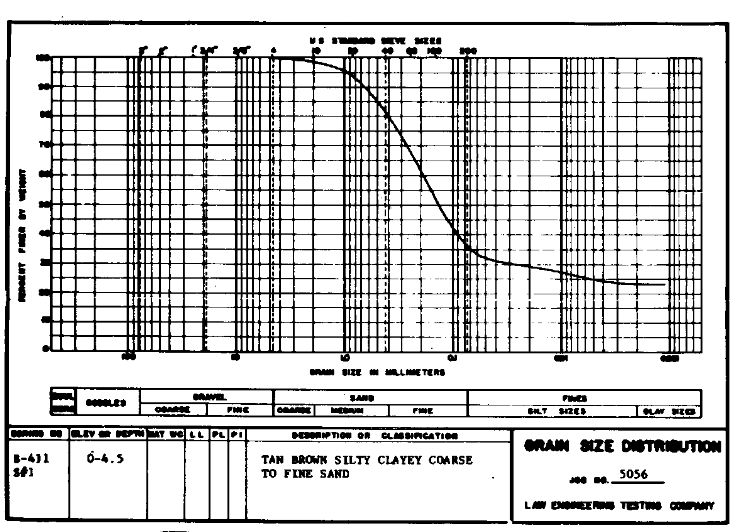
FIGURE 2A-7

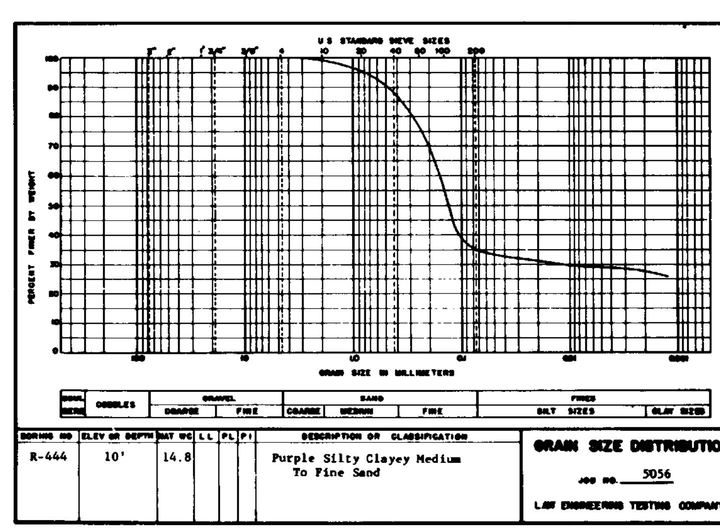












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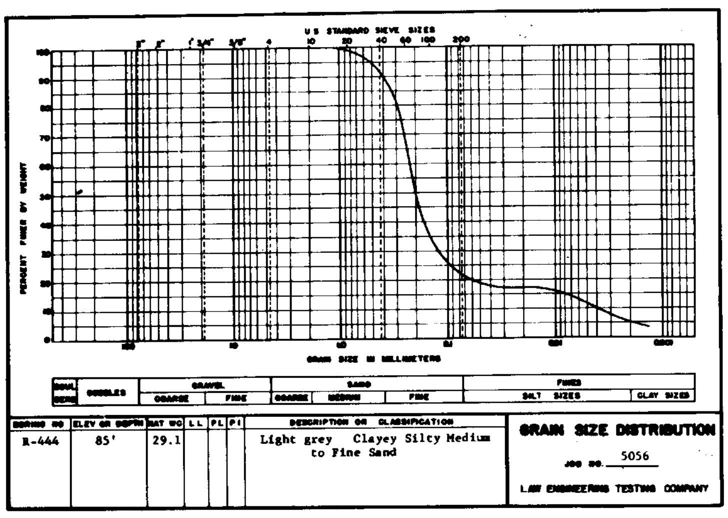
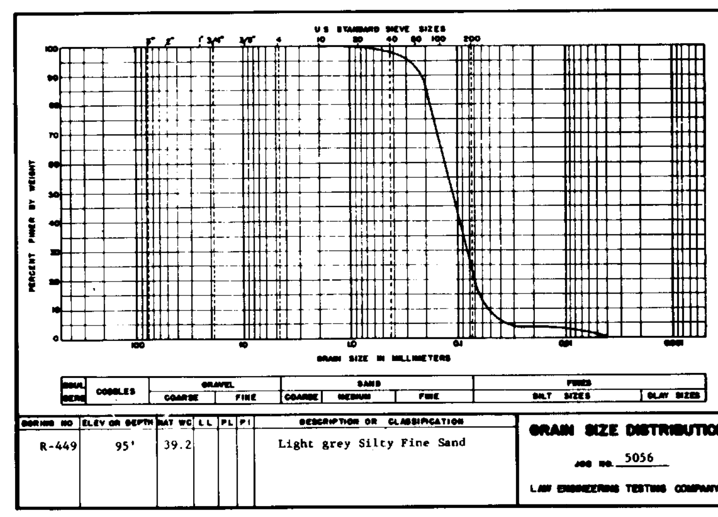
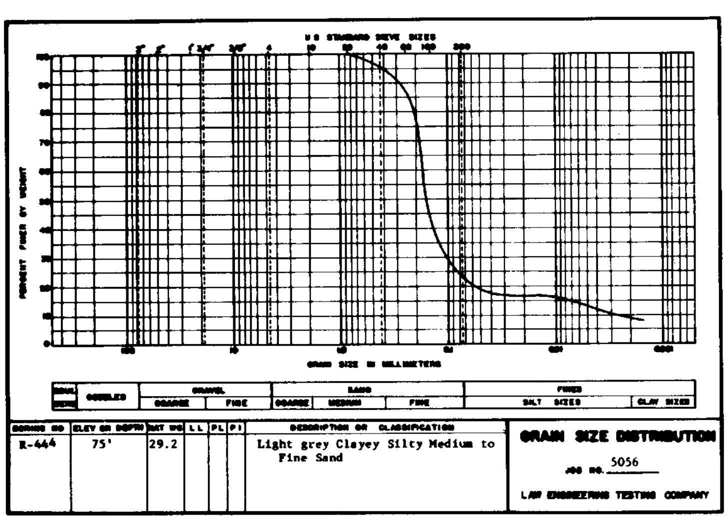
UNIT 2

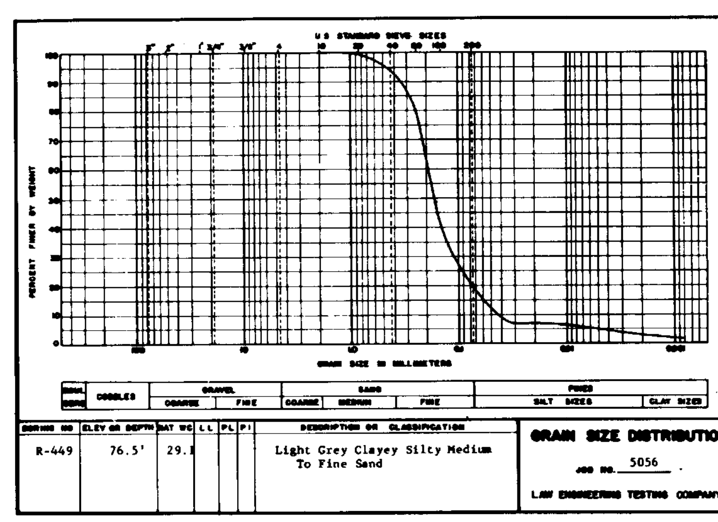
GRAIN SIZE DISTRIBUTION

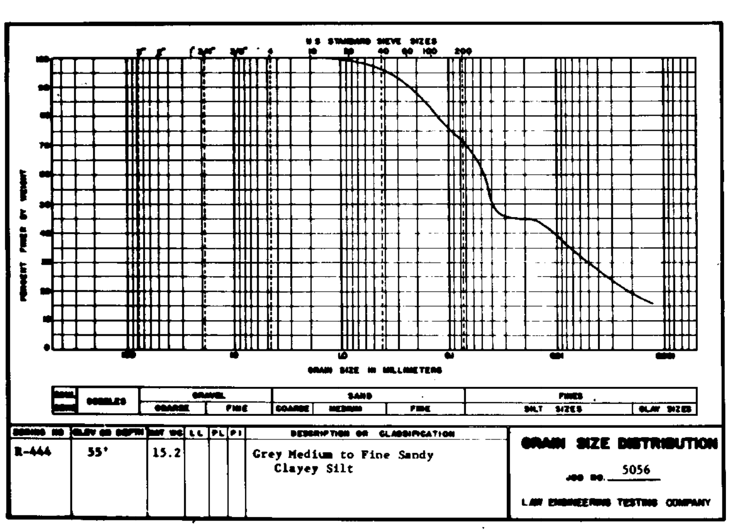
FIGURE 2A-8 (SHEET 1 OF 34)

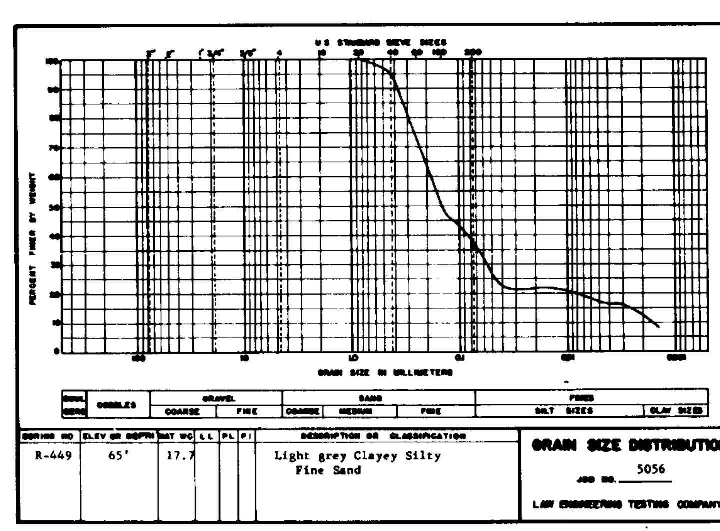












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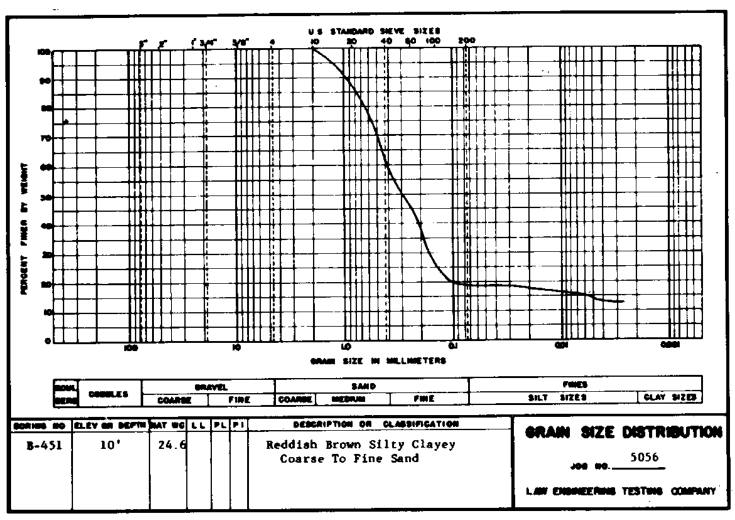
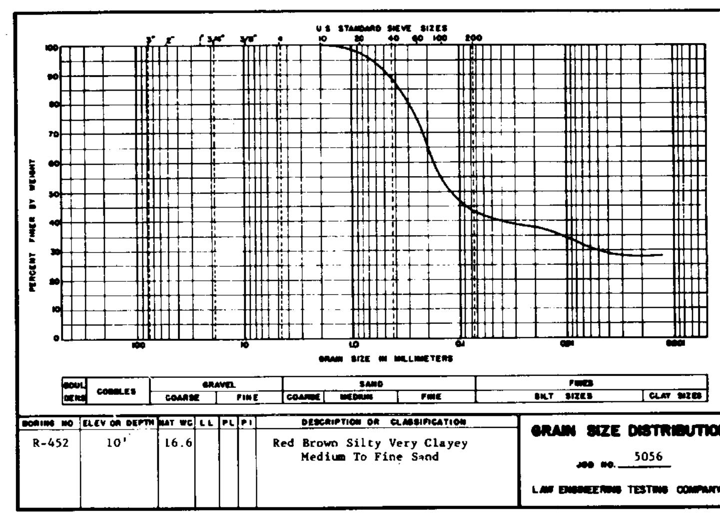
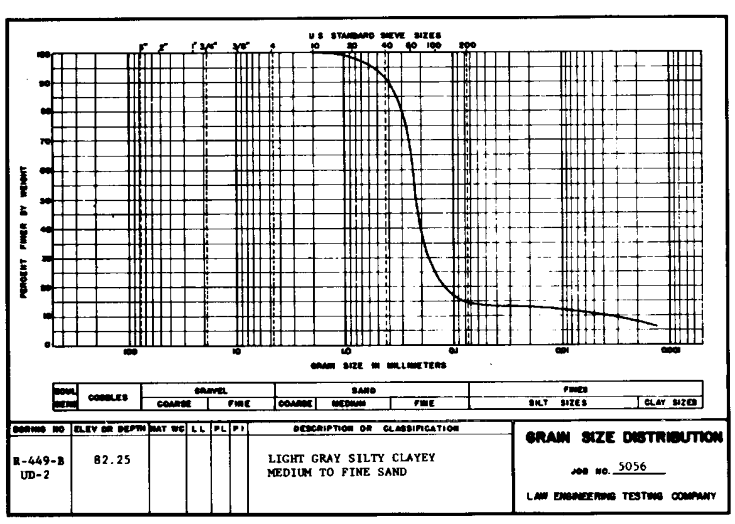
UNIT 2

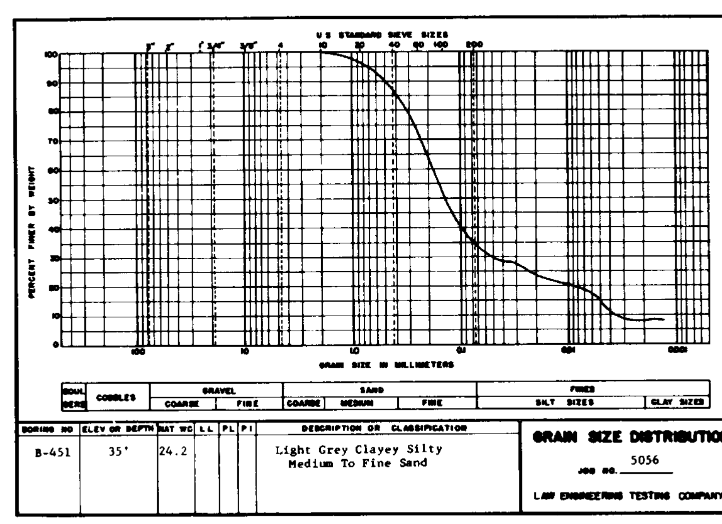
GRAIN SIZE DISTRIBUTION

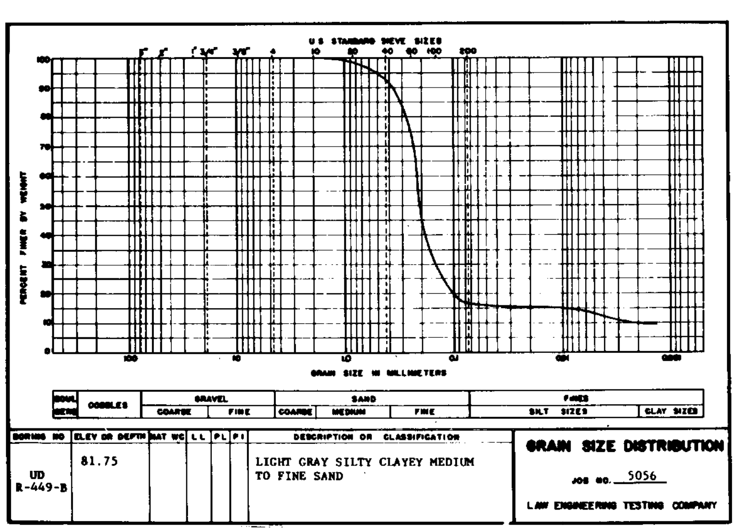
FIGURE 2A-8 (SHEET 2 OF 34)

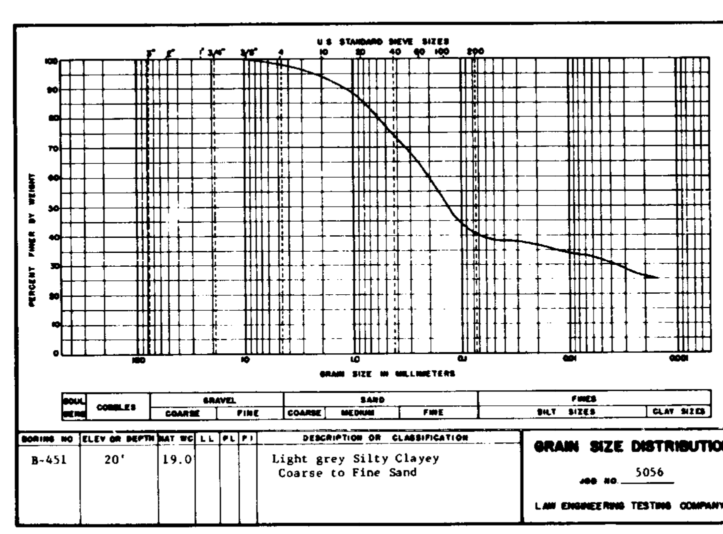












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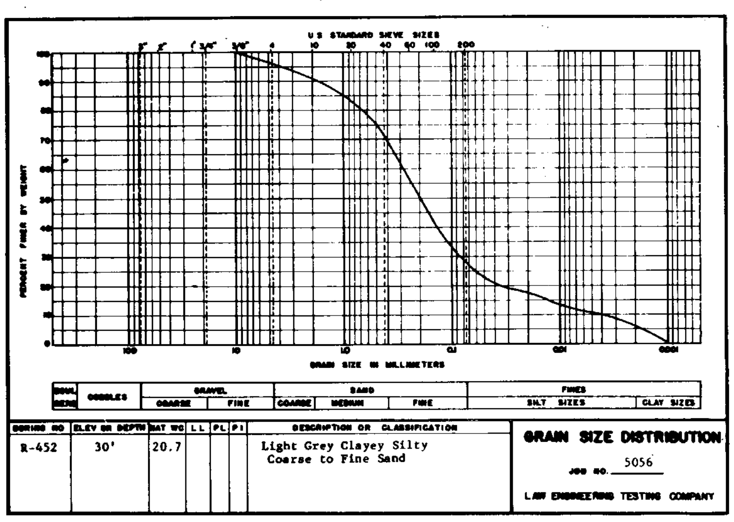
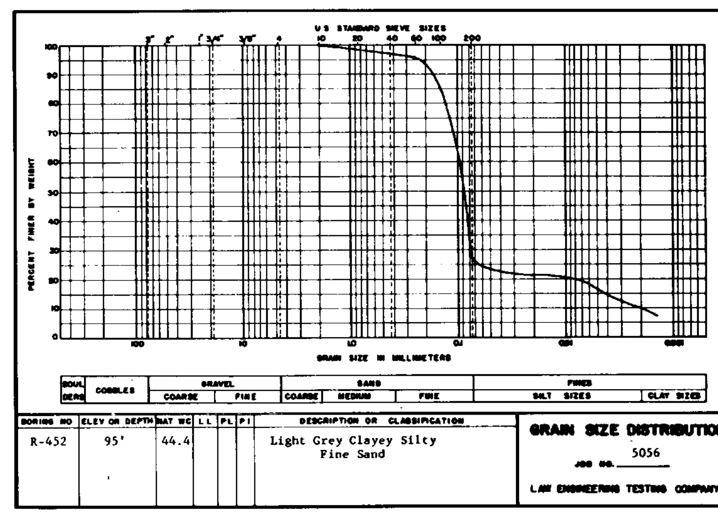
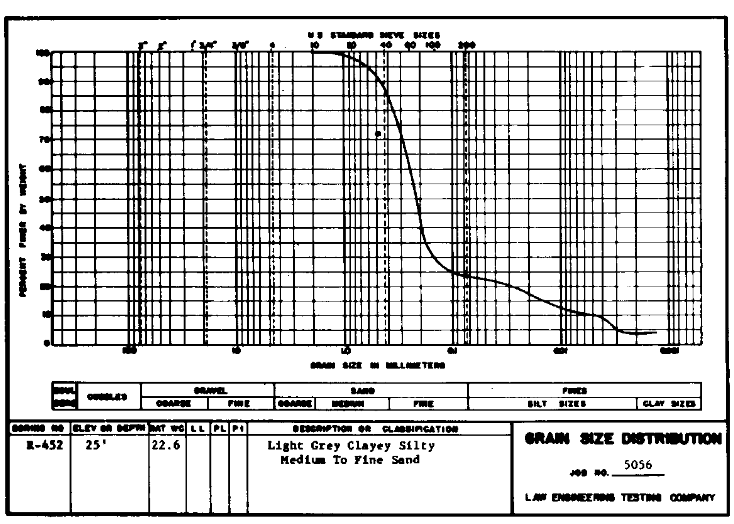
UNIT 2

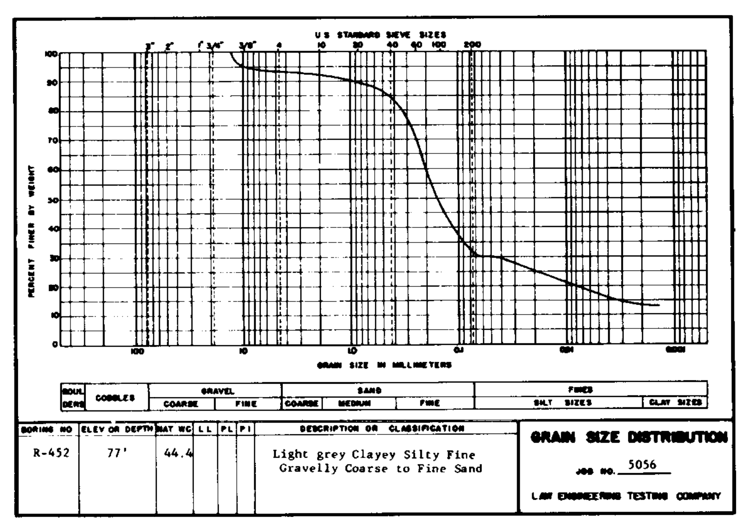
GRAIN SIZE DISTRIBUTION

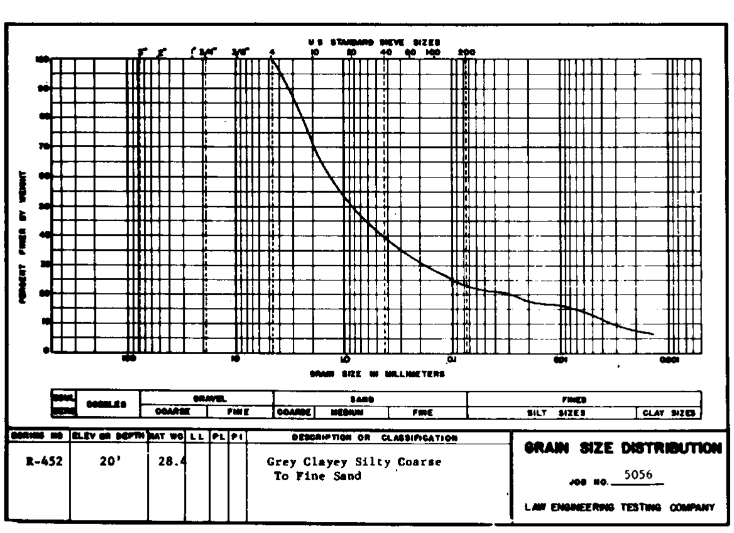
FIGURE 2A-8 (SHEET 3 OF 34)

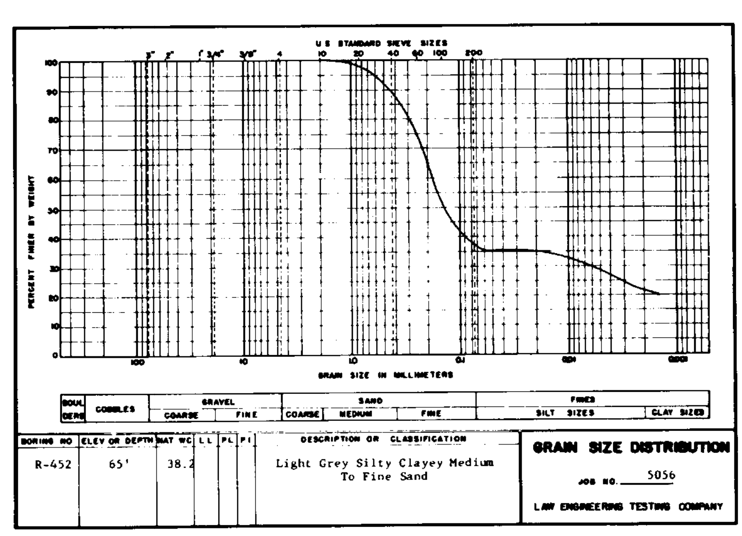












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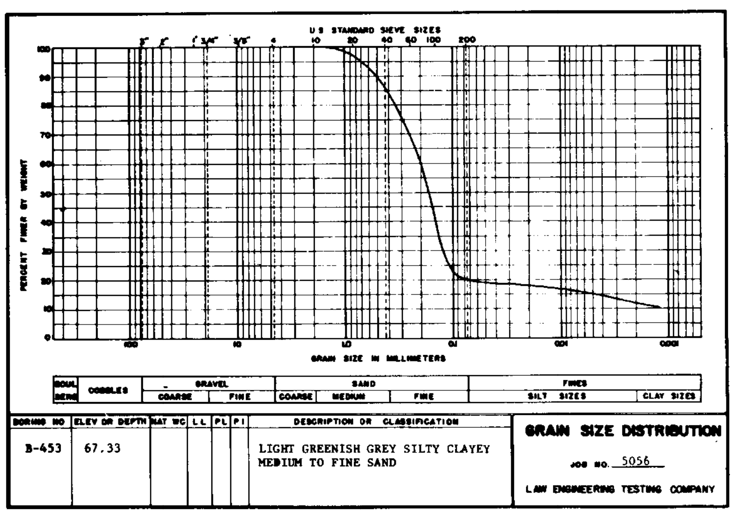
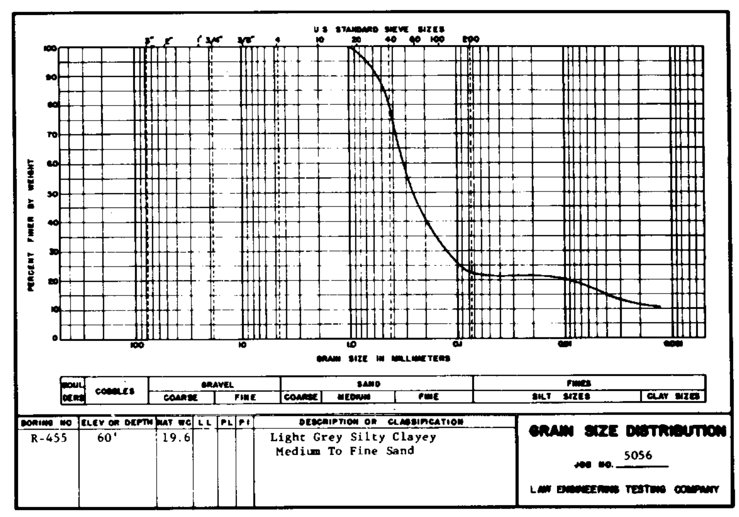
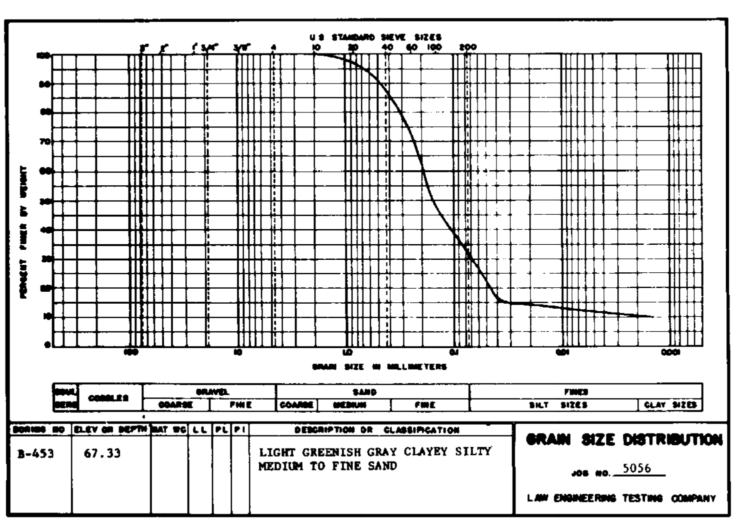
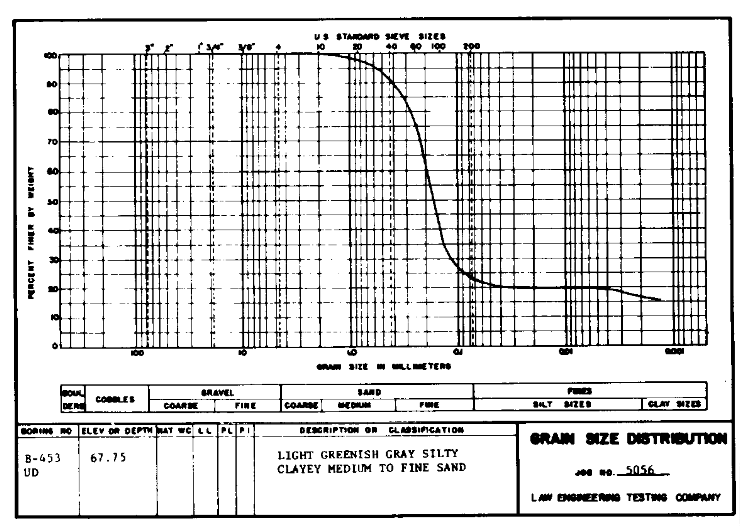
GRAIN SIZE DISTRIBUTION

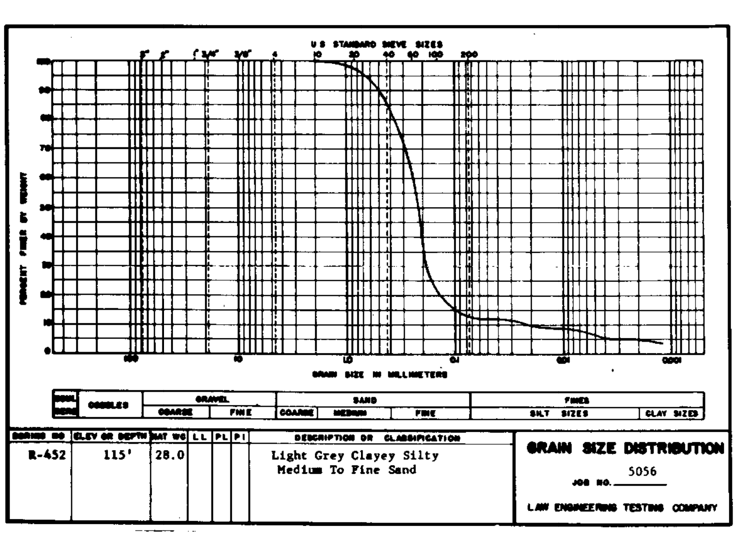
FIGURE 2A-8 (SHEET 4 OF 34)

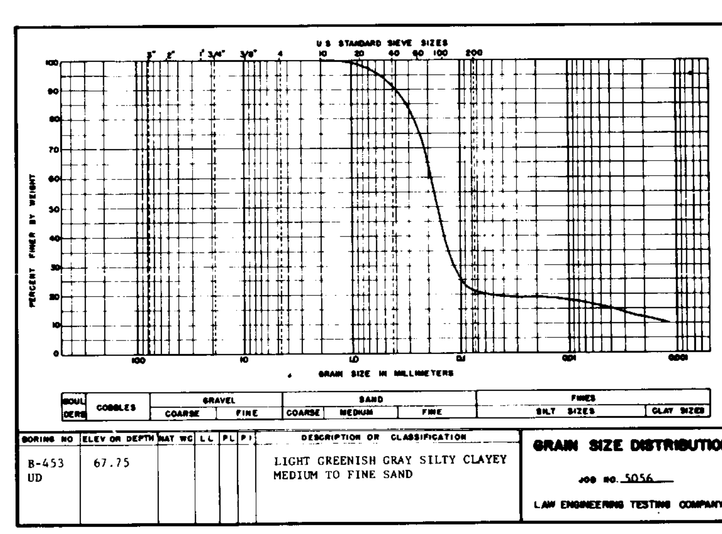












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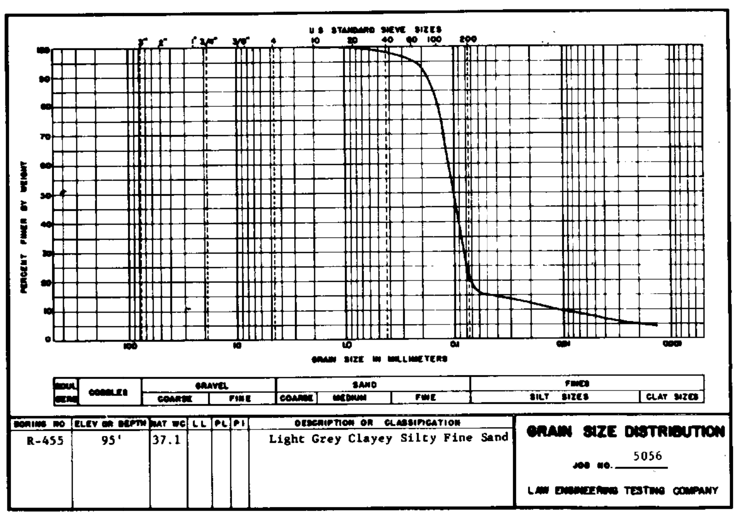
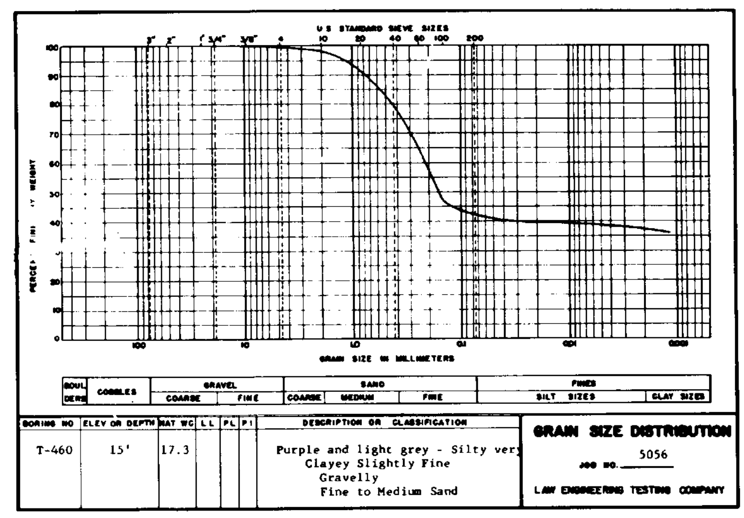
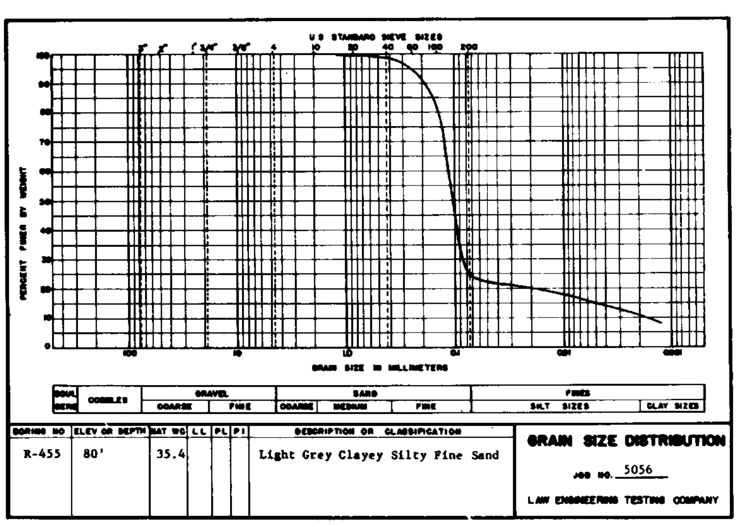
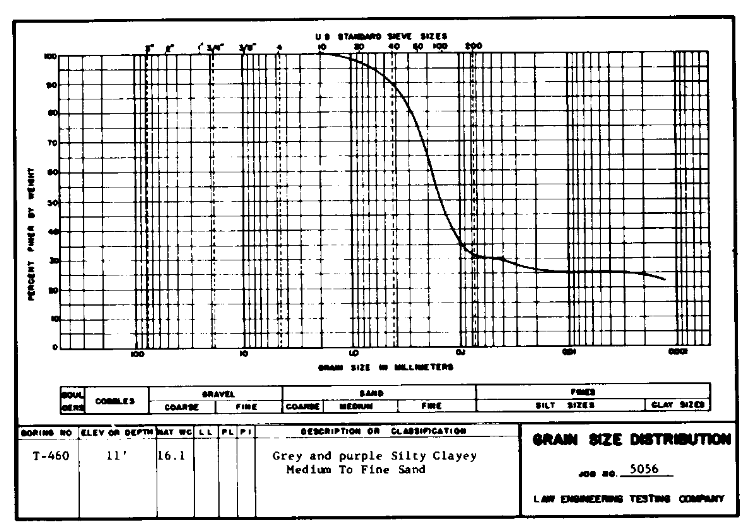
GRAIN SIZE DISTRIBUTION

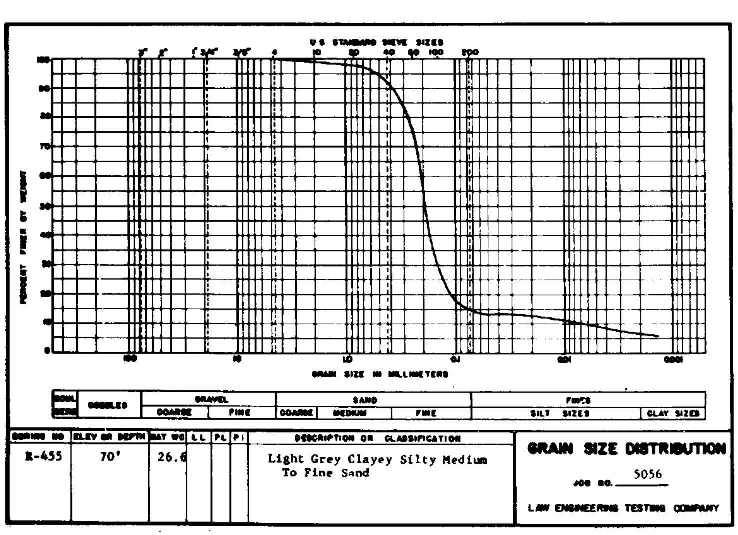
FIGURE 2A-8 (SHEET 5 OF 34)

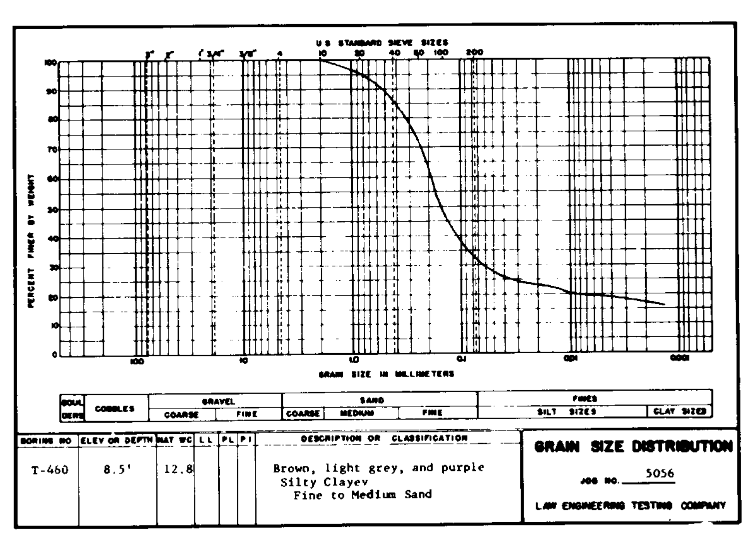












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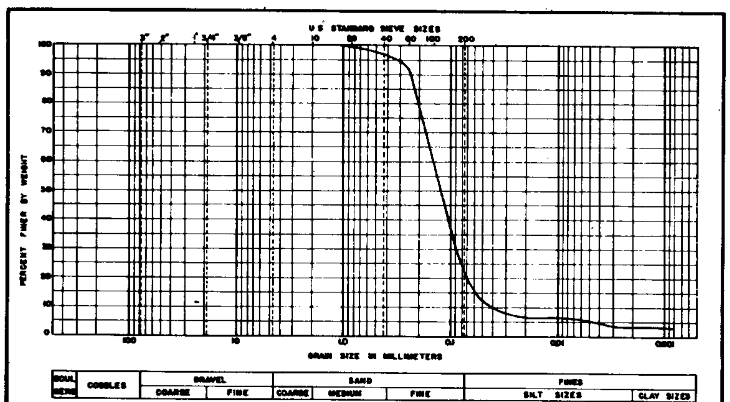
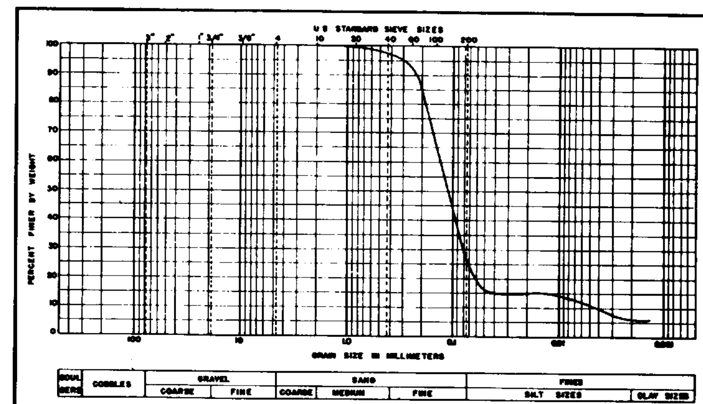
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 6 OF 34)



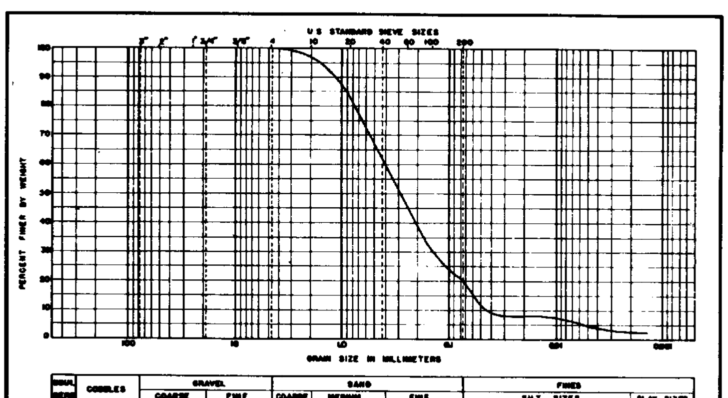
BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
R-465	55'	8.4				Light grey Clayey Silty Fine to Medium Sand

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



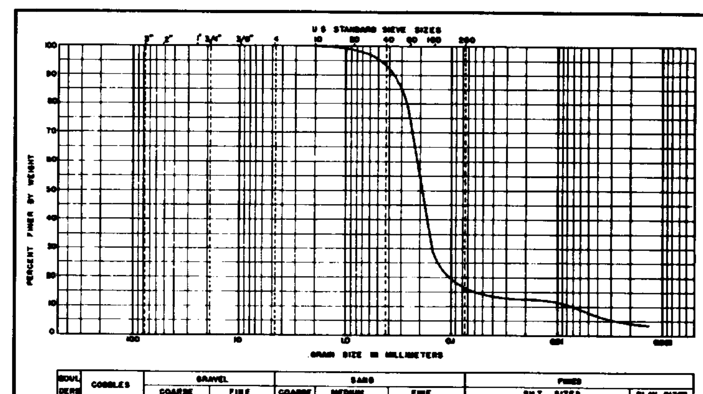
BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
R-465	95'	44.7				Light grey Clayey Silty Medium To Fine Sand

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



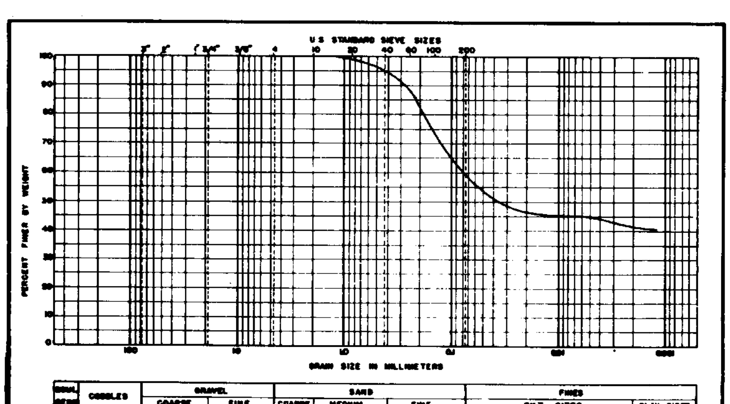
BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
R-465	35'	14.5				Light grey Clayey Silty Fine to Medium Sand

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



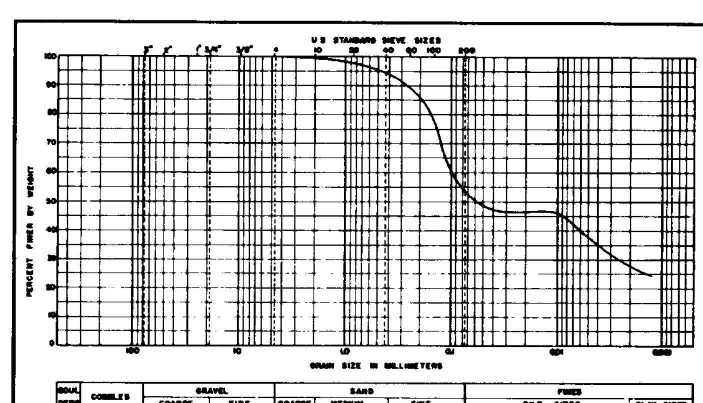
BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
R-465	80'	25.7				Light grey Clayey Silty Medium To Fine Sand

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
R-465	16'	22.9				Grey Fine To Medium Sandy Silty Clay

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
R-465	70'	45.1				Light grey Clayey Silty Medium To Fine Sand

GRAIN SIZE DISTRIBUTION
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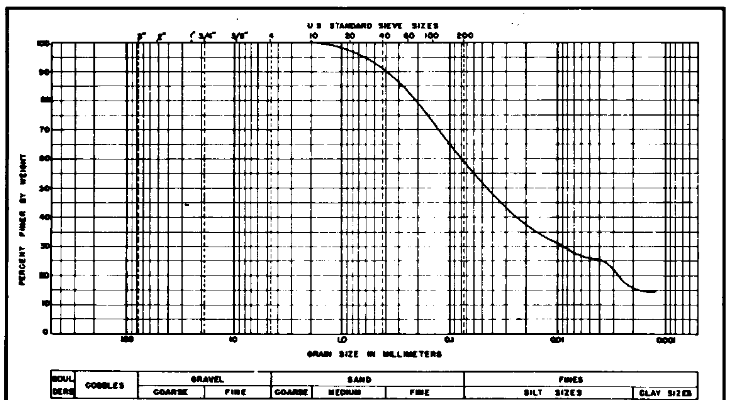
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 7 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
T-464	50'	18.1				Light grey Fine to Medium Sandy Clayey Silt

GRAIN SIZE DISTRIBUTION
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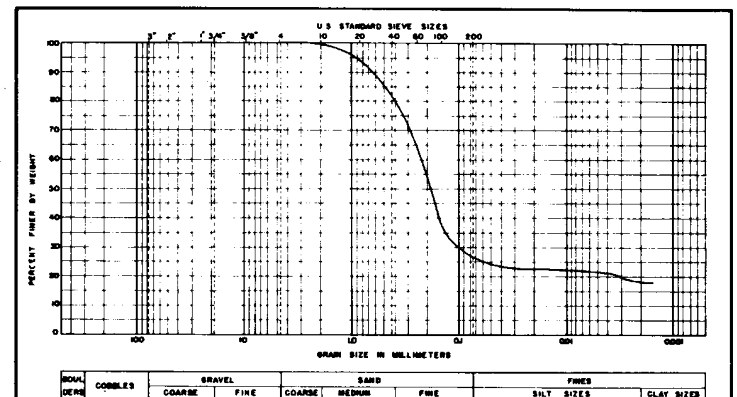
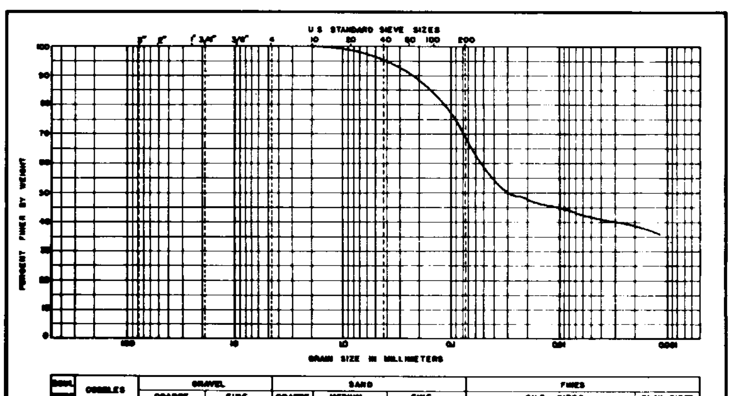


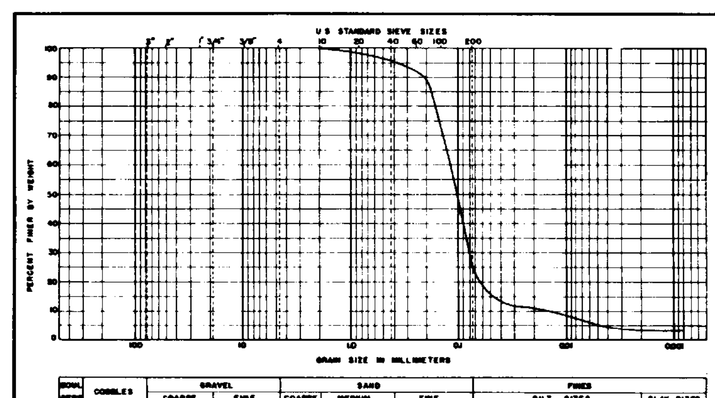
BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
R-465	8.5'	12.4				Light grey, purple, Silty Clayey Medium To Fine Sand

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



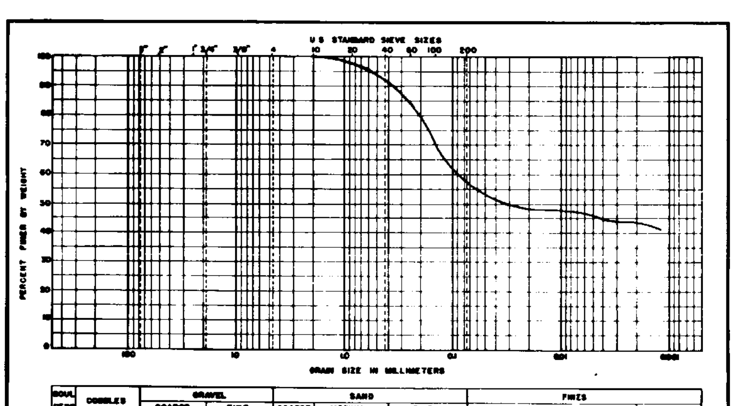
BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
T-464	15'	22.4				Light grey, purple and brown Fine Sandy Silty Clay

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



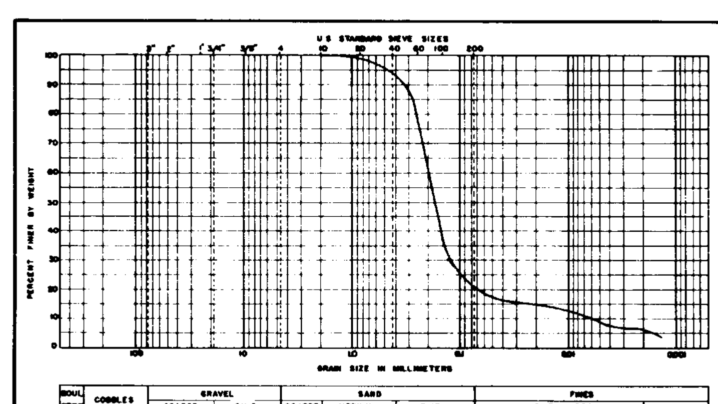
BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
T-464	90'	36.3				Light grey Clayey Silty Fine Sand

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
T-464	6'	17.6				Light grey, purple, and brown Fine to Medium Sandy Silty Clay

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
T-464	80'	24.7				Light Grey Clayey Silty Fine Sand

GRAIN SIZE DISTRIBUTION
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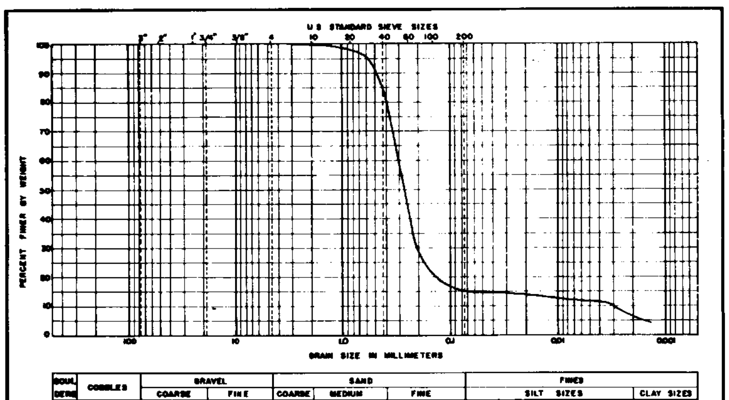
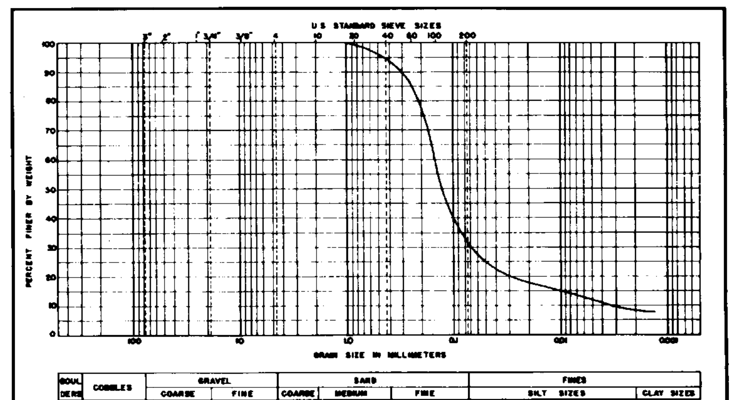
GRAIN SIZE DISTRIBUTION

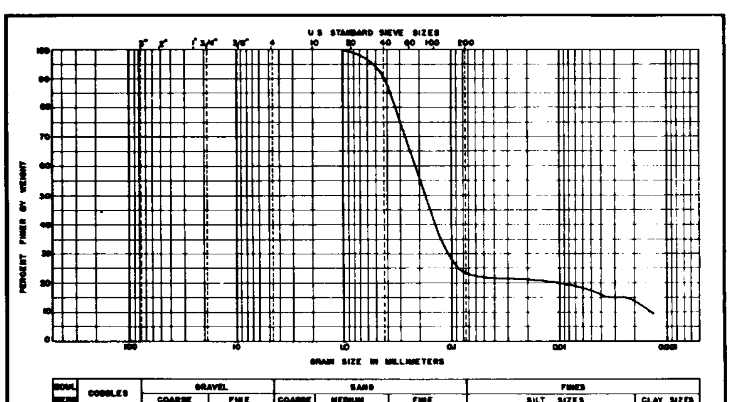
FIGURE 2A-8 (SHEET 8 OF 34)



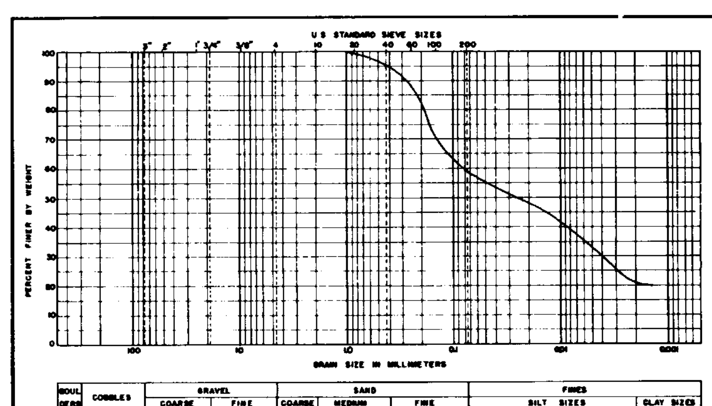
BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
T-460	72.5'					LIGHT GREENISH GRAY SLIGHTLY CLAYEY SLIGHTLY SILTY MEDIUM TO FINE SAND	JOB NO. 5056 LAW ENGINEERING TESTING COMPANY



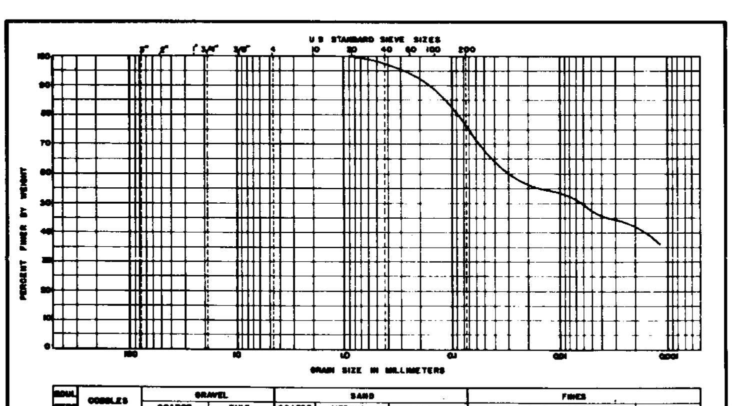
BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
T-460	90'	18.4				Light Grey Clayey Silty Fine Sand	JOB NO. 5056 LAW ENGINEERING TESTING COMPANY



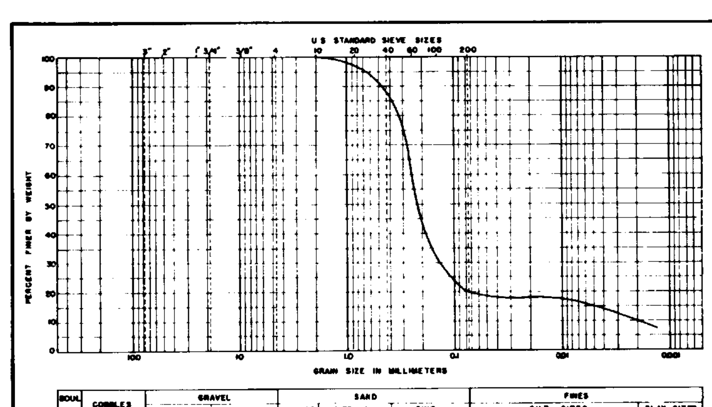
BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
T-460	70'	23.7				Light grey Silty Clayey Medium To Fine Sand	JOB NO. 5056 LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
T-460	80'	48.0				Light Grey Fine Sandy Clayey Silt	JOB NO. 5056 LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
T-460	20'	23.3				Light Grey and Brown Fine Sandy Silty Clay	JOB NO. 5056 LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
UD T-460	73.75					LIGHT GREENISH GRAY SILTY CLAYEY MEDIUM TO FINE SAND	JOB NO. 5056 LAW ENGINEERING TESTING COMPANY

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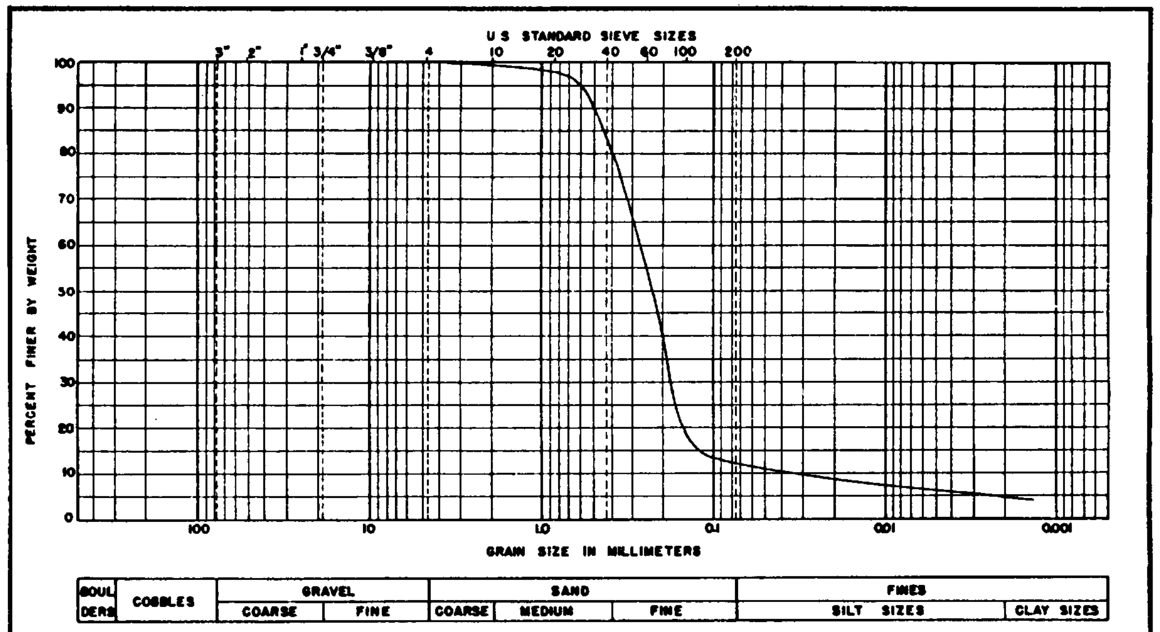
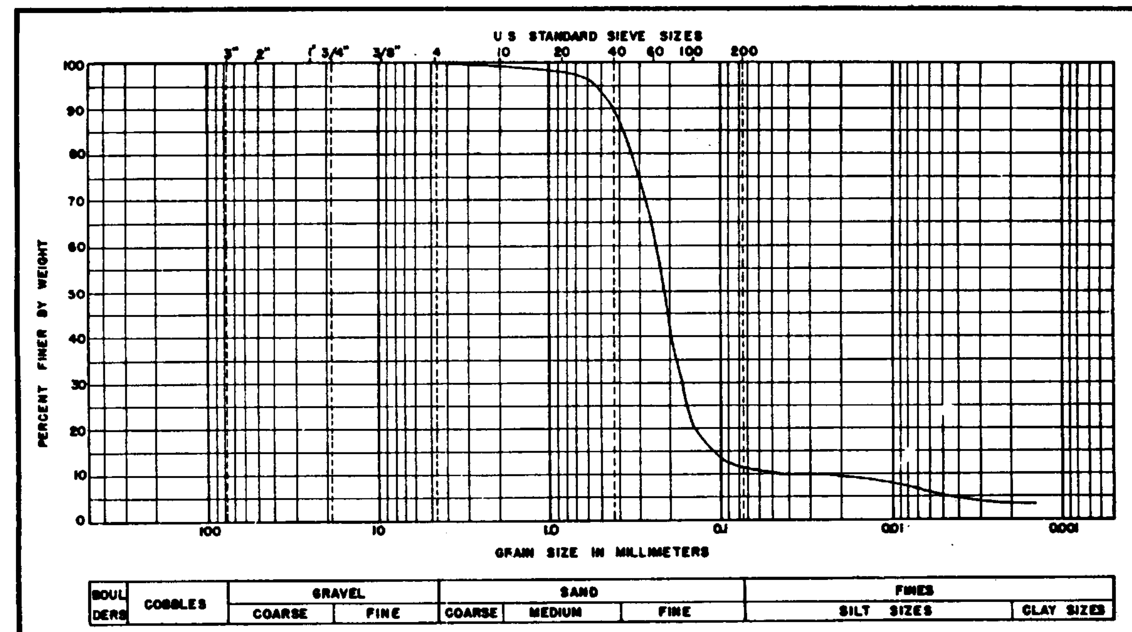
GRAIN SIZE DISTRIBUTION

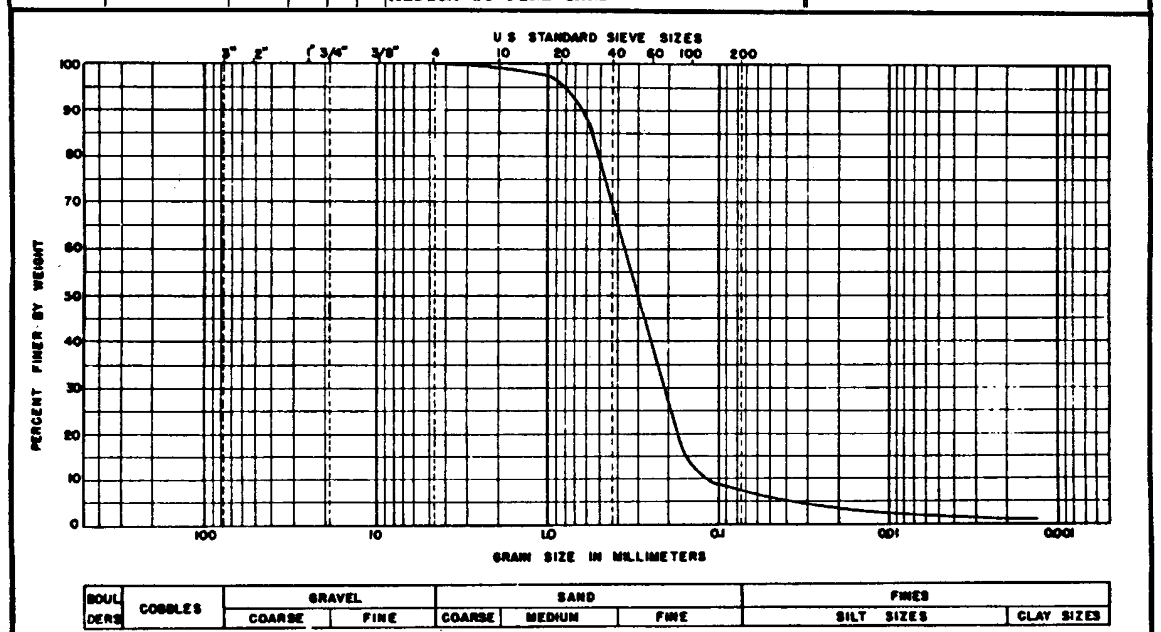
FIGURE 2A-8 (SHEET 9 OF 34)



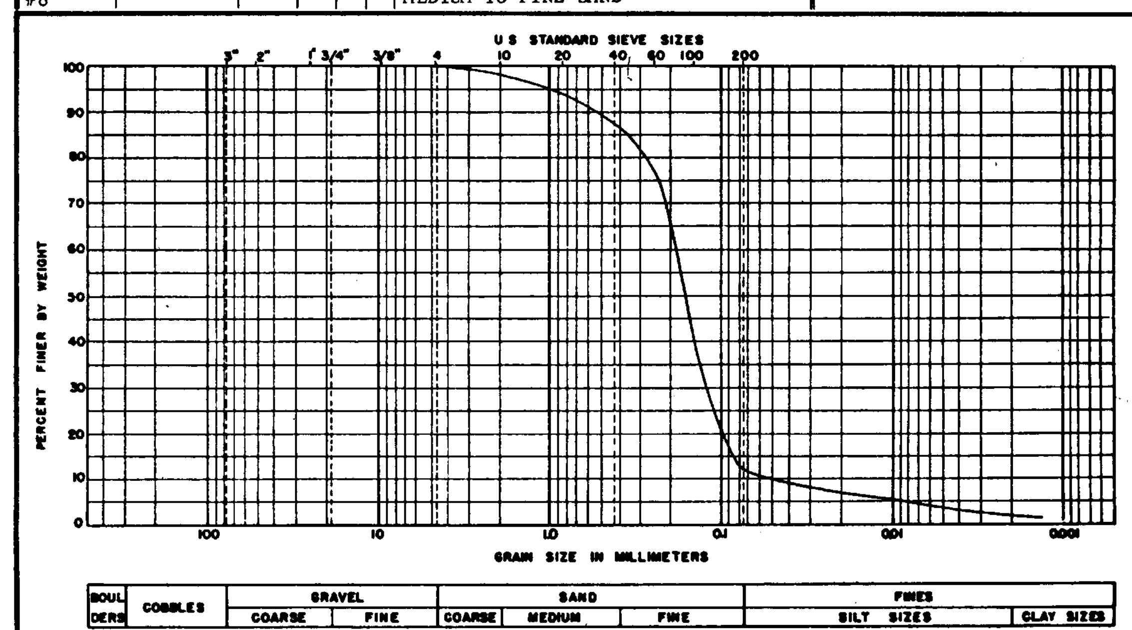
BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
B-578	Elev. 16					LIGHT GRAY BROWN SLIGHTLY CLAYEY MEDIUM TO FINE SAND



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
B-578 #8	Elev. 27					LIGHT GRAY SLIGHTLY CLAYEY MEDIUM TO FINE SAND



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
3579	Elev. 21					LIGHT GRAY MEDIUM TO FINE SAND



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
B-579	Elev. 46					LIGHT GRAY SLIGHTLY SILTY MEDIUM TO FINE SAND

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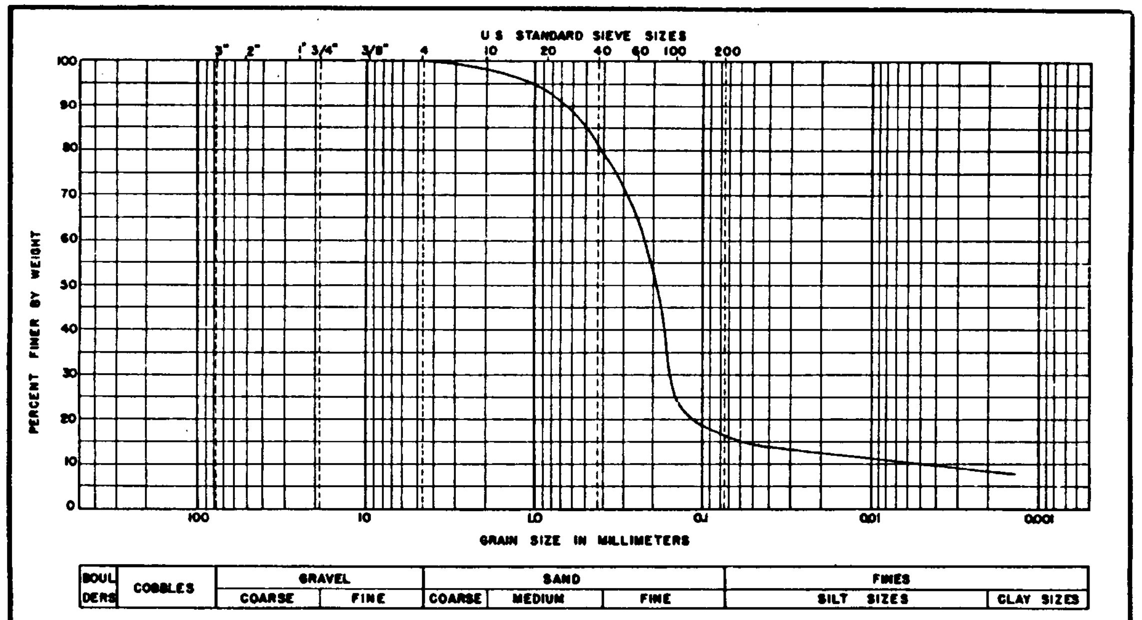
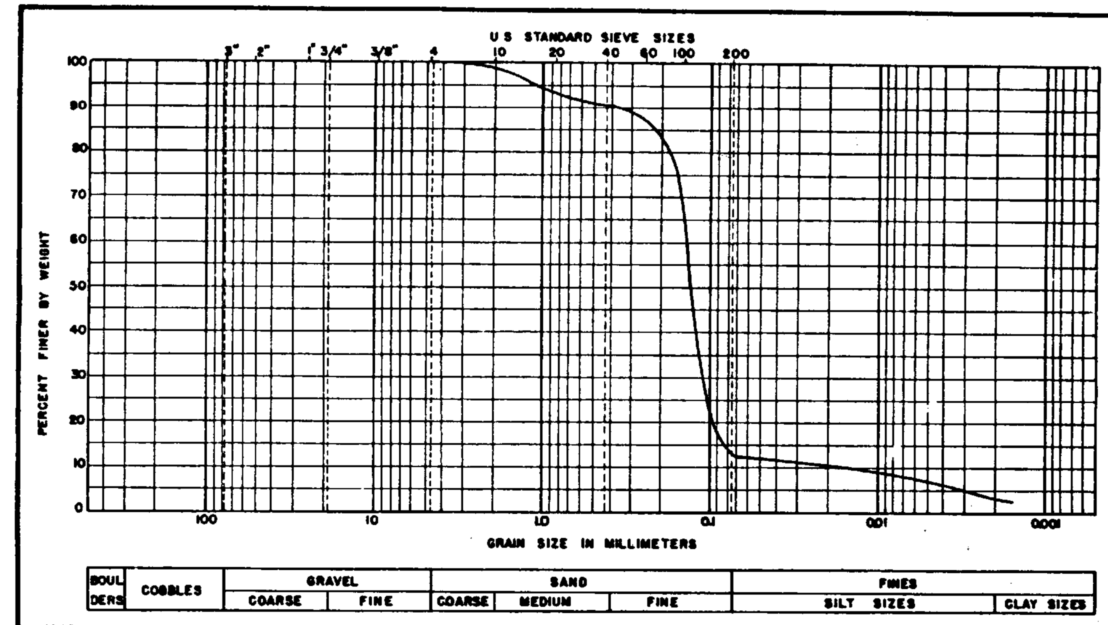
UNIT 2

GRAIN SIZE DISTRIBUTION

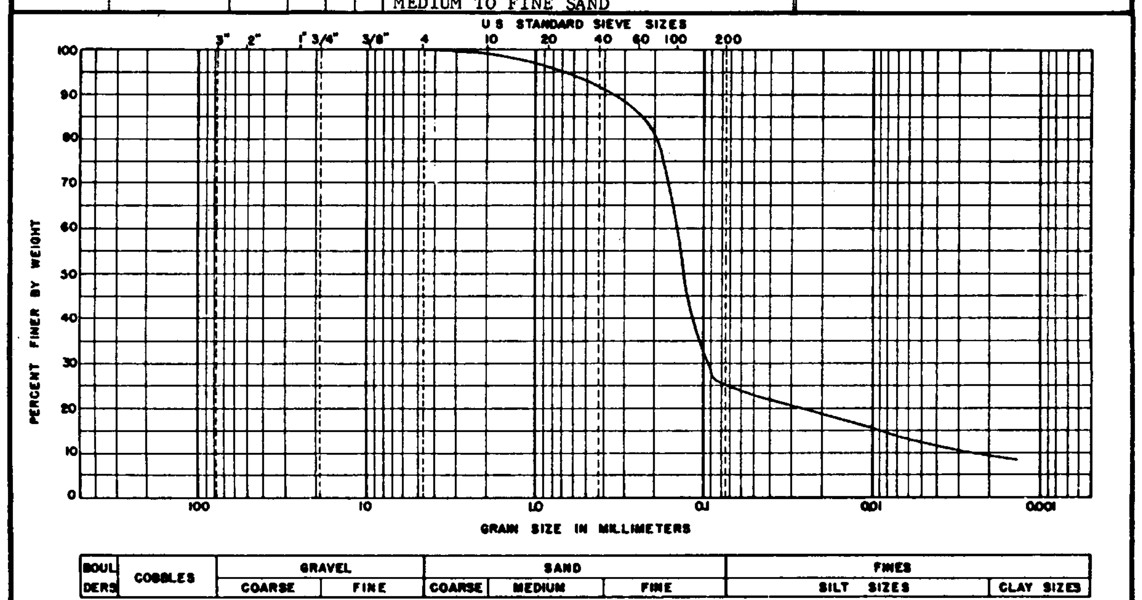
FIGURE 2A-8 (SHEET 10 OF 34)



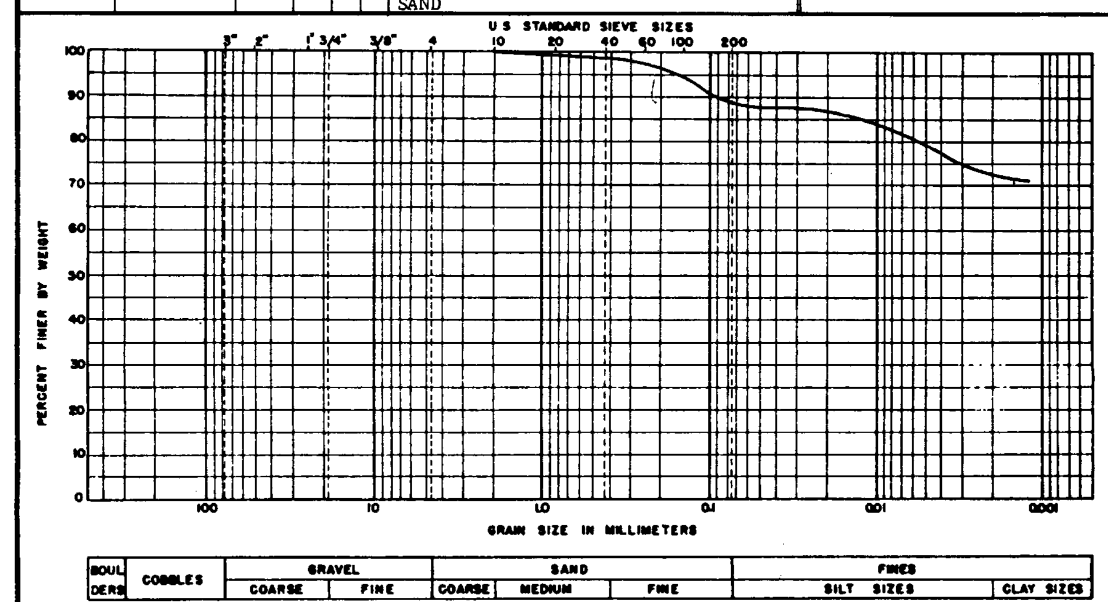
BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
B-580	Elev. 19					LIGHT GRAY BROWN SLIGHTLY CLAYEY MEDIUM TO FINE SAND



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
B-580	Elev. 35					LIGHT GRAY SLIGHTLY CLAYEY FINE SAND



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
3-580	Elev. 49					LIGHT GRAY TO GREEN SILTY CLAYEY FINE SAND.



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
3-580	Elev. 78	63.5	126	47	79	GRAY GREEN PLASTIC CLAY WITH FINE SAND INCLUSIONS

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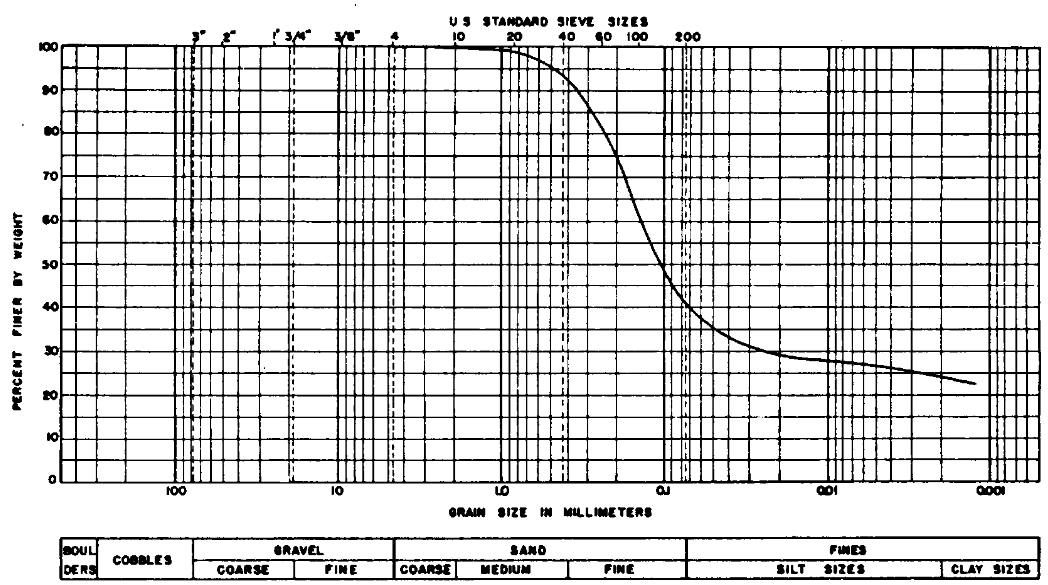
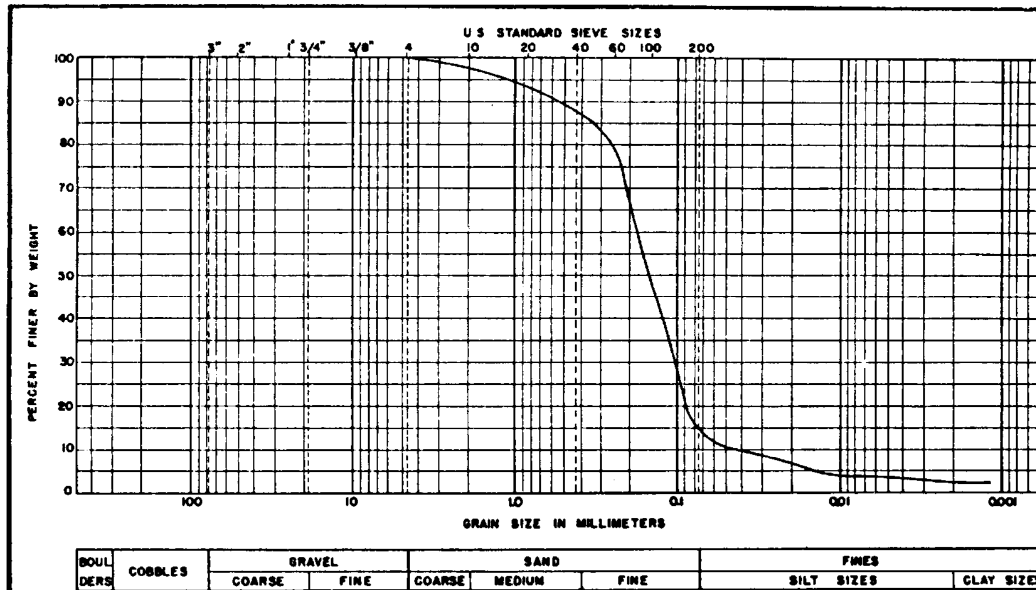
UNIT 2

GRAIN SIZE DISTRIBUTION

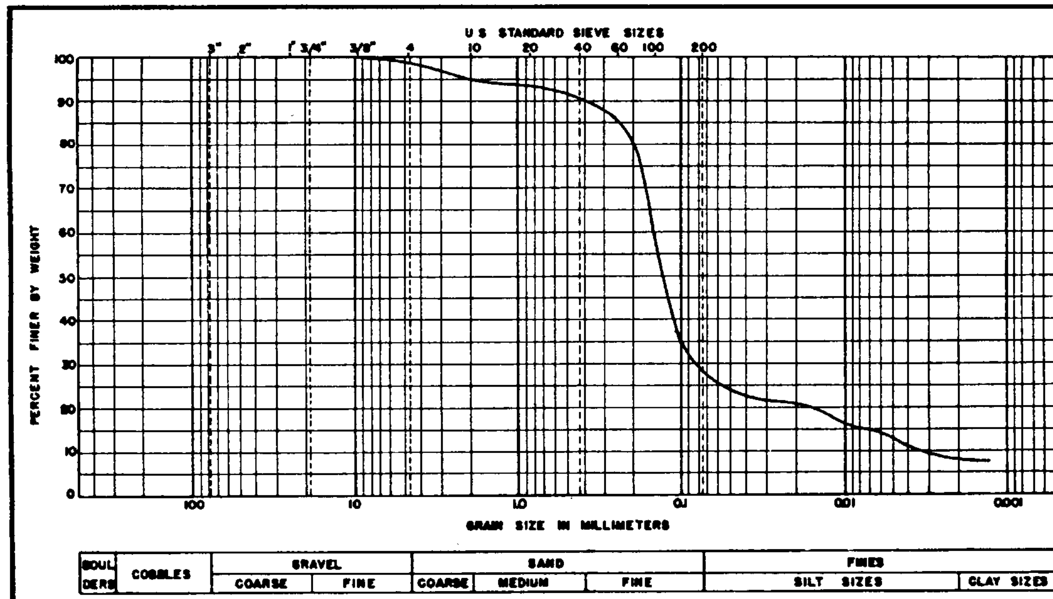
FIGURE 2A-8 (SHEET 11 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION	
B-581	Elev. 101					GRAY BROWN CLAYEY MEDIUM TO FINE SAND	



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION	
B-603	Elev. 43					GRAY SLIGHTLY SILTY SLIGHTLY CLAYEY MEDIUM TO FINE SAND	



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION	
B-603	Elev. 53					GRAY BROWN CLAYEY MEDIUM TO FINE SAND	

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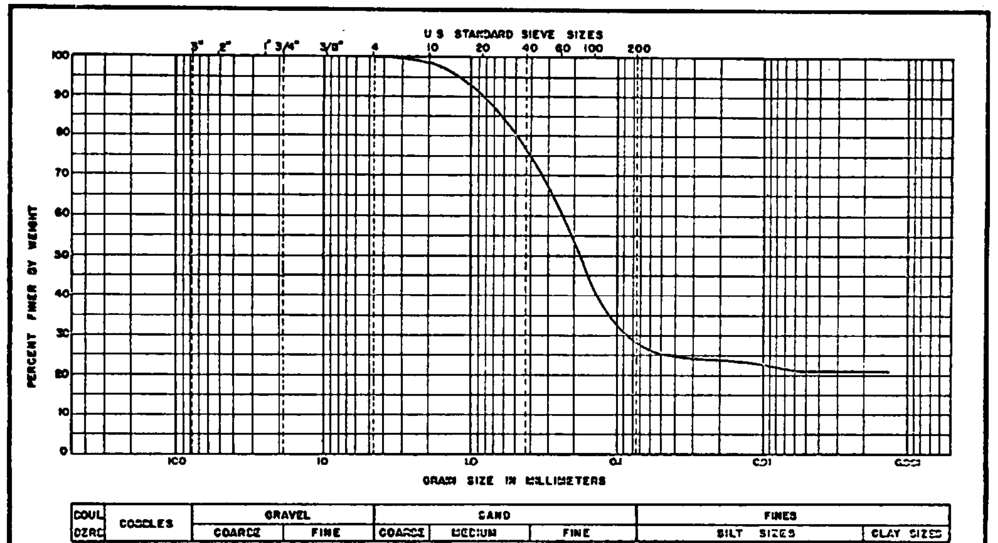
UNIT 2

GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 12 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
R-588 S#1	4.5-6'					Red brown and tan clayey medium to fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

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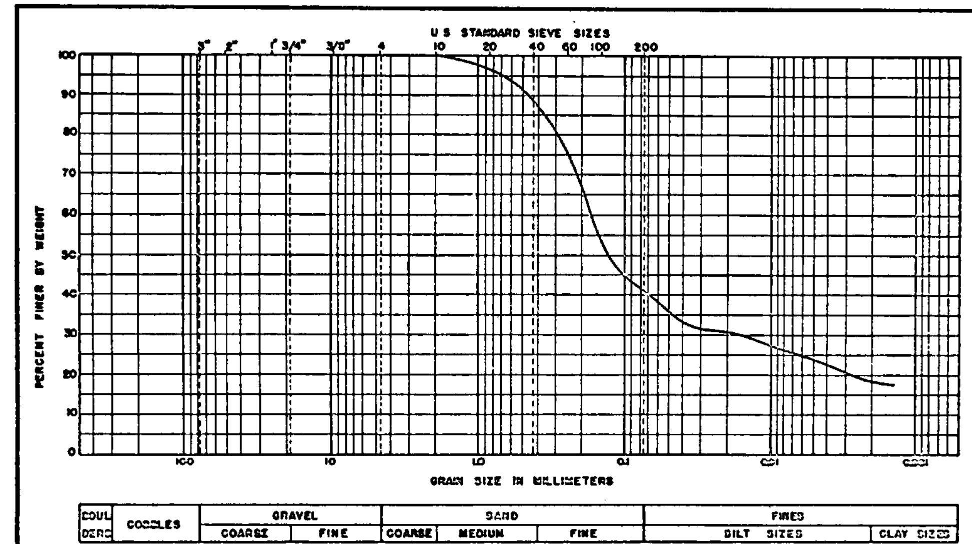
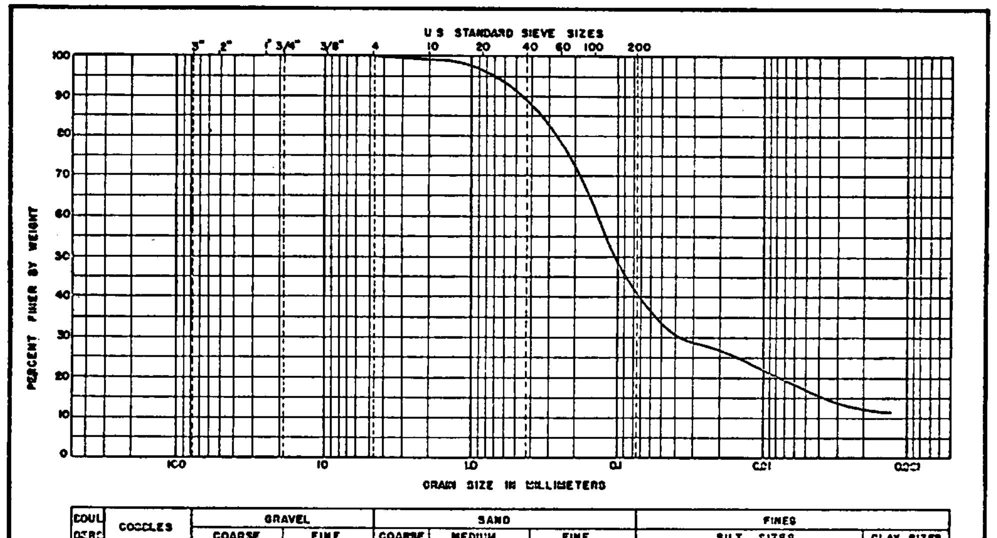


BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
R-588 S#11	54-55.5'					Light grey silty clayey medium to fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

LAW ENGINEERING TESTING COMPANY

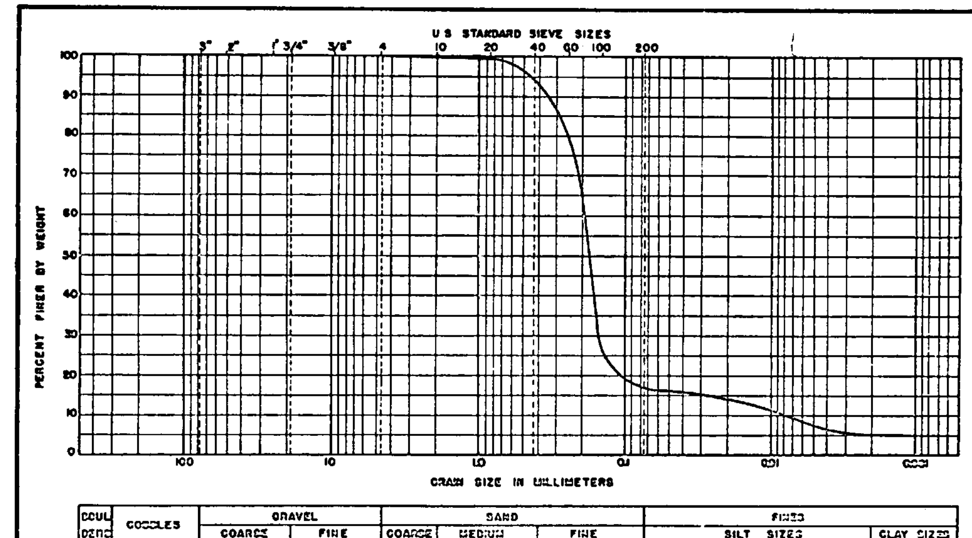


BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
R-588 S#4	19-20.5'					Green-grey clayey silty medium to fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
R-588 S#13	64-65.5'					Light grey slightly clayey slightly silty medium to fine sand

GRAIN SIZE DISTRIBUTION

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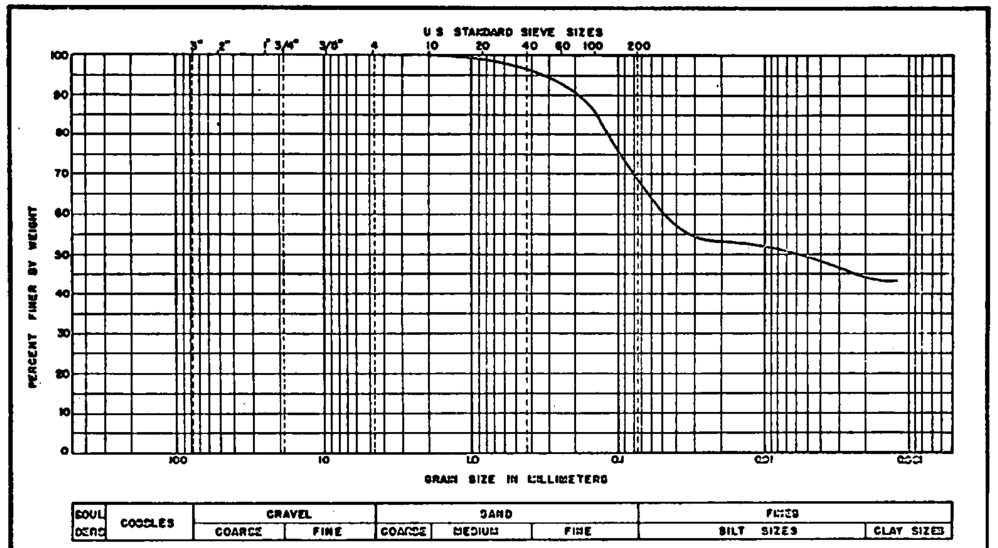
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 13 OF 34)

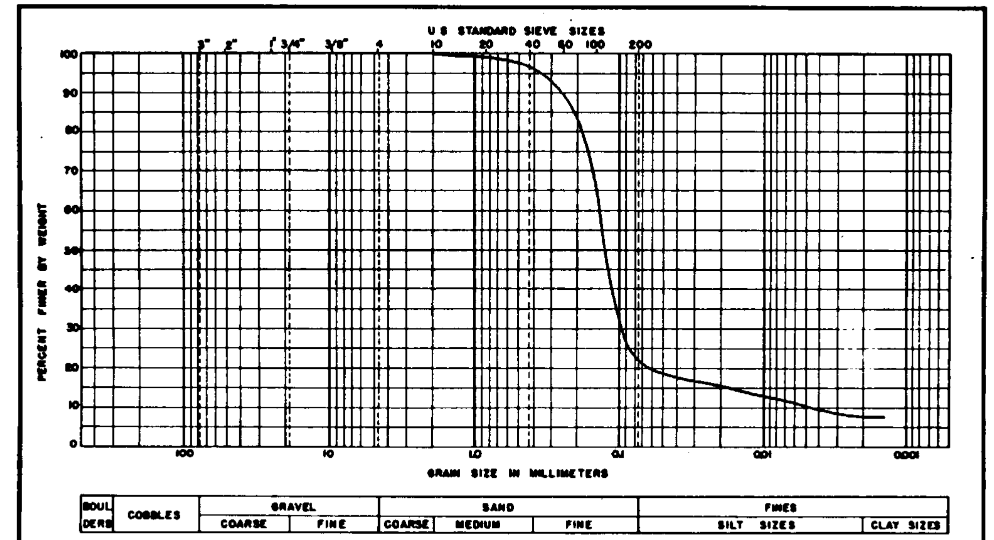


BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
R-593 S#1	4-5.5'					Grey dark red and brown fine sandy silty clay

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

LAW ENGINEERING TESTING COMPANY

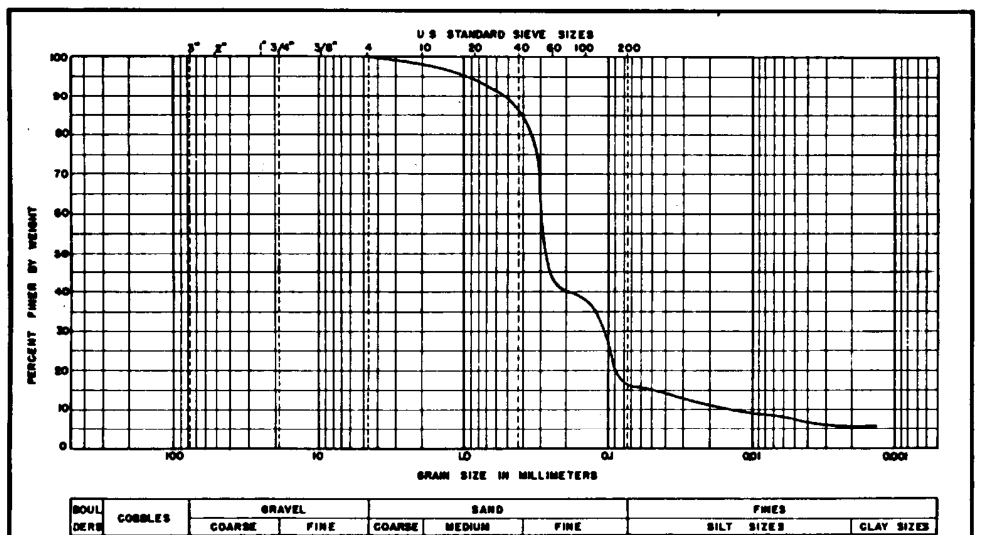


BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
R-588 S#16	79-80.5'					Light grey slightly clayey slightly silty medium to fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

LAW ENGINEERING TESTING COMPANY

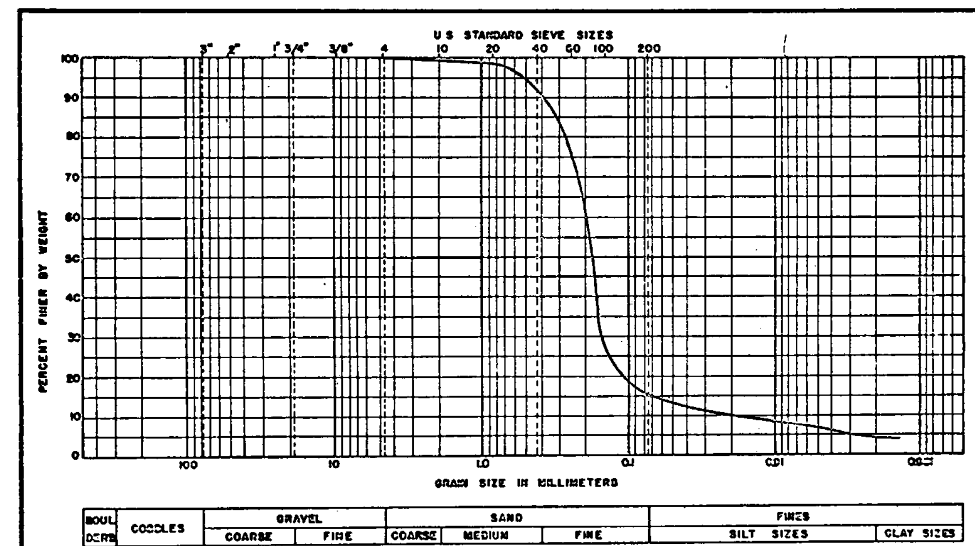


BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
R-588 S#15	74-75.5'					Light grey slightly clayey slightly silty medium to fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
R-588 S#23	114-115.5'					Light grey slightly clayey slightly silty medium to fine sand

GRAIN SIZE DISTRIBUTION

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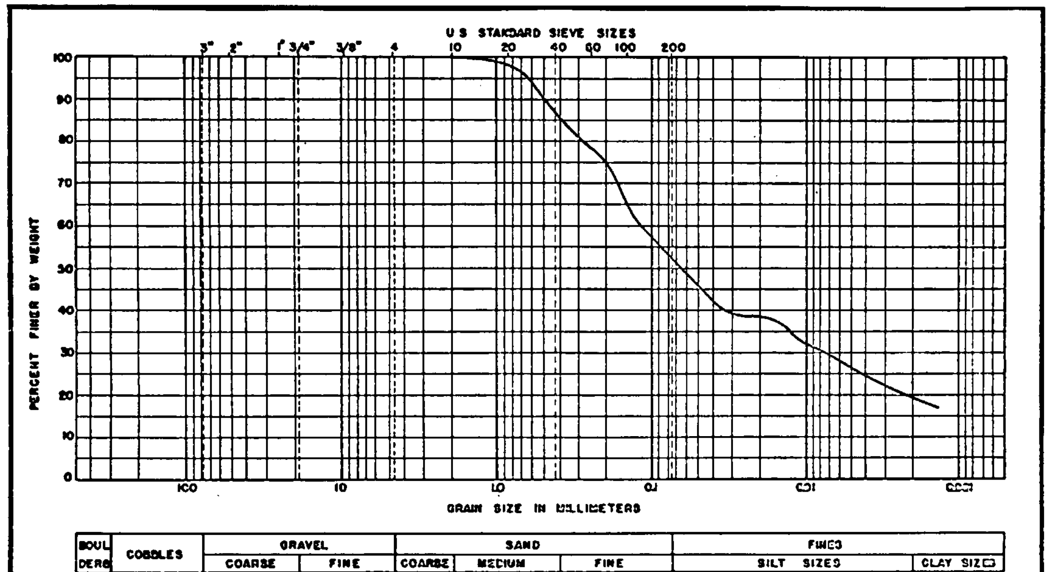
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 14 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
R-593 S#3	19-20.5'					Grey clayey silty medium to fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

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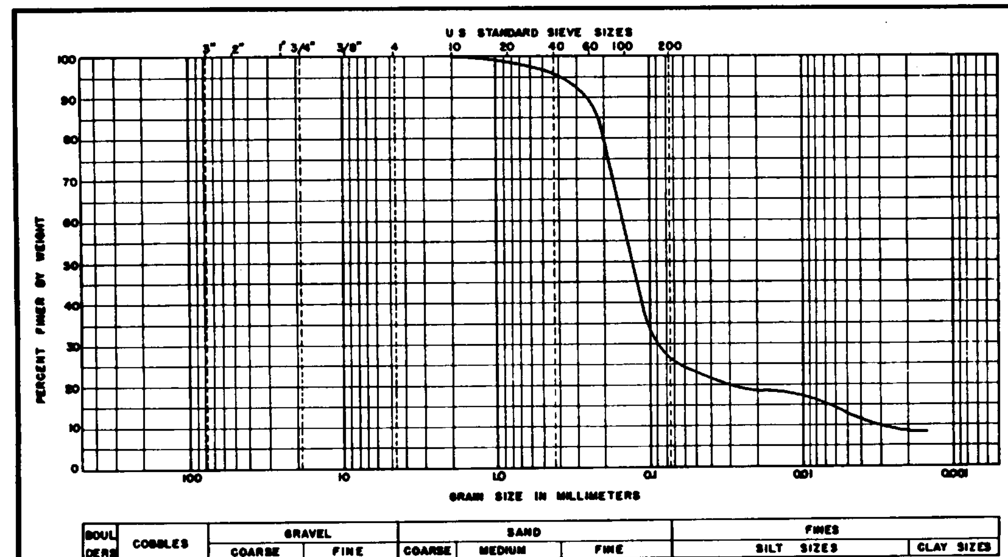
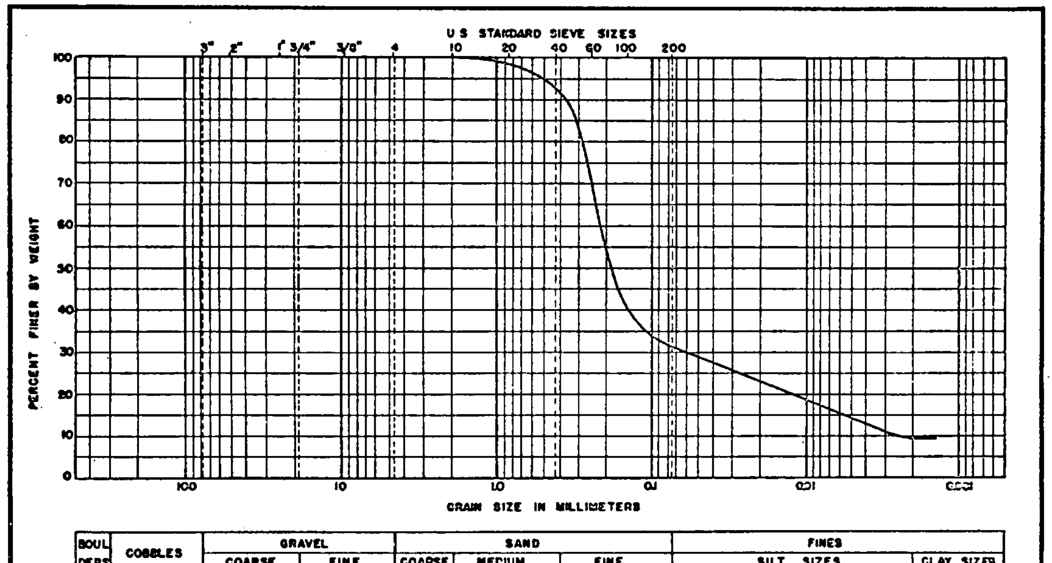


BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
R-593 S#13	79-80.5'					Light grey slightly clayey silty fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

LAW ENGINEERING TESTING COMPANY

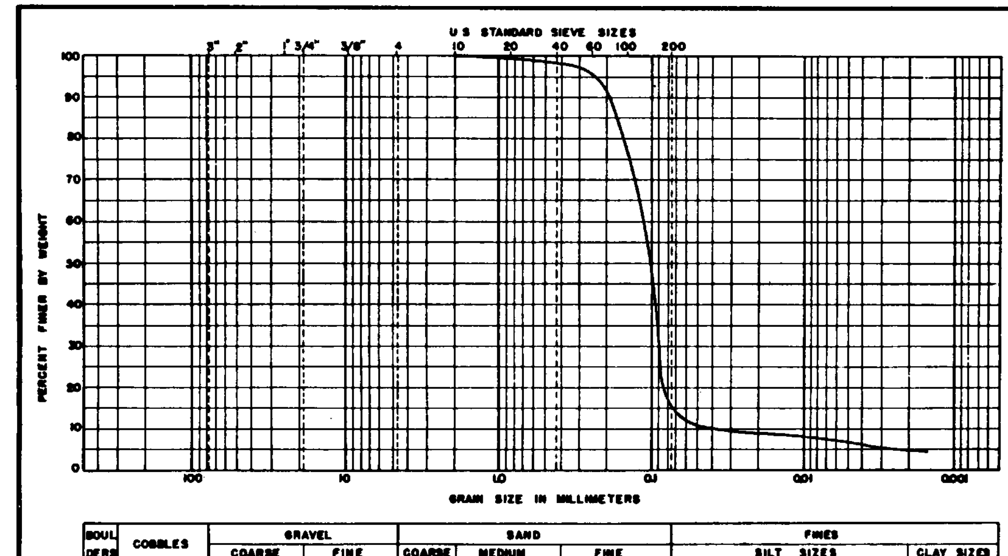


BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
R-593 S#5	29-30.5'					Light grey clayey silty medium to fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
R-593 S#16	94-95.5					Light grey slightly clayey slightly silty fine sand

GRAIN SIZE DISTRIBUTION

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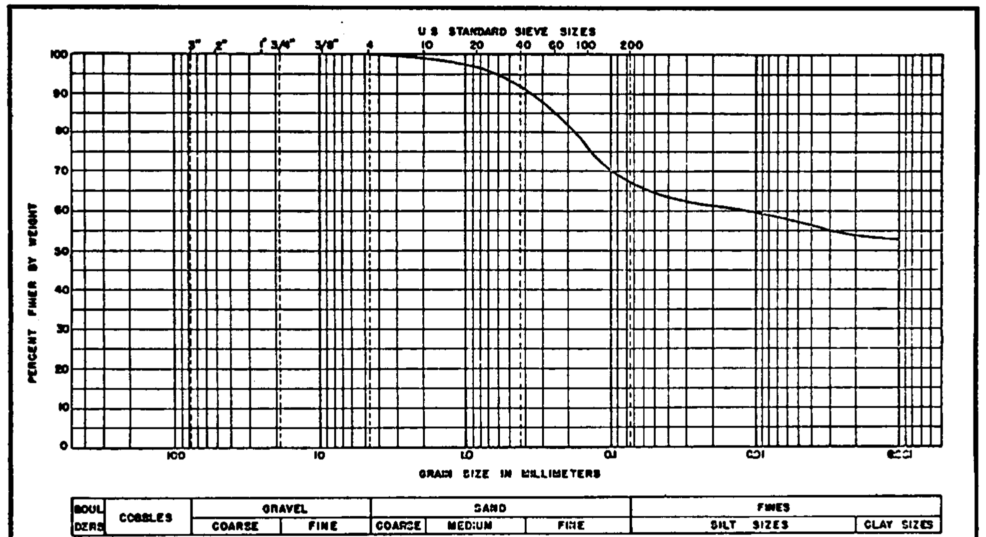
UNIT 2

GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 15 OF 34)

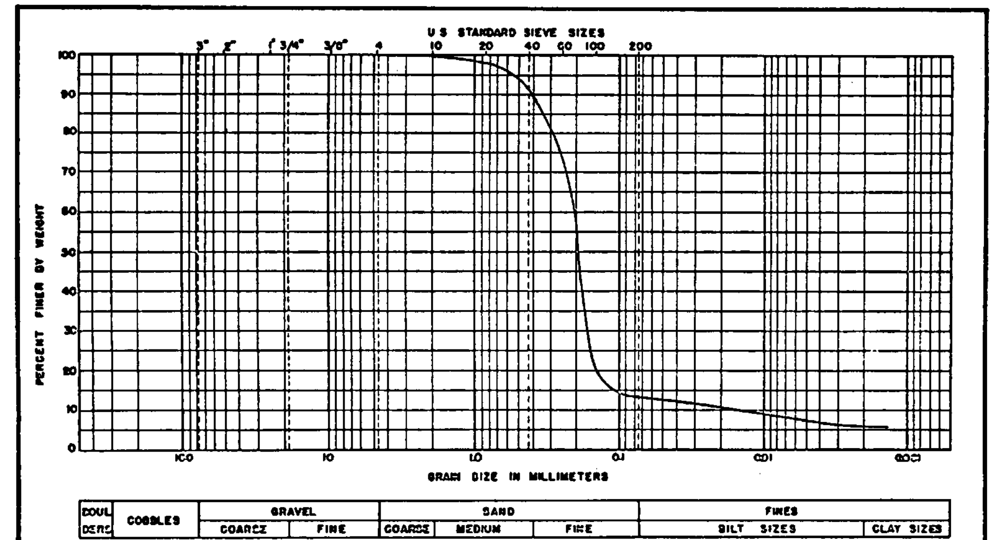


BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
T-600 S#1	4.5-6'					Pink,brown and light grey medium to fine sandy silty clay

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

LAW ENGINEERING TESTING COMPANY

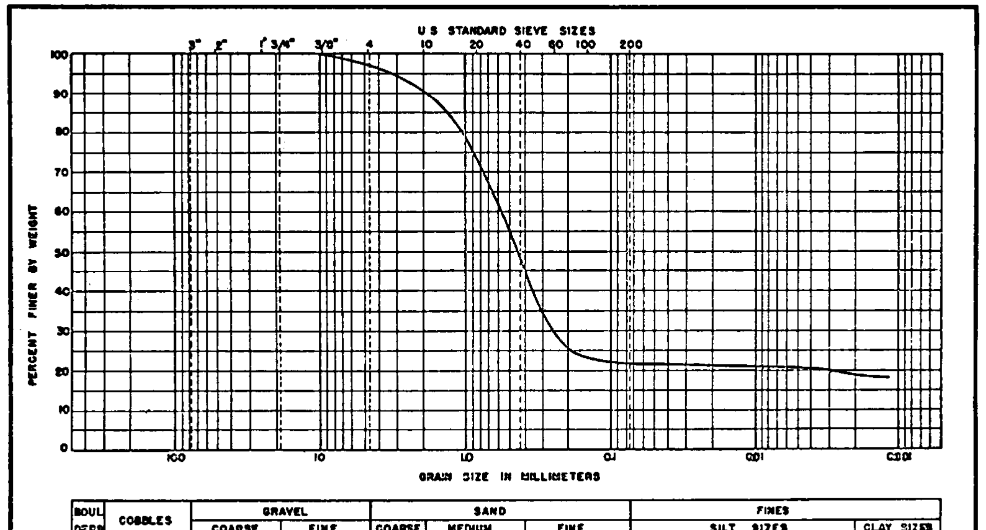


BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
R-593 S#19	109-110.5'					Light grey slightly silty slightly clayey medium to fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
T-600 S#3	14-15.5'					Light green grey clayey coarse to fine sand

GRAIN SIZE DISTRIBUTION

JOB NO. 5056

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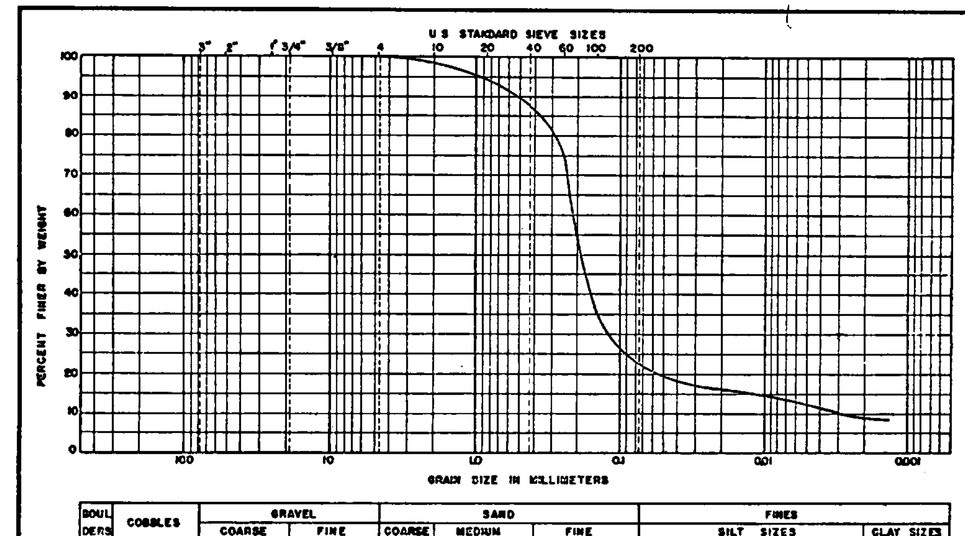

BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
R-593 S#21	119-120.5'					Light grey slightly clayey silty medium to fine sand

GRAIN SIZE DISTRIBUTION

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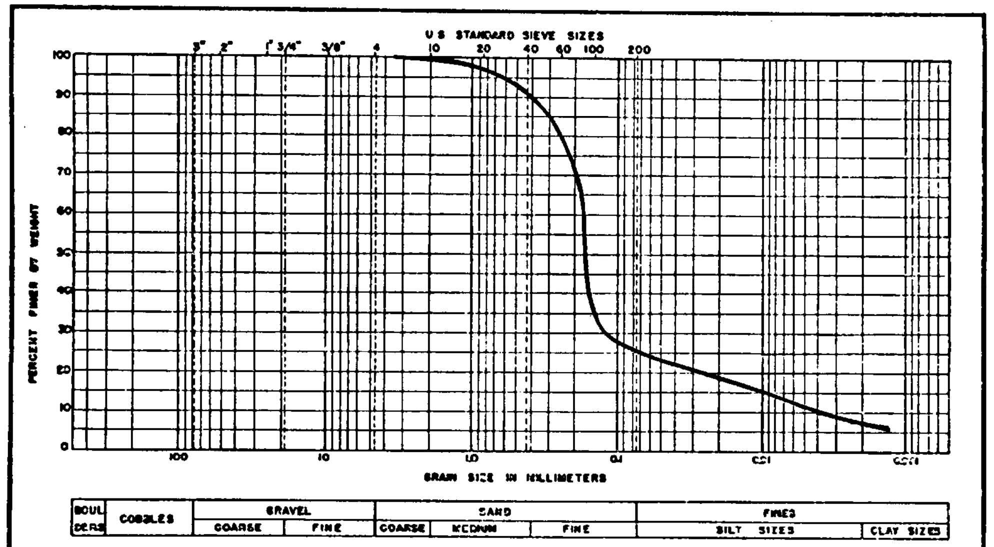
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

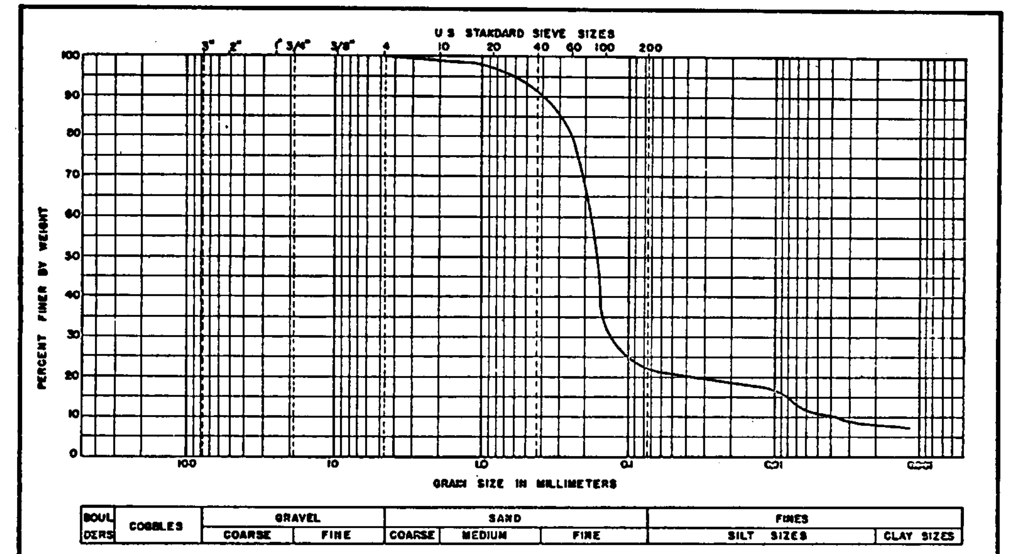
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 16 OF 34)



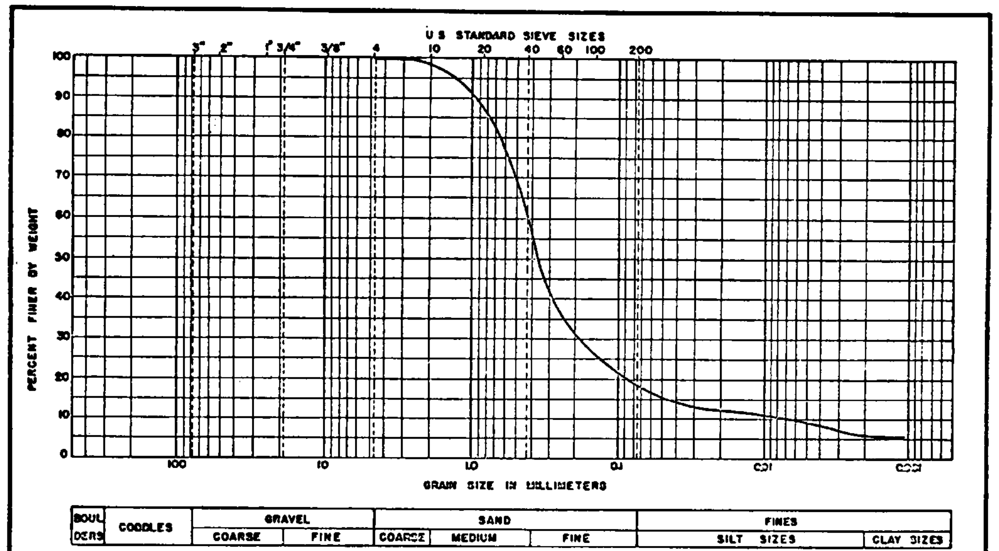
BORING NO	~~ELEV OR~~ DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
RFI-1 S-1 (UNIT II REACTOR)	3.5'-5.0'					GREEN GRAY SILTY CLAYEY FINE SAND N=11

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



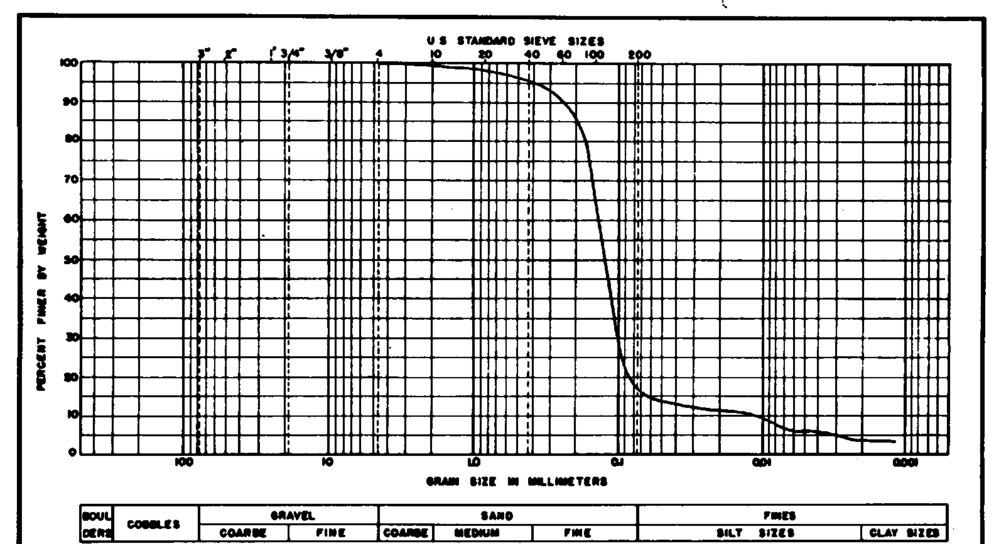
BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
T-600 S#13	64-65.5'					Green grey slightly clayey silty medium to fine sand

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
T-600 S#7	34-35.5'					Greyish white slightly clayey slightly silty coarse to fine sand

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
T-600 S#16	79-80.5'					Light grey slightly silty fine sand

GRAIN SIZE DISTRIBUTION
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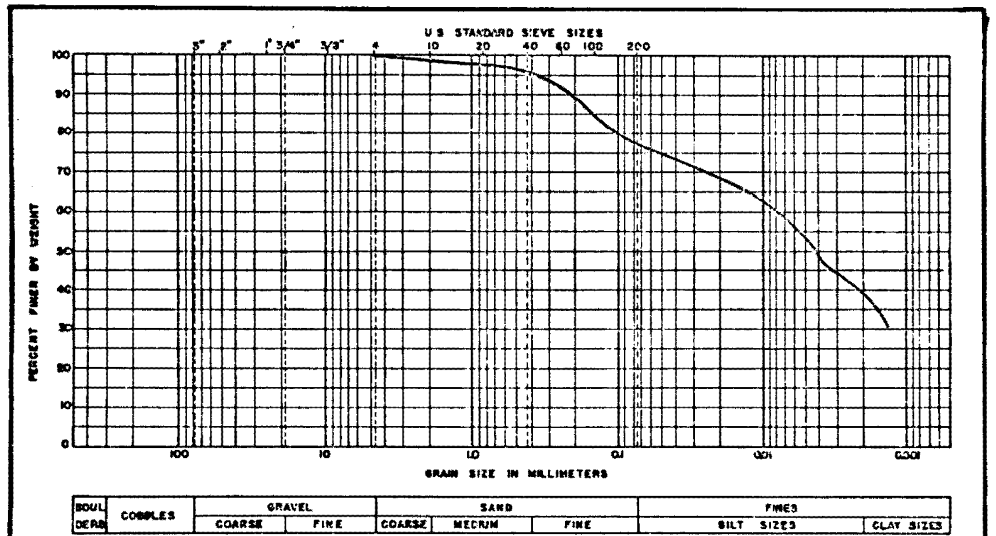
UNIT 2

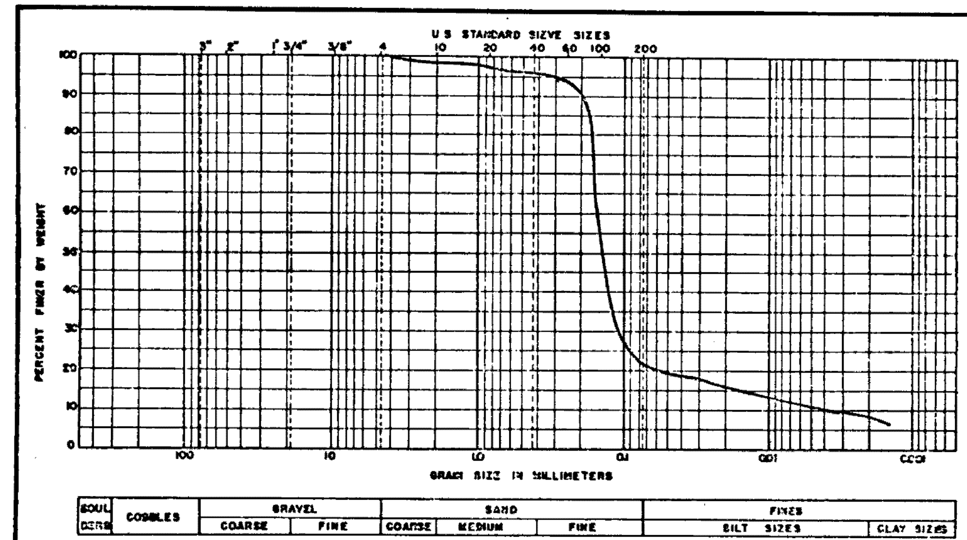
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 17 OF 34)



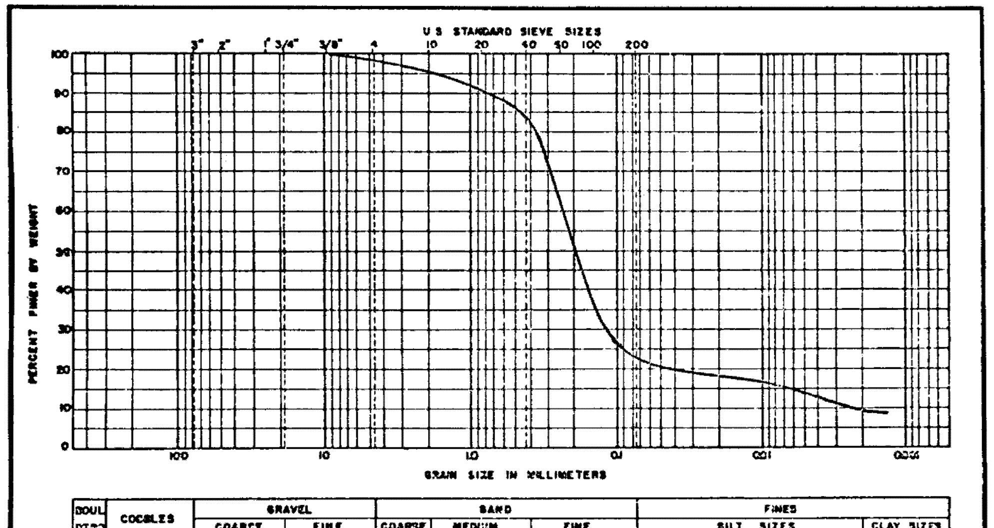
BORING NO	~~ELEV OR~~ DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
RFI-1 S-2 (UNIT II REACTOR)	8.5'-10'					GRAY GREEN FINE SANDY SILTY CLAY N=16

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



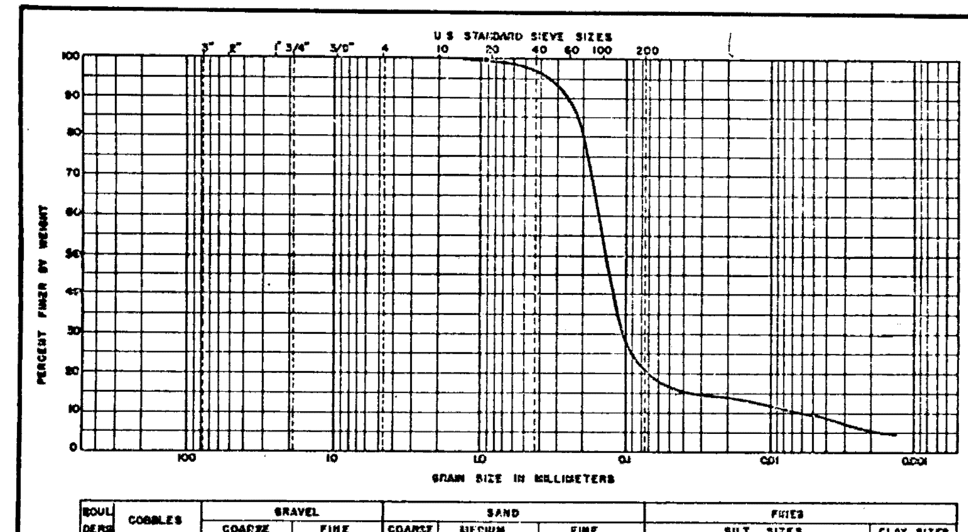
BORING NO	~~ELEV OR~~ DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
RFI-1 S-4 (UNIT II REACTOR)	18.5'-20'					LIGHT GRAY SILTY CLAYEY FINE SAND N=7

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



BORING NO	~~ELEV OR~~ DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
RFI-1 S-3 (UNIT II REACTOR)	13.5-15'					LIGHT GRAY AND GRAY SILTY CLAYEY FINE TO MEDIUM SAND N=8

GRAIN SIZE DISTRIBUTION
JOB NO. 5056
LAW ENGINEERING TESTING COMPANY



BORING NO	~~ELEV OR~~ DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
RFI-1 S-5 (UNIT II REACTOR)	23.5'-25'					LIGHT GRAY SILTY CLAYEY FINE SAND N-20

GRAIN SIZE DISTRIBUTION
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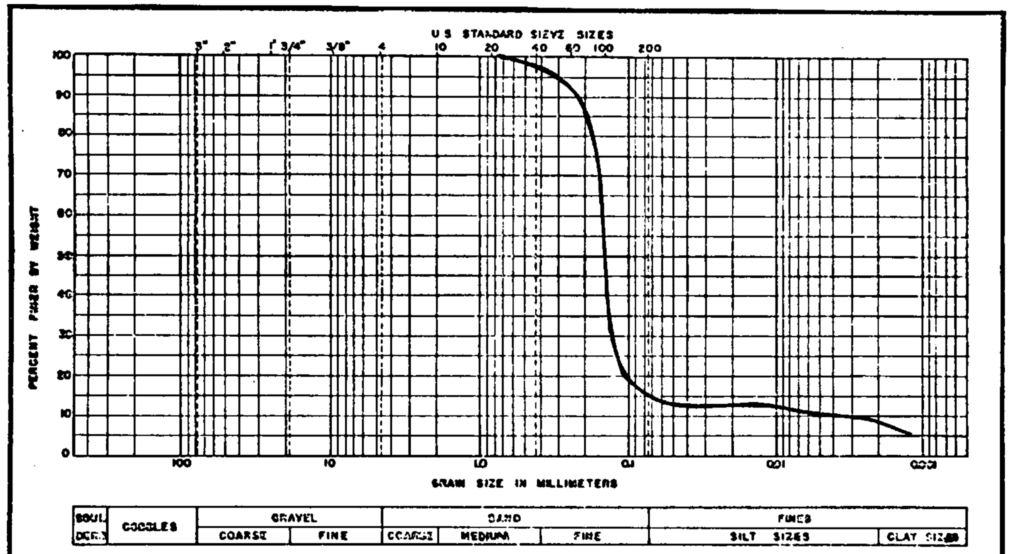
UNIT 2

GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 18 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
RFI-4 S-1 (UNIT II REACTOR)	3.5'-5'					GREEN GRAY CLAYEY FINE SAND N=15

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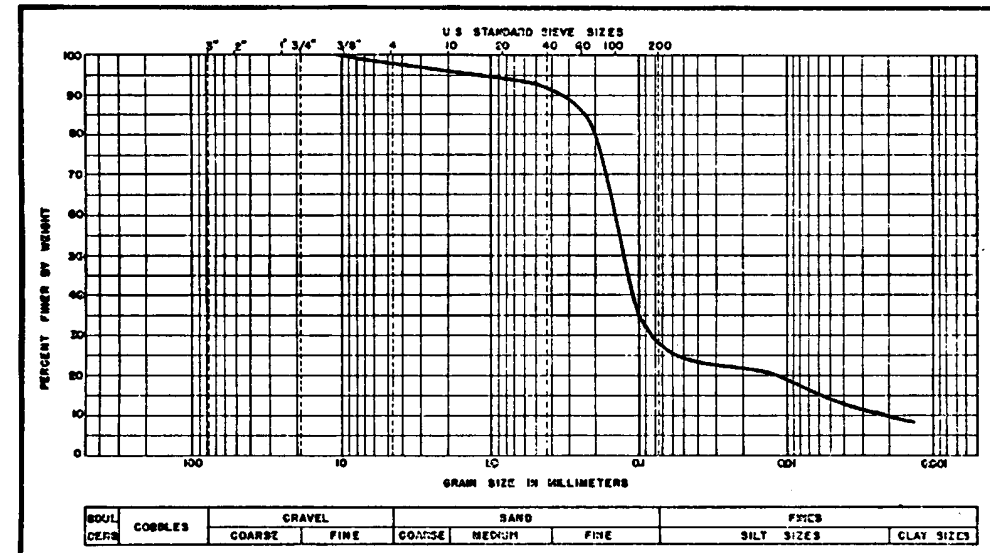

BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
RFI-4 S-6 (UNIT II REACTOR)	23.5-25'					LIGHT GRAY CLAYEY SILTY FINE SAND N=9

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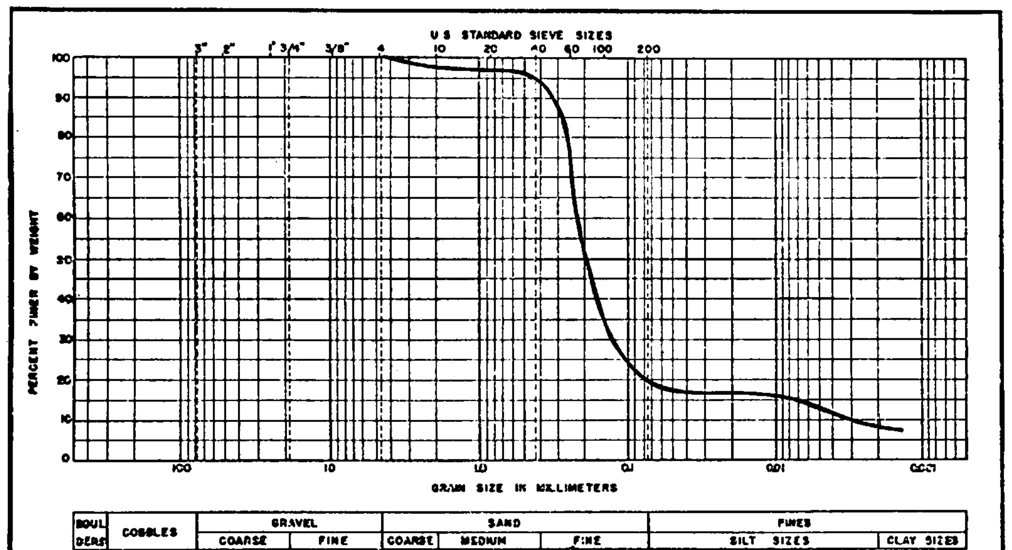

BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
RFI-4 S-4 (UNIT II REACTOR)	13.5'-15'					LIGHT GREEN GRAY CLAYEY SILTY FINE SAND N=12

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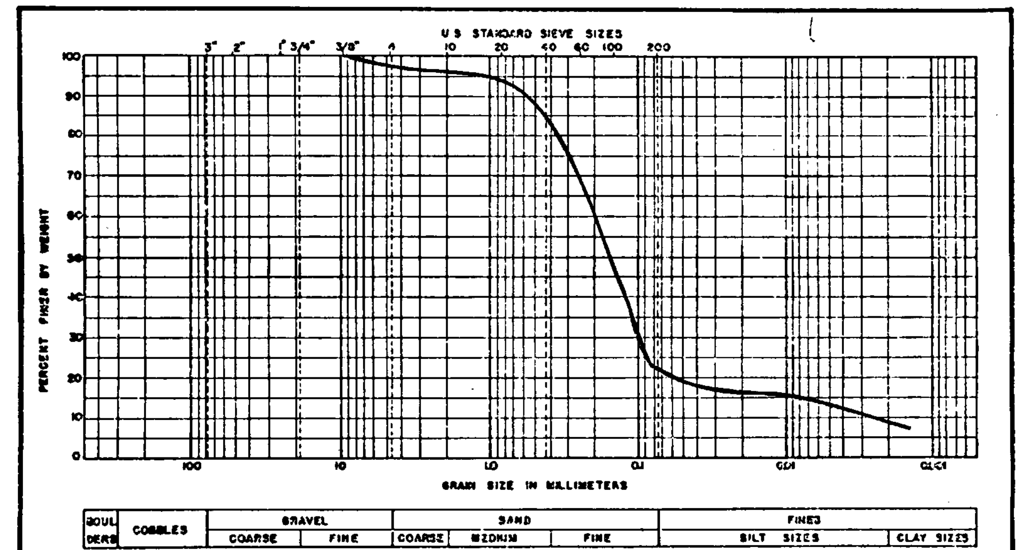

BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
RFI-1 S-8 (UNIT II REACTOR)	38.5-40'					LIGHT GRAY SILTY CLAYEY MEDIUM TO FINE SAND N=45

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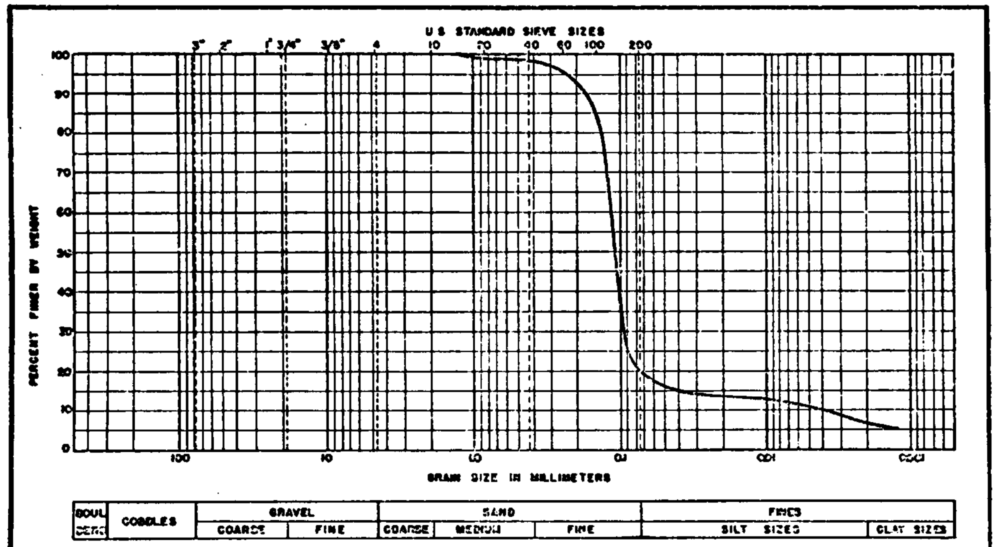
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 19 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
RFI-4 S-7 (UNIT II REACTOR)	33.5'-35'					LIGHT GRAY SILTY FINE SAND N=22

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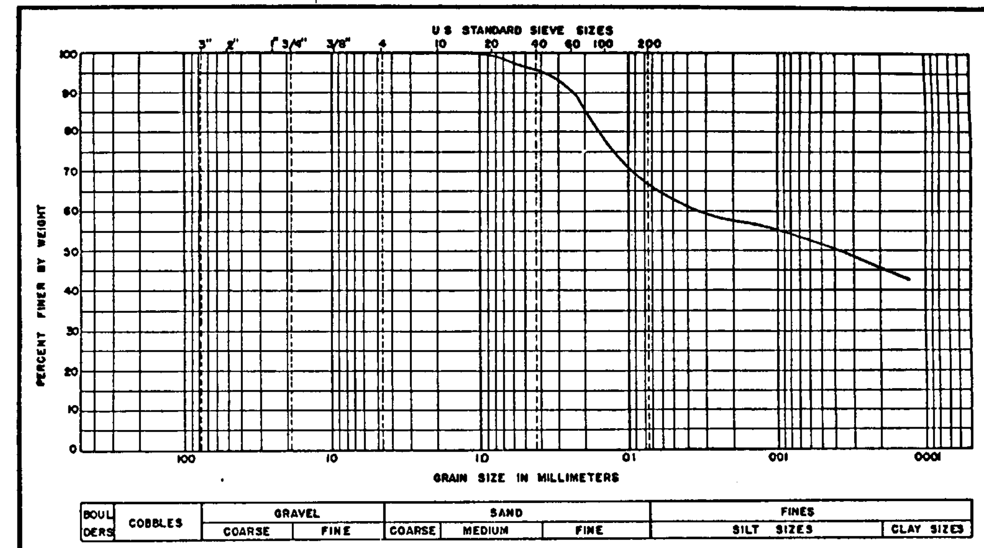
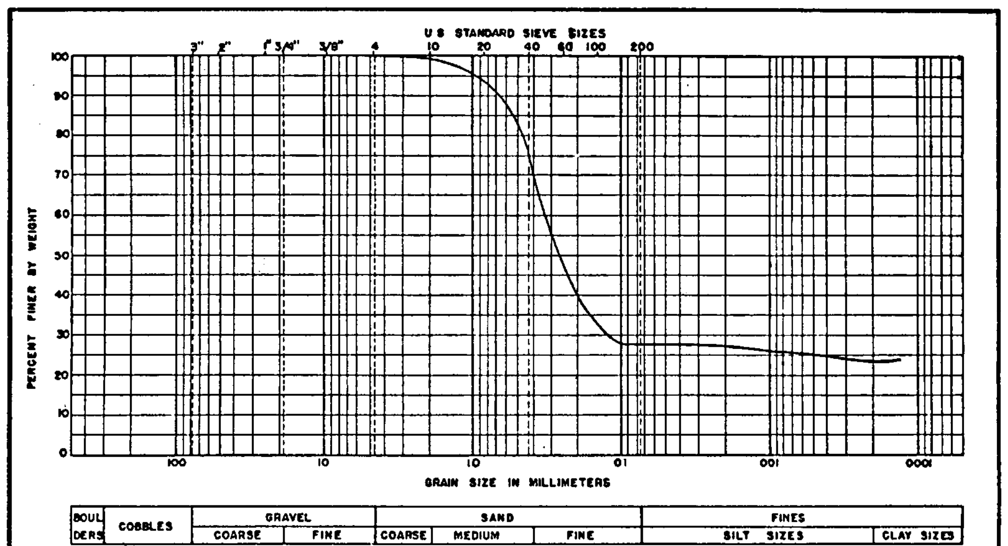


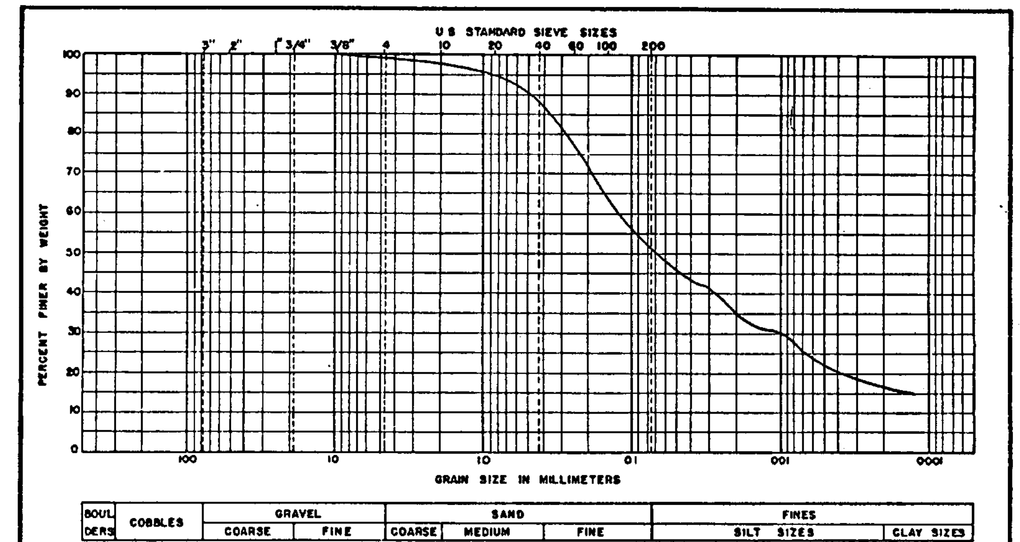
BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
A-2001 UD-1	10' - 12'	19.5	57	14	43	FINE SANDY SILTY CLAY

GRAIN SIZE DISTRIBUTION
JOB NO. #5056
LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
A-2001 UD-2	20' - 22'	14.8	37	10	27	SLIGHTLY SILTY CLAYEY MEDIUM TO FINE SAND

GRAIN SIZE DISTRIBUTION
JOB NO. #5056
LAW ENGINEERING TESTING COMPANY



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
A-2001 UD-3	30' - 32.5'	17.9	34	18	16	CLAYEY SILTY COARSE TO FINE SAND

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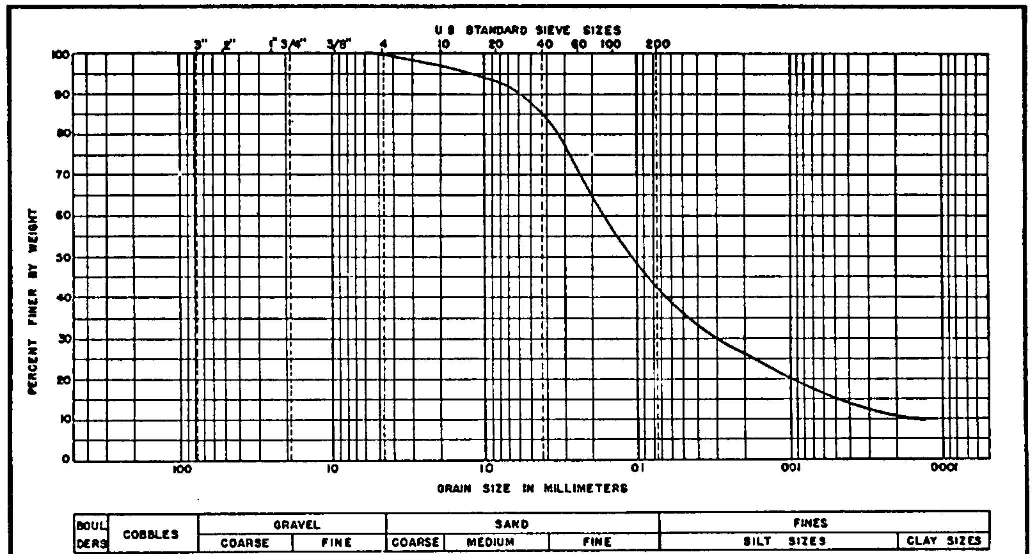
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 20 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
A-2001 UD-3&4	30' - 42.5'	15.2	33	18	15	CLAYEY SILTY COARSE TO FINE SAND

GRAIN SIZE DISTRIBUTION

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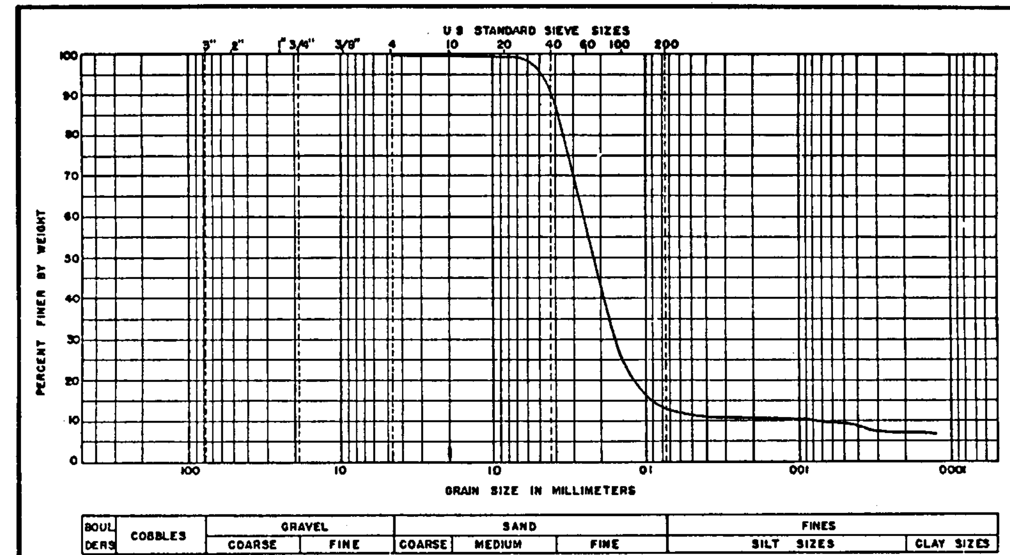

BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
A-2001 UD-5	50' - 52.5'	17.8	N/L	N/P	N/P	SLIGHTLY CLAYEY SLIGHTLY SILTY MEDIUM TO FINE SAND

GRAIN SIZE DISTRIBUTION

JOB NO. #5056

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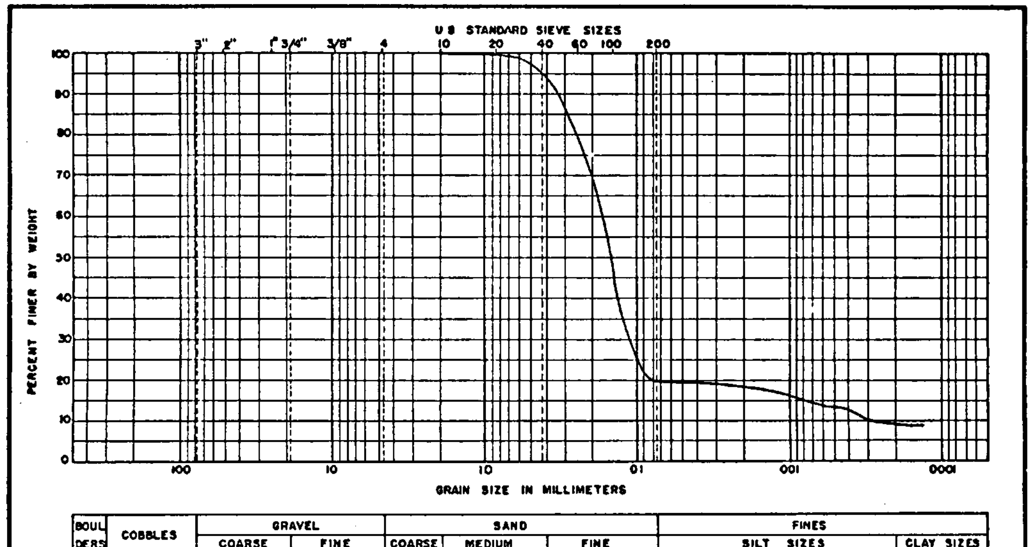

BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
A-2001 UD-6	55' - 57'	23.6	30	19	11	SLIGHTLY CLAYEY SILTY MEDIUM TO FINE SAND

GRAIN SIZE DISTRIBUTION

JOB NO. #5056

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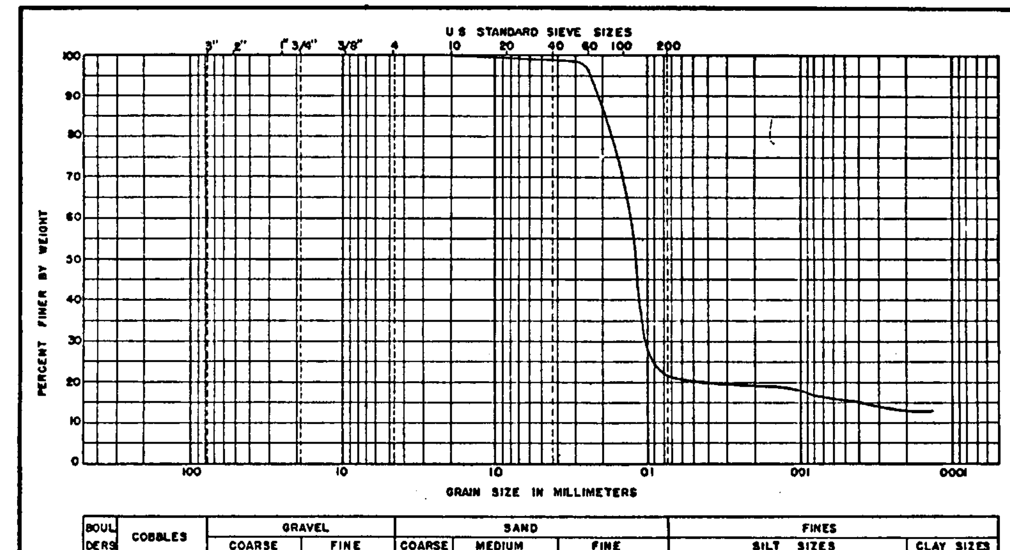

BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
A-2001 UD-7	60' - 62'	30.8	33	21	12	CLAYEY SILTY MEDIUM TO FINE SAND

GRAIN SIZE DISTRIBUTION

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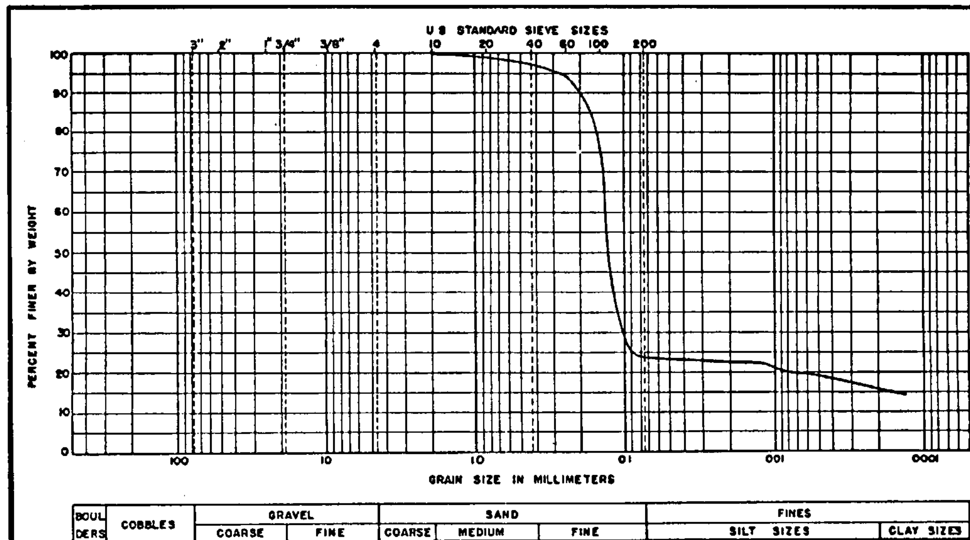
UNIT 2

GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 21 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
A-2001 UD-8	72' - 74.5'	27.7	34	19	15	SLIGHTLY SILTY CLAYEY MEDIUM TO FINE SAND

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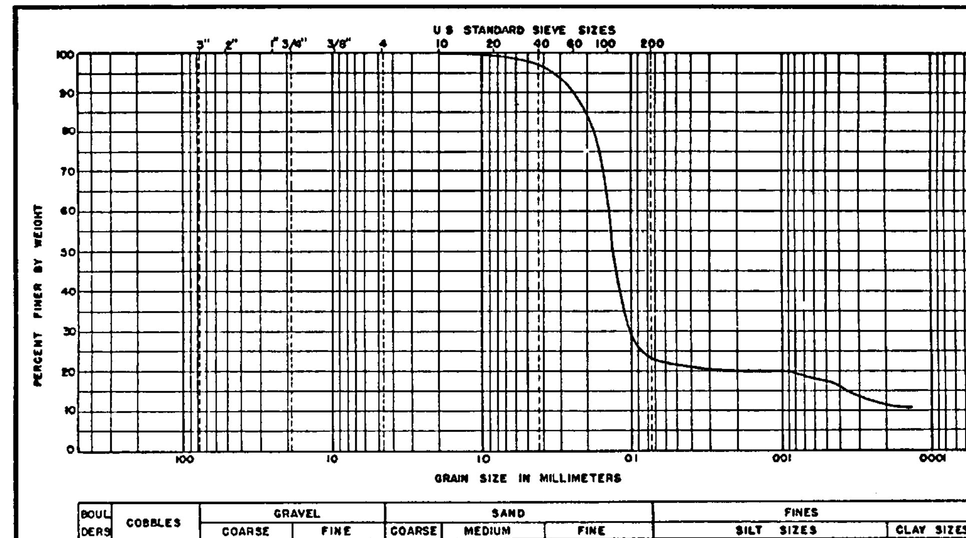

BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
A-2001 UD-9	75' - 77.5'	47.3	36	16	20	CLAYEY SILTY MEDIUM TO FINE SAND

GRAIN SIZE DISTRIBUTION

JOB NO. #5056

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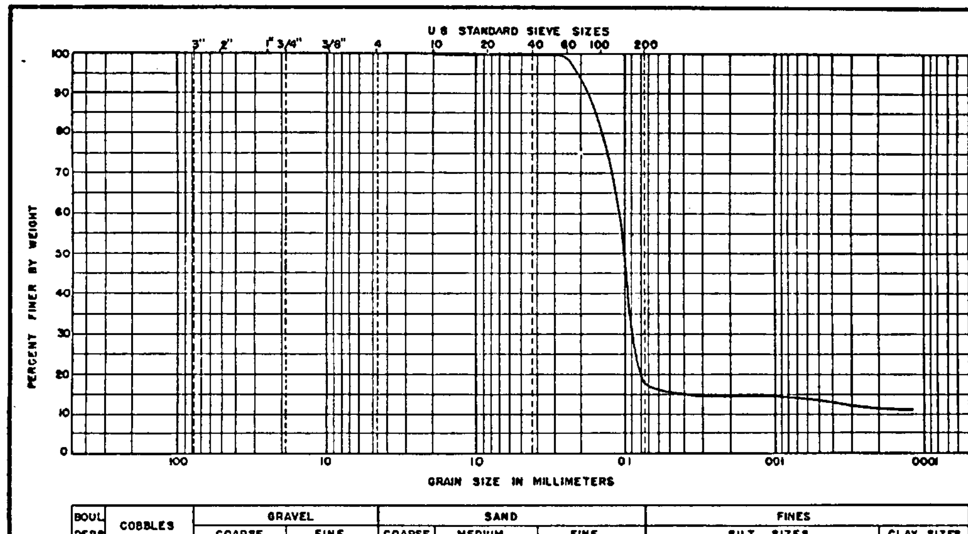

BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
UD #10	80' - 82.5'	Dry Sample	N/L	N/P	N/P	SLIGHTLY SILTY CLAYEY FINE SAND

GRAIN SIZE DISTRIBUTION

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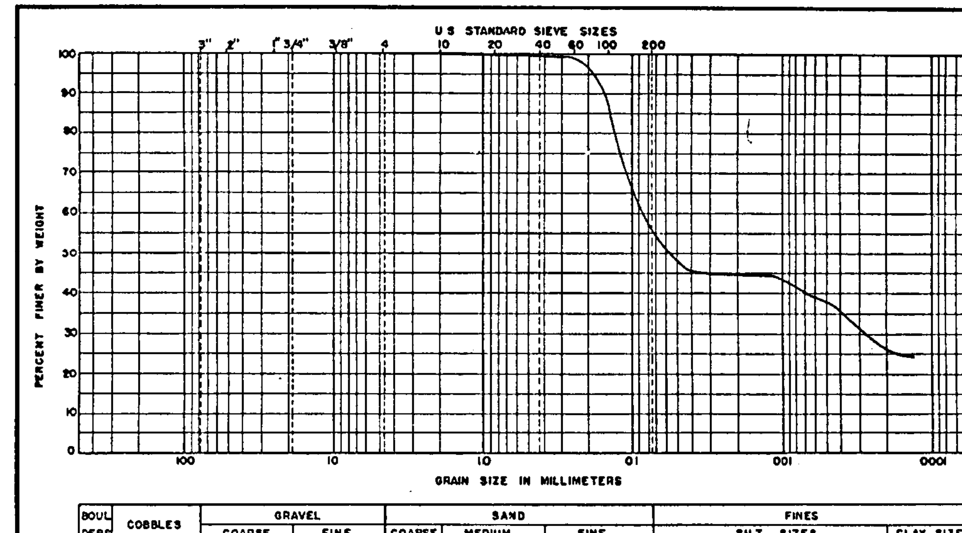

BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
A-2001 UD-11	85' - 87.5'	78.0	75	21	54	CLAYEY SILTY FINE SAND

GRAIN SIZE DISTRIBUTION

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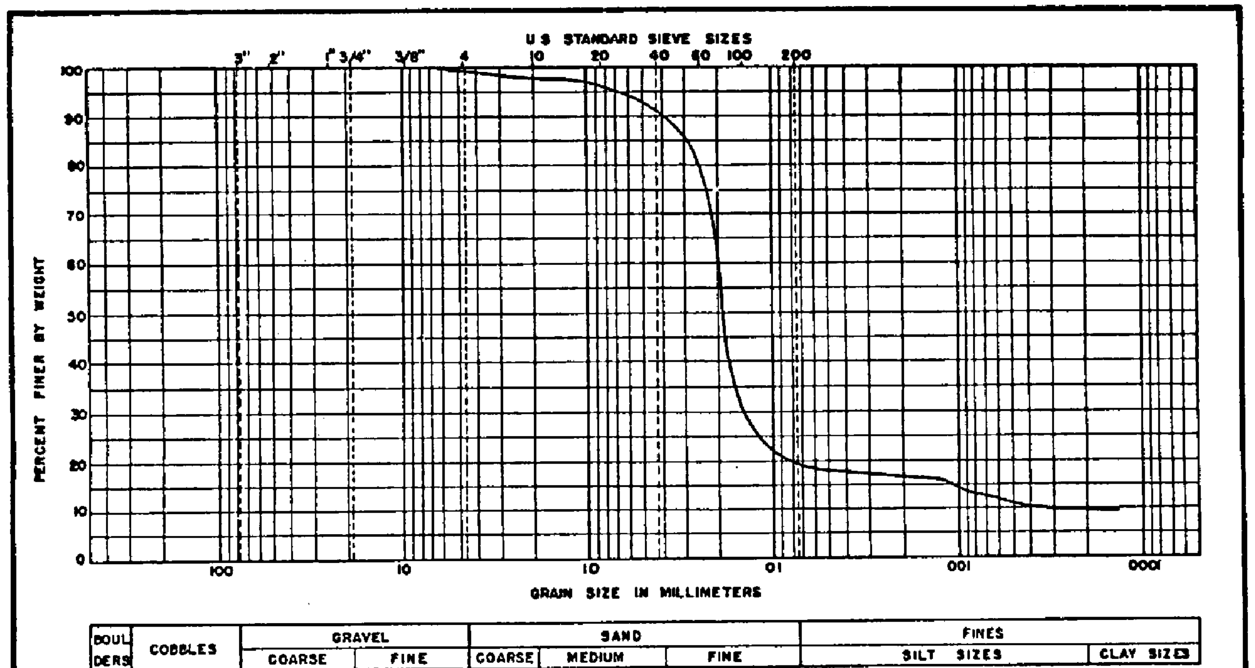
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 22 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
A-2001 UD-12	90' - 92.5'	23.3	28	19	9	SLIGHTLY CLAYEY SILTY COARSE TO FINE SAND

GRAIN SIZE DISTRIBUTION

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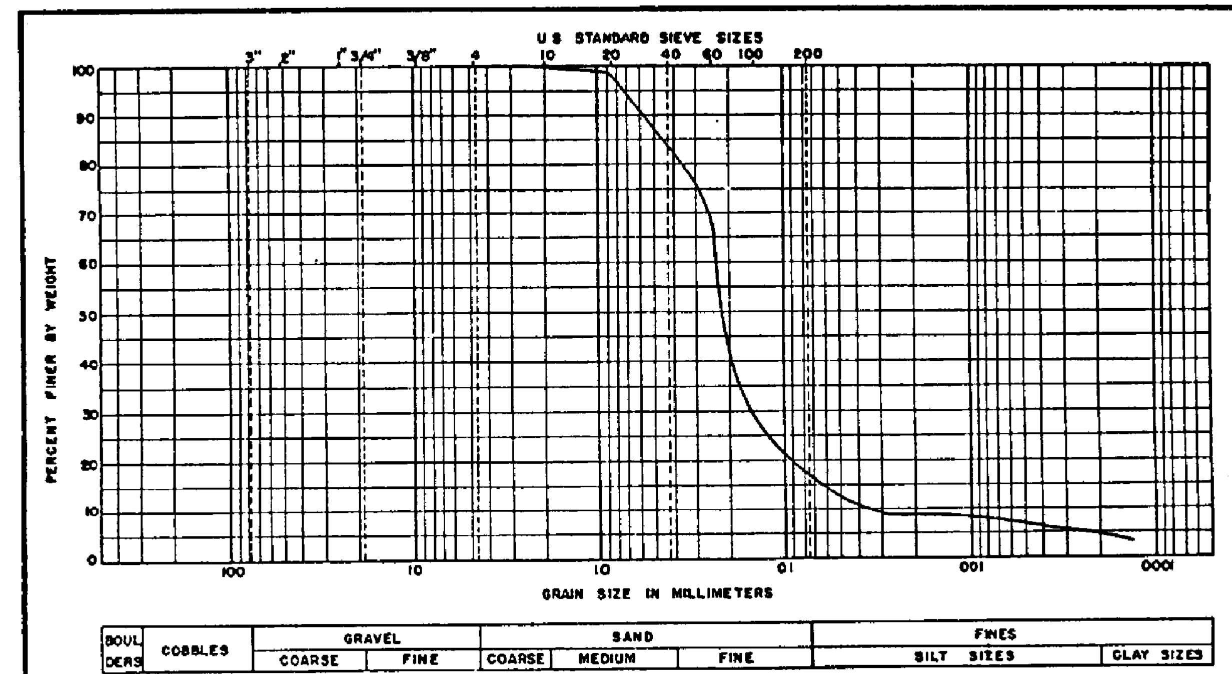

BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
A-2001 UD-13	100'-102.5'	26.2	38	21	17	SLIGHTLY CLAYEY SILTY MEDIUM TO FINE SAND

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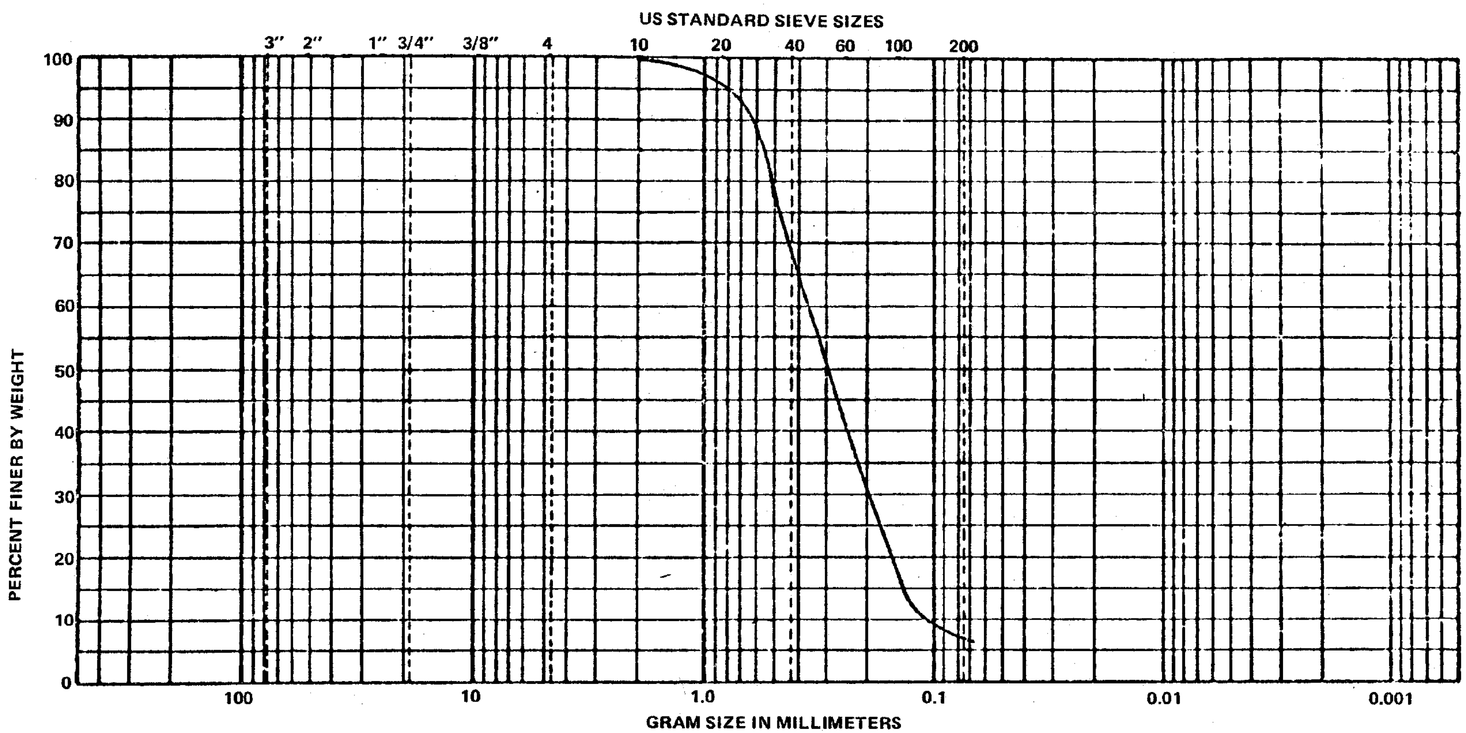
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 23 OF 34)



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV OR DEPTH	MAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
IFI-1 (INTAKE)	13.5					GRAY TAN FINE TO MEDIUM SAND N= 24

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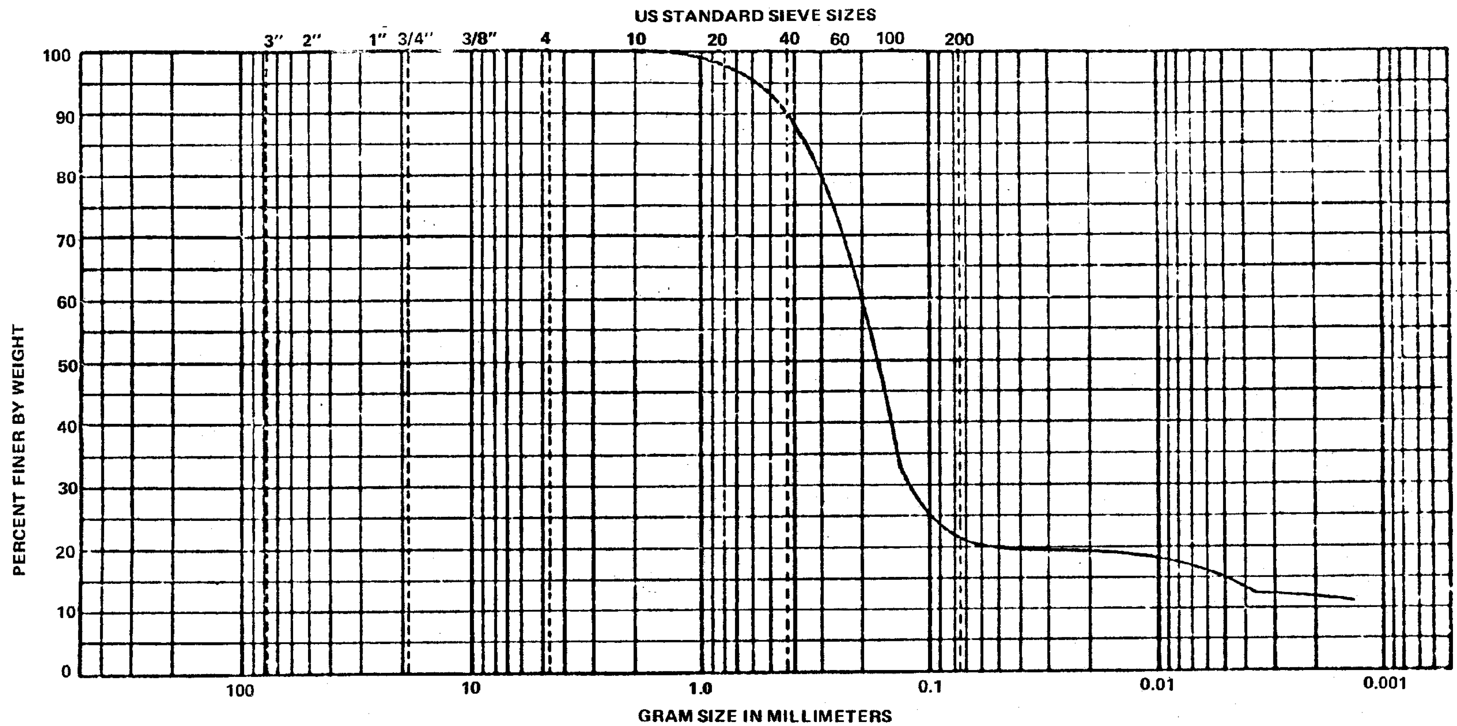
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 24 OF 34)



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV OR DEPTH	MAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
IFI–1 (INTAKE)	23.5					GRAY TAN SILTY CLAYEY FINE TO MEDIUM SAND N =17

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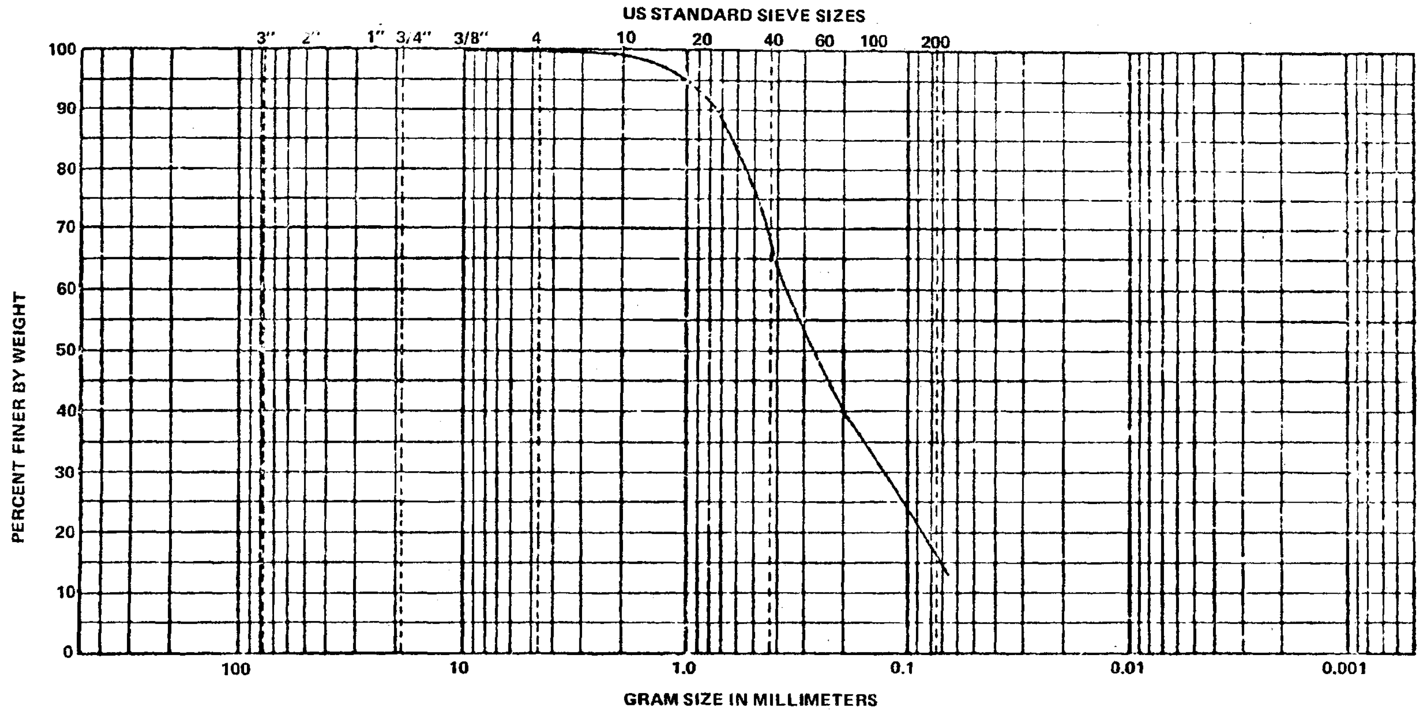
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 25 OF 34)



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV OR DEPTH	MAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
IFI-1 (INTAKE)	28.5					GRAY TAN CLAYEY FINE TO MEDIUM SAND N=14

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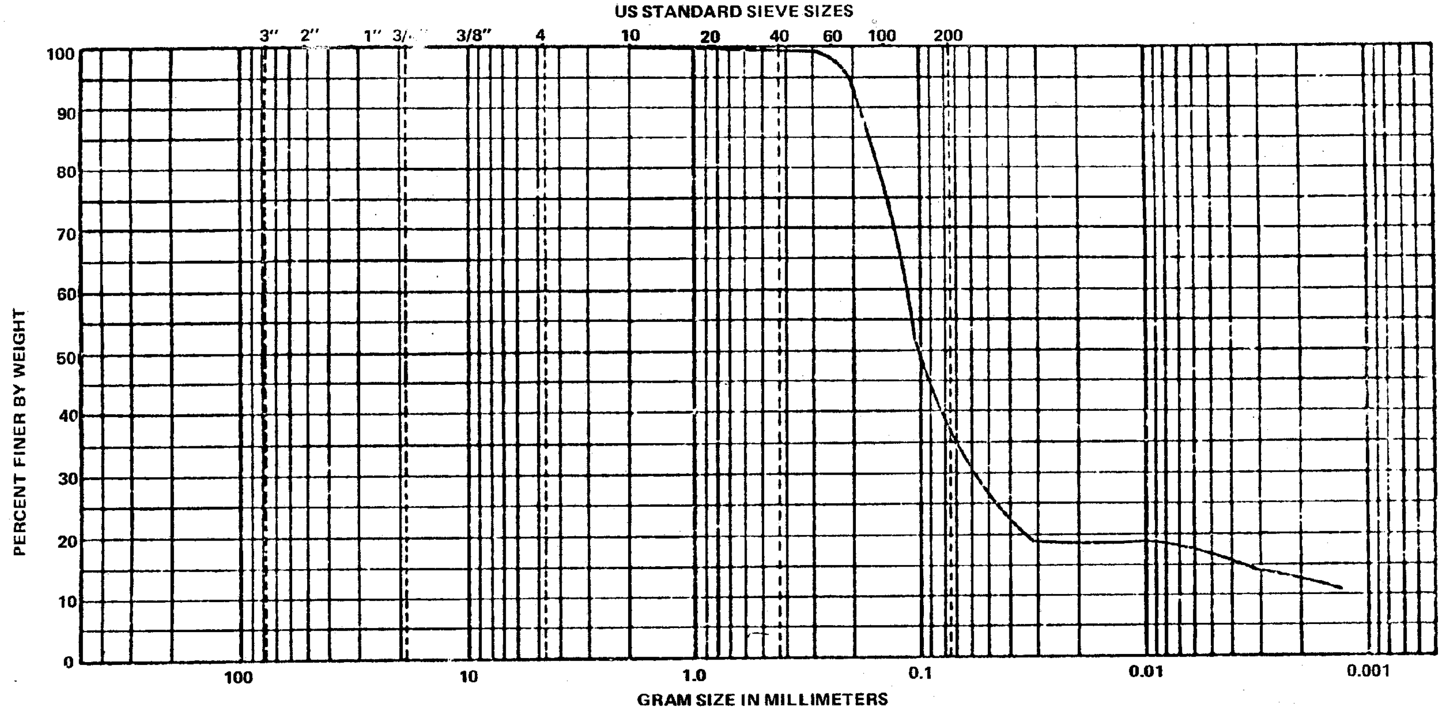
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 26 OF 34)



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV OR DEPTH	MAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
IFI-1 (INTAKE)	33.5					LIGHT GRAY SILTY CLAYEY FINE SAND N=15

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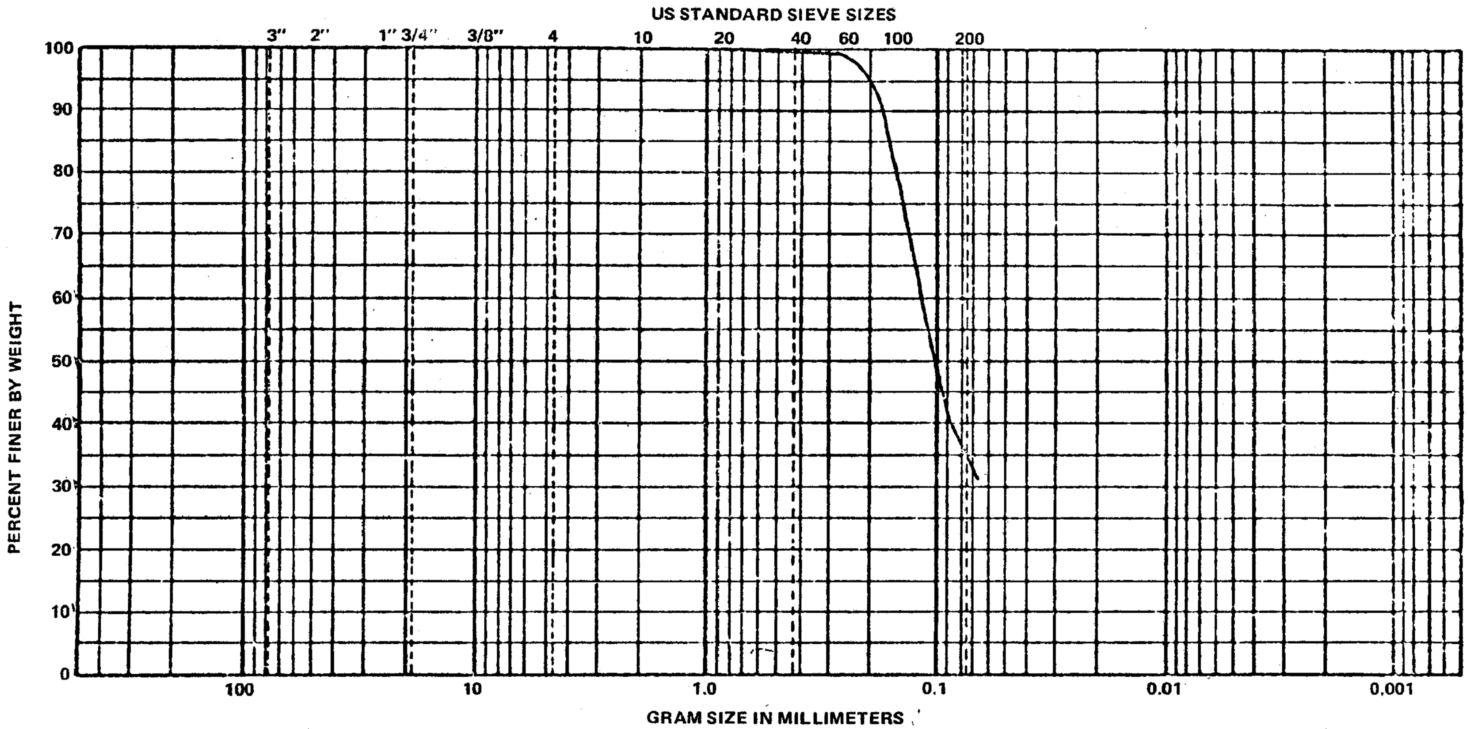
FIGURE 2A-8 (SHEET 27 OF 34)



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
IFI-1 (INTAKE)	38.5					LIGHT GRAY CLAYEY FINE SAND N=18

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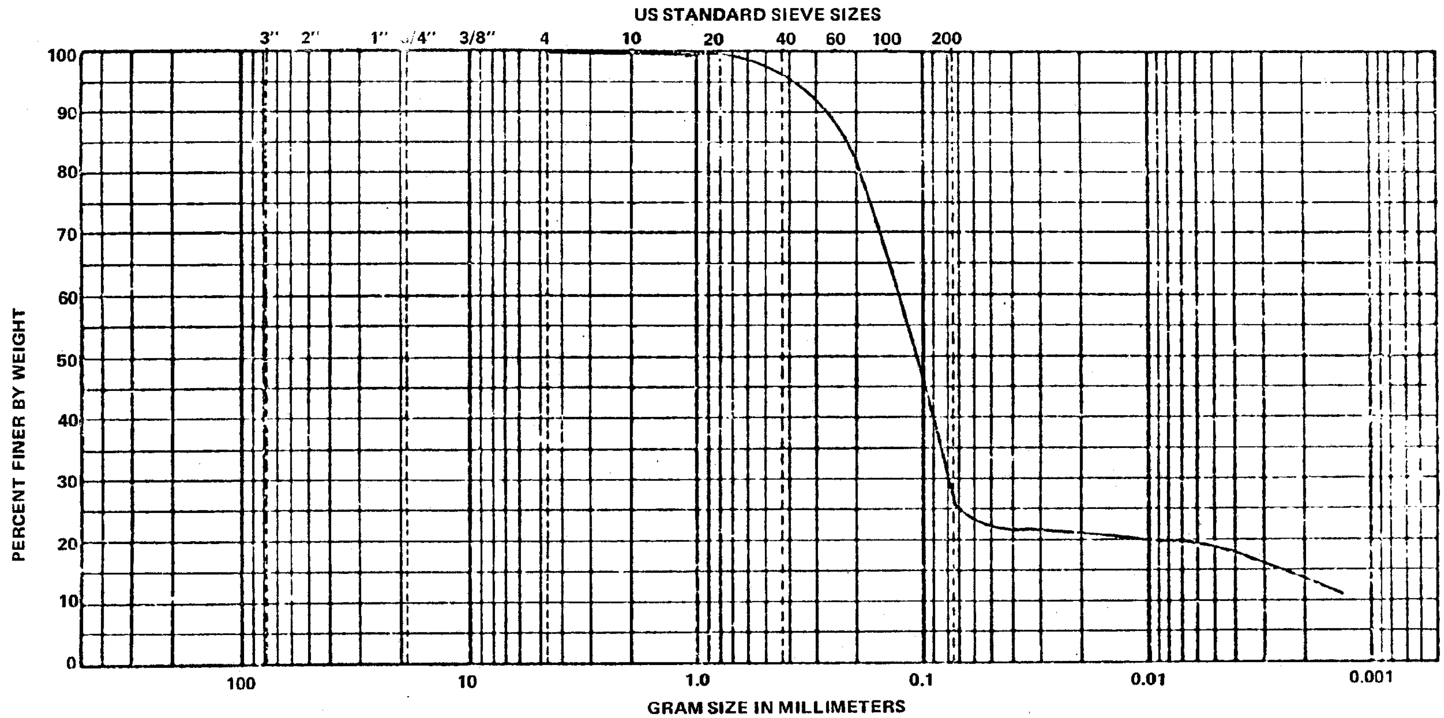
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 28 OF 34)



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT SIZES	CLAY SIZES

BORING NO	ELEV OR DEPTH	MAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
IFI-1 (INTAKE)	43.5					LIGHT GRAY SILTY CLAYEY FINE SAND N=20

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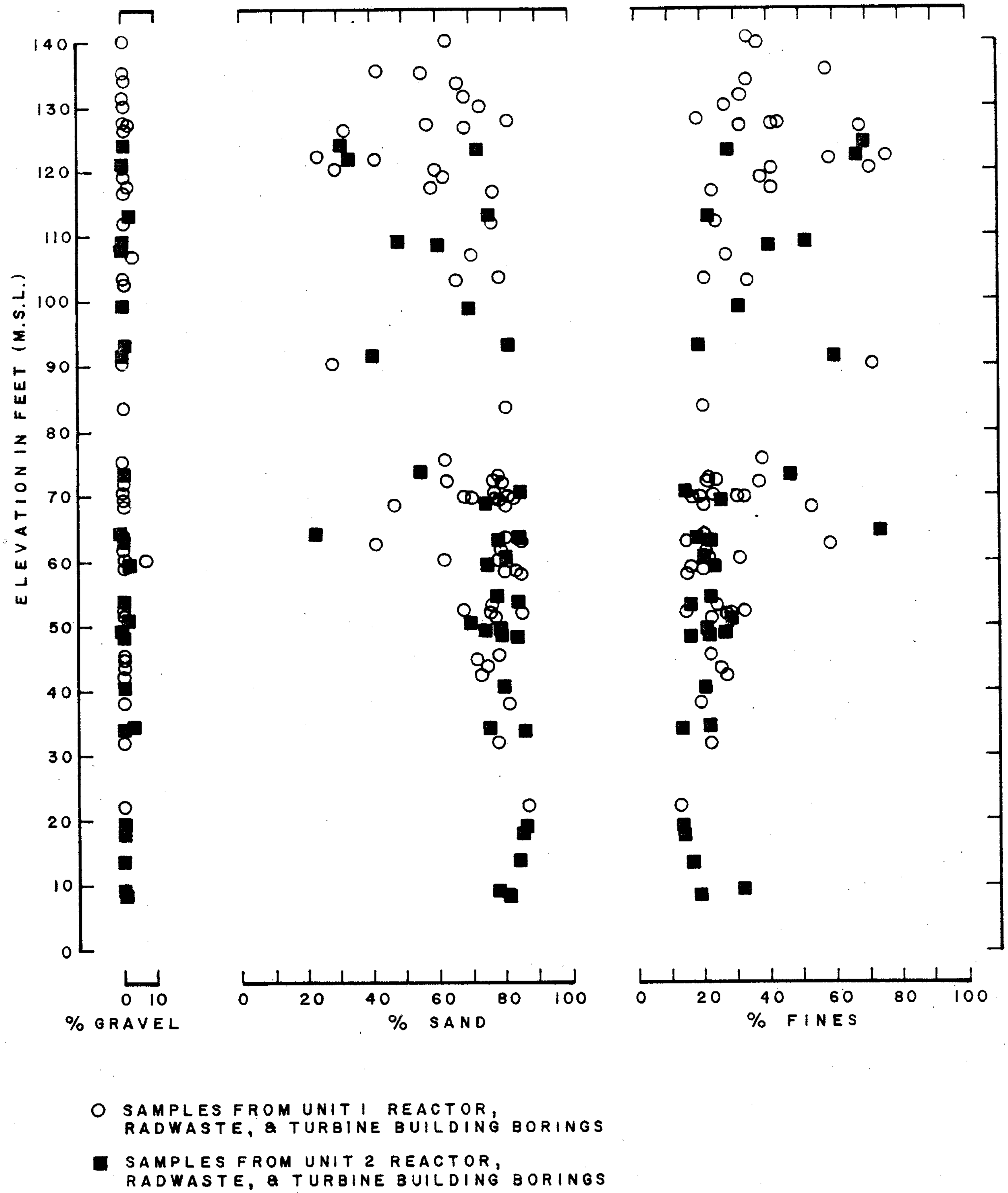
GRAIN SIZE DISTRIBUTION

FIGURE 2A-8 (SHEET 29 OF 34)



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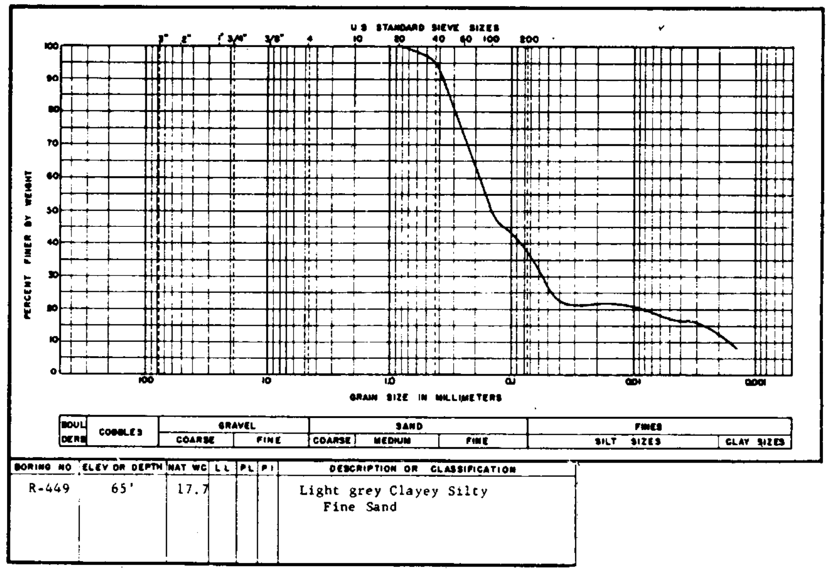
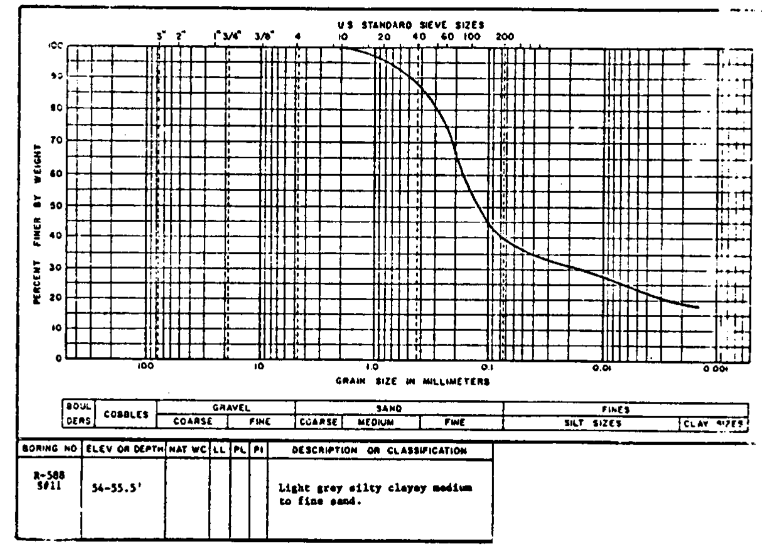
UNIT 2

COMPARISON OF RESULTS OF GRAIN SIZE TESTS WITH DEPTH FOR HNP-1 AND HNP-2

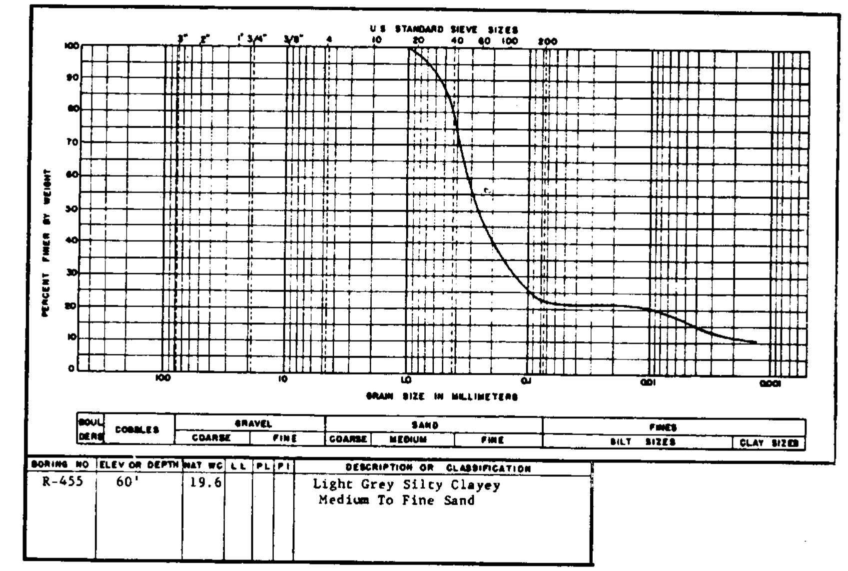
FIGURE 2A-8 (SHEET 30 OF 34)



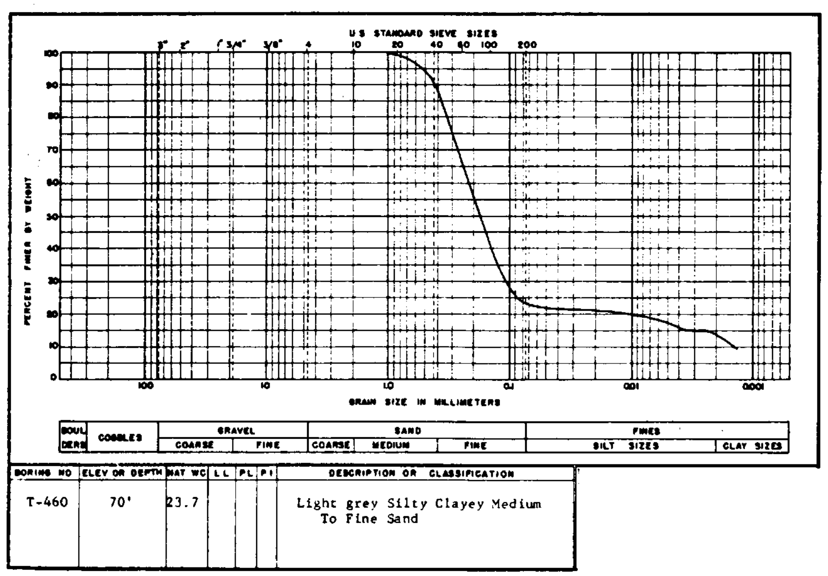
Unit 1 Elevation 75.4



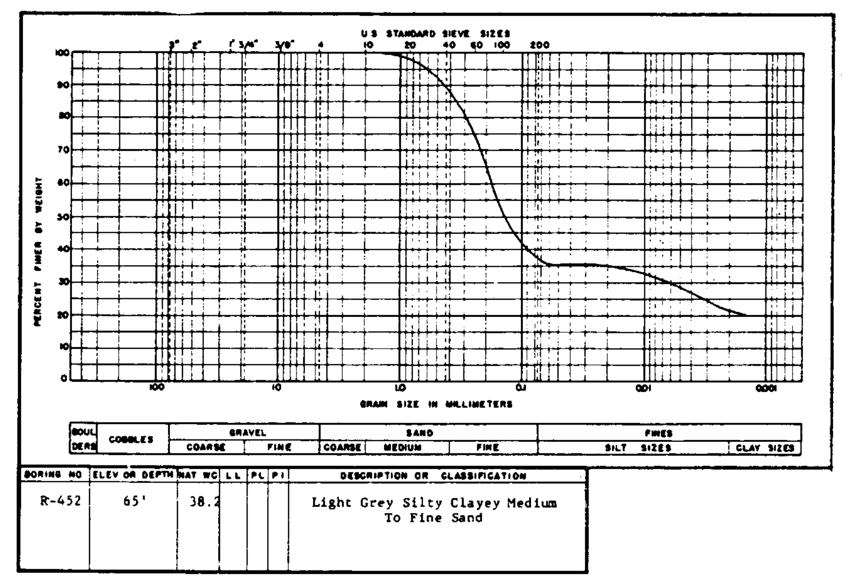
Unit 2 Elevation 73.5



Unit 1 Elevation 73.0



Unit 1 Elevation 72.5



Unit 1 Elevation 72.2

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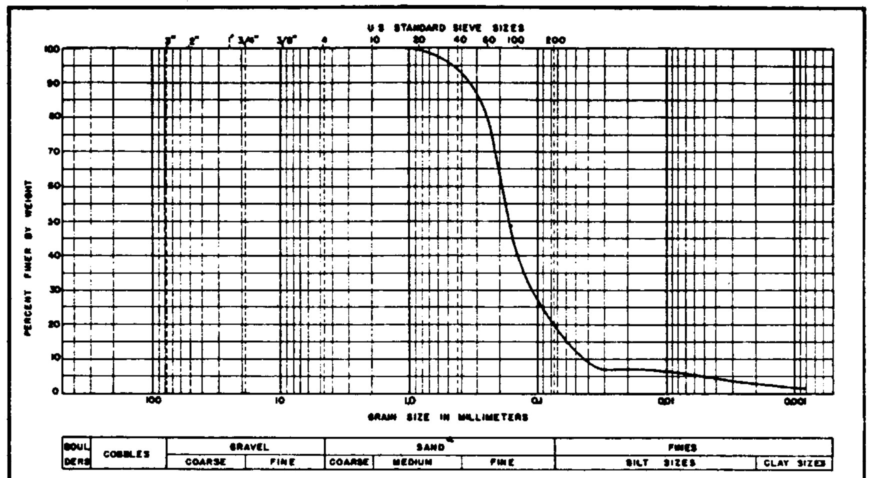
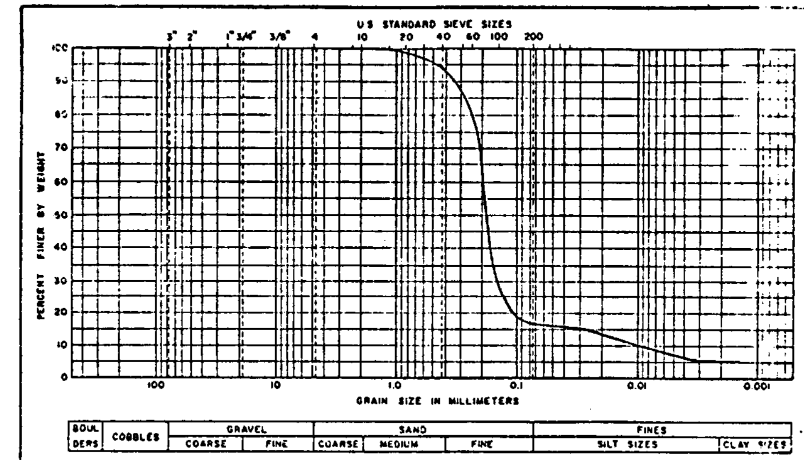
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	COMPARISON OF HNP-1 AND HNP-2 GRAIN SIZE CURVES BETWEEN el 72 ft AND 76 ft FIGURE 2A-8 (SHEET 31 OF 34)



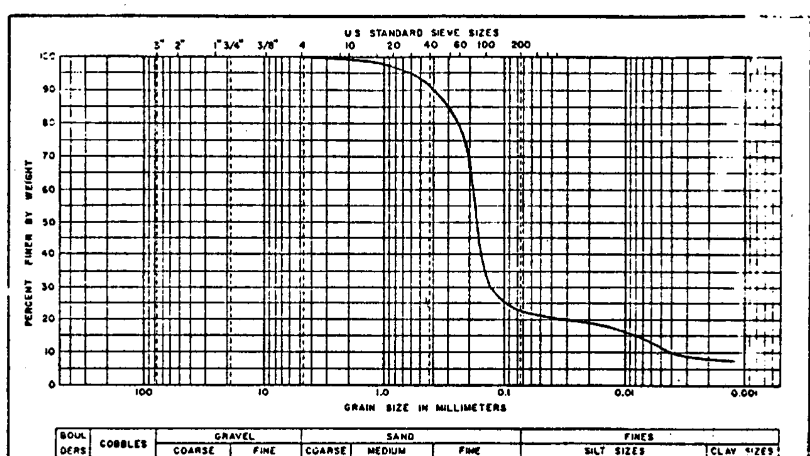
BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
R-449	76.5'	29.1				Light Grey Clayey Silty Medium To Fine Sand

Unit 1 Elevation 63.9



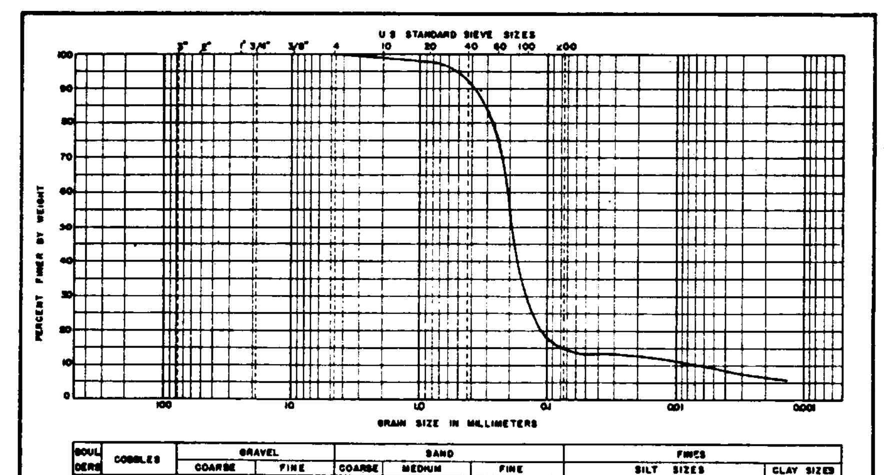
BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
R-588 S#13	64.65.5'					Light grey slightly clayey slightly silty medium to fine sand

Unit 2 Elevation 63.5



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
T-600 S#13	64-65.5'					Green grey slightly clayey silty medium to fine sand

Unit 2 Elevation 63.1



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
R-455	70'	26.6				Light Grey Clayey Silty Medium To Fine Sand

Unit 1 Elevation 63.0

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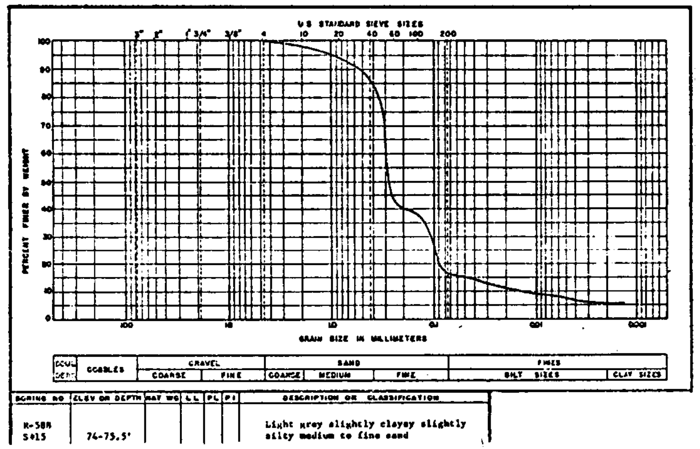
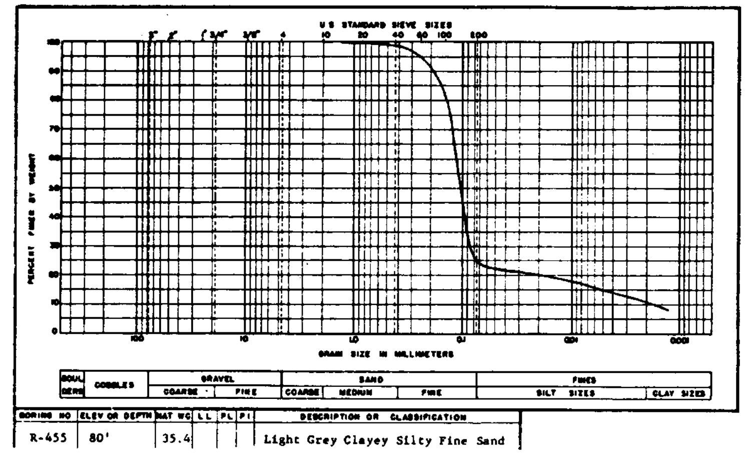
UNIT 2

COMPARISON OF HNP-1 AND HNP-2 GRAIN SIZE CURVES BETWEEN el 63 ft AND 64 ft

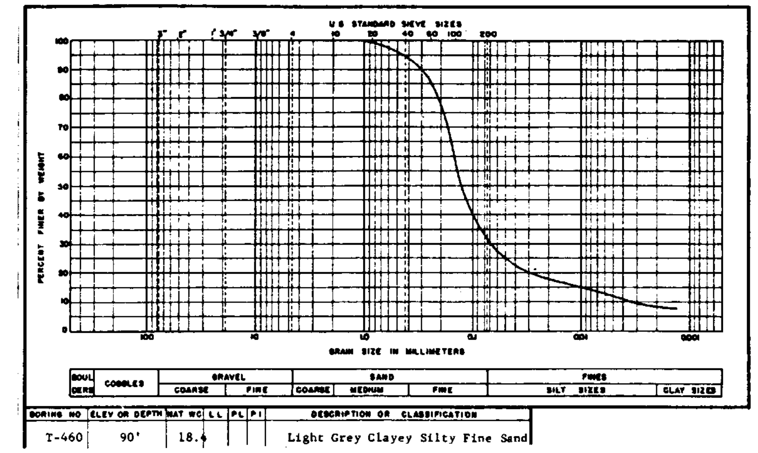
FIGURE 2A-8 (SHEET 32 OF 34)



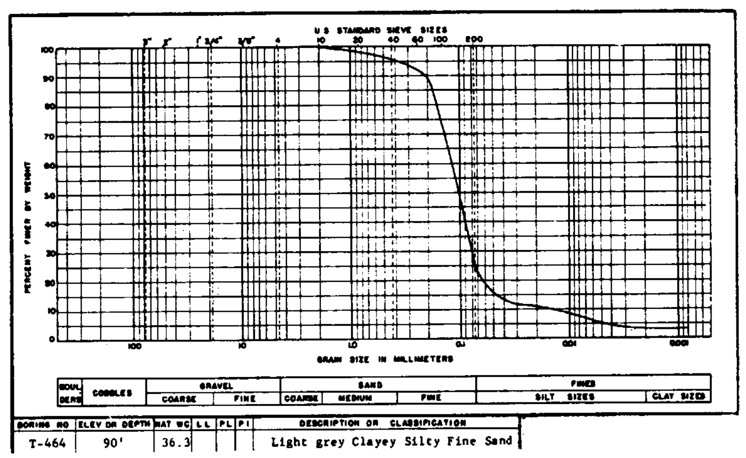
Unit 2 Elevation 53.5



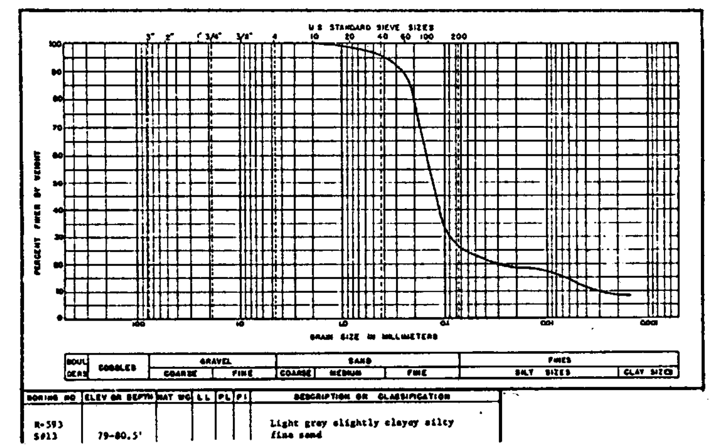
Unit 1 Elevation 53.0



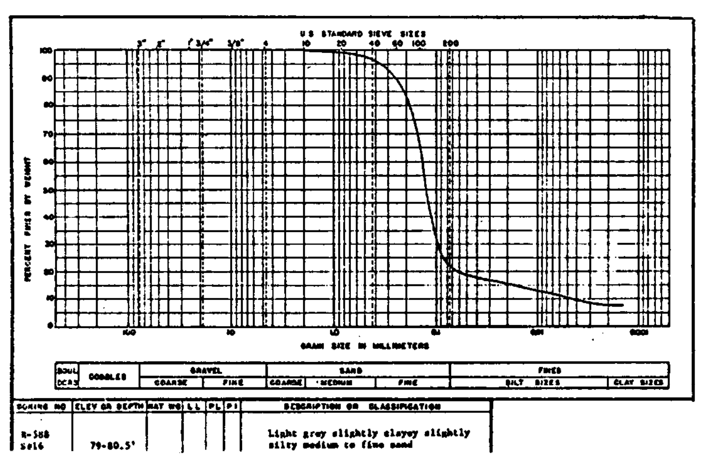
Unit 1 Elevation 52.5



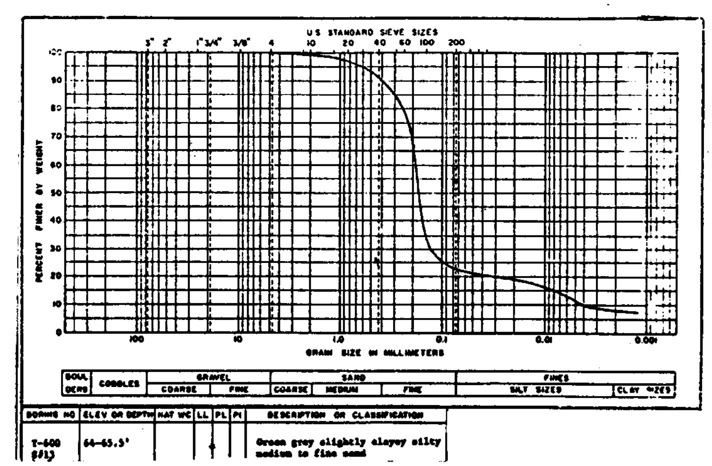
Unit 1 Elevation 51.7



Unit 2 Elevation 49.1



Unit 2 Elevation 48.5



Unit 2 Elevation 48.1

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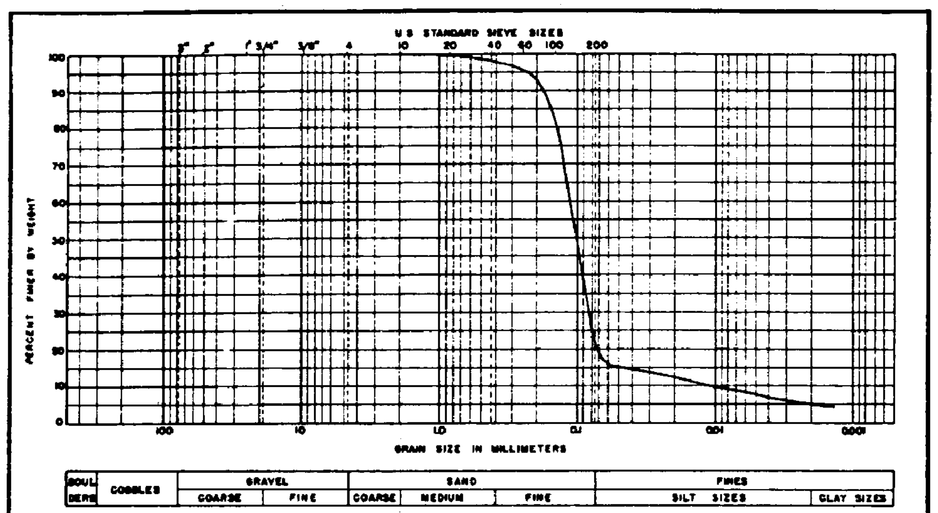
UNIT 2

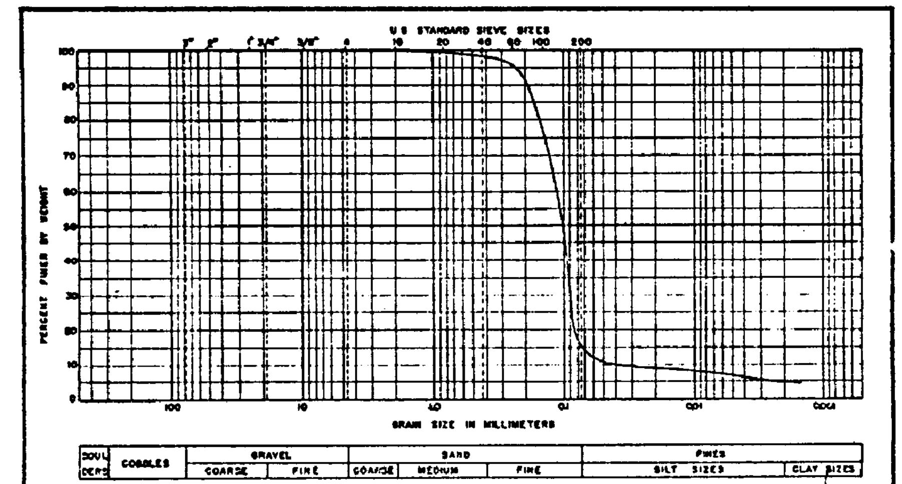
COMPARISON OF HNP-1 AND HNP-2 GRAIN SIZE CURVES BETWEEN el 48 ft AND 54 ft

FIGURE 2A-8 (SHEET 33 OF 34)



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
R-455	95'	37.1				Light Grey Clayey Silty Fine Sand

Unit 1 Elevation 38.0



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
R-593 S#16	94-95.5					Light grey slightly clayey slightly silty fine sand

Unit 2 Elevation 34.1

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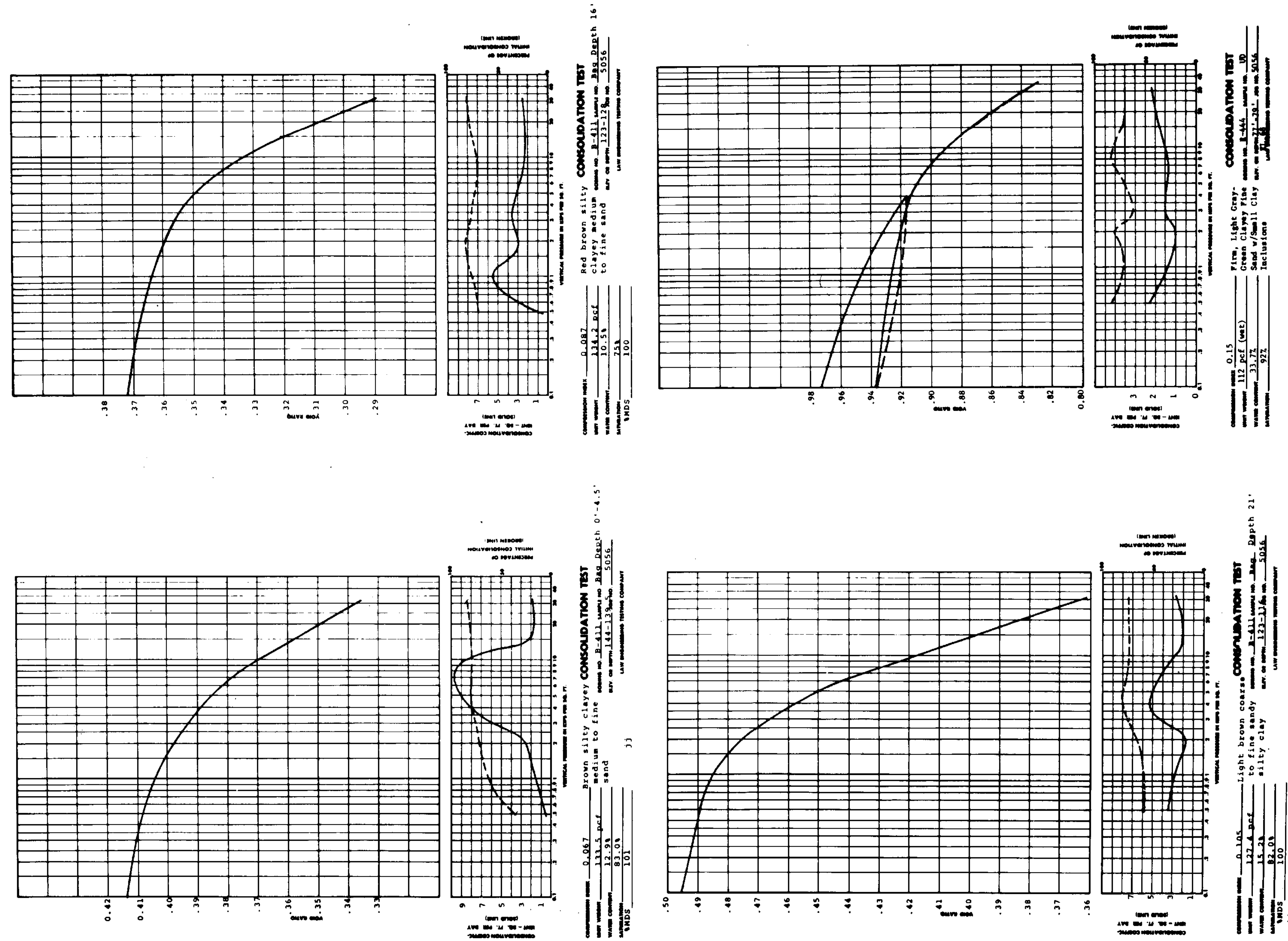
COMPARISON OF HNP-1 AND HNP-2 GRAIN SIZE CURVES BETWEEN el 34 ft AND 38 ft

FIGURE 2A-8 (SHEET 34 OF 34)



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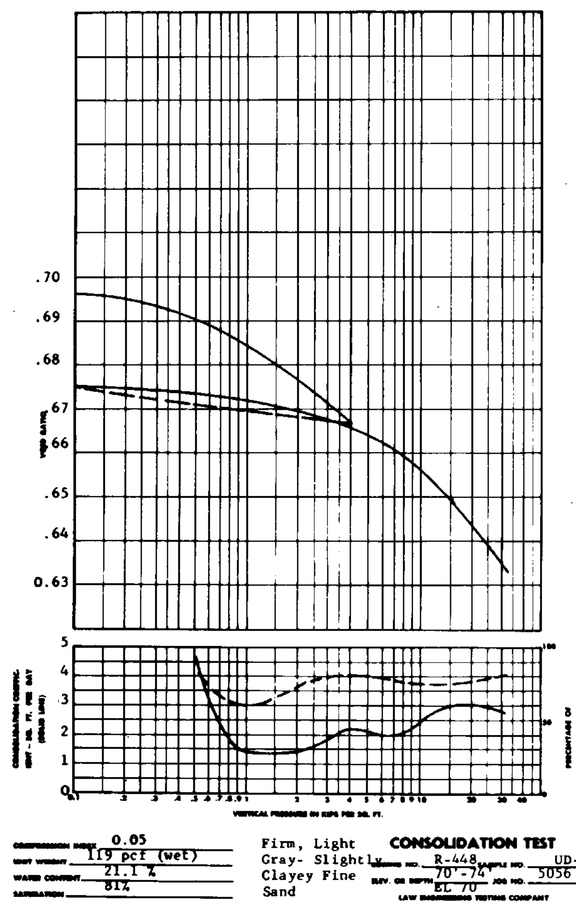
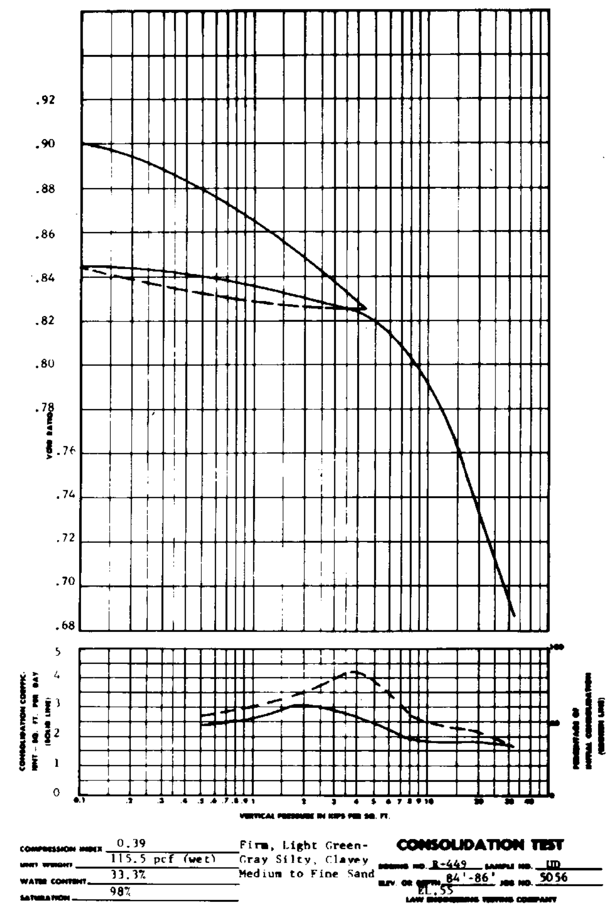
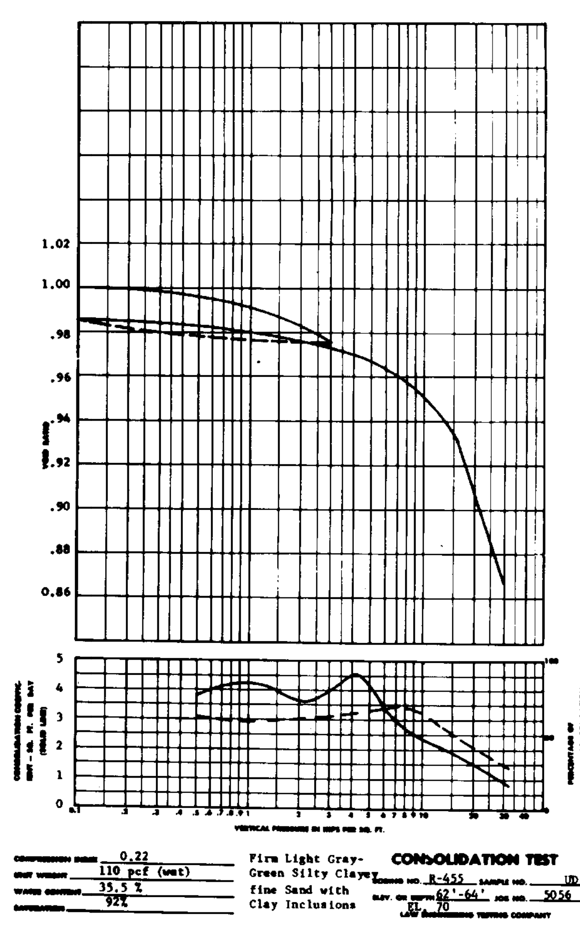
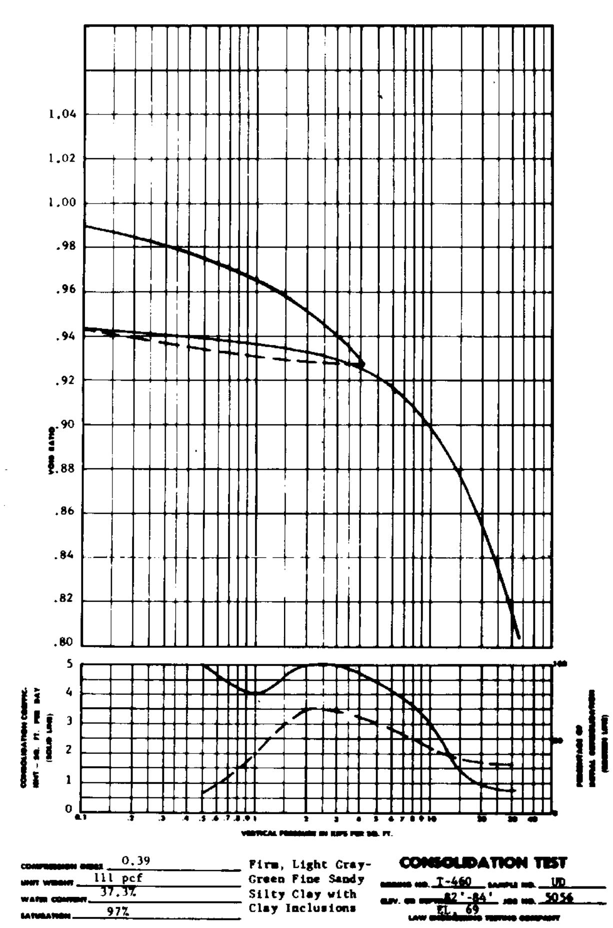
CONSOLIDATION TEST RESULTS

FIGURE 2A-9 (SHEET 1 OF 4)









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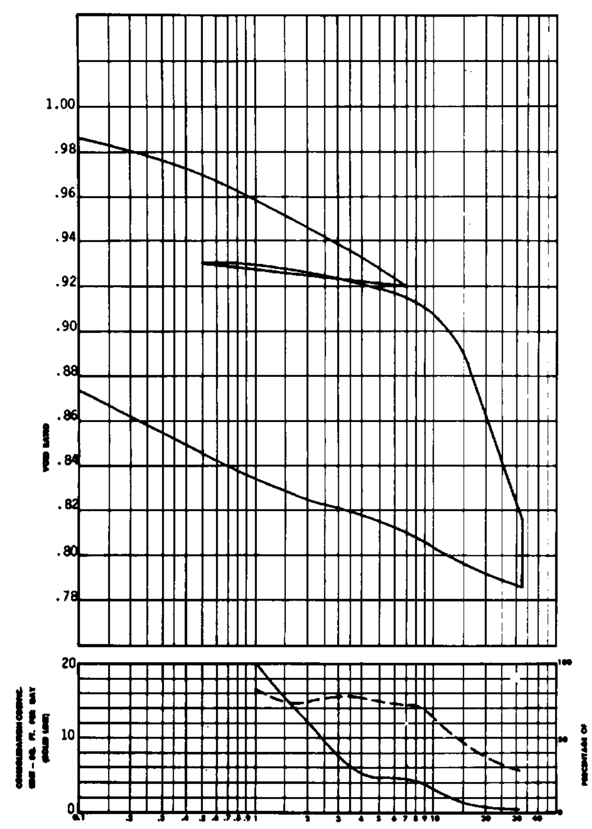
CONSOLIDATION TEST RESULTS

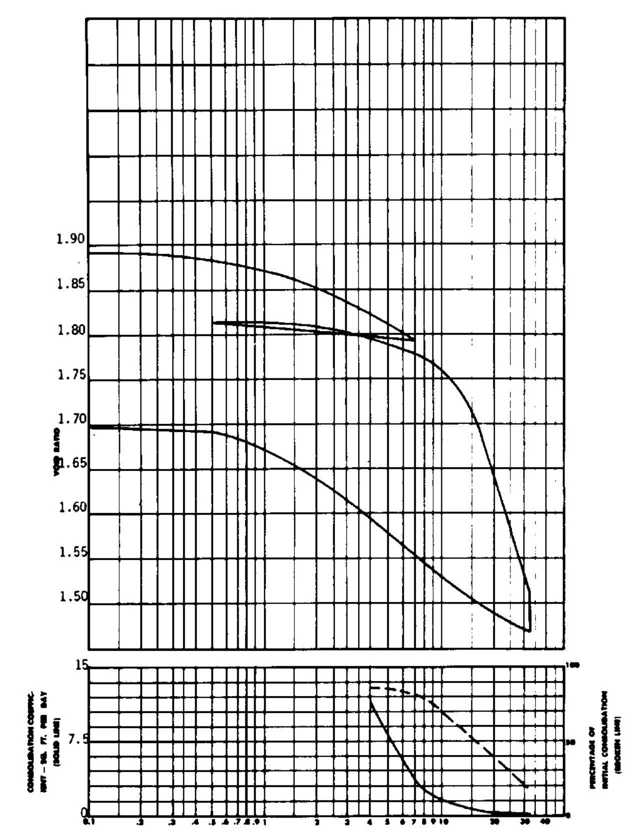
FIGURE 2A-9 (SHEET 2 OF 4)



GRAY-GREEN VERY CLAYEY FINE SAND

COMPRESSION INDEX 0.23
UNIT WEIGHT (Dry) 82.6 pcf, (Wet) 110.5 pcf
WATER CONTENT Wo 33.8%, Wf 33.1% L.L. = 110
SATURATION So 90.1%, Sf 99.5% P.L. = 51
SPECIFIC GRAVITY 2.63 P.I. = 59

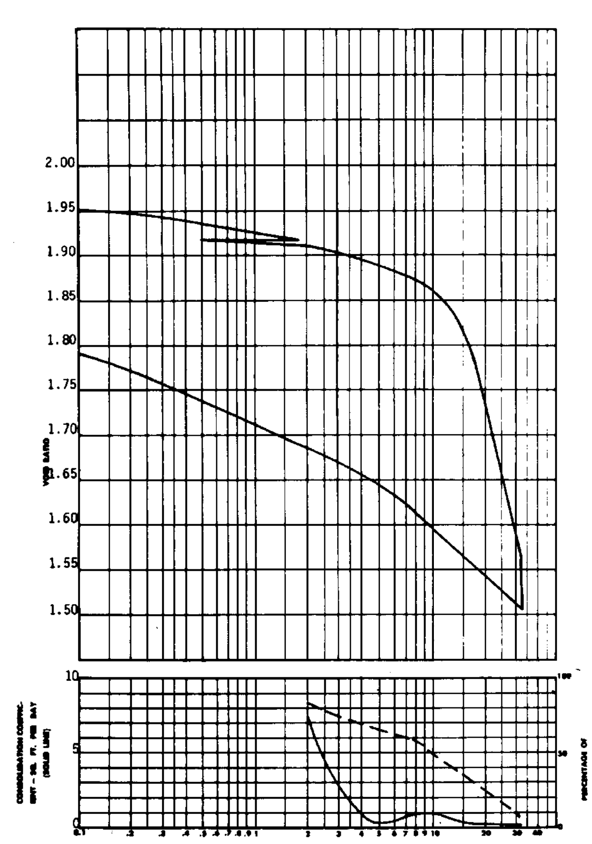
CONSOLIDATION TEST
BORING NO. R-594 SAMPLE NO. UD-1
ELEV. OR DEPTH 61.5 JOB NO. 5056
LAW ENGINEERING TESTING COMPANY
Elev. 66.7



GRAY-GREEN FINE SANDY CLAY

COMPRESSION INDEX 0.61
UNIT WEIGHT (Dry) 57.0 pcf, (Wet) 95.2 pcf
WATER CONTENT Wo 67.0%, Wf 64.7% L.L. = 88
SATURATION SO 93.6%, Sf 100% P.L. = 38
SPECIFIC GRAVITY 2.64 P.I. = 50

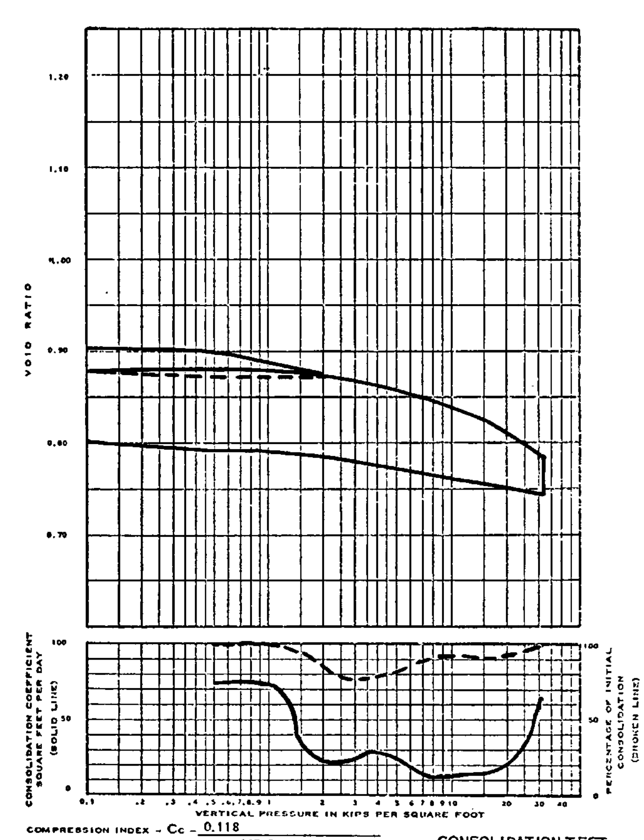
CONSOLIDATION TEST
BORING NO. T-597 SAMPLE NO. UD-1
ELEV. OR DEPTH 62.25 JOB NO. 5056
LAW ENGINEERING TESTING COMPANY
Elev. 63.85



GRAY-GREEN SLIGHTLY FINE SANDY CLAY

COMPRESSION INDEX 0.84
UNIT WEIGHT (Dry) 56.0 pcf, (Wet) 95.5 pcf
WATER CONTENT Wo 70.5%, Wf 67.1% L.L. = 64
SATURATION SO 95.7%, Sf 99.3% P.L. = 31
SPECIFIC GRAVITY 2.65 P.I. = 33

CONSOLIDATION TEST
BORING NO. R-587 SAMPLE NO. UD-1
ELEV. OR DEPTH 17.75 JOB NO. 5056
LAW ENGINEERING TESTING COMPANY
Elev. 67.25



COMPRESSION INDEX - Cc - 0.118
UNIT WEIGHT (PCF) - γM (INITIAL) 115.0 γM (FINAL) 119.1
γD (INITIAL) 88.6 γD (FINAL) 93.6
WATER CONTENT (%) - W (INITIAL) 29.8 W (FINAL) 27.3
SATURATION (%) - S (INITIAL) 89 S (FINAL) 92
POROSITY 0.474 SPECIFIC GRAVITY 2.70
DESCRIPTION LIGHT GRAY CLAYEY FINE SAND WITH CLAY INCLUSIONS
N=12

CONSOLIDATION TEST
SAMPLE LOCATION RFI-2
ELEVATION OR DEPTH 20'-22'
JOB NUMBER 5056

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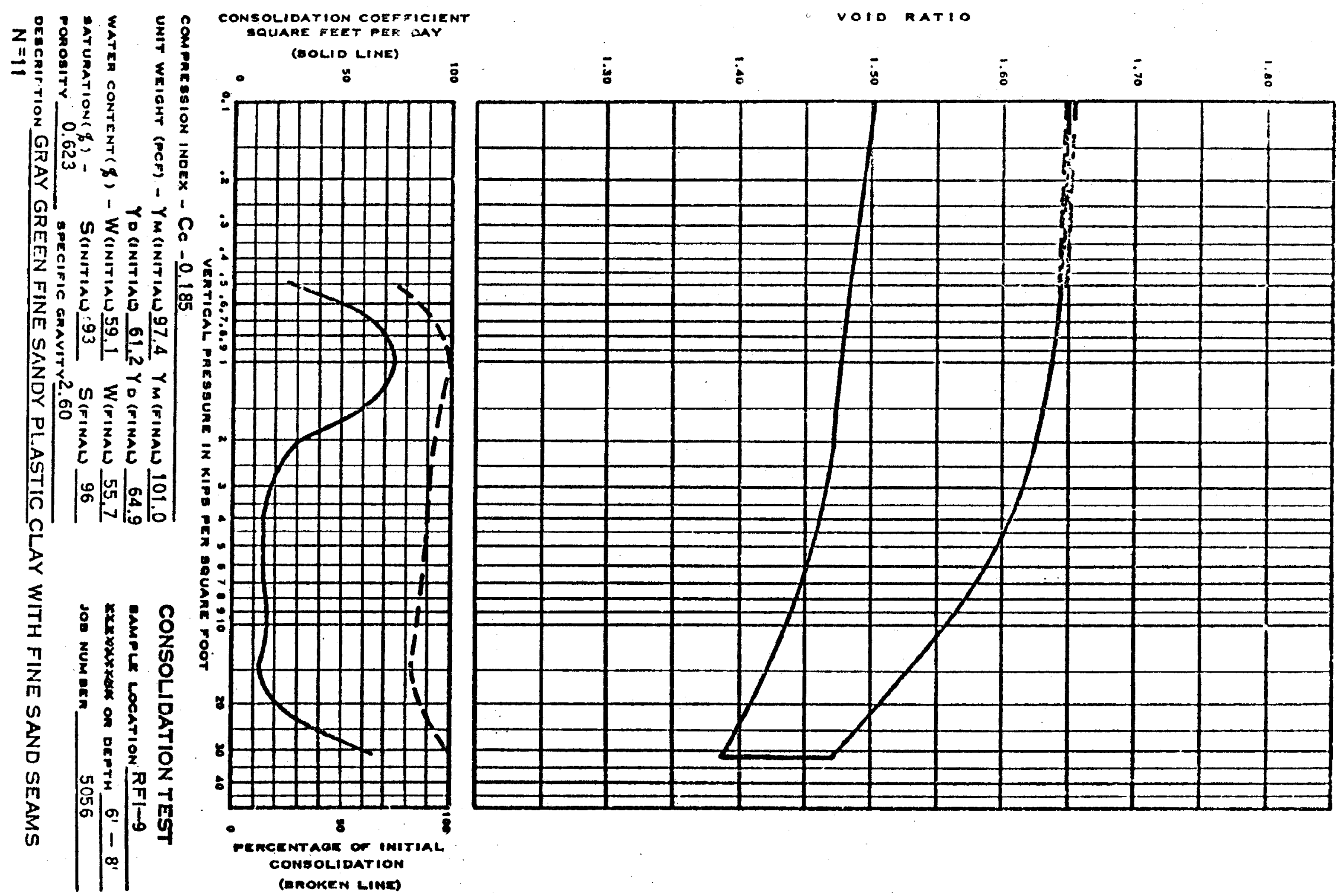
CONSOLIDATION TEST RESULTS

FIGURE 2A-9 (SHEET 3 OF 4)



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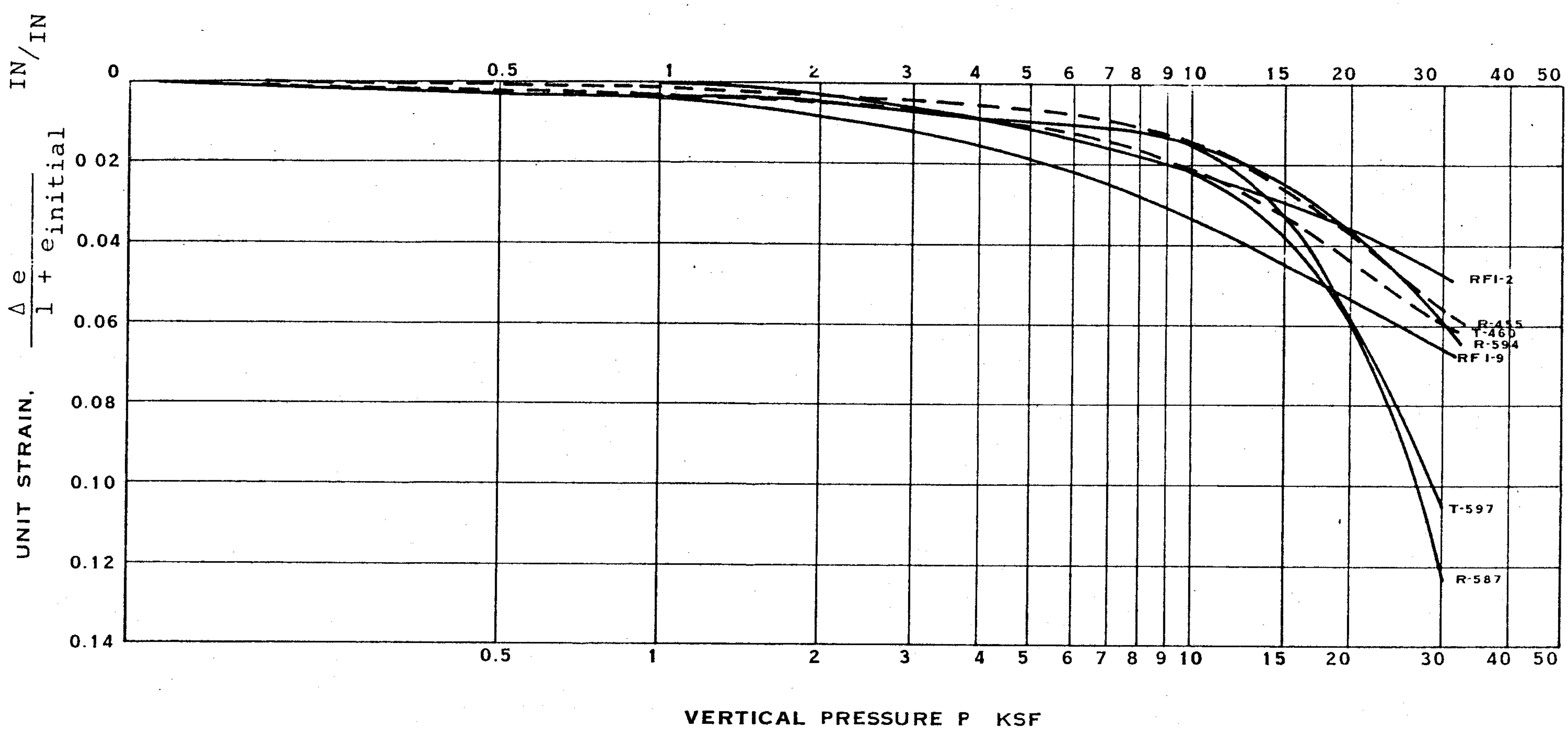
CONSOLIDATION TEST RESULTS

FIGURE 2A-9 (SHEET 4 OF 4)



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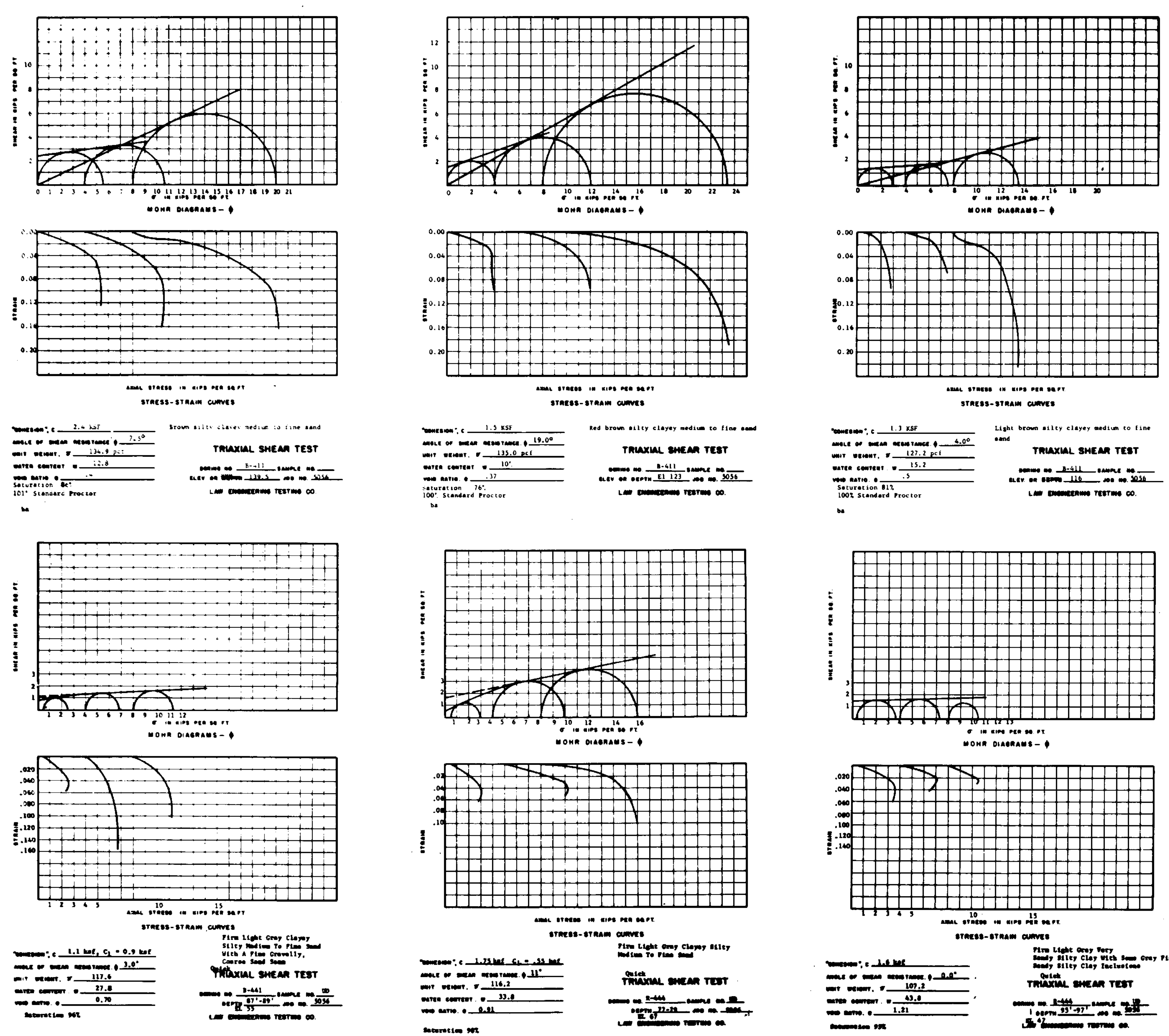
COMPARISON OF CONSOLIDATION TEST CURVES FOR HNP-1 AND HNP-2

FIGURE 2A-10



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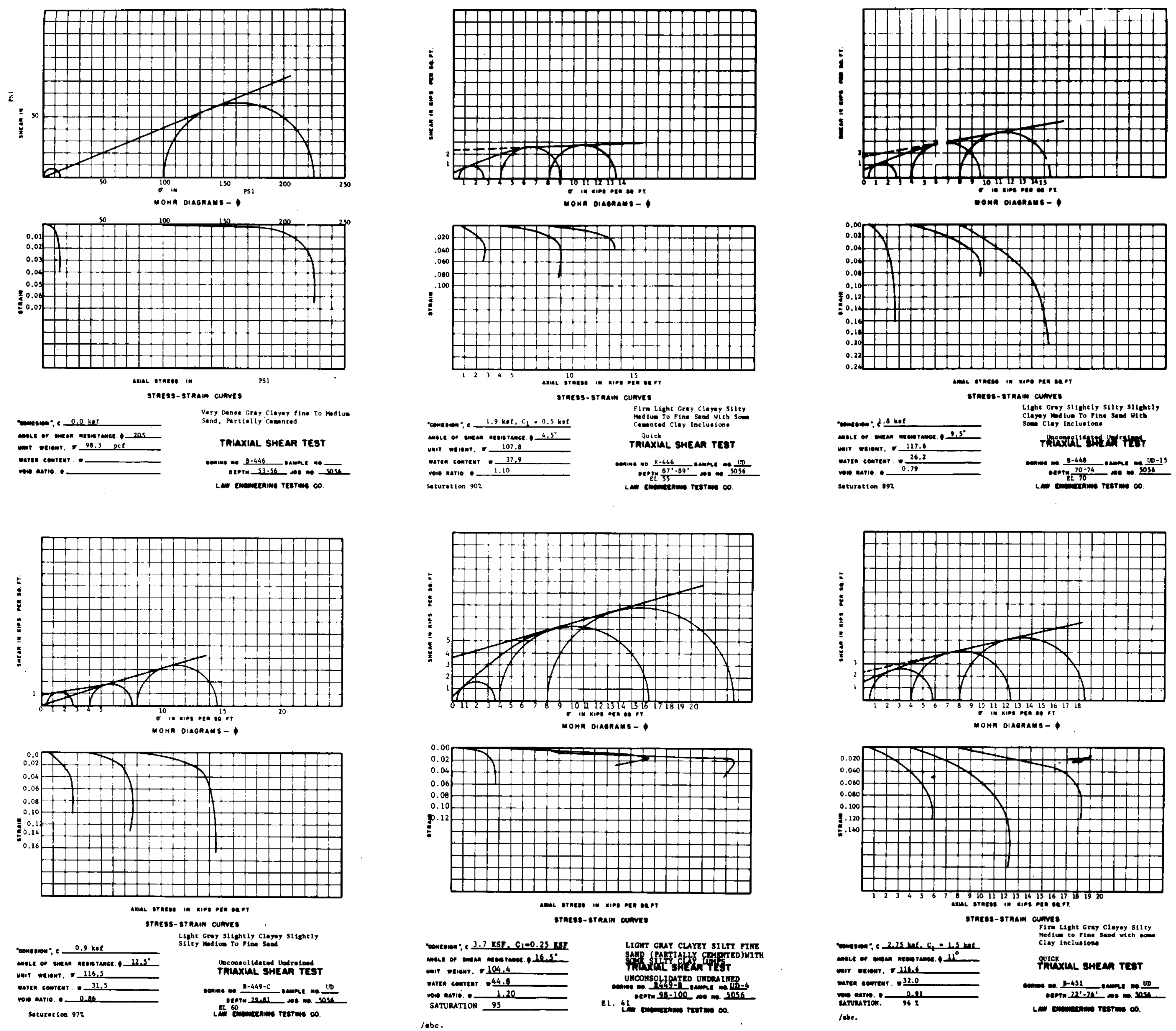
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TRIAXIAL SHEAR TEST RESULTS

FIGURE 2A-11 (SHEET 1 OF 5)



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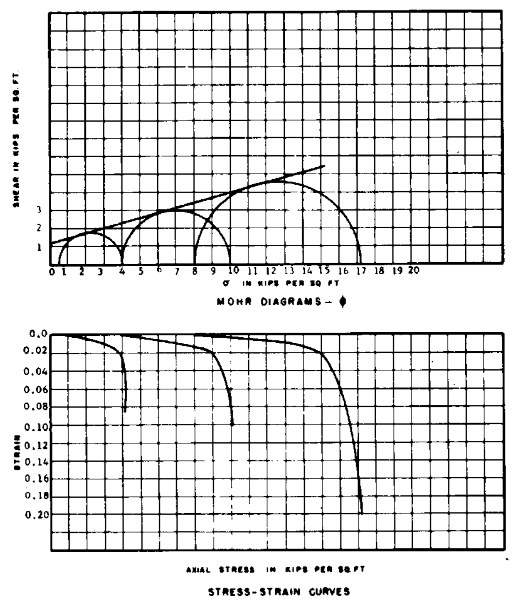
SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TRIAXIAL SHEAR TEST RESULTS
	FIGURE 2A-11 (SHEET 2 OF 5)

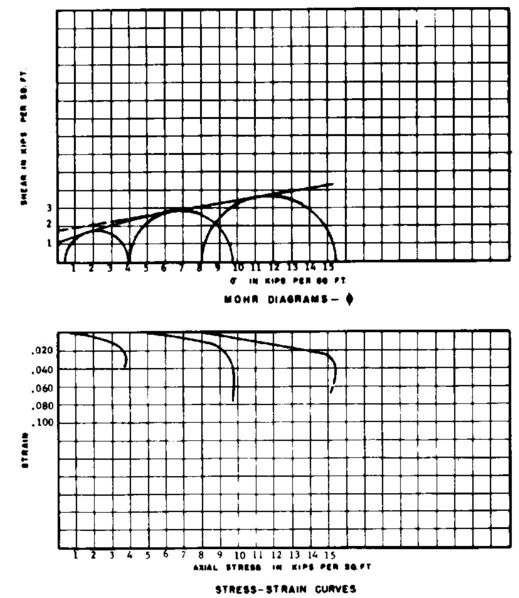


"COHESION", c 1.2 ksf
ANGLE OF SHEAR RESISTANCE φ 16.0°
UNIT WEIGHT, γ 129.6
WATER CONTENT w 16.6
VOID RATIO, e 0.52
Saturation 87%

Light Grey, Red-Brown, and Yellowish Brown Silty Clayey Medium To Fine Sand

Unconsolidated Undrained
TRIAXIAL SHEAR TEST

BORING NO R-452-B SAMPLE NO UD-1
DEPTH 9-11 JOB NO. 5056
EL. 127
LAW ENGINEERING TESTING CO.

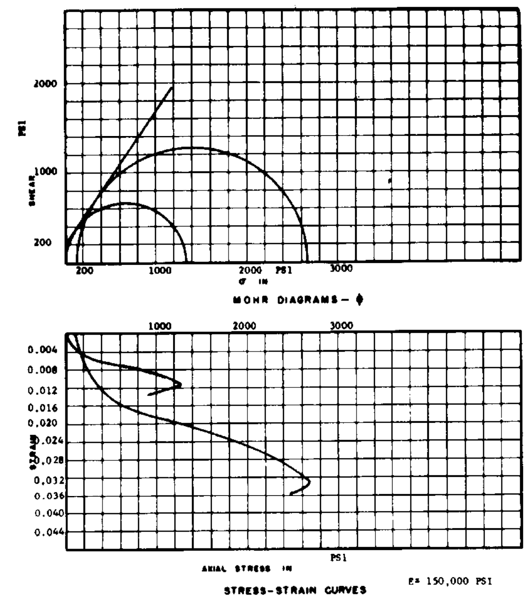


"COHESION", c 1.85ksf, C_1 = 1.05 ksf
ANGLE OF SHEAR RESISTANCE φ 9.5°
UNIT WEIGHT, γ 105.8
WATER CONTENT w 44.7
VOID RATIO, e 1.26
Saturation 94%

Firm Light Gray Clayey Silty Medium To Fine Sand With Some Cemented Clay Inclusions

Quick
TRIAXIAL SHEAR TEST

BORING NO R-452 SAMPLE NO UD
DEPTH 85'-87' JOB NO. 5056
EL 51
LAW ENGINEERING TESTING CO.

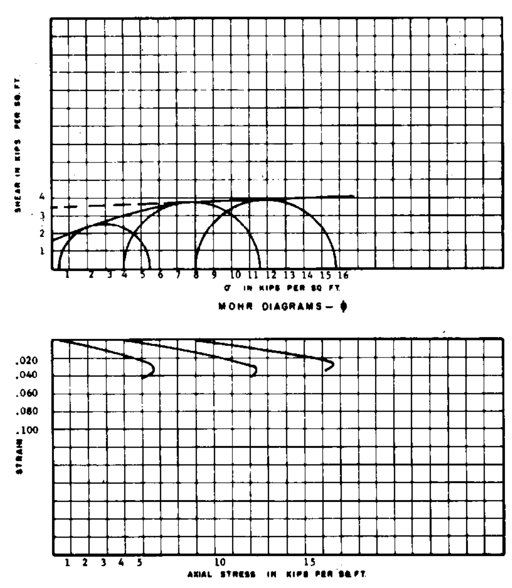


"COHESION", c 0.2
ANGLE OF SHEAR RESISTANCE φ 59°
UNIT WEIGHT, γ 138.9
WATER CONTENT w
VOID RATIO, e

Very Dense Tan and Gray Clayey Silt Sand, Partially Cemented

TRIAXIAL SHEAR TEST

BORING NO. B-456 SAMPLE NO.
DEPTH 23'-25' JOB NO. 5056
LAW ENGINEERING TESTING CO.

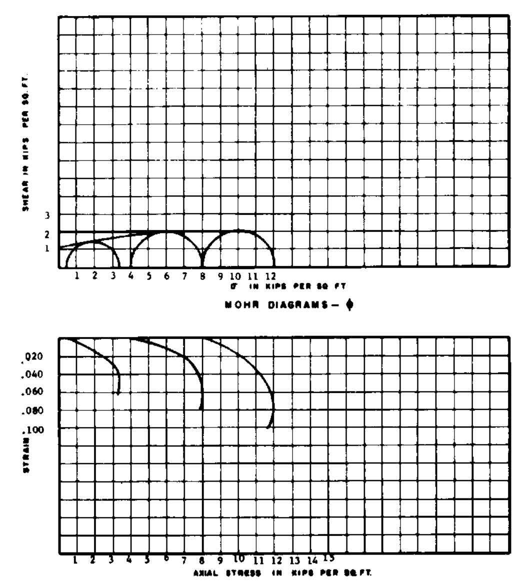


"COHESION", c 3.3 ksf, C_1 = 1.6 ksf
ANGLE OF SHEAR RESISTANCE φ 3.0°
UNIT WEIGHT, γ 95.2
WATER CONTENT, w 77.7
VOID RATIO, e 1.44
Saturation 100%

Stiff Light Gray Slightly Fine Sandy Silty Clay With Fine Sand Seams

Quick
TRIAXIAL SHEAR TEST

BORING NO. B-456 SAMPLE NO. UD
DEPTH 42'-44' JOB NO. 5056
EL 63
LAW ENGINEERING TESTING CO.

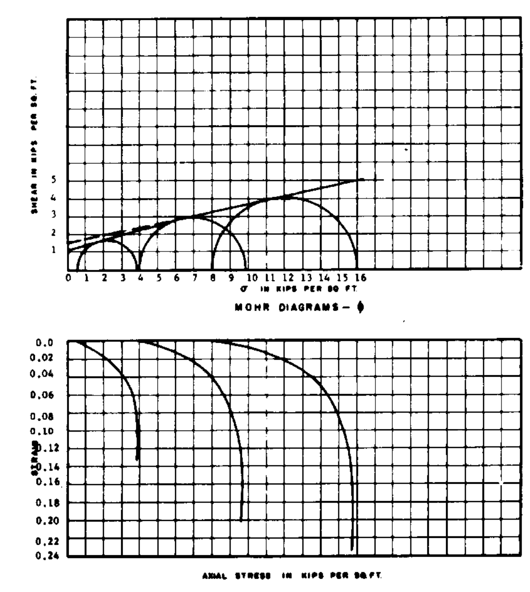


"COHESION", c 2.0 ksf, C_1 = 1.2 ksf
ANGLE OF SHEAR RESISTANCE φ 0.0°
UNIT WEIGHT, γ 110.2
WATER CONTENT, w 39.4
VOID RATIO, e 1.07
Saturation 98%

Firm Light Gray Clayey Silty Fine Sand With Gray Silty Clay Inclusions

Quick
TRIAXIAL SHEAR TEST

BORING NO. B-456 SAMPLE NO. UD
DEPTH 44'-46' JOB NO. 5056
EL. 61
LAW ENGINEERING TESTING CO.



"COHESION", c 1.6 ksf; c_1 = 1.1ksf
ANGLE OF SHEAR RESISTANCE φ 11.5°
UNIT WEIGHT, γ 118.8
WATER CONTENT, w 29.2
VOID RATIO, e 0.81
Saturation 96%

Light Grey Slightly Clayey Slightly Silty Fine Sand With Some Greenish Grey Silty Clay

TRIAXIAL SHEAR TEST

BORING NO. B-456 SAMPLE NO. UD-1
DEPTH 56-58 JOB NO. 5056
EL. 62
LAW ENGINEERING TESTING CO.

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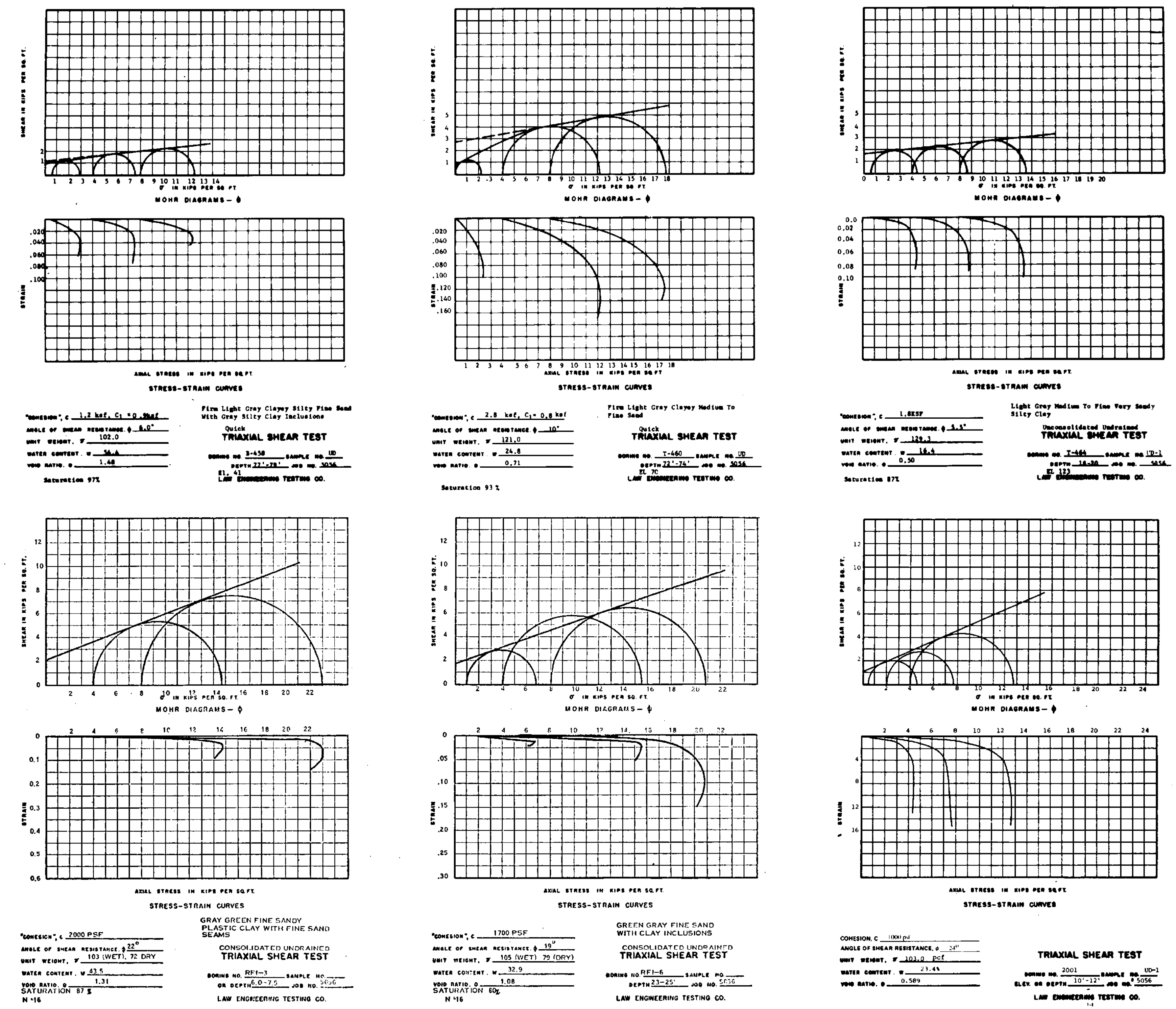
SOUTHERN COMPANY Energy to Serve Your World®	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TRIAXIAL SHEAR TEST RESULTS FIGURE 2A-11 (SHEET 3 OF 5)



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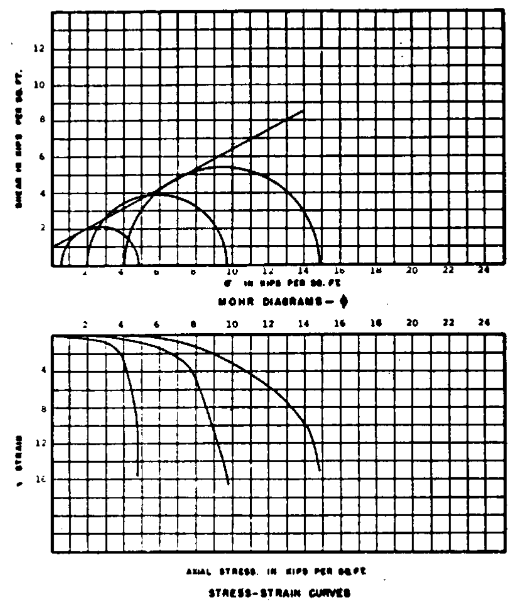
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

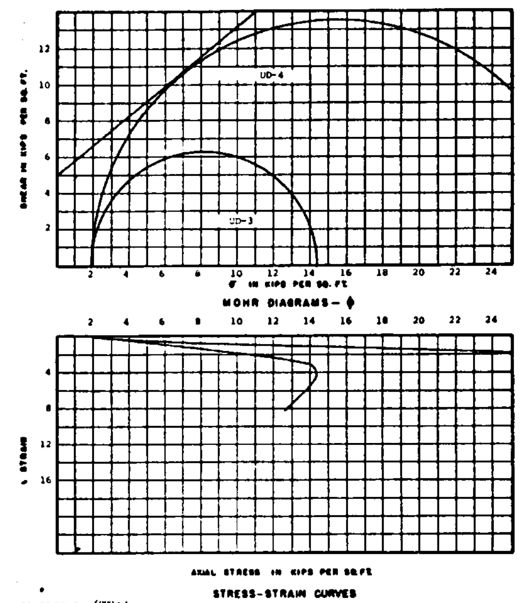
TRIAXIAL SHEAR TEST RESULTS

FIGURE 2A-11 (SHEET 4 OF 5)



COHESION, C 1000 psf
ANGLE OF SHEAR RESISTANCE, φ [illegible]
UNIT WEIGHT, γ 112.9 pcf
WATER CONTENT, w 15.7%
VOID RATIO, e 0.468

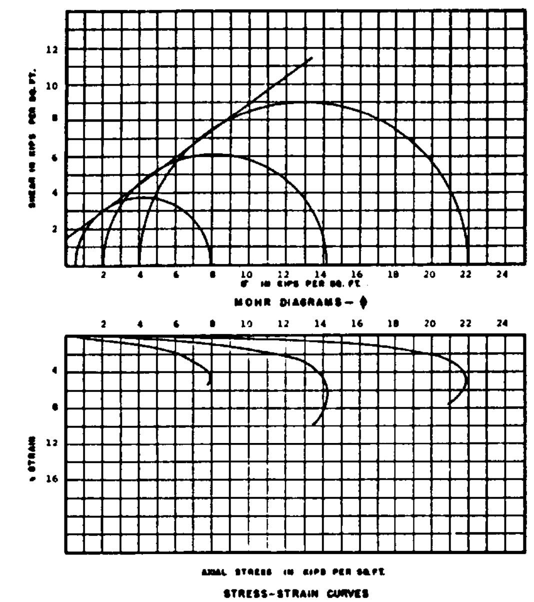
TRIAXIAL SHEAR TEST
BORING NO. 2001 SAMPLE NO. UD-2
ELEV. OR DEPTH 20'-22' JOB NO. #5056
LAW ENGINEERING TESTING CO.



COHESION, C 5000 psf
ANGLE OF SHEAR RESISTANCE, φ 40°

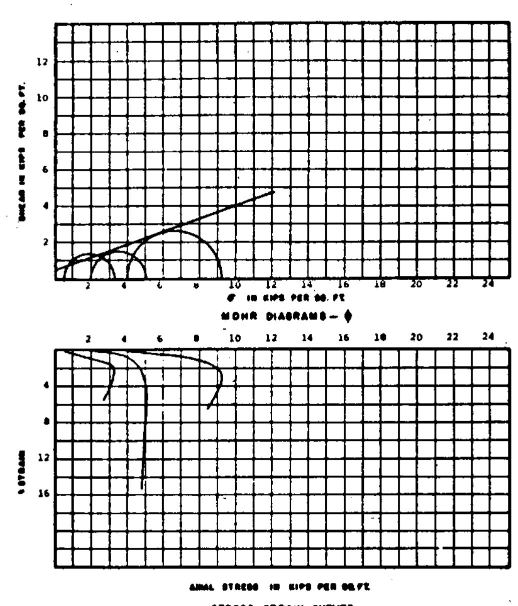
	UD-3	UD-4
UNIT WEIGHT, γ	106.1 pcf	111.7 pcf
WATER CONTENT, w	19.2%	14.5%
VOID RATIO, e	0.576	0.497

TRIAXIAL SHEAR TEST
BORING NO. 2001 SAMPLE NO. UD-3 & UD-4
ELEV. OR DEPTH 30'-32.5' 40'-42.5' JOB NO. #5056
LAW ENGINEERING TESTING CO.



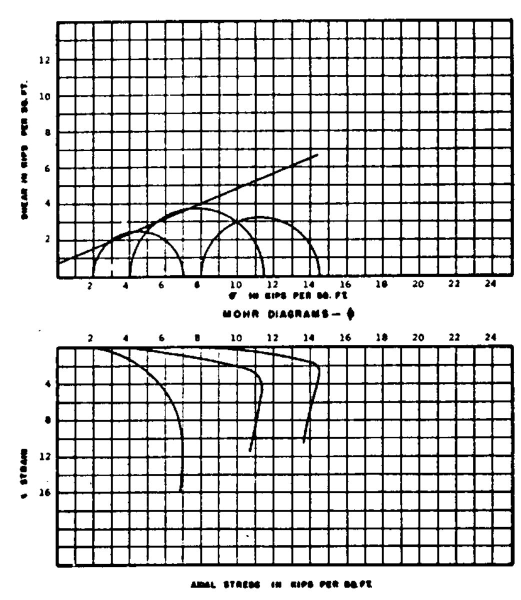
COHESION, C 1500 psf
ANGLE OF SHEAR RESISTANCE, φ 36°
UNIT WEIGHT, γ 107.7 pcf
WATER CONTENT, w 19.4%
VOID RATIO, e 0.566

TRIAXIAL SHEAR TEST
BORING NO. 2001 SAMPLE NO. UD-5
ELEV. OR DEPTH 50'-52.5' JOB NO. #5056
LAW ENGINEERING TESTING CO.



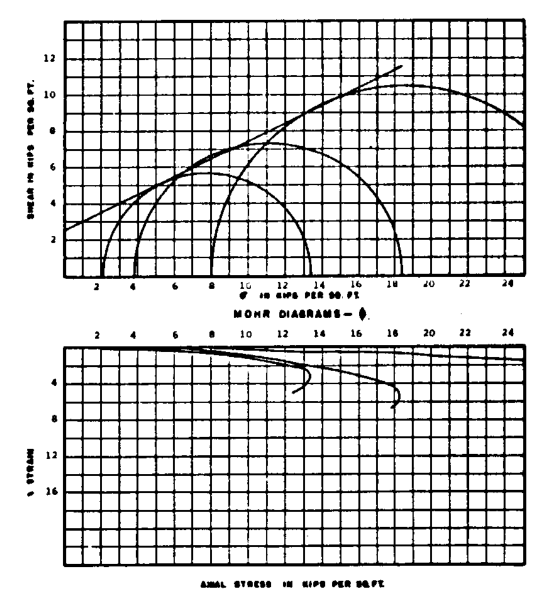
COHESION, C 300 psf
ANGLE OF SHEAR RESISTANCE, φ 20°
UNIT WEIGHT, γ 67.6 pcf
WATER CONTENT, w 48.2%
VOID RATIO, e 1.456

TRIAXIAL SHEAR TEST
BORING NO. 2001 SAMPLE NO. UD-9
ELEV. OR DEPTH 75'-77.5' JOB NO. #5056
LAW ENGINEERING TESTING CO.



COHESION, C 900 psf
ANGLE OF SHEAR RESISTANCE, φ 22°
UNIT WEIGHT, γ 72.7 pcf
WATER CONTENT, w 42.5%
VOID RATIO, e 1.315

TRIAXIAL SHEAR TEST
BORING NO. 2001 SAMPLE NO. UD-10
ELEV. OR DEPTH 80'-82.5' JOB NO. #5056
LAW ENGINEERING TESTING CO.



COHESION, C 2500 psf
ANGLE OF SHEAR RESISTANCE, φ 26°
UNIT WEIGHT, γ 82.4 pcf
WATER CONTENT, w 30.2%
VOID RATIO, e 1.008

TRIAXIAL SHEAR TEST
BORING NO. 2001 SAMPLE NO. UD-13
ELEV. OR DEPTH 100'-102.5' JOB NO. #5056
LAW ENGINEERING TESTING CO.

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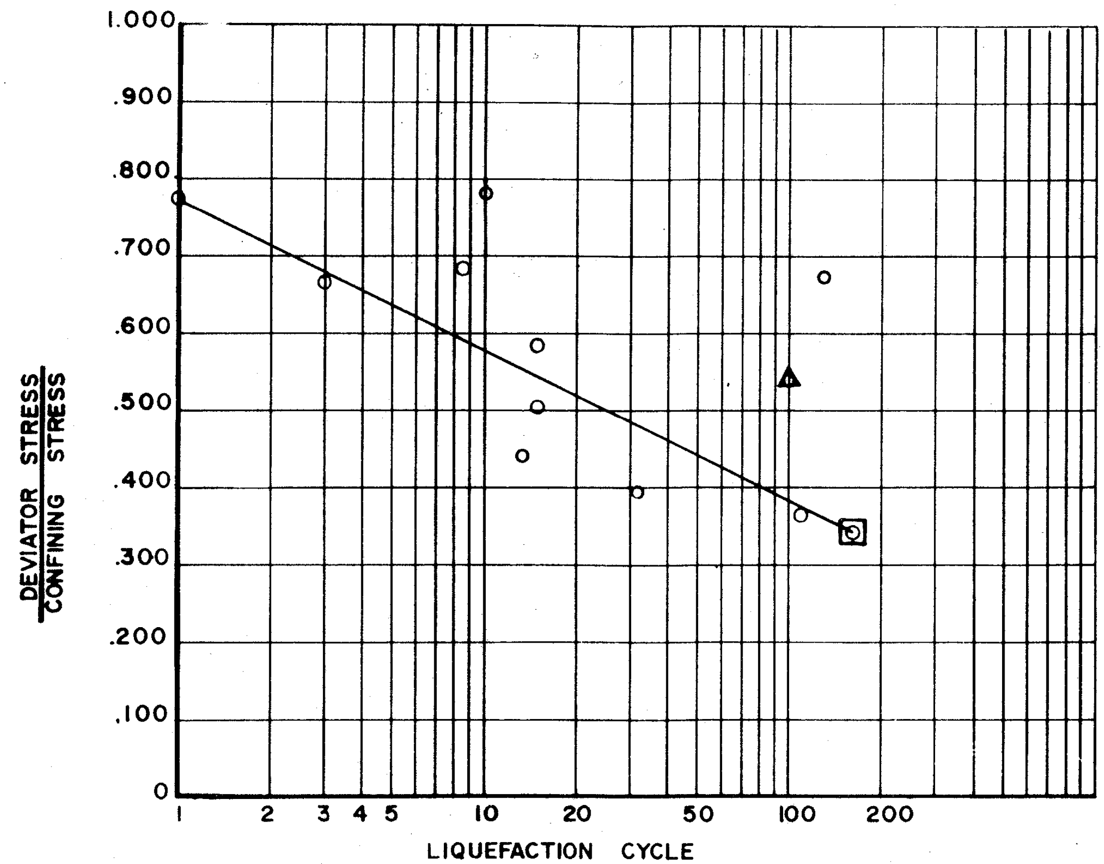
UNIT 2

TRIAXIAL SHEAR TEST RESULTS

FIGURE 2A-11 (SHEET 5 OF 5)



LEGEND

△ NO LIQUEFACTION AT 100 CYCLES

□ NO LIQUEFACTION AT 160 CYCLES

$\sigma_d = .580\ \overline{\sigma_0}$ AT 10 CYCLES

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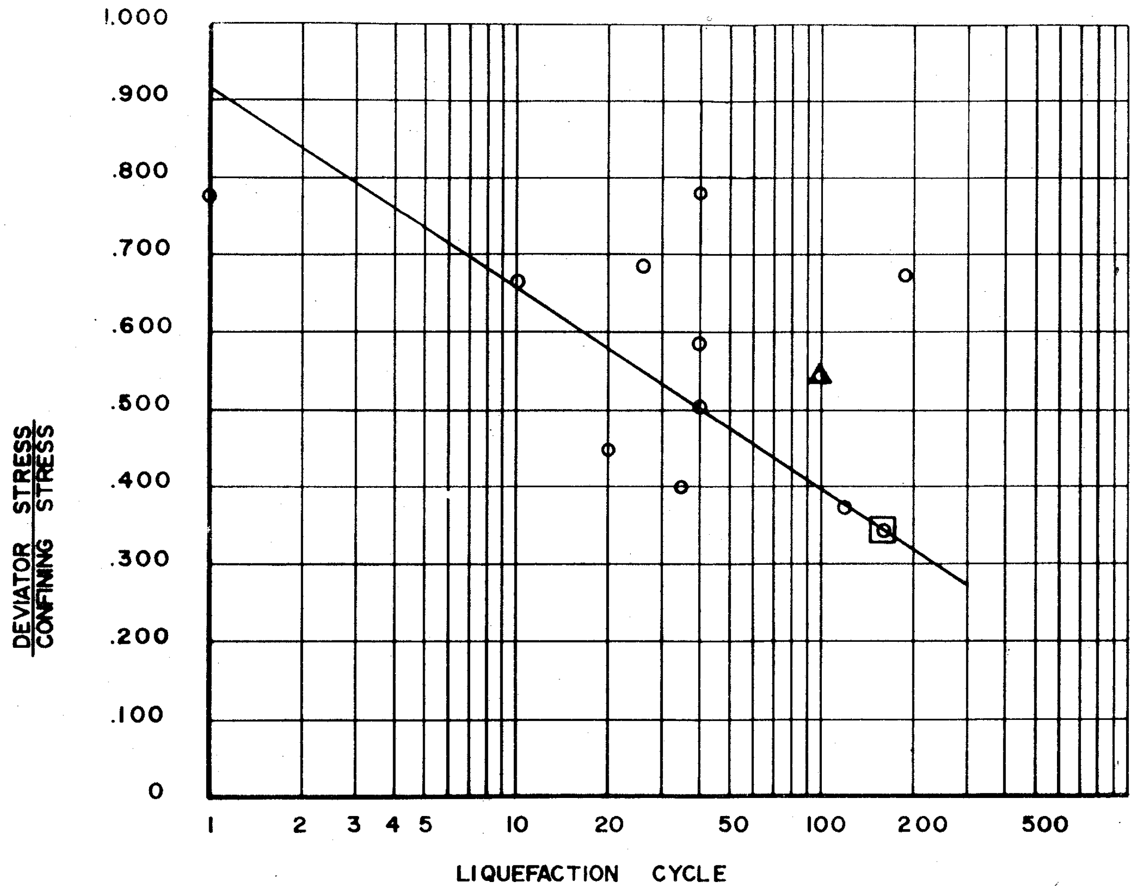
UNIT 2

INITIAL-TO-PARTIAL LIQUEFACTION

FIGURE 2A-12



LEGEND

△ NO LIQUEFACTION AT 100 CYCLES

□ NO LIQUEFACTION AT 160 CYCLES

σ_d =.660 $\overline{\sigma_o}$ AT 10 CYCLES

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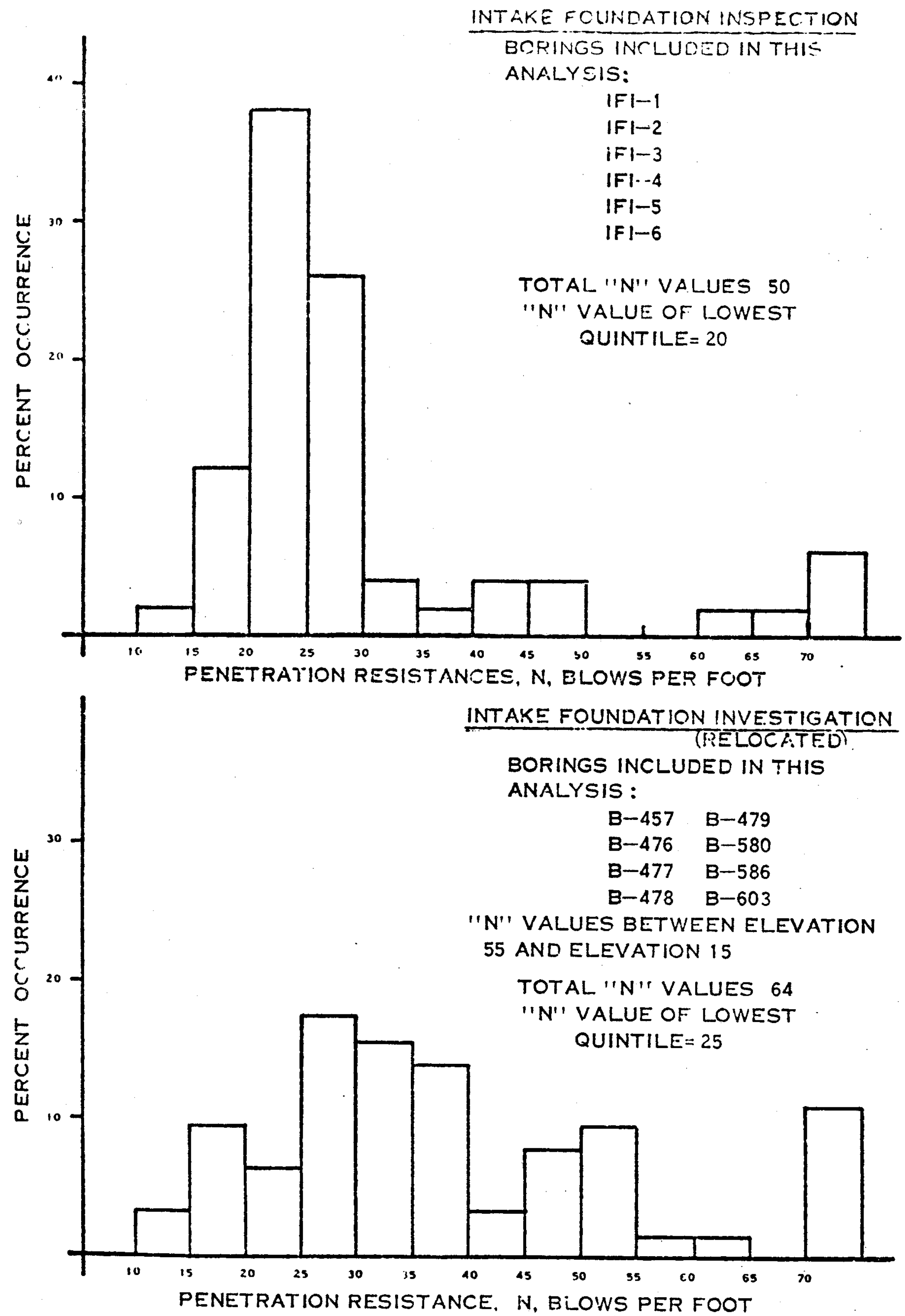
UNIT 2

COMPLETE LIQUEFACTION

FIGURE 2A-13



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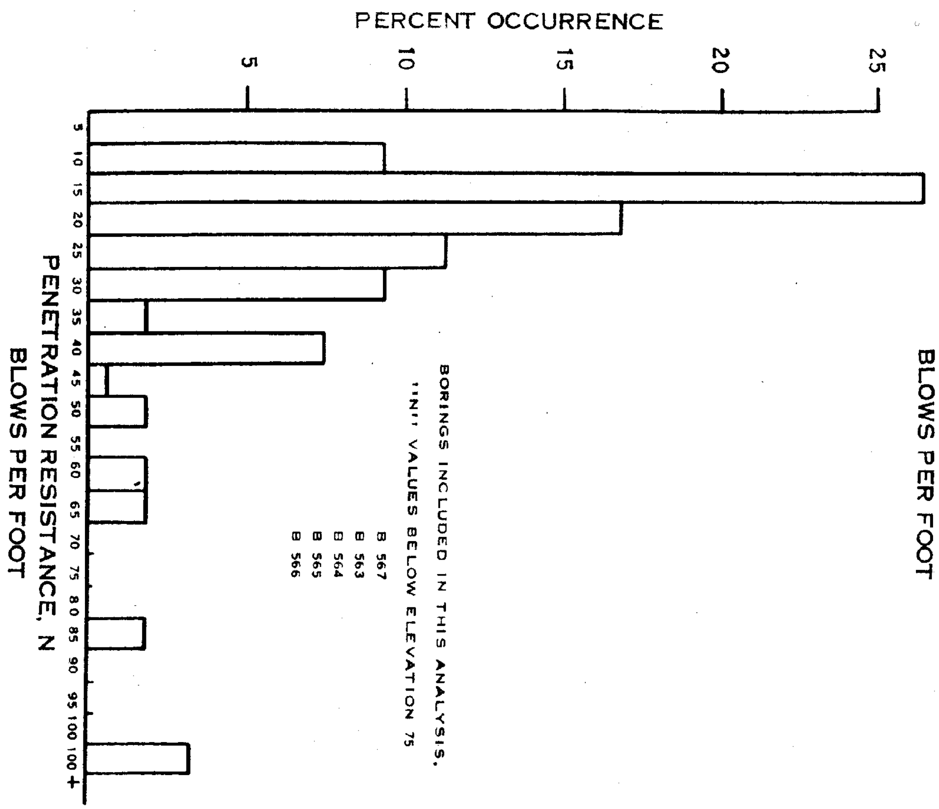
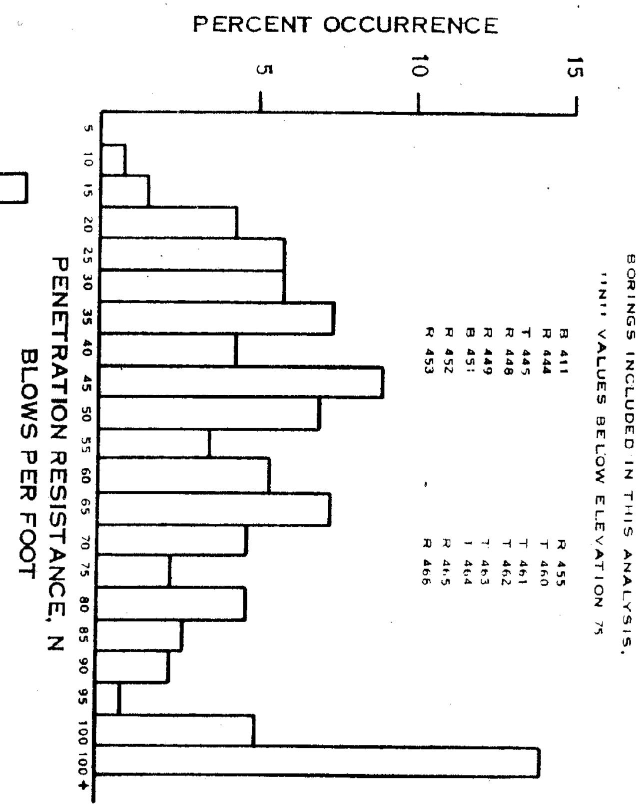
UNIT 2

INTAKE FOUNDATION INSPECTION

FIGURE 2A-14



AFTER EXCAVATION
TOTAL "N" VALUES - 53
"N" VALUE, LOWEST QUINTILE - 13



BEFORE EXCAVATION
TOTAL "N" VALUES - 254
"N" VALUE, LOWEST QUINTILE - 31

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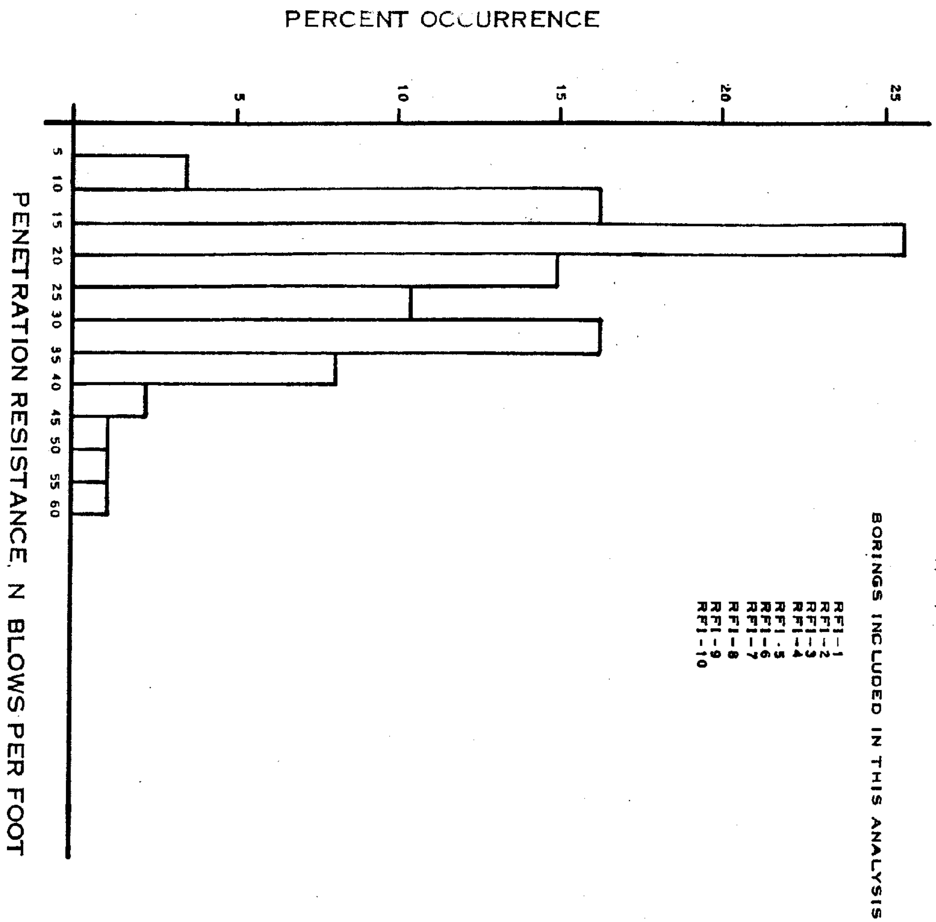
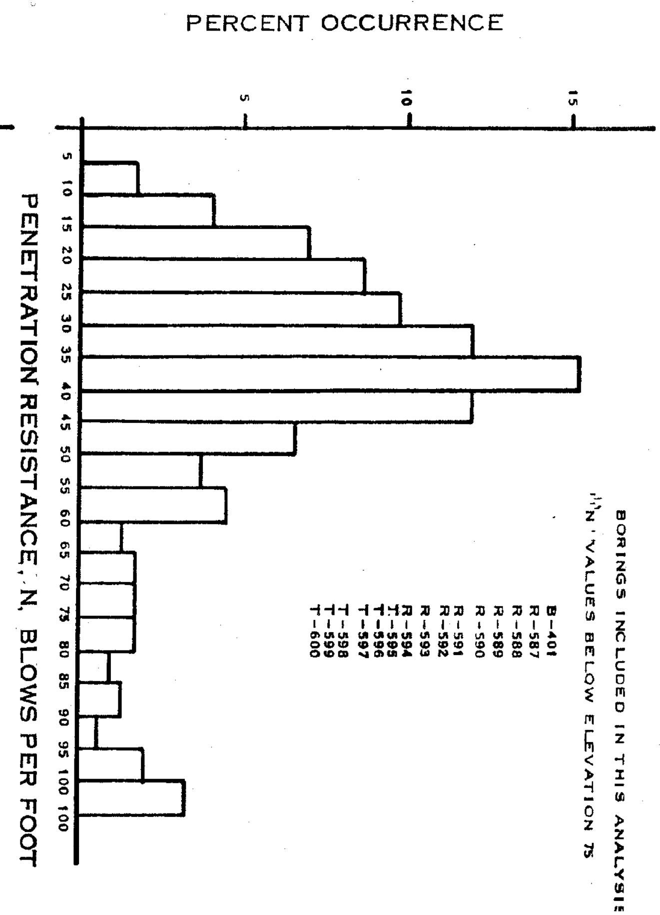
HNP-1 HISTOGRAMS BEFORE AND AFTER EXCAVATION

FIGURE 2A-15



AFTER EXCAVATION
TOTAL "N" VALUES – 86
"N" VALUE OF LOWEST QUINTILE = 1.5



BEFORE EXCAVATION
TOTAL "N" VALUES – 241
"N" VALUE OF LOWEST QUINTILE = 24

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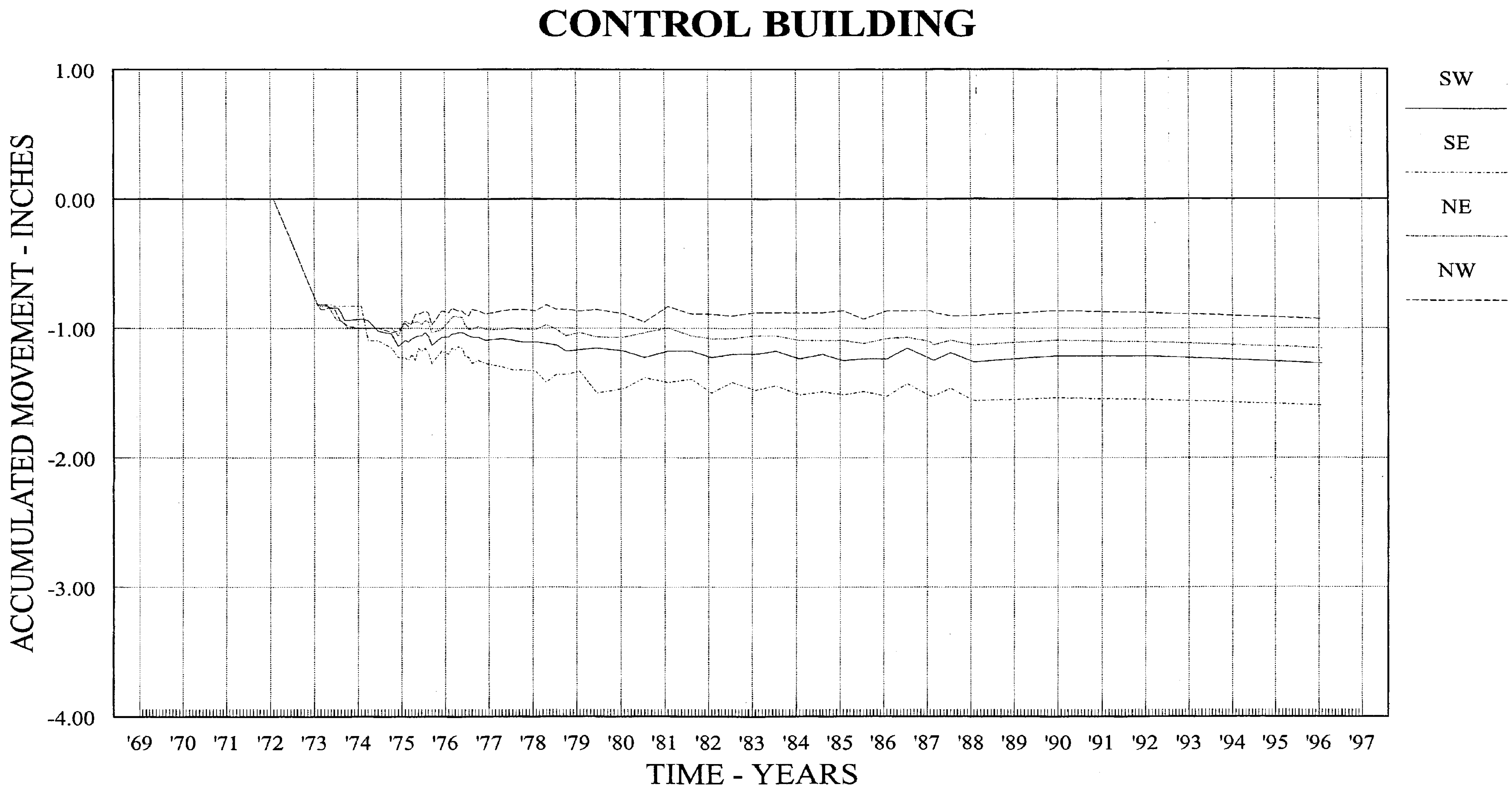
HNP-2 HISTOGRAMS BEFORE AND AFTER EXCAVATION

FIGURE 2A-16



<u>BENCHMARK NUMBERS AND LOCATIONS</u>

(Located on EL. 130' Floor)

No. 9 - NE - 10'-2" West of TA, 1'-3" North of T10
No. 10 - SE - 5'-3 1/2" West of TA, 4'-0" North of T13
No. 11 - NW - 4'-11" East of TH, 4'-5" South of T10
No. 12 - SW - 2'-10" East of TI, 7'-11" South of T12

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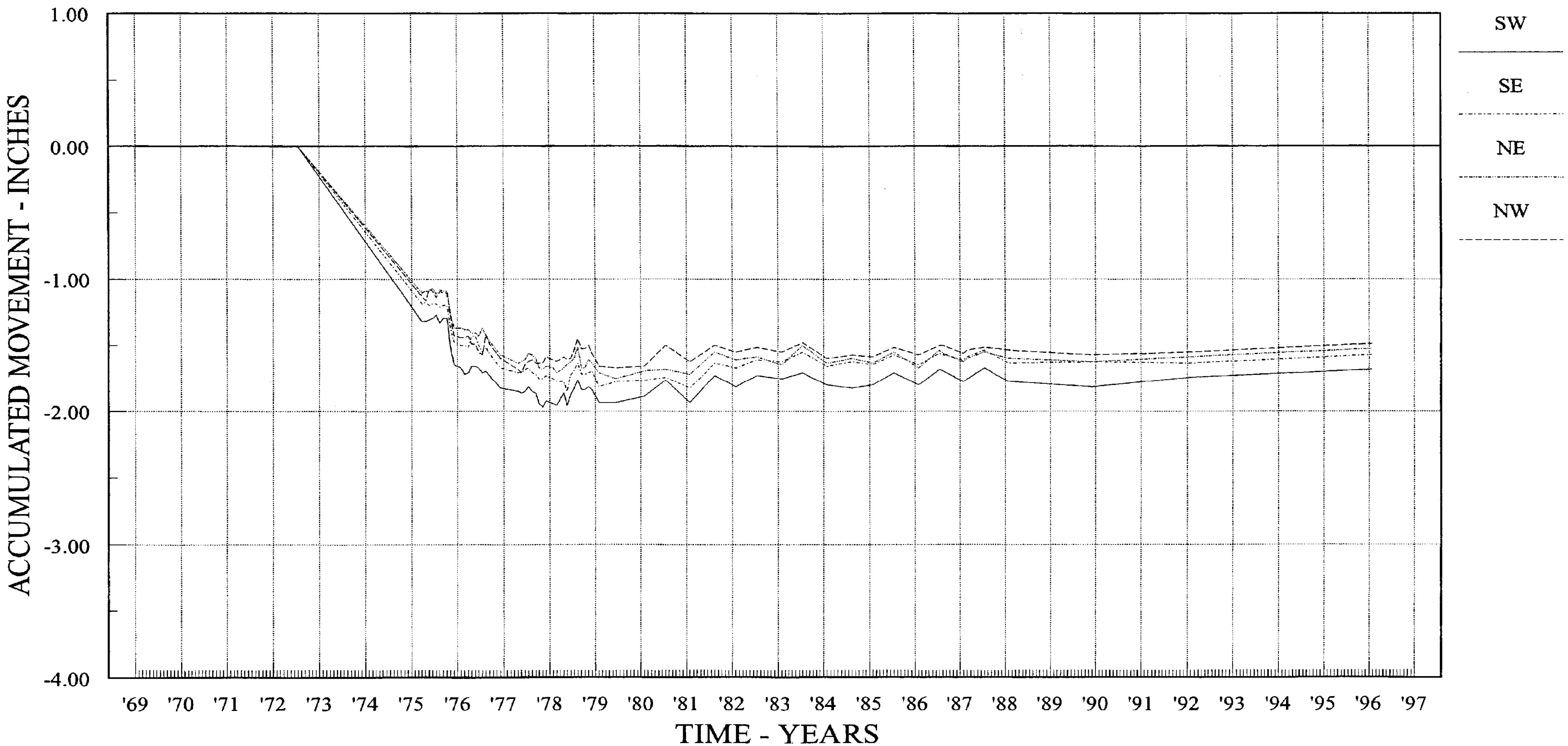
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	HNP-2 BUILDING SETTLEMENT FIGURE 2A-17 (SHEET 1 OF 7)

REACTOR BUILDING UNIT 2



BENCHMARK NUMBERS AND LOCATIONS

(Located on EL. 130' Floor)

No. 1 - NE - 24'-2 3/8" West of RL, on R15

No. 2 - SE - 15'-0" East of RL, 14'-3 1/2" South of R24

No. 3 - NW - 13'-0" West of RA, 13'-7" South of R14

No. 4 - SW - 24'-8" East of RA, on R23

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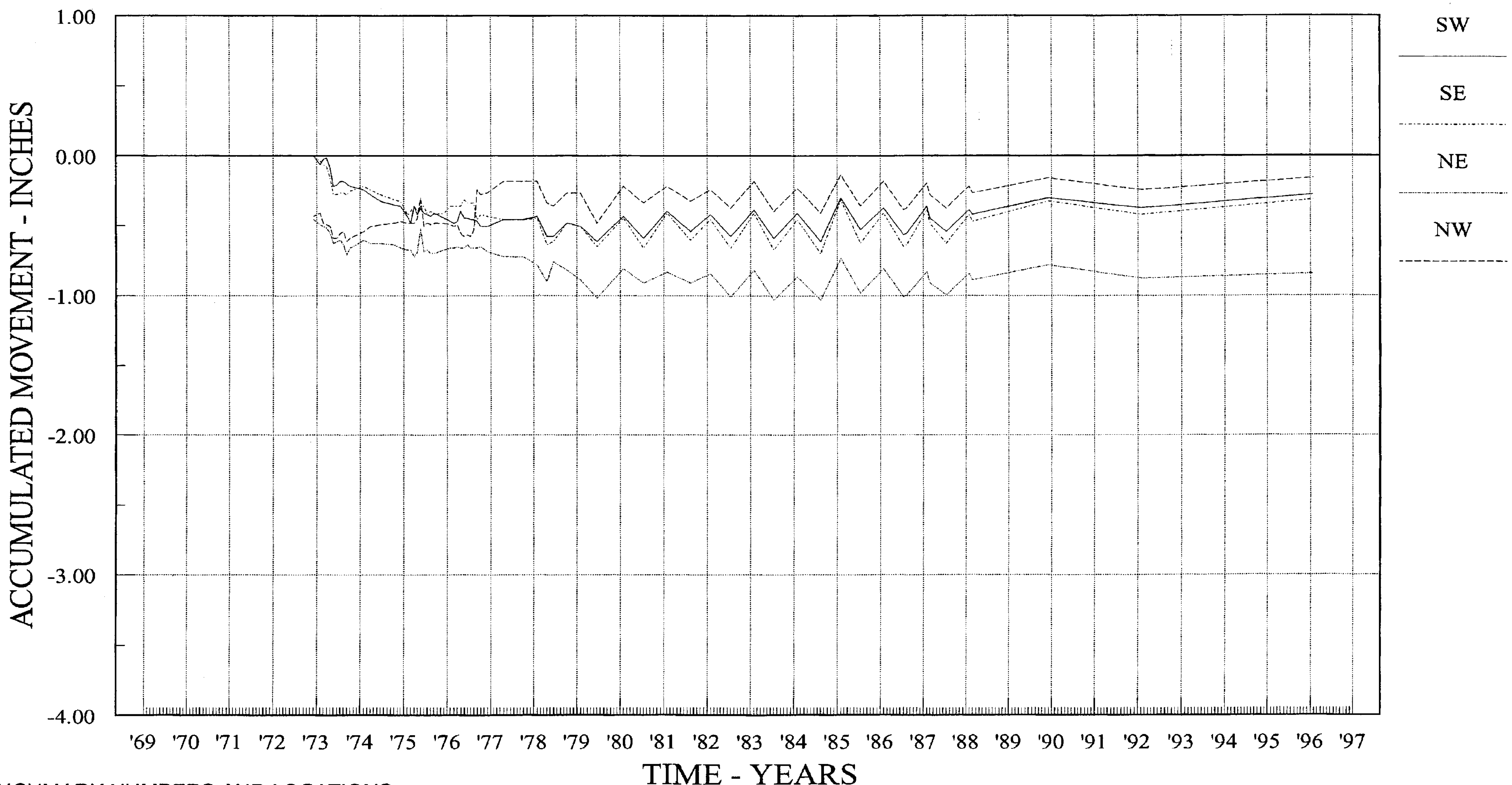
HNP-2 BUILDING SETTLEMENT

FIGURE 2A-17 (SHEET 2 OF 7)

DIESEL GENERATOR BUILDING



BENCHMARK NUMBERS AND LOCATIONS
(Located on Exterior Walls of Building)

No. 17 - NE - East Wall, 0'-11" South of NE Corner

No. 18 - SE - East Wall, 0'-1" North of SE Corner

No. 19 - NW - North Wall, 1'-1" East of NW Corner

No. 20 - SW - South Wall, 0'-11" East of SW Corner

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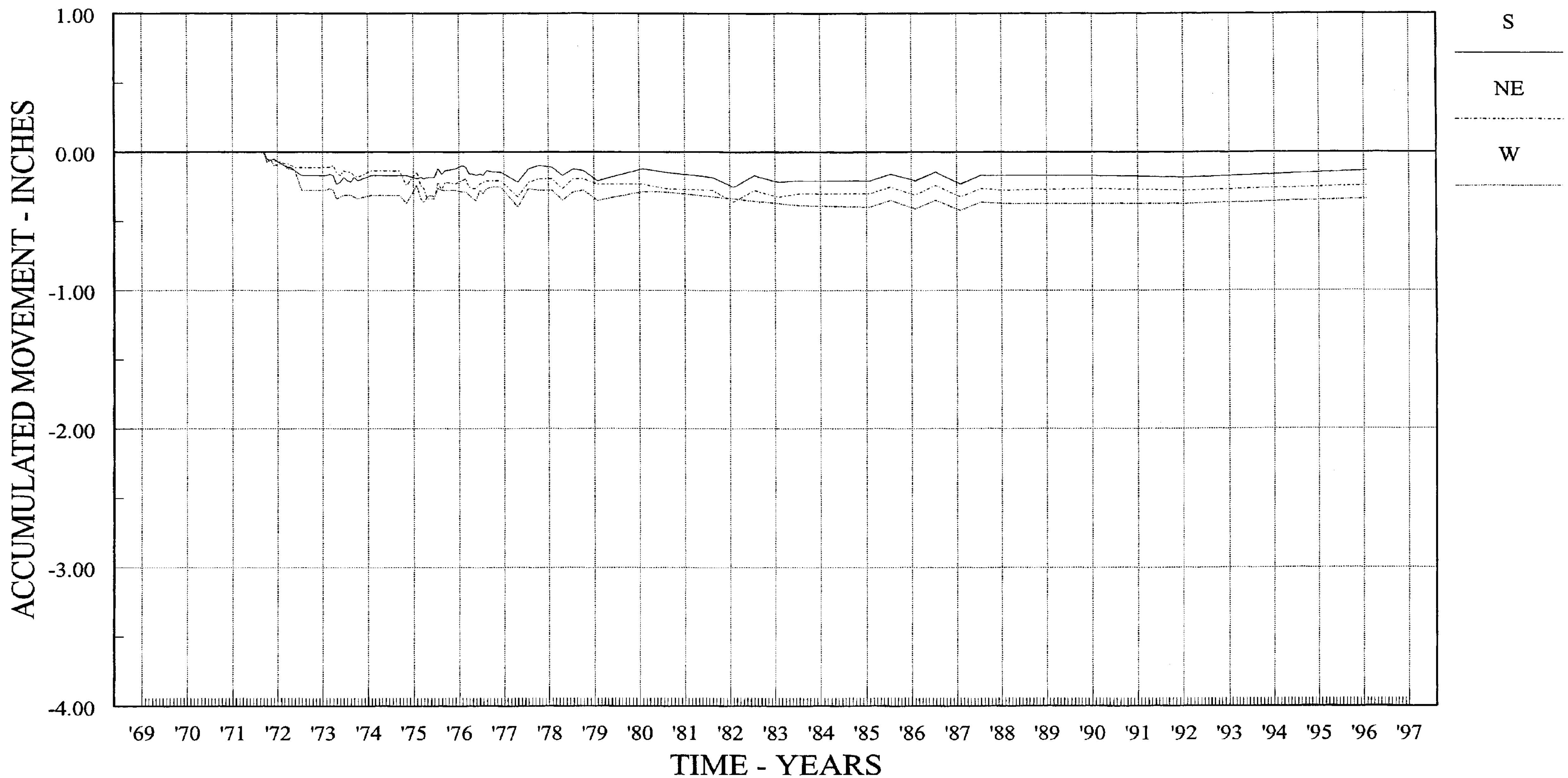
UNIT 2

HNP-2 BUILDING SETTLEMENT

FIGURE 2A-17 (SHEET 3 OF 7)

OFFGAS STACK



BENCHMARK NUMBERS AND LOCATIONS
(Located on EL. 120' Floor)
No. 21 - NE - 130 Degree Azimuth
No. 22 - S - 245 Degree Azimuth
No. 23 - W - 0 Degree Azimuth

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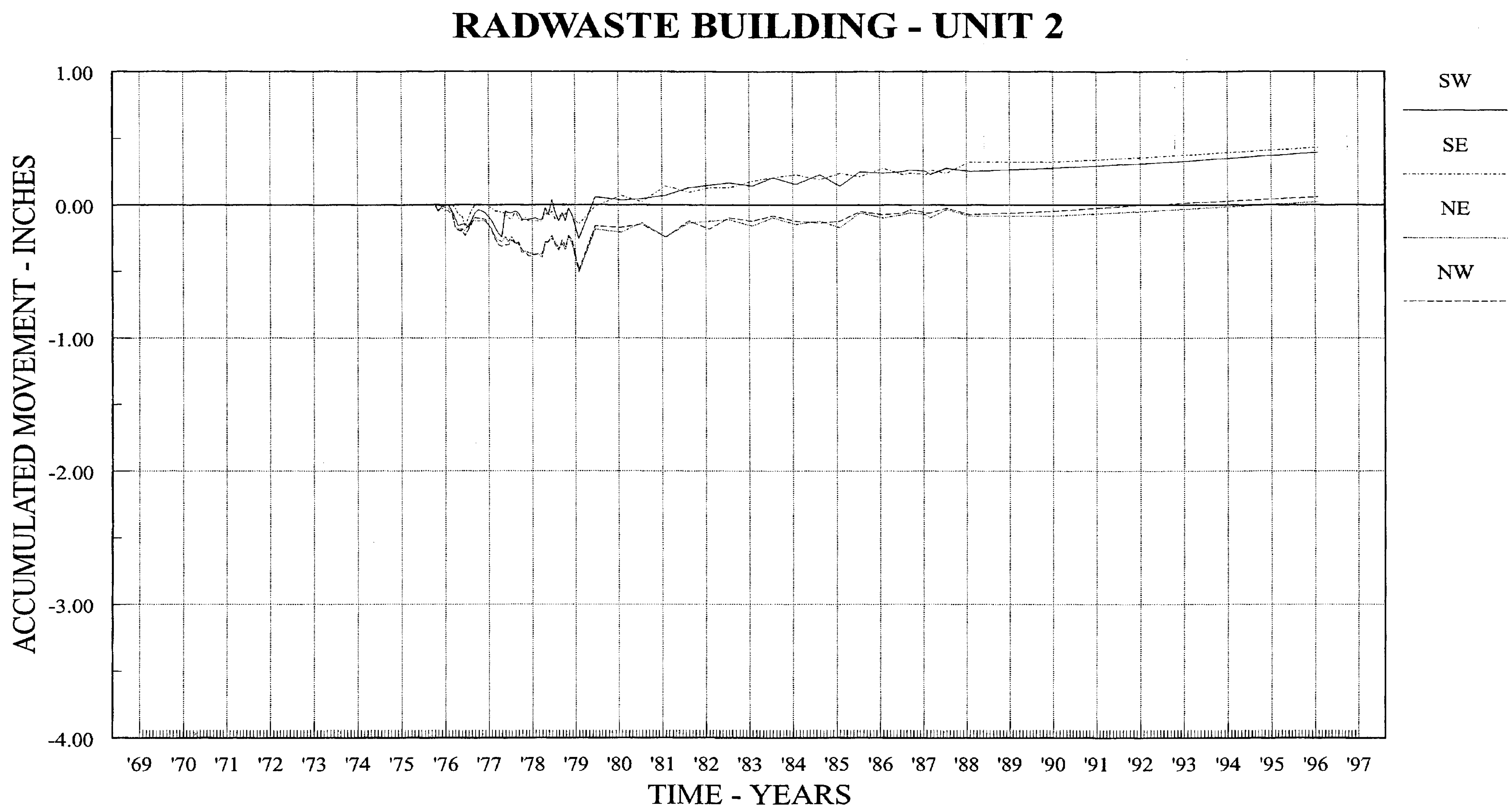
HNP-2 BUILDING SETTLEMENT

FIGURE 2A-17 (SHEET 4 OF 7)



BENCHMARK NUMBERS AND LOCATIONS
(Located on EL. 132'-4" Floor)
No. 5 - NE - 1'-3" East of BD, 3'-0" South of B1
No. 6 - SE - 0'-9" West of BJ, 5'-6" North of B11
No. 7 - NW - 2'-6" West of BA, 2'-6" South of B1
No. 8 - SW - 7'-3" West of BC, 0'-1" North of B11

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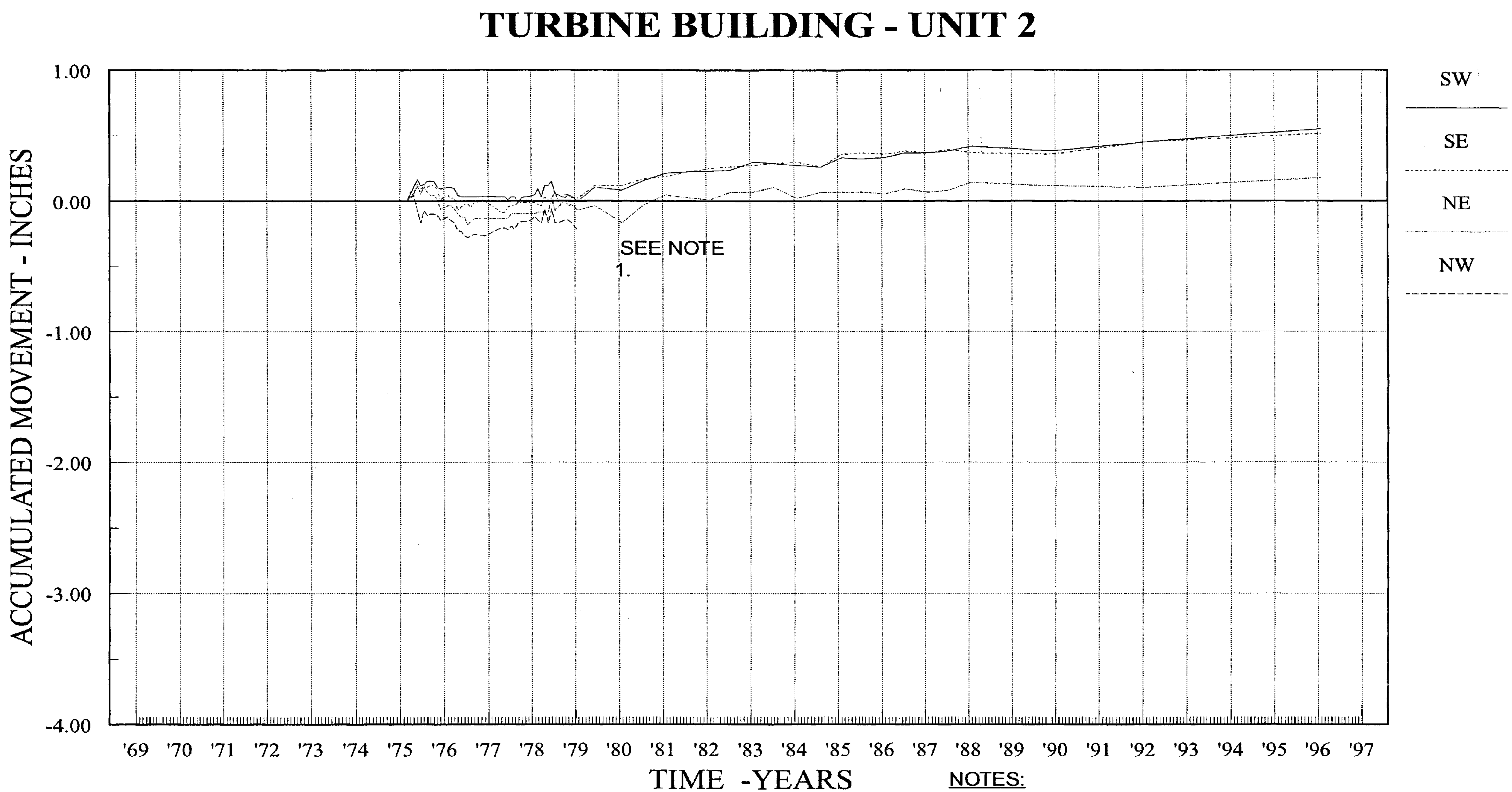
HNP-2 BUILDING SETTLEMENT

FIGURE 2A-17 (SHEET 5 OF 7)



NOTES:
1. NW MARKER INACCESSABLE SINCE 6-27-79.

BENCHMARK NUMBERS AND LOCATIONS
(Located on EL. 130' Floor)
No. 13 - NE - 10'-2" West of TA, 1'-10" North of T15
No. 14 - SE - 13'-2" West of TA, 0'-3" North of T23
No. 15 - NW - 2'-5" West of TH, 3'-7 1/2" South of T15
No. 16 - SW - 2'-7" East of TI, 17'-10" North of T23

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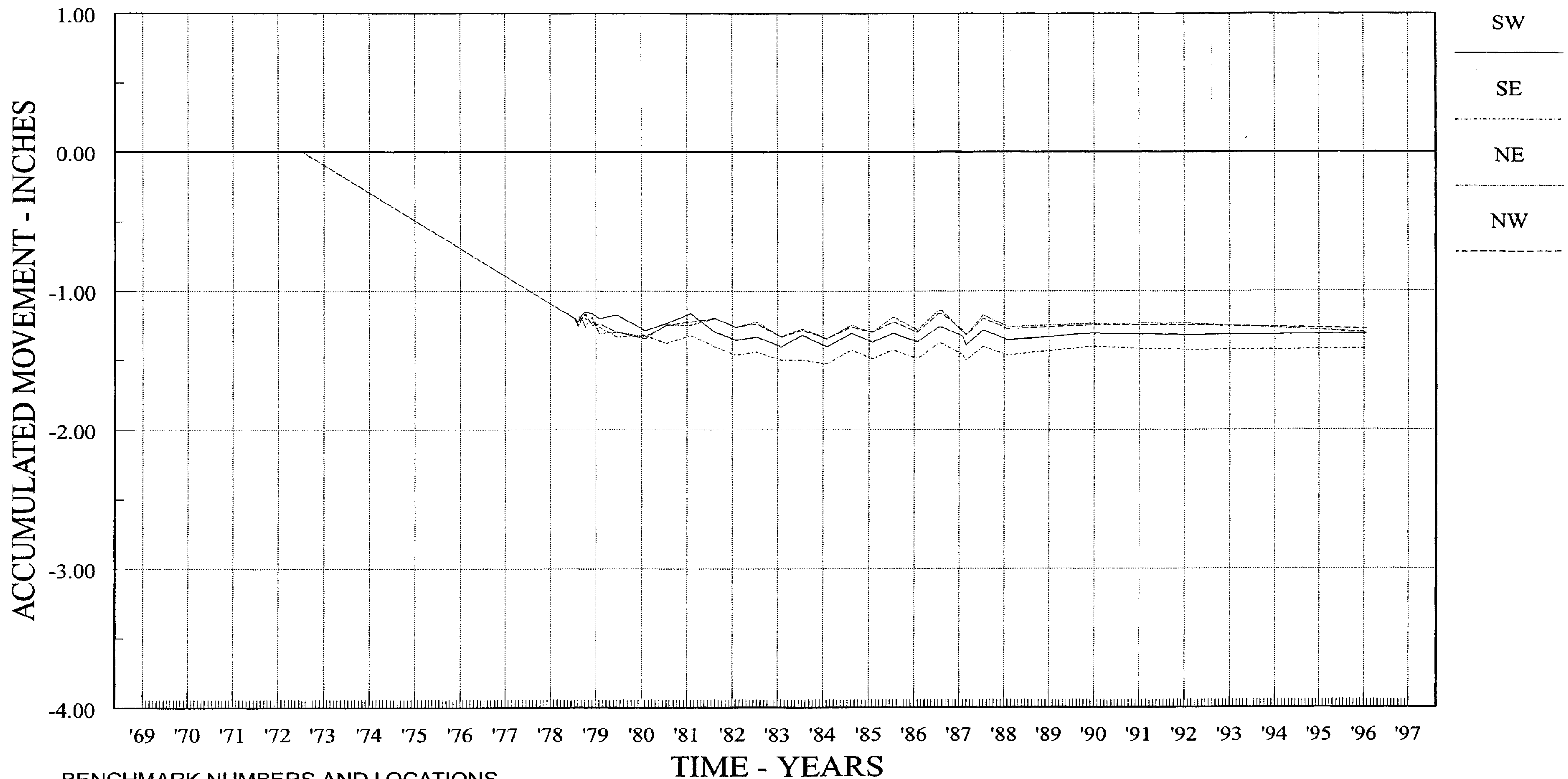
INTAKE STRUCTURE



<u>BENCHMARK NUMBERS AND LOCATIONS</u>
(Located at Building Corners)
No. 24 - NE - 1'-3" South
No. 25 - SE - 1'-0" West, 1'-0" North
No. 26 - NW - 1'-3" South
No. 27 - SW - 1'-0" East, 1'-0" North

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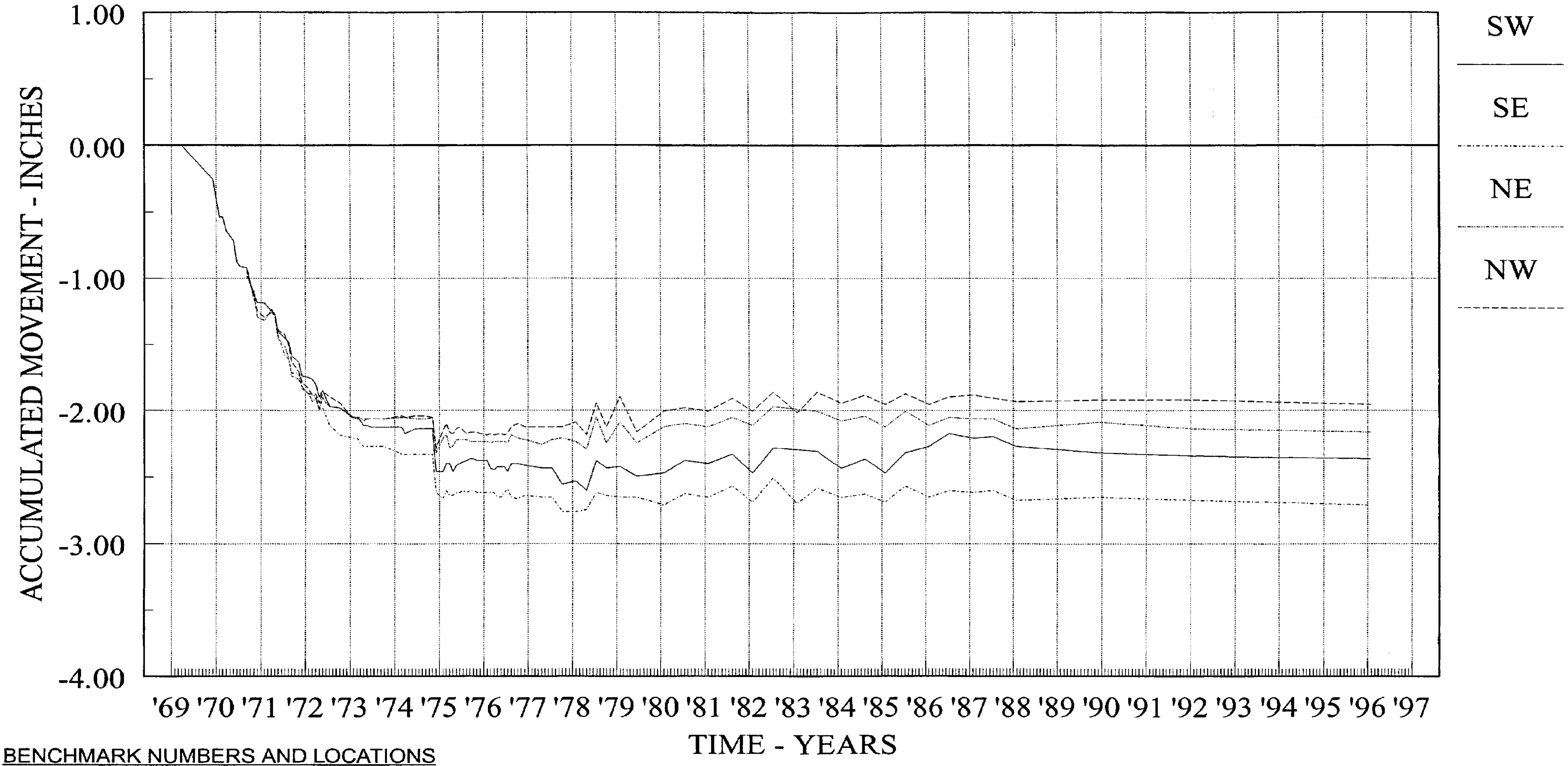
HNP-2 BUILDING SETTLEMENT

FIGURE 2A-17 (SHEET 7 OF 7)

REACTOR BUILDING - UNIT 1



BENCHMARK NUMBERS AND LOCATIONS
(Located on EL. 130' Floor)
No. 28 - NE - 23'-0" West of RL, on R3
No. 29 - SE - 23'-11" West of RL, 18'-5" North of R13
No. 30 - NW - 11'-5" East of RA, 6'-0" South of R2
(Southeast Anchor for Cable Tray Support 1U-7B)
No. 31 - SW - 23'-11" East of RA, on R11

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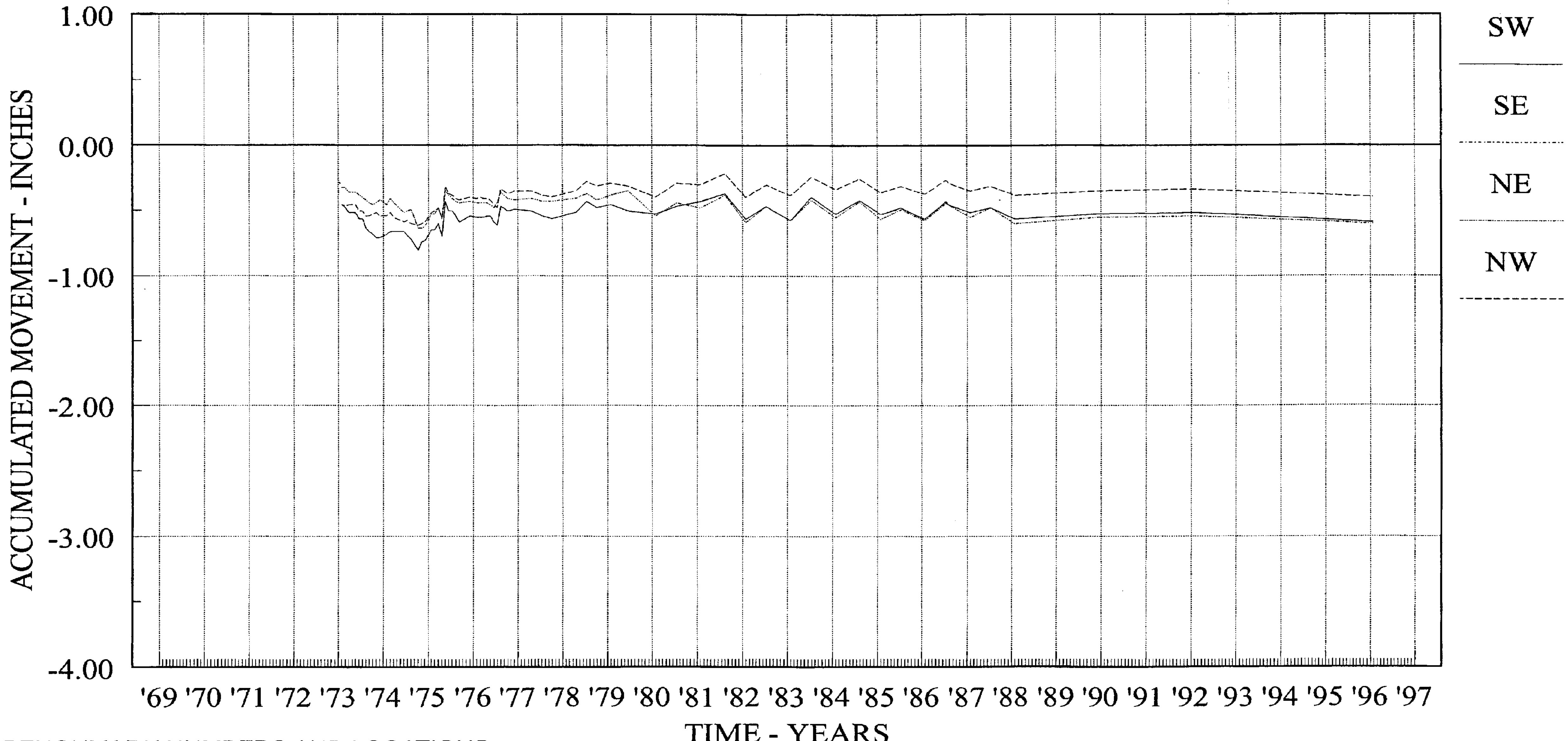
SOUTHERN COMPANY Energy to Serve Your World®	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	HNP-1 BUILDING SETTLEMENT
		FIGURE 2A-18 (SHEET 1 OF 3)

RADWASTE BUILDING - UNIT 1



BENCHMARK NUMBERS AND LOCATIONS

(Located on EL. 132'-4" Floor)

No. 32 - NE - 7'-8" West of WC, 6'-5 1/2" South of W2

No. 33 - SE - 9'-9 1/2" West of WC, 4'-8 3/4" North of W4

No. 34 - NW - 2'-5 1/2" East of WA, 7'-4 1/2" South of W2

No. 35 - SW - 3'-0" East of WA, 1'-0" North of W4

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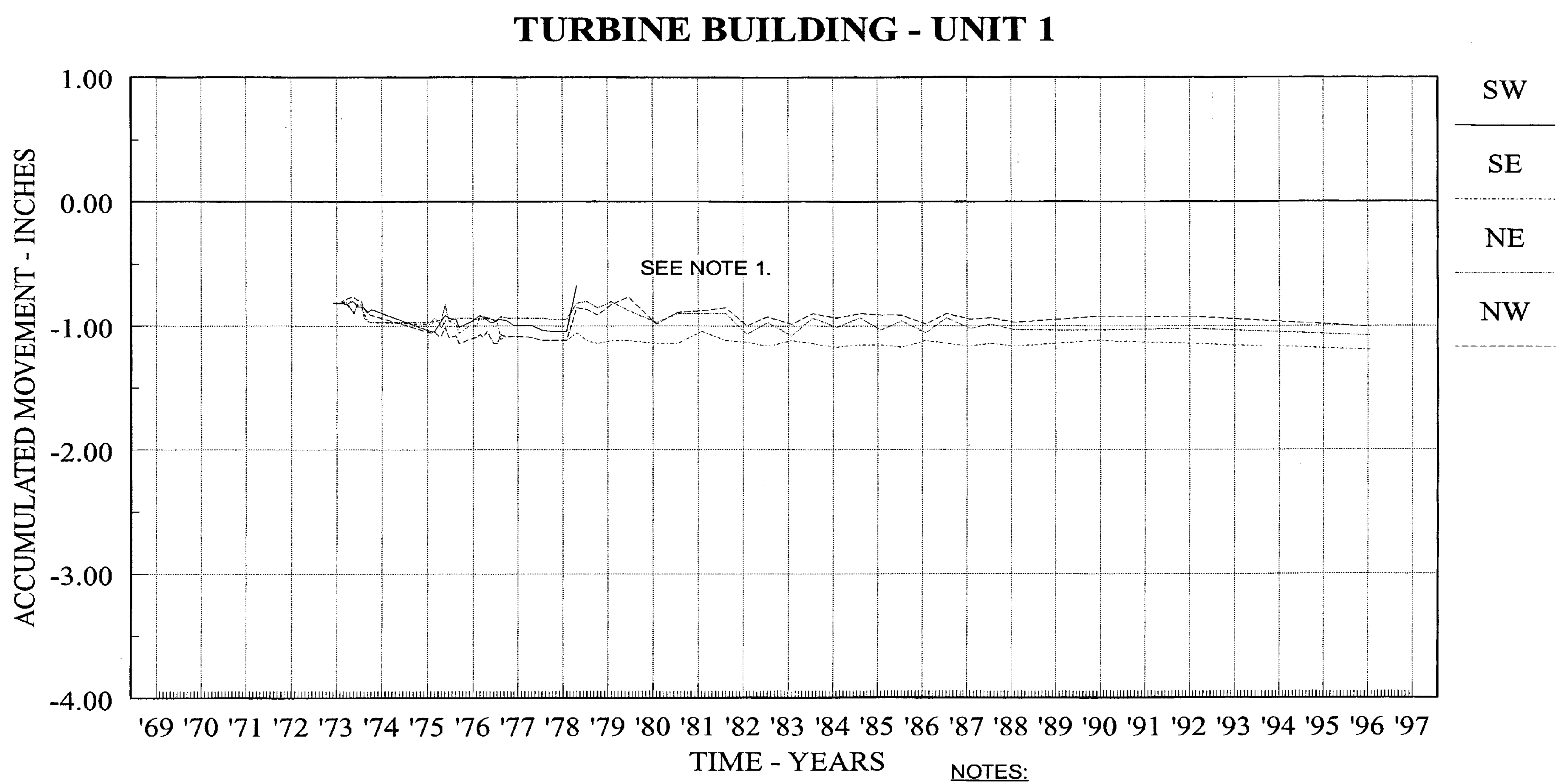
SOUTHERN COMPANY Energy to Serve Your World®	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	HNP-1 BUILDING SETTLEMENT FIGURE 2A-18 (SHEET 2 OF 3)



NOTES:

1. MARKER INACCESSABLE, FROM 7-10-78 TO 1-24-79 AND SINCE 1-16-80.

BENCHMARK NUMBERS AND LOCATIONS

(Located on EL. 130' Floor)

No. 36 - NE - 7'-9" West of TA, 14'-5" South of T2
No. 37 - SE - 10'-2" West of TA, 0'-2 1/4" South of T9
No. 38 - NW - 5'-3 1/2" East of TH, 15'-0" South of T1
No. 39 - SW - 7'-8 1/2" East of TI, 3'-2 1/2" North of T9

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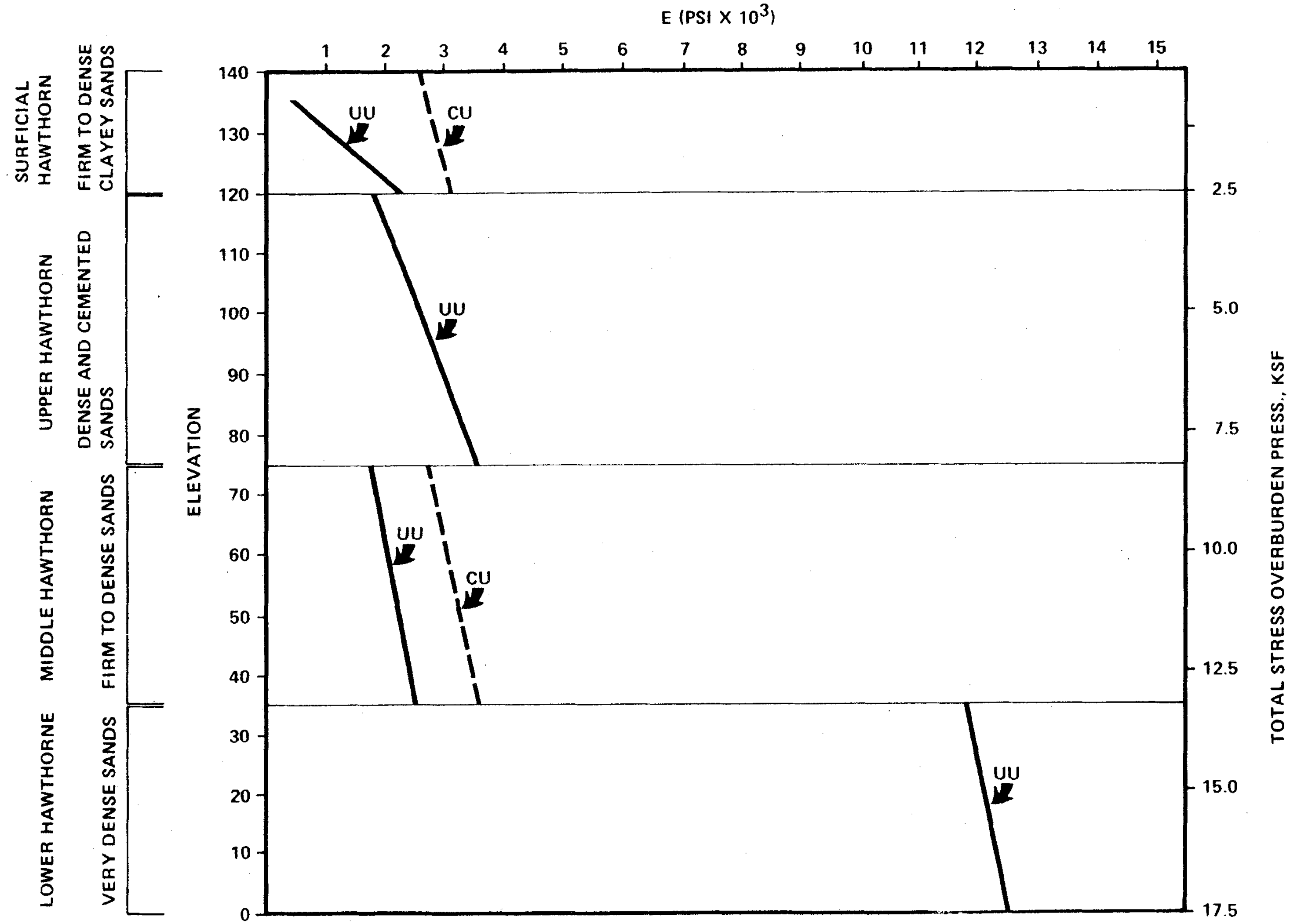
UNIT 2

HNP-1 BUILDING SETTLEMENT

FIGURE 2A-18 (SHEET 3 OF 3)



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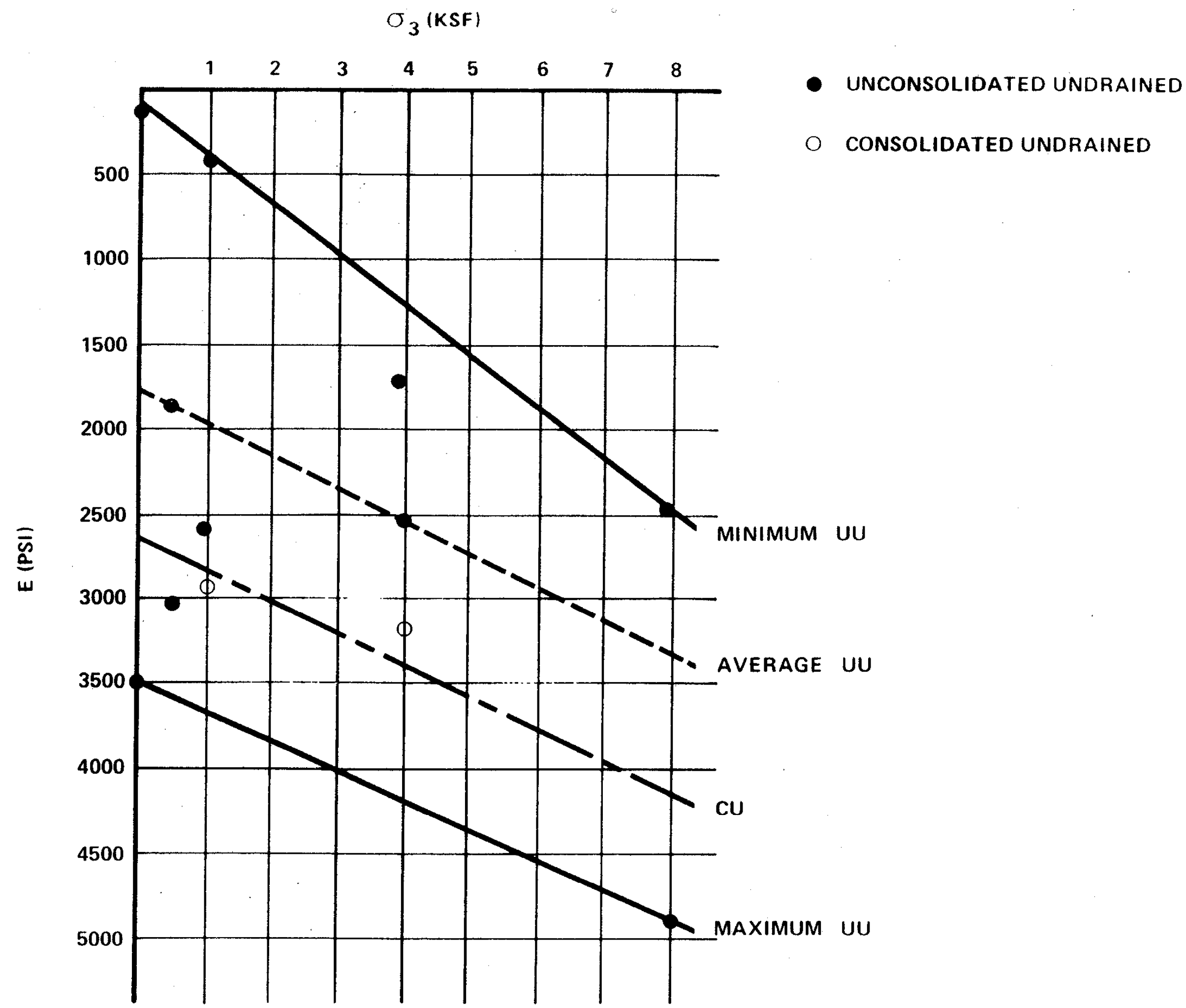
AVERAGE ELASTIC MODULI

FIGURE 2A-19



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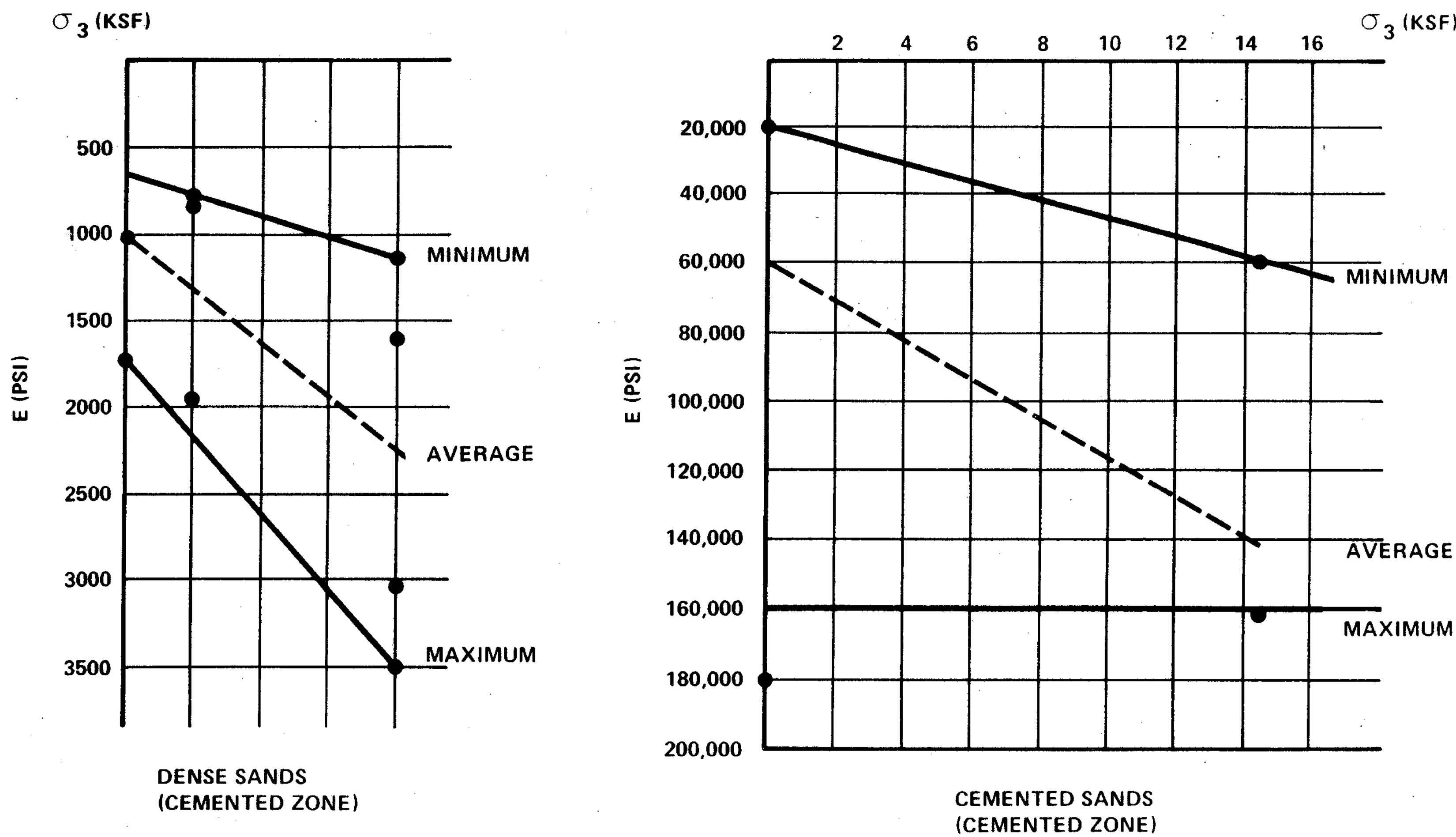
ELASTIC MODULI – SURFICIAL HAWTHORN

FIGURE 2A-20



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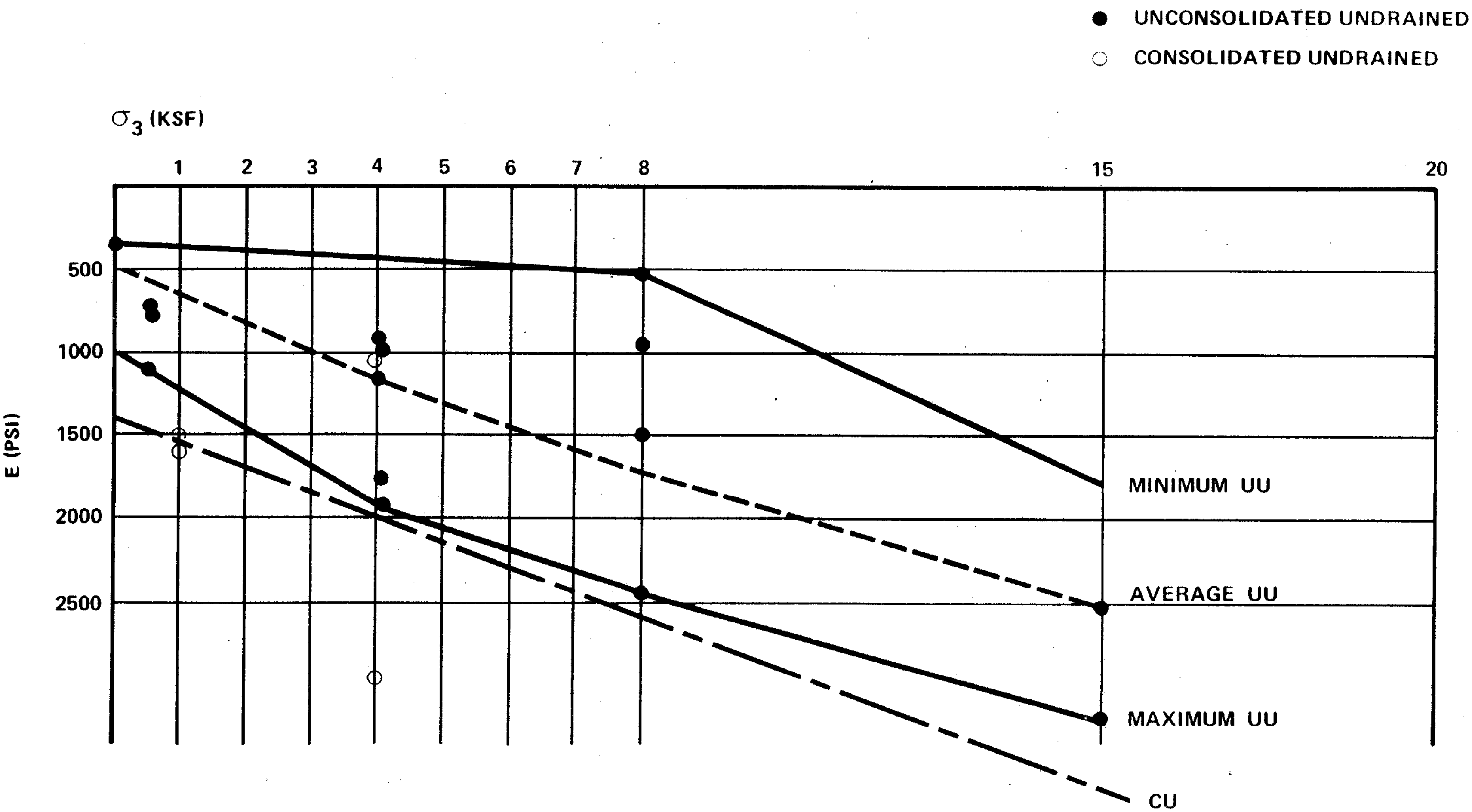
ELASTIC MODULI – UPPER HAWTHORN

FIGURE 2A-21



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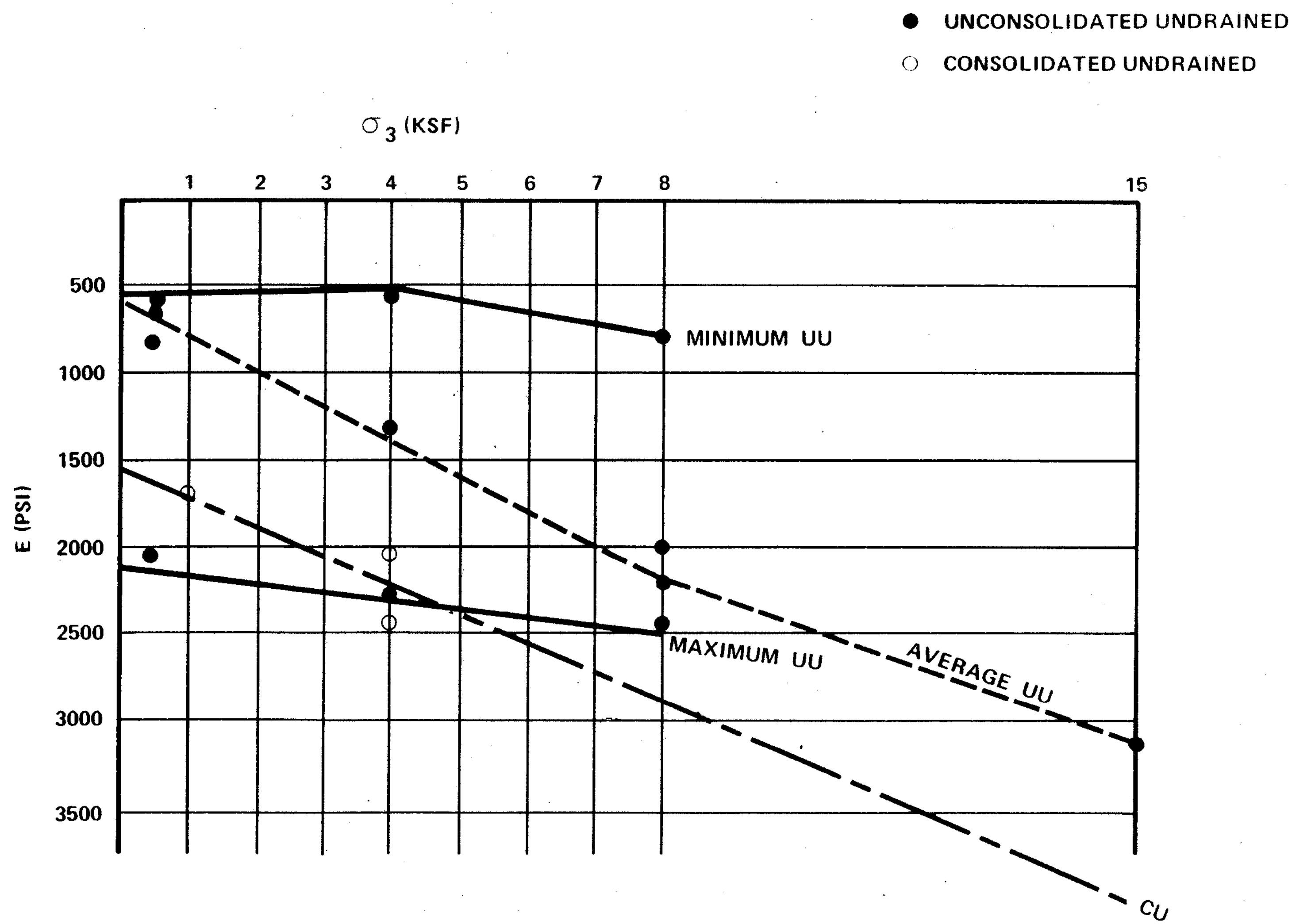
ELASTIC MODULI – MIDDLE HAWTHORN

FIGURE 2A-22 (SHEET 1 OF 2)



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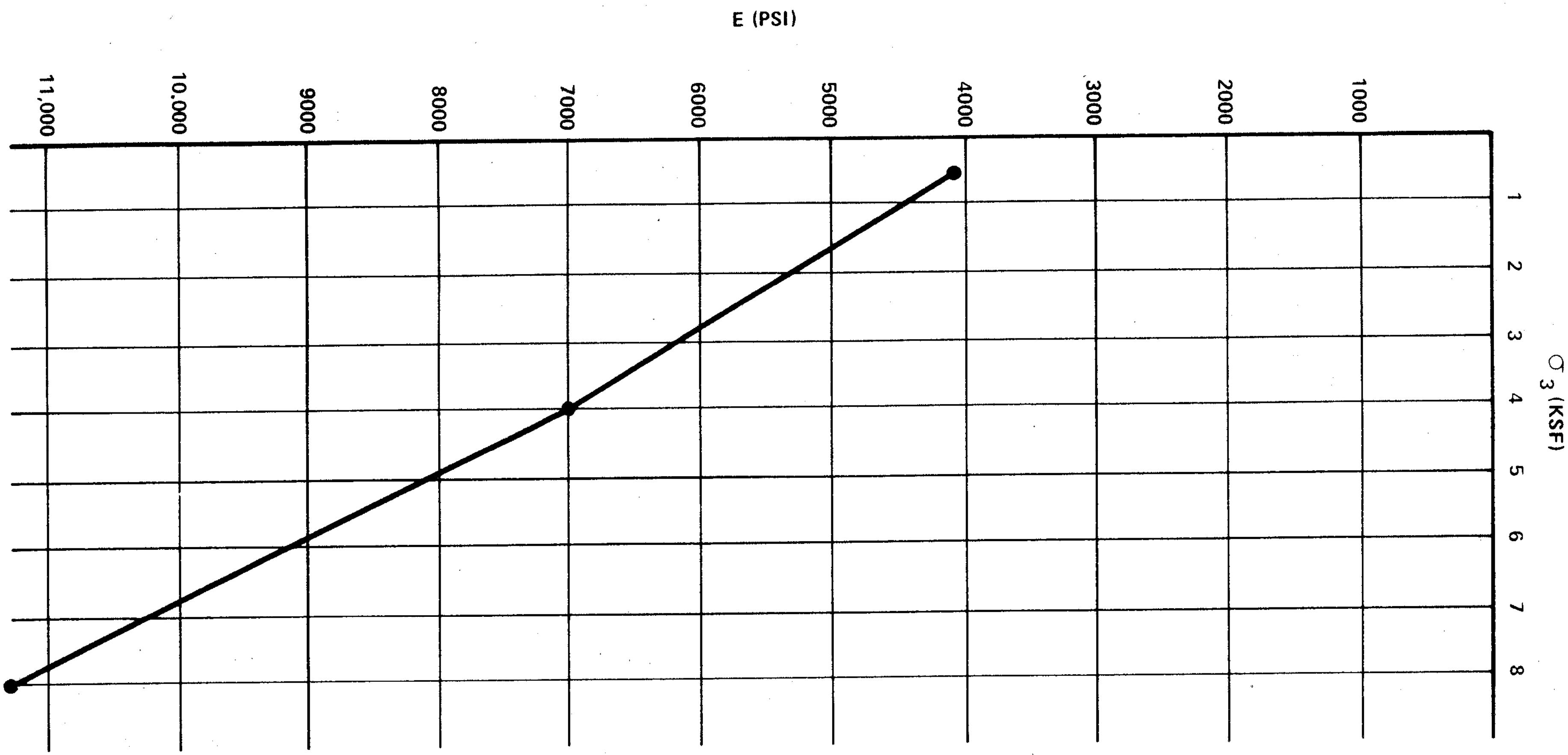
ELASTIC MODULI – MIDDLE HAWTHORN

FIGURE 2A-22 (SHEET 2 OF 2)



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ELASTIC MODULI – LOWER HAWTHORN

FIGURE 2A-23

SAFETY FACTORS WITH
RESPECT TO MOMENTARY LIQUEFACTION

A. Corrected for 60 percent Relative Density

Location See Figure

Piezometric Level	Yard	B	D	F	K
Max. Observed (77)	2.12	2.36	2.36	2.20	2.40
10 Year Flood (85)	1.99	2.23	2.22	2.08	2.28
1000 Year Flood (93)	1.84	2.09	2.08	1.90	2.10
Max. Theoretical Flood (105)	1.66	1.90	1.90	1.73	1.94

B. Based on Test of Loosest Materials - Uncorrected for Relative Density

Location See Figure

Piezometric Level	Yard	B	D	F	K
Max. Observed (77)	1.76	1.96	1.96	1.83	2.00
10 Year Flood (85)	1.64	1.85	1.84	1.71	1.89
1000 Year Flood (93)	1.53	1.73	1.73	1.57	1.74
Max. Theoretical Flood (105)	1.37	1.58	1.57	1.44	1.61

SAFETY FACTOR WITH
RESPECT TO COMPLETE LIQUEFACTION

A. Corrected for 60 percent Relative Density

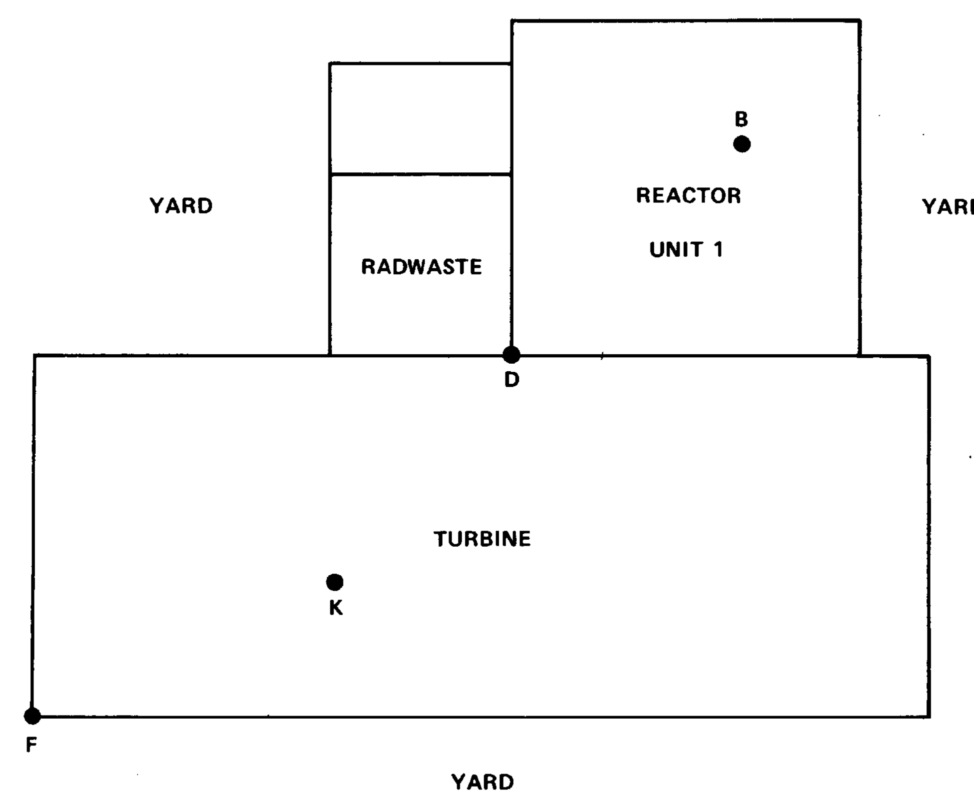
Location See Figure

Piezometric Level	Yard	B	D	F	K
Max. Observed (77)	2.40	2.69	2.68	2.50	2.74
10 Year Flood (85)	2.25	2.53	2.52	2.34	2.58
1000 Year Flood (93)	2.09	2.38	2.37	2.16	2.39
Max. Theoretical Flood (105)	1.87	2.16	2.15	1.97	2.20

B. Based on Tests of Loosest Material - Uncorrected for Relative Density

Location Figure

Piezometric Level	Yard	B	D	F	K
Max. Observed (77)	2.00	2.22	2.22	2.07	2.26
10 Year Flood (85)	1.88	2.10	2.09	1.96	2.15
1000 Year Flood (93)	1.74	1.97	1.97	1.79	1.98
Max. Theoretical Flood (105)	1.57	1.79	1.78	1.64	1.82



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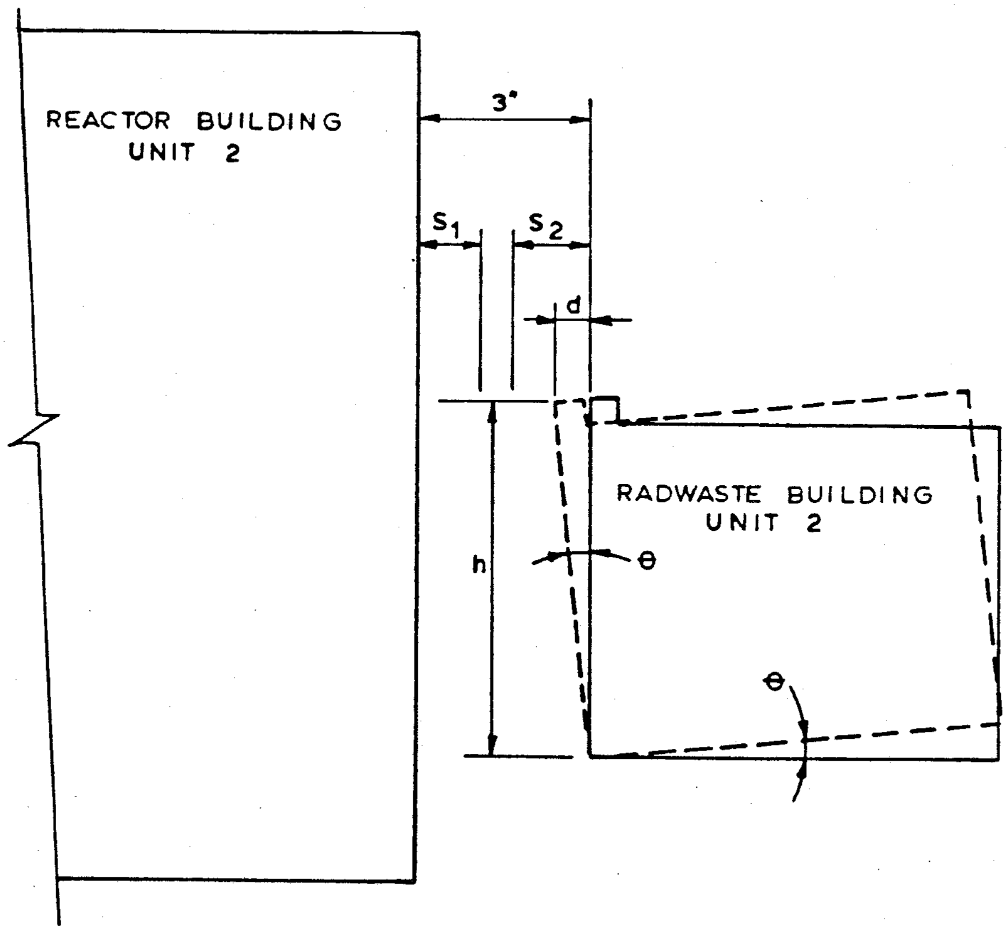
LIQUEFACTION STUDY

FIGURE 2A-24



ALLOWABLE SLOPE $\theta = \frac{d}{h}$ (FOR SMALL ANGLES)

AND $d = \frac{3'' - (S_1 + S_2)}{2}$

WHERE: θ = ALLOWABLE SLOPE OF THE BUILDING

d = ALLOWABLE HORIZONTAL COMPONENT OF TILT TOWARDS AN ADJACENT BUILDING

h = HEIGHT OF BUILDING

S_1 = OBE DEFLECTION OF BUILDING AT TOP

S_2 = OBE DEFLECTION OF ADJACENT BUILDING AT SAME ELEVATION

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ALLOWABLE SETTLEMENT PROFILE SLOPE CALCULATION PROCEDURE

FIGURE 2A-26

KEY

SOIL 1 - SANDY SILTY CLAY AND CLAYEY COARSE TO FINE SAND

SOIL 2 - CLAYEY SILTY COARSE TO FINE SAND (CEMENTED)

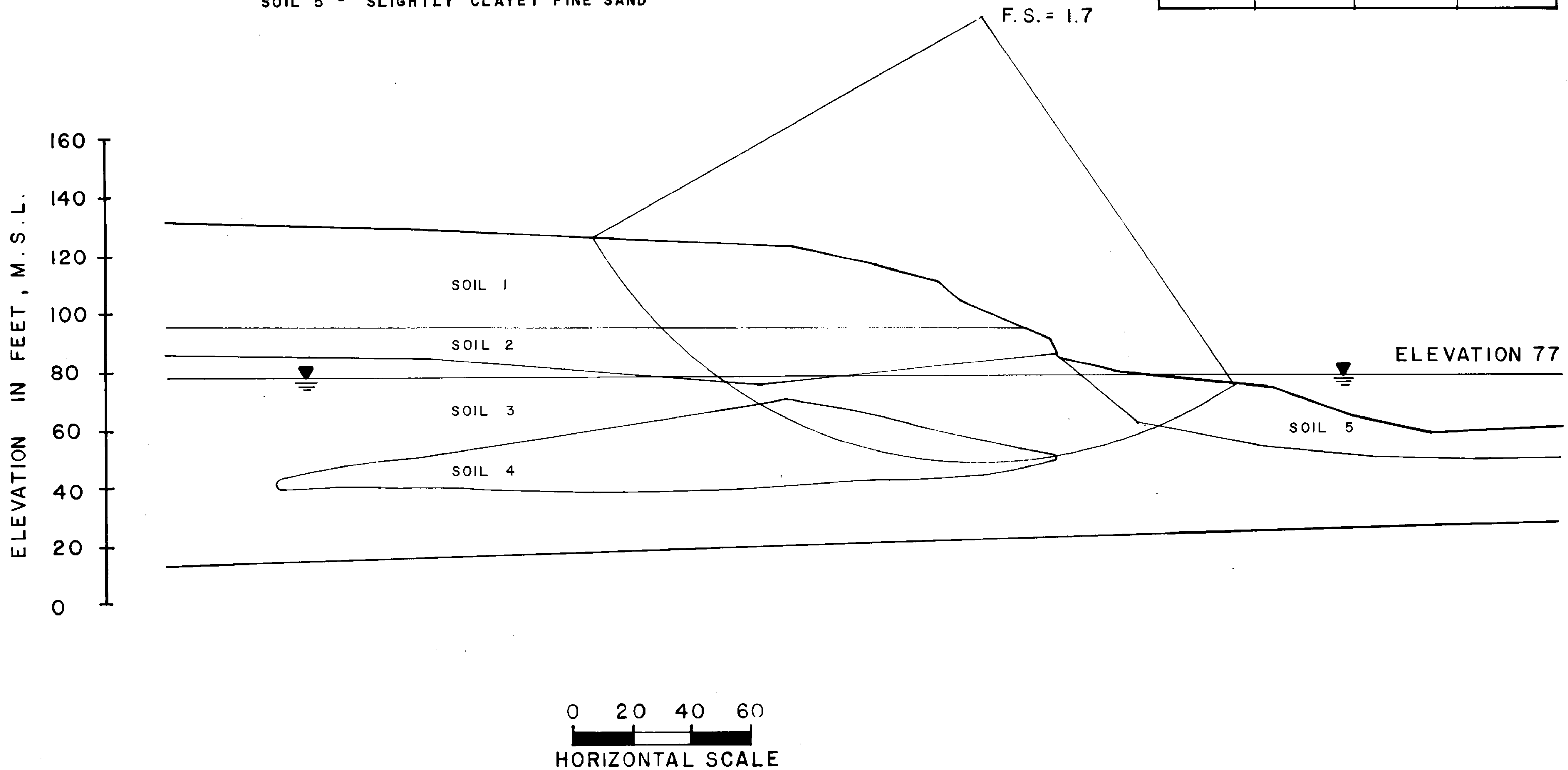
SOIL 3 - SILTY FINE SAND, MEDIUM TO FINE SAND AND CLAYEY SILTY FINE SAND

SOIL 4 - CLAYEY SILTY FINE SAND AND SILTY FINE SAND

SOIL 5 - SLIGHTLY CLAYEY FINE SAND

SOIL PROPERTY TABLE

SOIL	DENSITY	TOTAL STRENGTH	
	γ_T = PCF	ϕ - DEG.	C - PSF
1	135	24	1000
2	130	40	5000
3	130	36	1500
4	110	20	300
5	115	25	100



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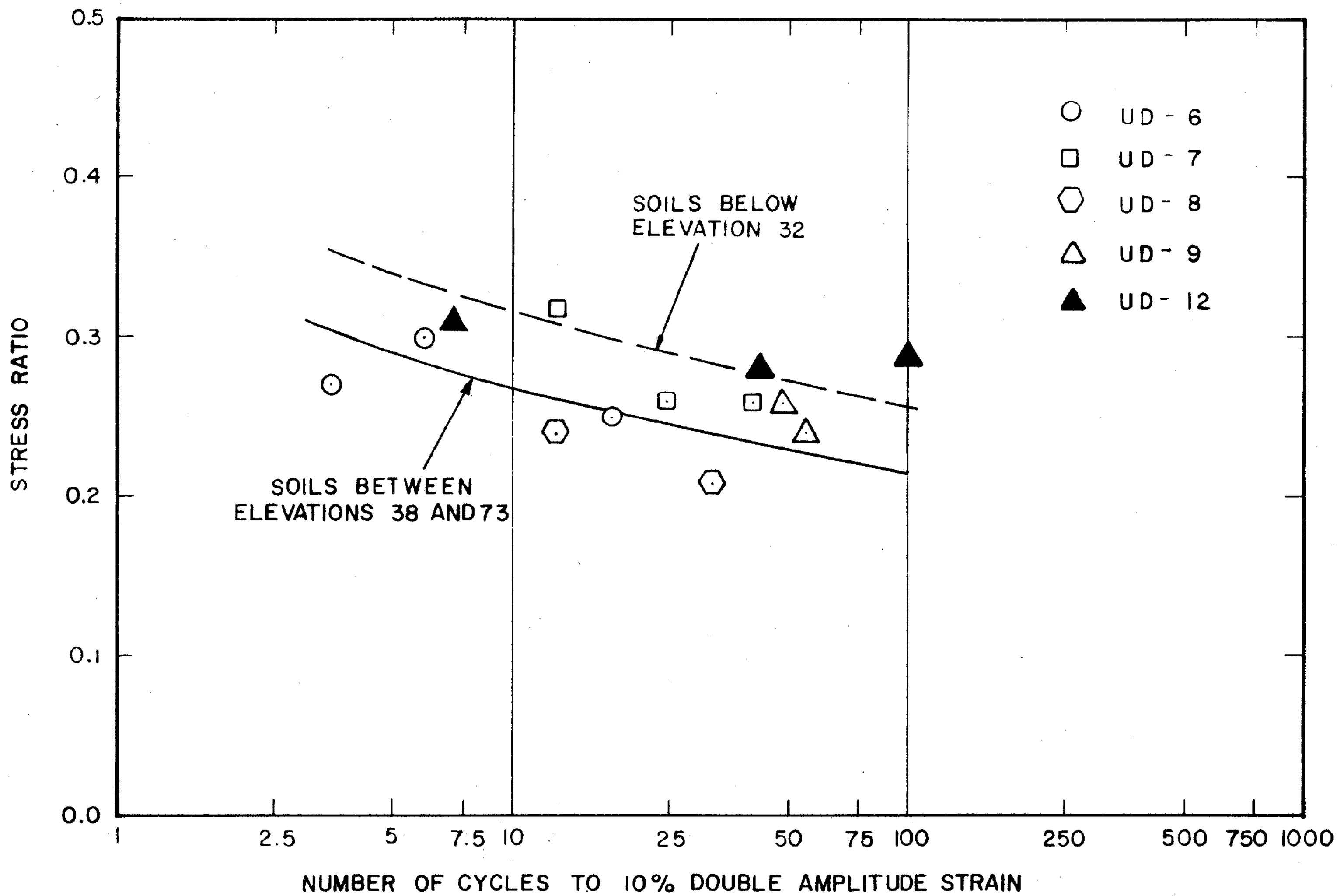
PSEUDOSTATIC STABILITY ANALYSIS OF RIVER BLUFF FOR DBE

FIGURE 2A-27



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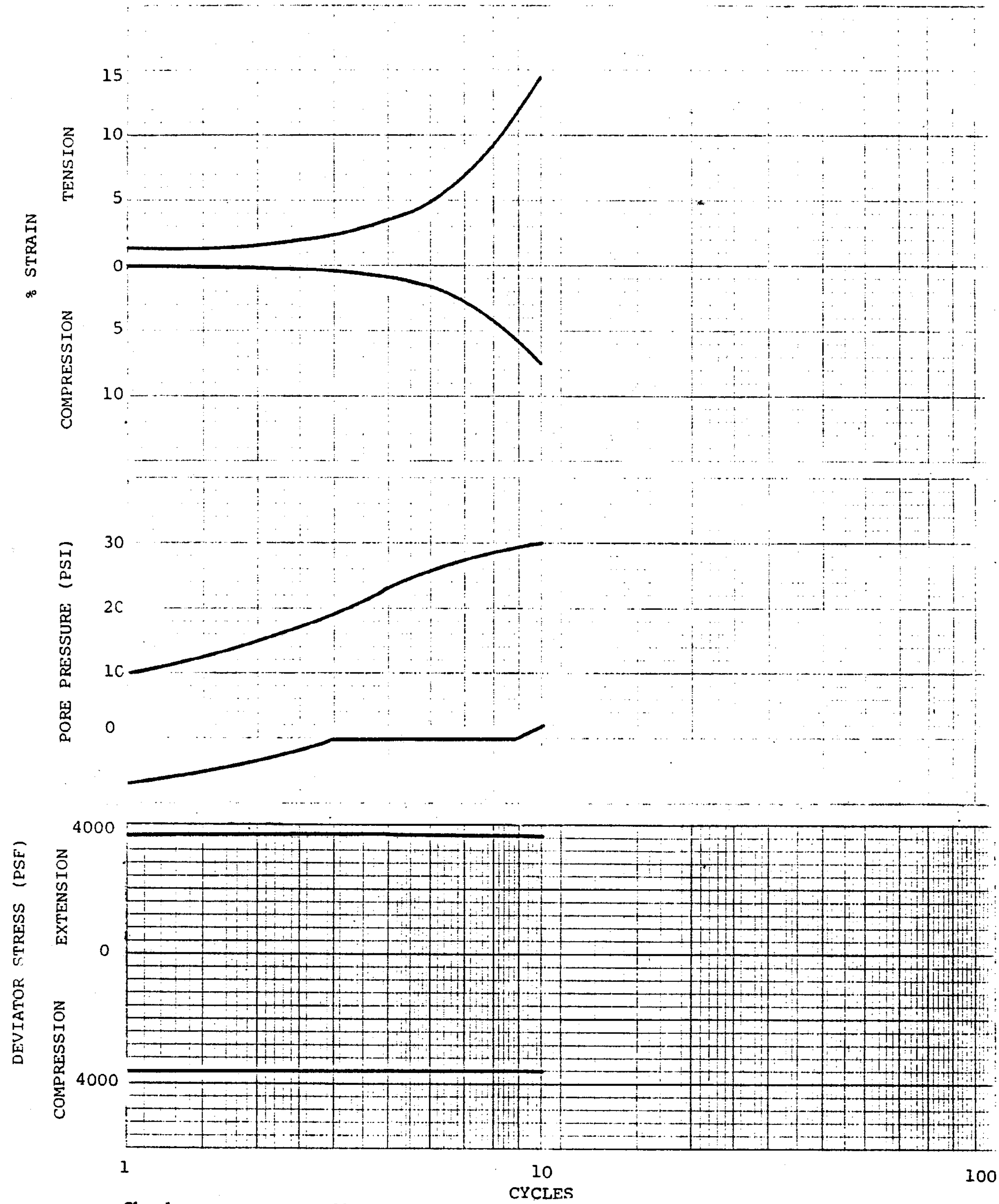
CYCLIC STRENGTH FOR SOILS AT RIVER BLUFF

FIGURE 2A-28



Chamber pressure **6000 psf**
Unit dry weight **95.1 pcf**
Water content **27.7%**

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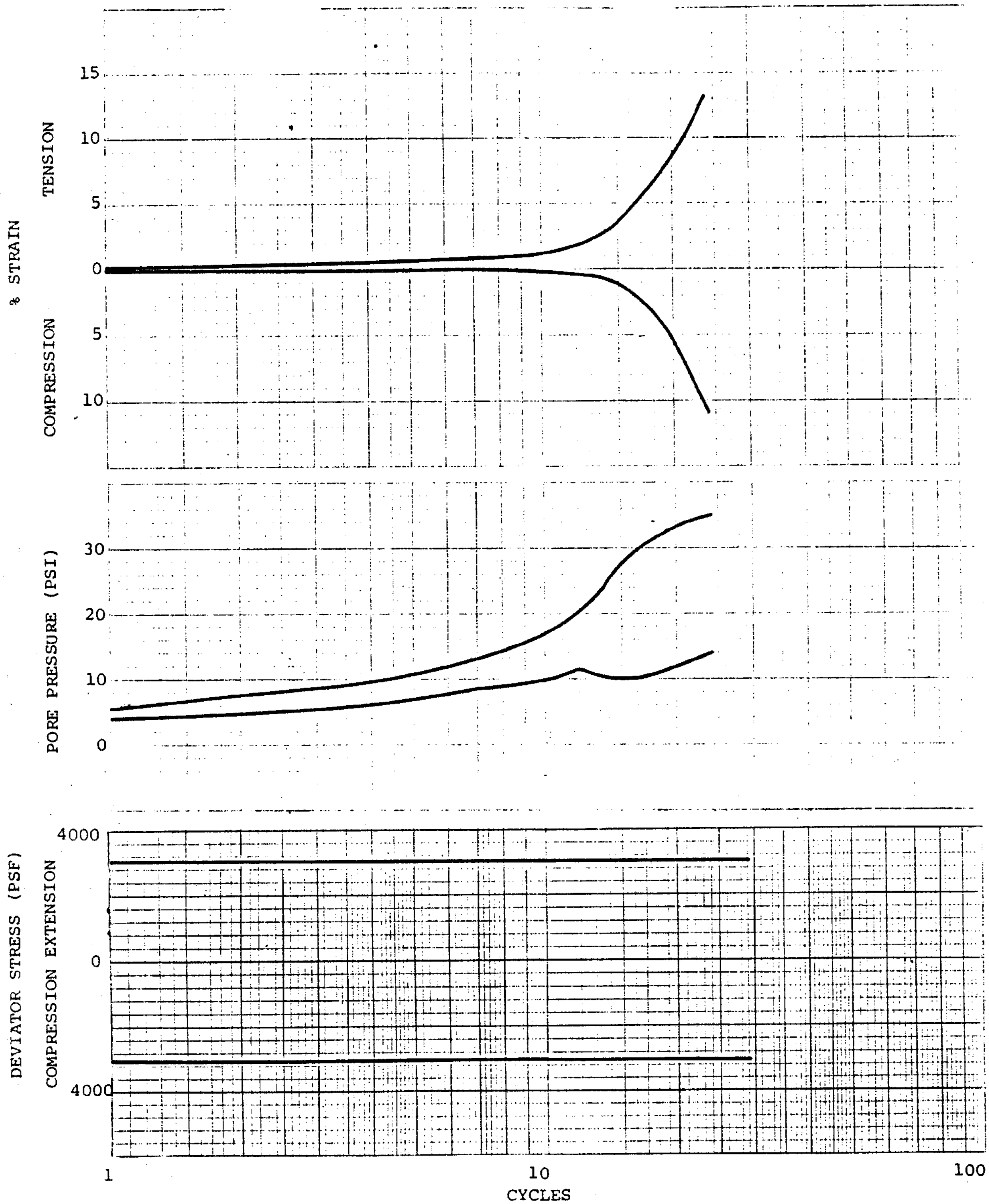
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 1 OF 13)



Chamber pressure	600 psf
Unit dry weight	94.2 pcf
Water content	26.8%

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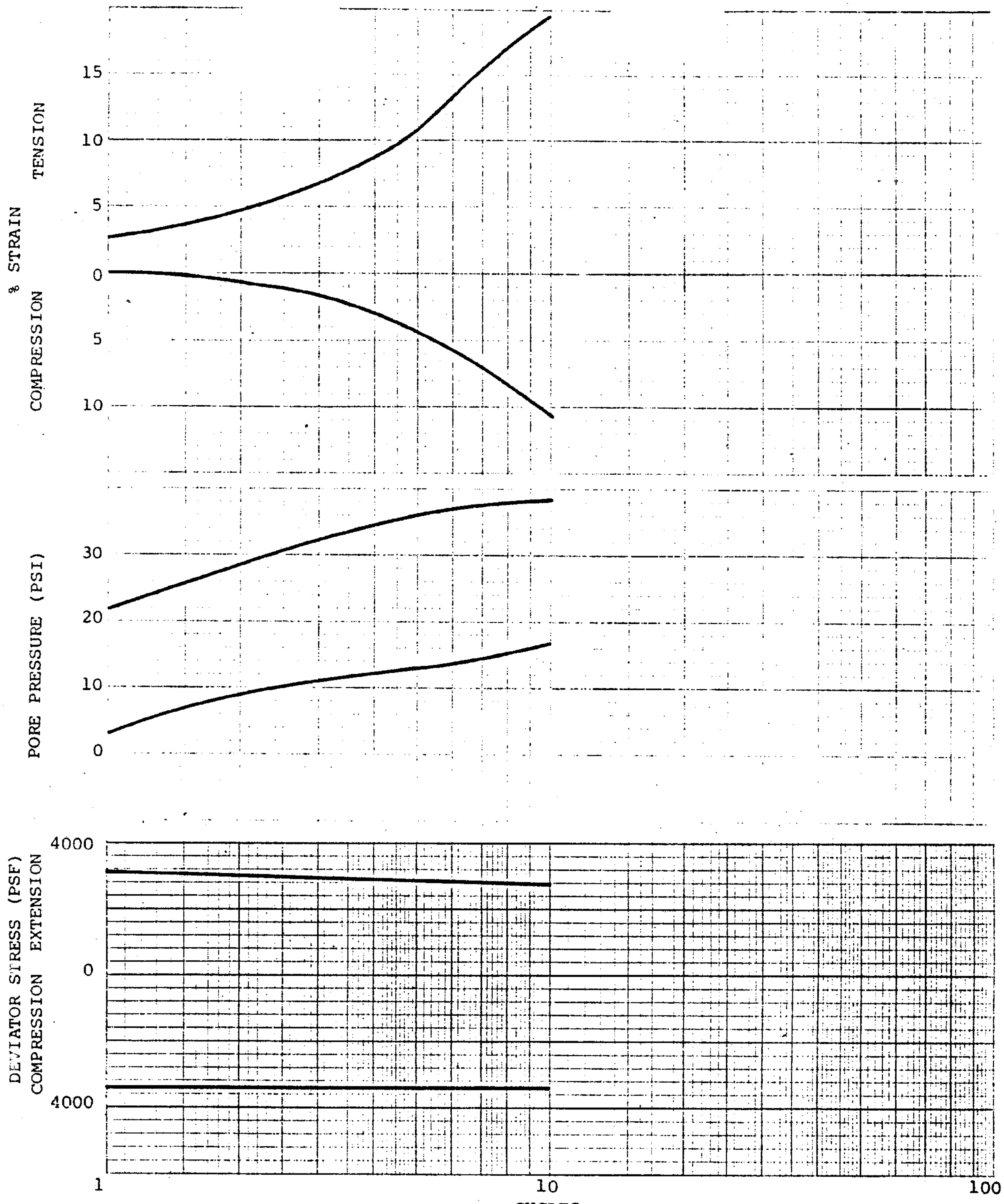
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 2 OF 13)



Chamber pressure 6000 osf
Unit dry weight 91.3 pcf
Water content 25.0%

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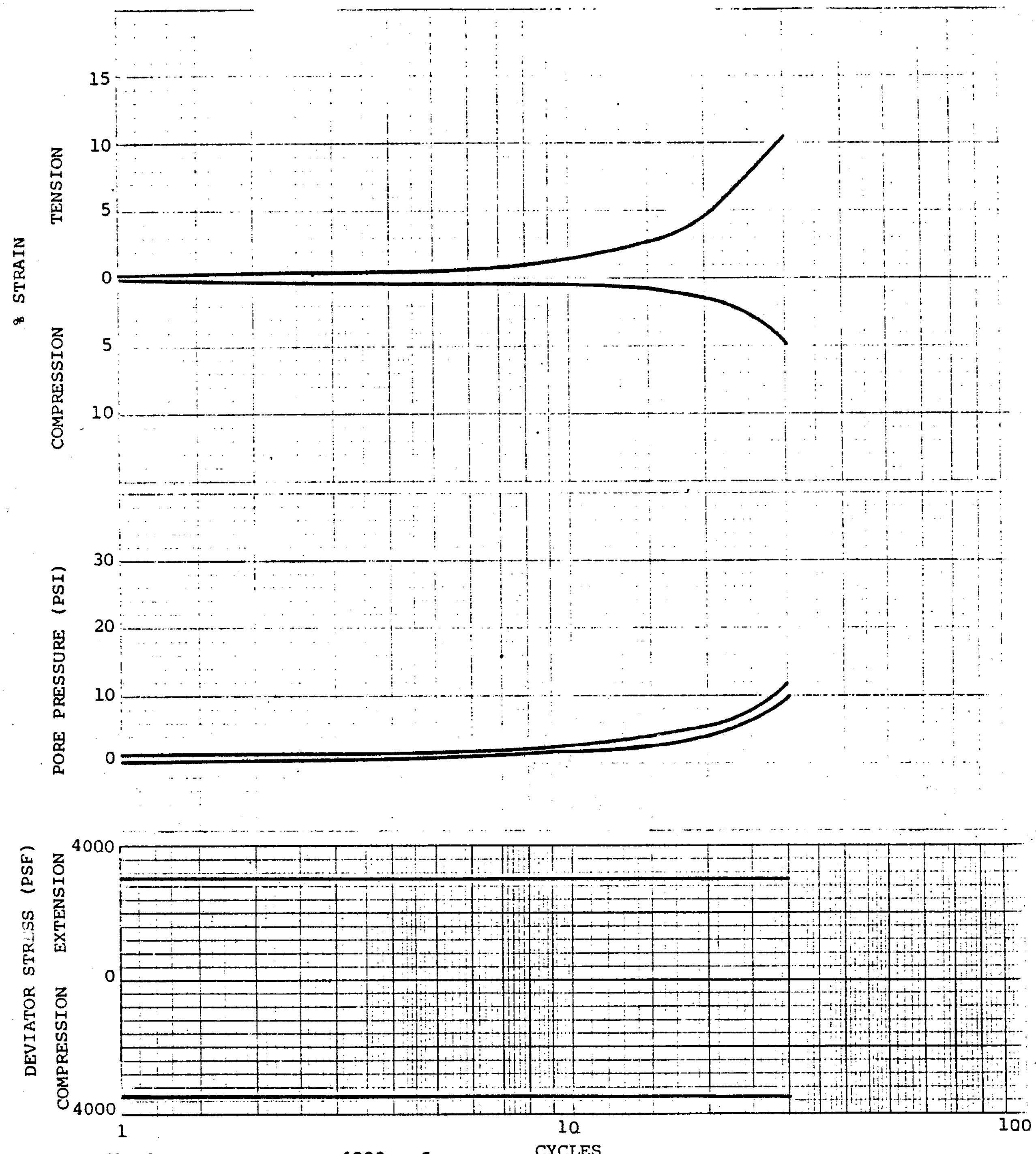
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 3 OF 13)



Chamber pressure **6000 psf**
Unit dry weight **94.5 pcf**
Water content **28.9%**

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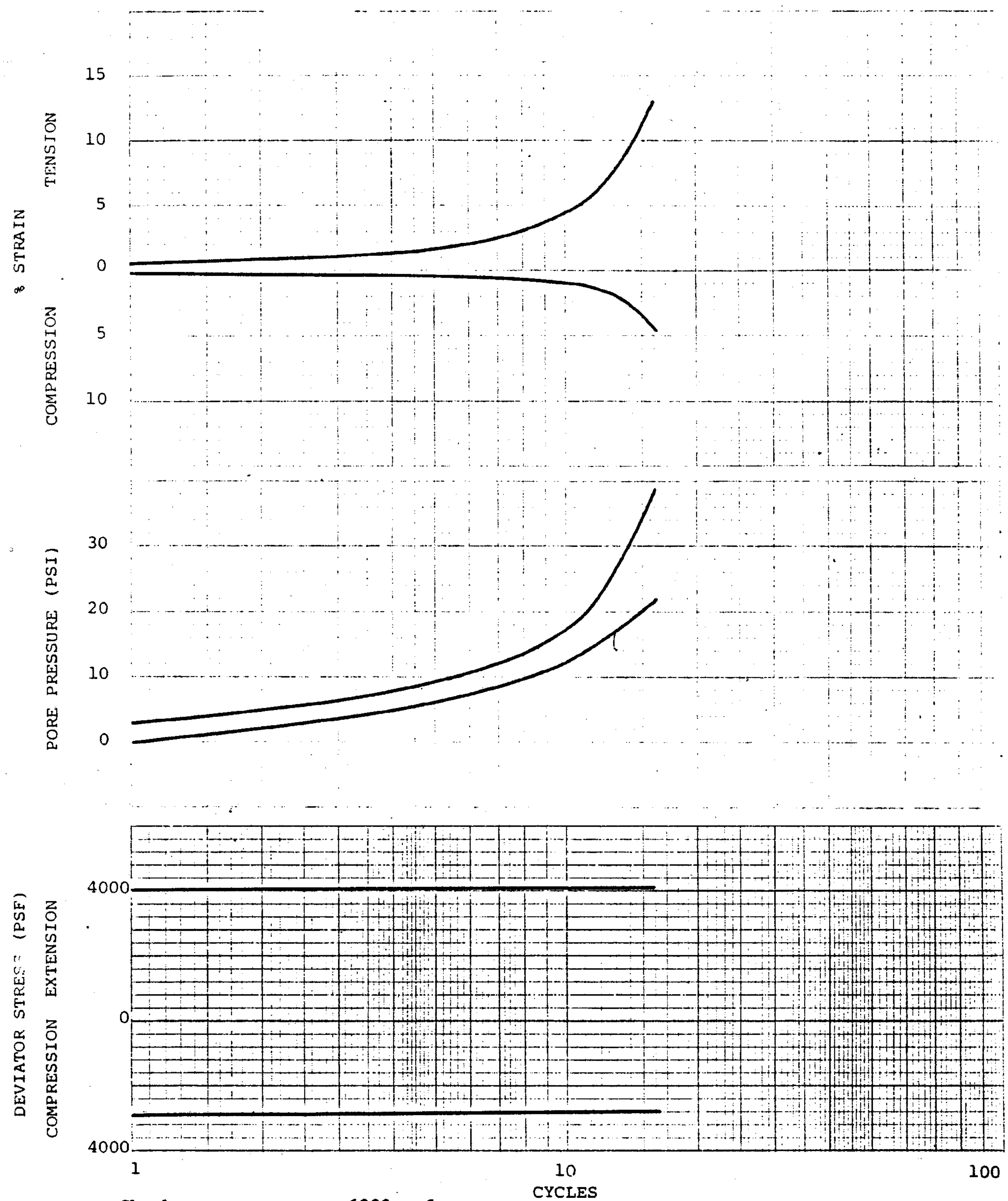
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 4 OF 13)



Chamber pressure 6000 psf
Unit dry weight 86.2 pcf
Water content 33.3%

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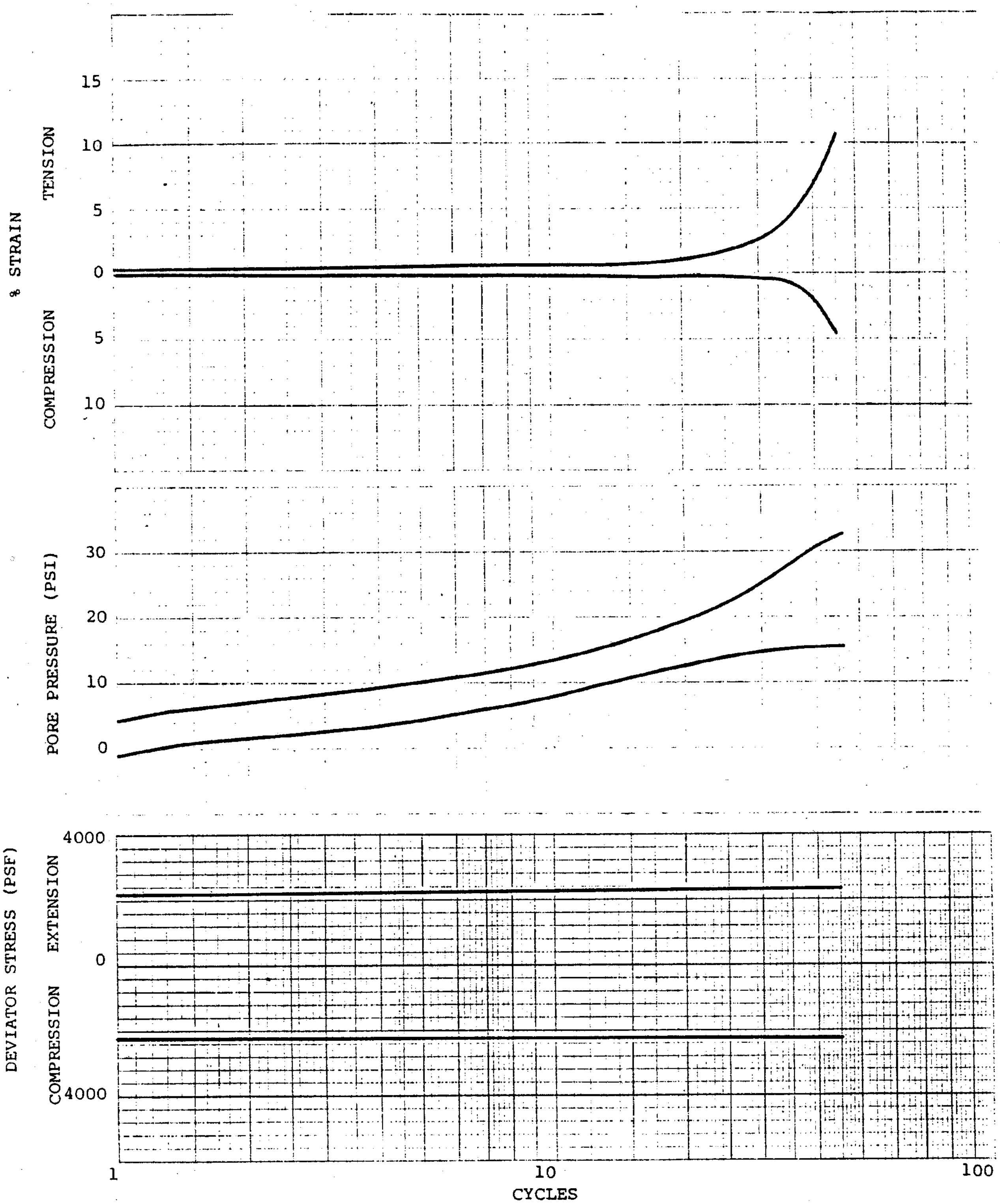
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 5 OF 13)



Chamber pressure **6000 psf**
Unit dry weight **90.9 pcf**
Water content **31.8%**

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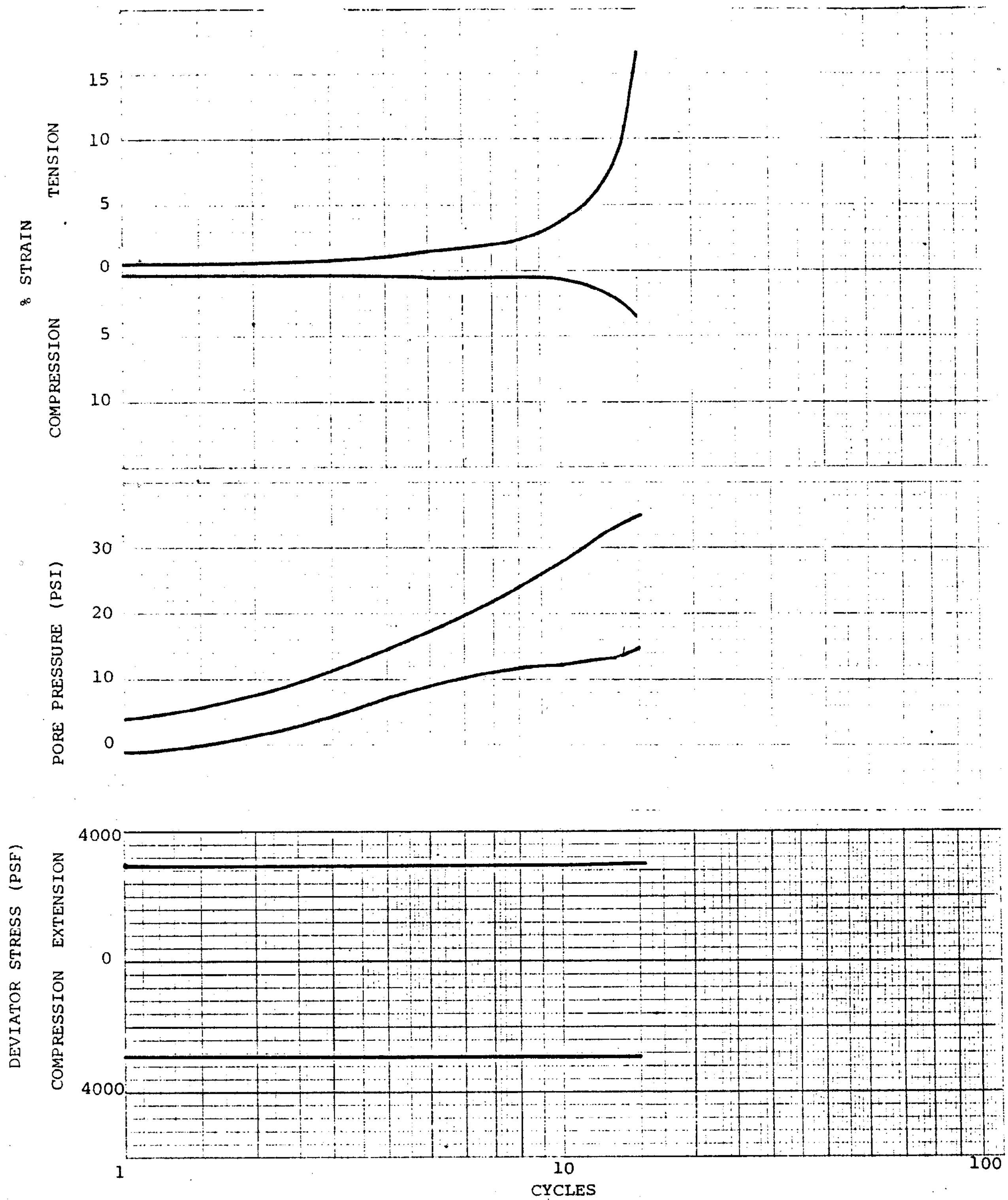
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 6 OF 13)



Chamber pressure **6000 psf**
Unit dry weight **76.0 pcf**
Water content **43.1%**

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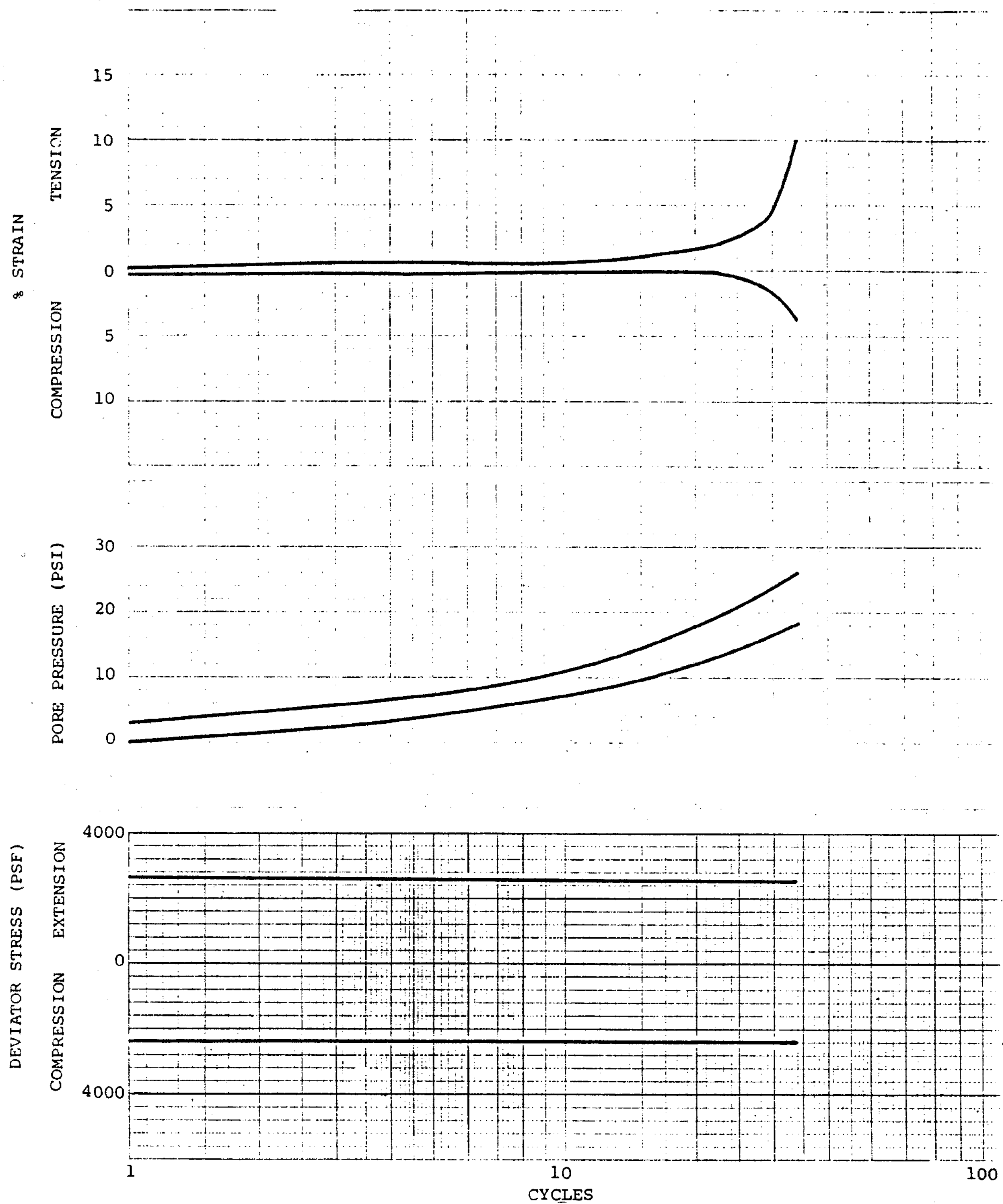
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 7 OF 13)



Chamber pressure	**6000 psf**
Unit dry weight	**74.8 pcf**
Water content	**46.3%**

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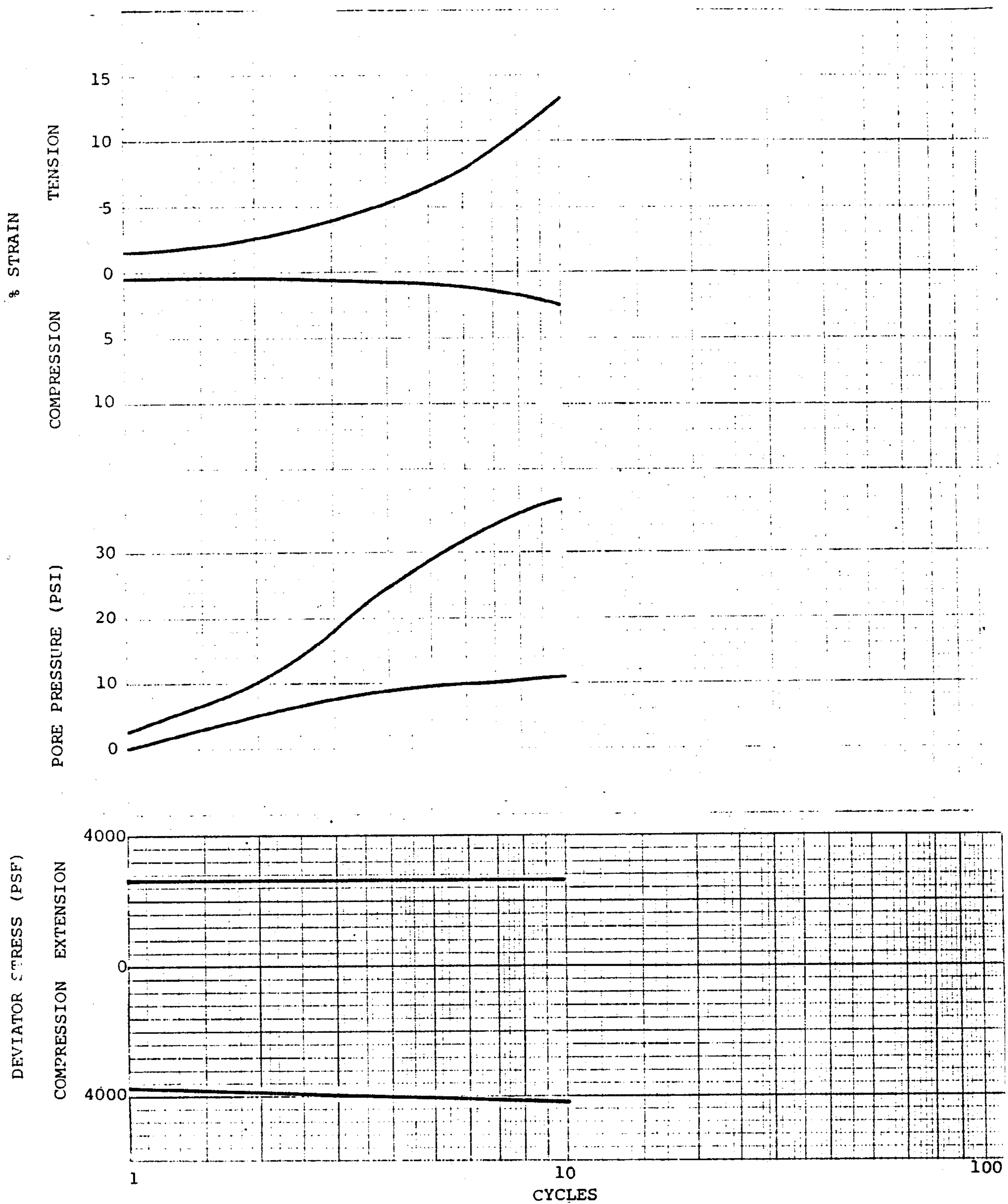
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 8 OF 13)



Chamber pressure 6000 psf
Unit dry weight 101.0 pcf
Water content 23.5%

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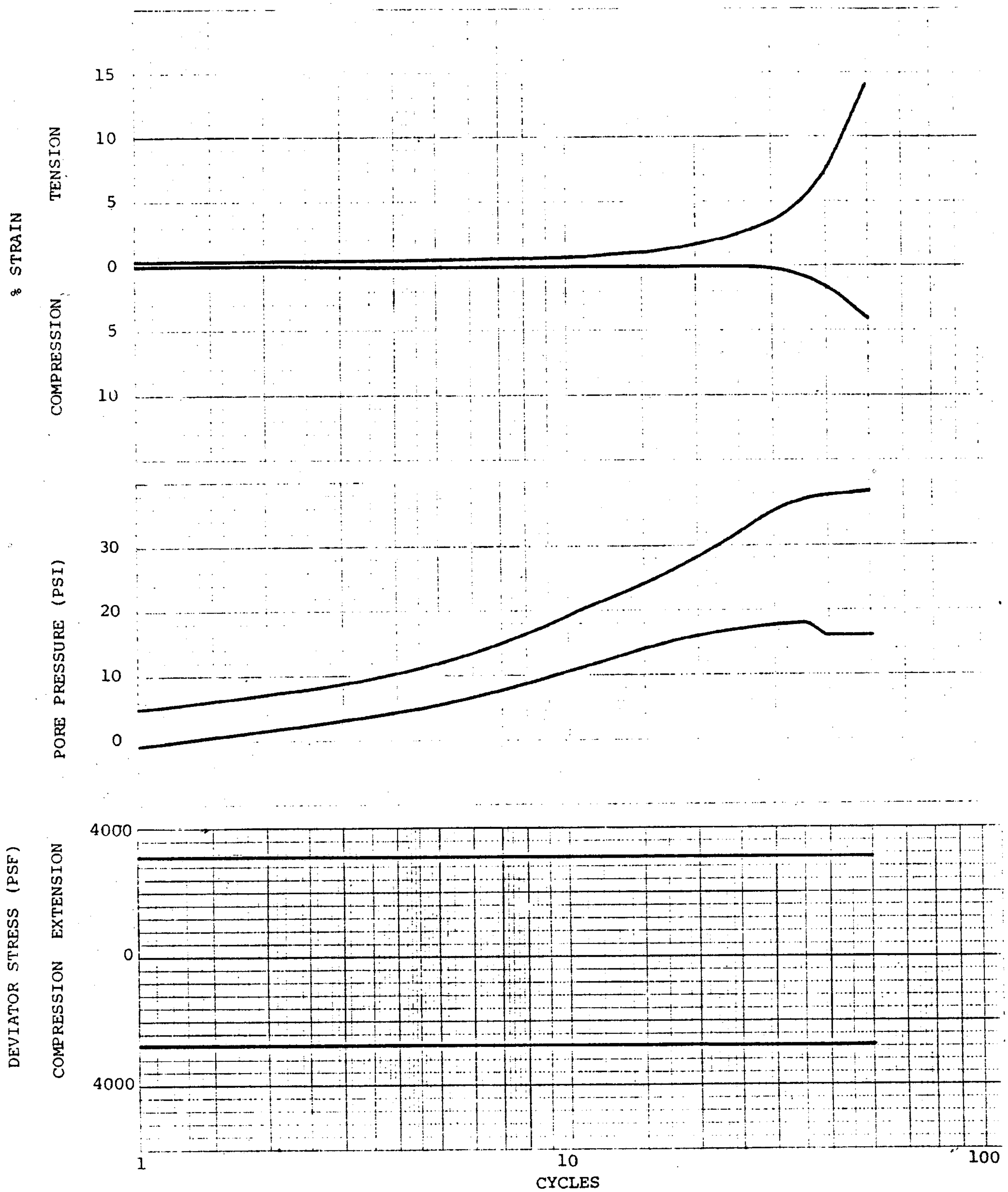
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 9 OF 13)



Chamber pressure	**6000 psf**
Unit dry weight	**92.5 pcf**
Water content	**27.4%**

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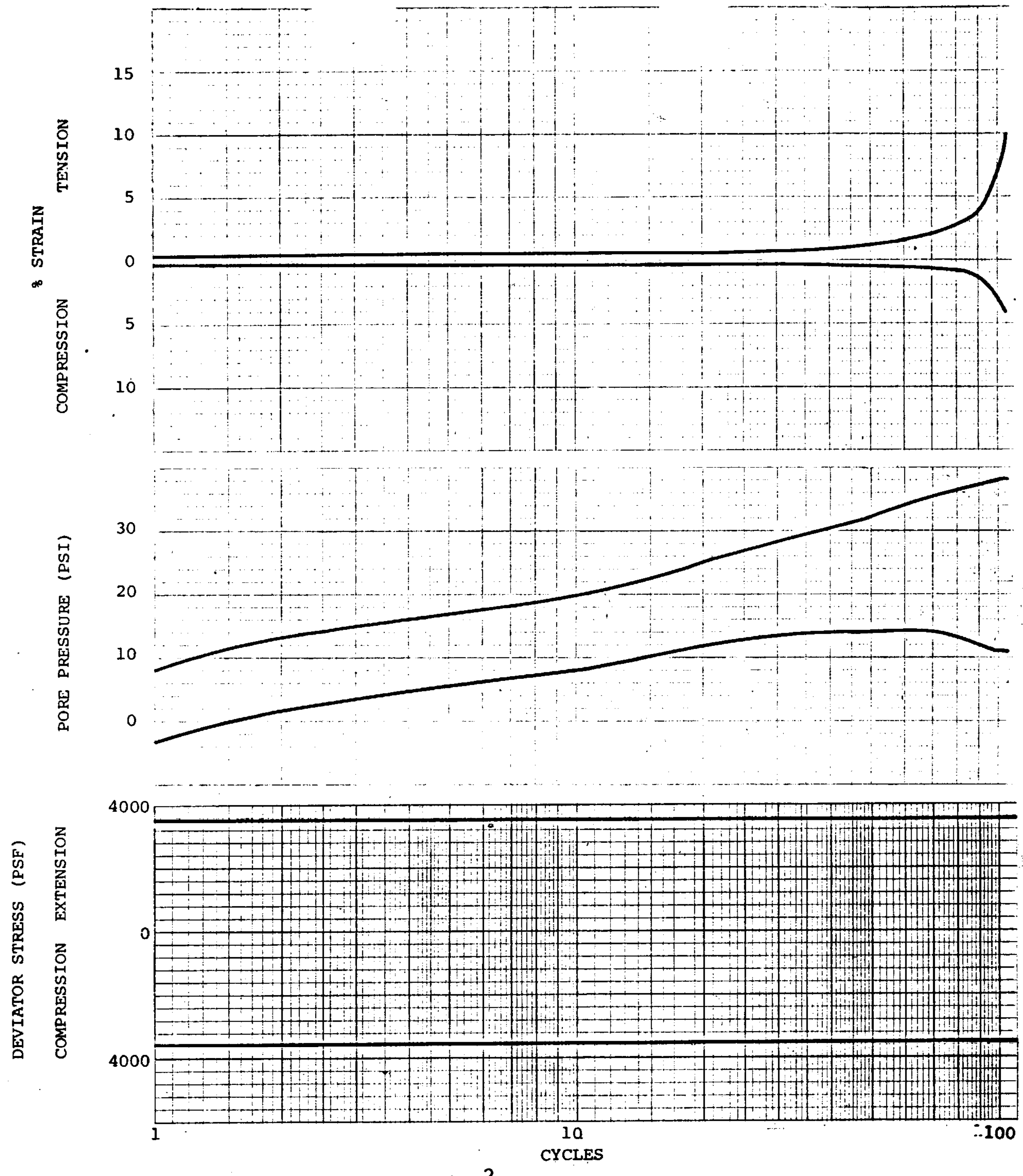
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 10 OF 13)



CHAMBER PRESSURE 6000 lb/ft^2
UNIT DRY WEIGHT 91.3 percent
WATER CONTENT 27.3 percent

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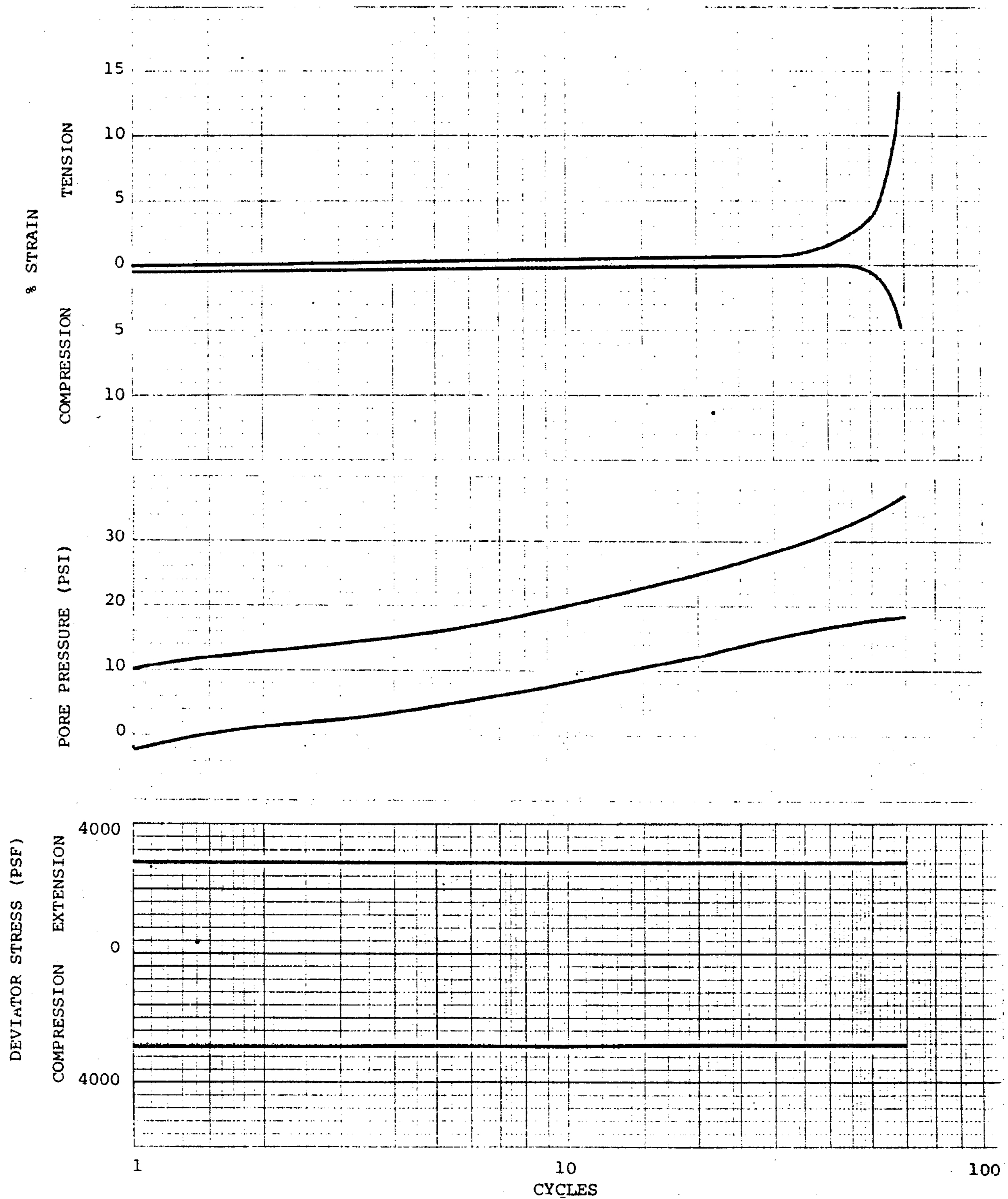
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 11 OF 13)



Chamber pressure	6000 psf
Unit dry weight	69.7 pcf
Water content	46.2%

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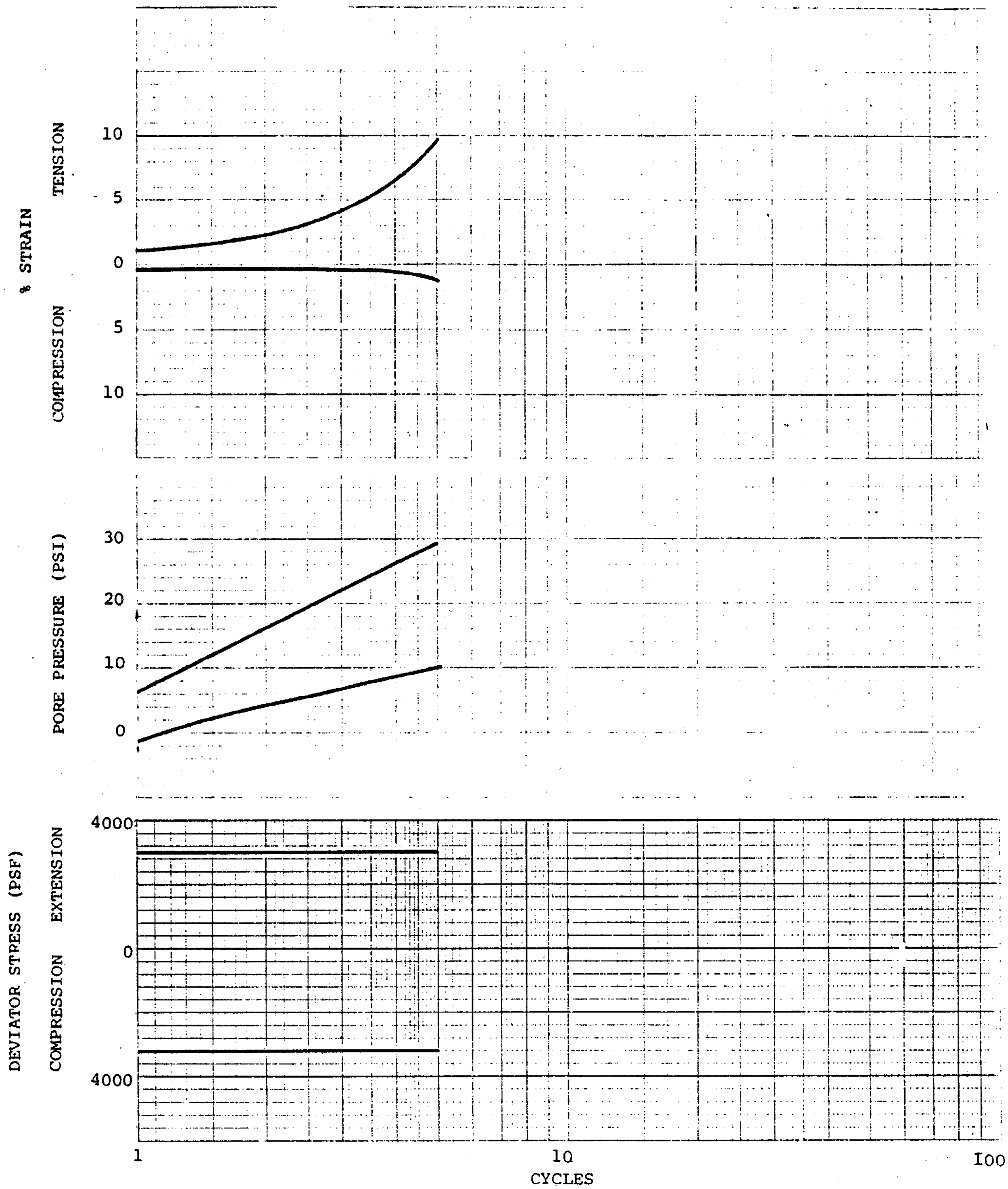
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 12 OF 13)



Chamber pressure **6000 psf**
Unit dry weight **71.0 pcf**
Water content **45.4%**

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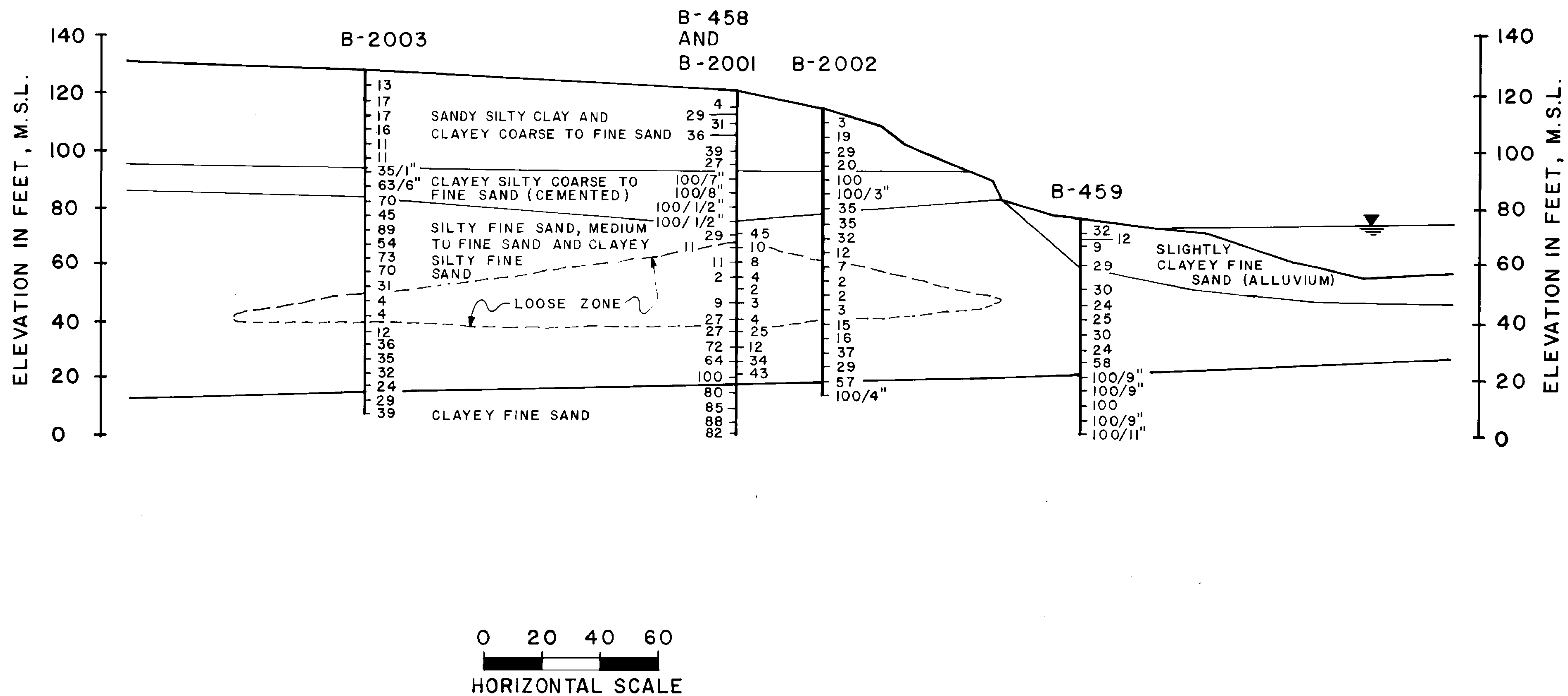
CYCLIC TRIAXIAL TEST BORING NO. 2001
SAMPLE NO. UD-6 TEST SAMPLE A

FIGURE 2A-29 (SHEET 13 OF 13)



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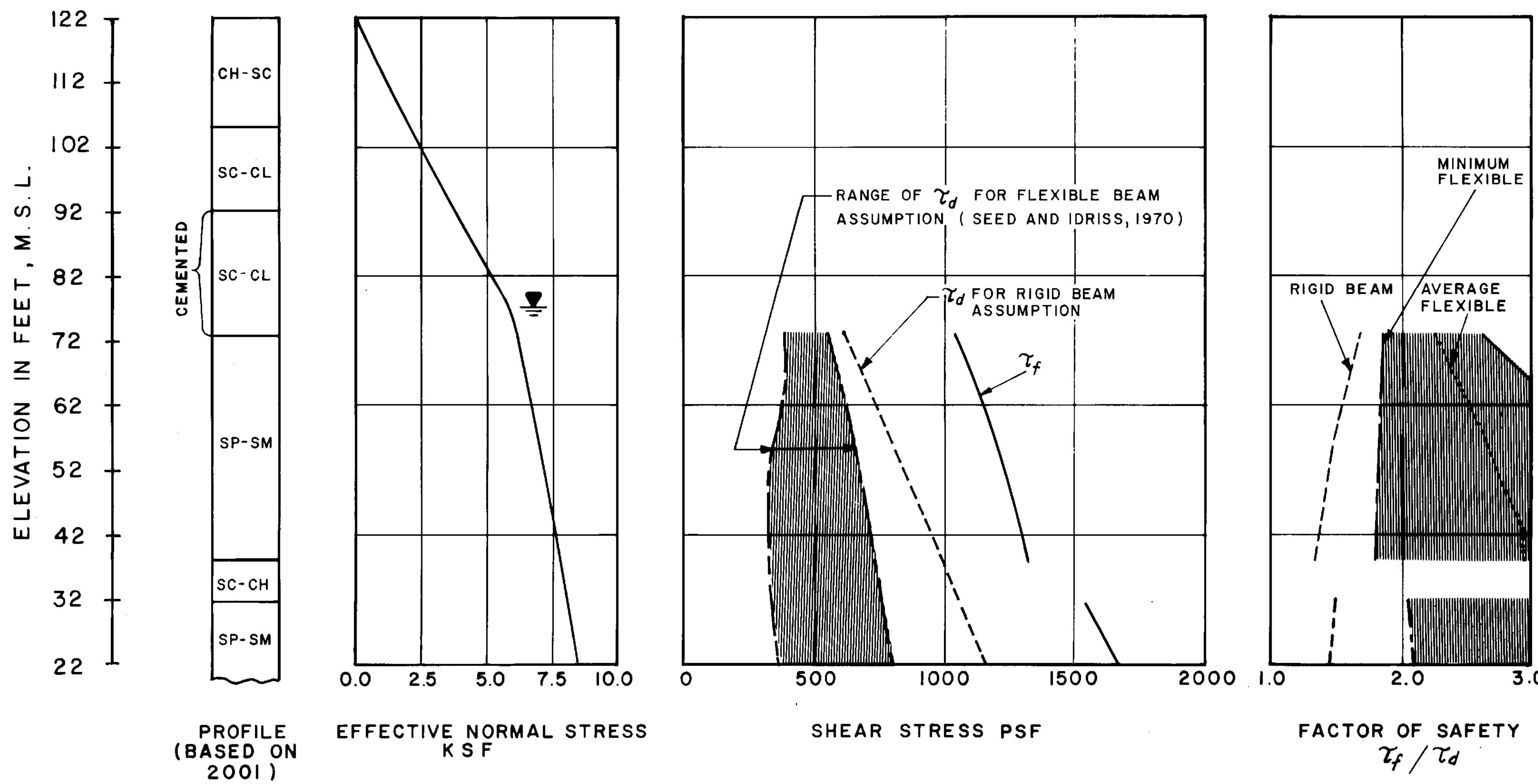
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	RIVER BANK SOIL PROFILE FIGURE 2A-30



τ_f CYCLIC SHEAR STRESS REQUIRED TO CAUSE 10 PERCENT DOUBLE AMPLITUDE STRAIN IN 5 UNIFORM STRESS CYCLES

τ_d EQUIVALENT AVERAGE UNIFORM SHEAR STRESS INDUCED BY THE DESIGN BASIS EARTHQUAKE

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FACTORS OF SAFETY AGAINST LIQUEFACTION FOR SANDS AT RIVER BLUFF

FIGURE 2A-31

KEY

SOIL 1 - SANDY SILTY CLAY AND CLAYEY COARSE TO FINE SAND

SOIL 2 - CLAYEY SILTY COARSE TO FINE SAND (CEMENTED)

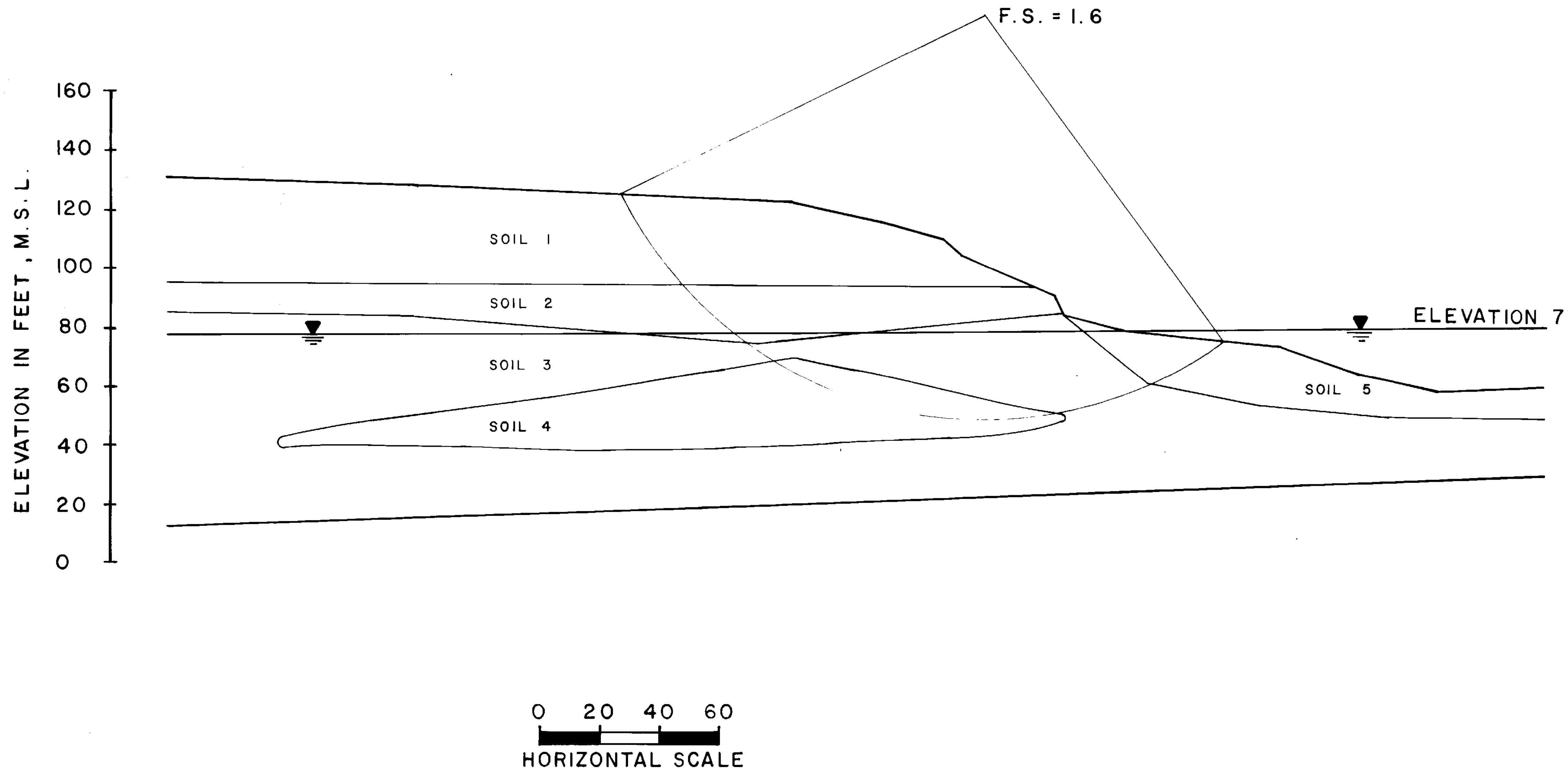
SOIL 3 - SILTY FINE SAND, MEDIUM TO FINE SAND AND CLAYEY FINE SAND

SOIL 4 - CLAYEY SILTY FINE SAND AND SILTY FINE SAND

SOIL 5 - SLIGHTLY CLAYEY FINE SAND

SOIL PROPERTY TABLE

SOIL	DENSITY	TOTAL STRESS	
	γ_T = PCF	ϕ - DEG.	C - PSF
1	135	24	1000
2	130	40	5000
3	130	36	1500
4	110	0	0
5	115	0	0



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ZERO STRENGTH ANALYSIS OF RIVER BLUFF

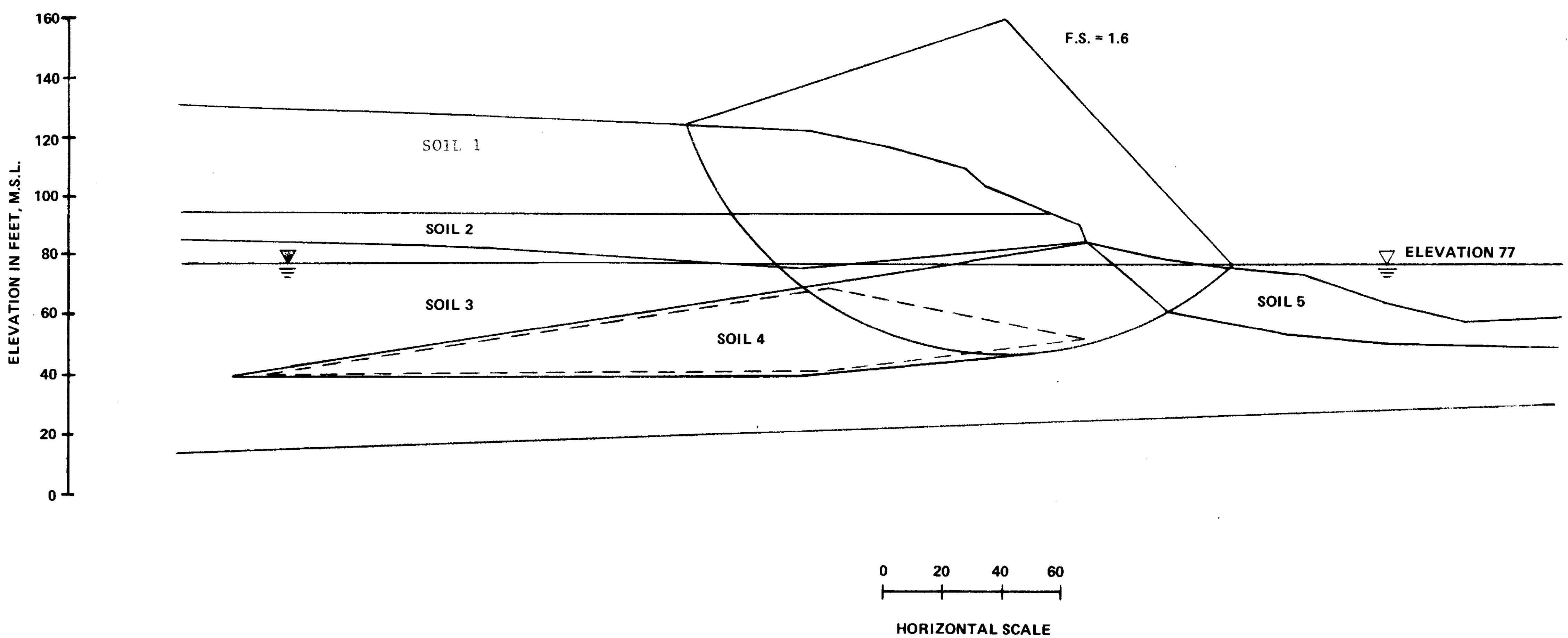
FIGURE 2A-32

KEY

SOIL 1 – SANDY SILTY CLAY AND CLAYEY COARSE TO FINE SAND
SOIL 2 – CLAYEY SILTY COARSE TO FINE SAND (CEMENTED)
SOIL 3 – SILTY FINE SAND, MEDIUM TO FINE SAND AND CLAYEY SILTY FINE SAND
SOIL 4 – CLAYEY SILTY FINE SAND AND SILTY FINE SAND
SOIL 5 – SLIGHTLY CLAYEY FINE SAND

SOIL PROPERTY TABLE

SOIL	DENSITY γ_+ – pcf	TOTAL STRENGTH ϕ – DEG.	TOTAL STRENGTH c – psf
1	135	24	1000
2	130	40	5000
3	130	36	1500
4	110	20	300
5	115	25	100



PSEUDOSTATIC STABILITY ANALYSIS OF
RIVER BLUFF FOR DESIGN BASIS
EARTHQUAKE ASSUMING EXTENDED
LOOSE ZONE OF SOIL 4

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UNIT 2

STABILITY ANALYSIS FOR DBE ASSUMING EXTENDED LOOSE ZONE OF SOIL 4

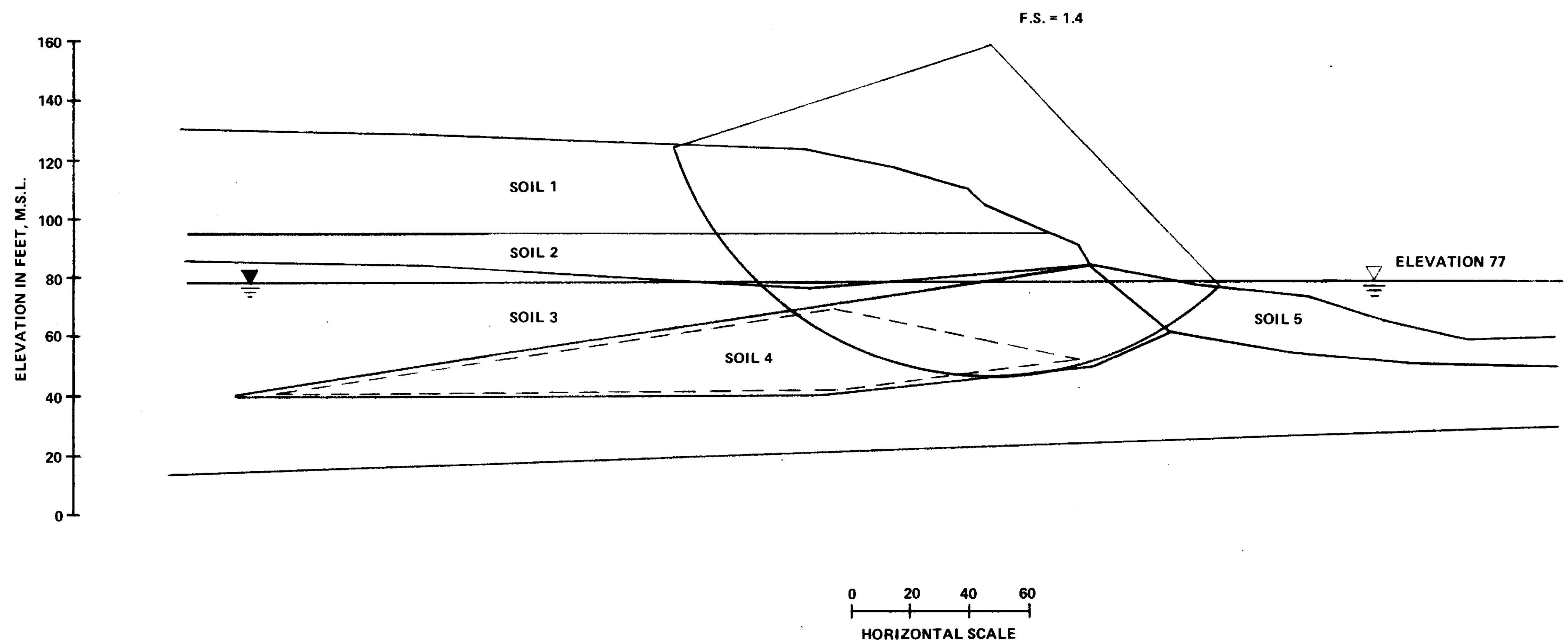
FIGURE 2A-33

KEY

SOIL 1 – SANDY SILTY CLAY AND CLAYEY COARSE TO FINE SAND
SOIL 2 – CLAYEY SILTY COARSE TO FINE SAND (CEMENTED)
SOIL 3 – SILTY FINE SAND, MEDIUM TO FINE SAND AND CLAYEY SILTY FINE SAND
SOIL 4 – CLAYEY SILTY FINE SAND AND SILTY FINE SAND
SOIL 5 – SLIGHTLY CLAYEY FINE SAND

SOIL PROPERTY TABLE

SOIL	DENSITY γ_+ – pcf	TOTAL STRENGTH ϕ – DEG.	c – psf
1	135	24	1000
2	130	50	0
3	130	36	1500
4	110	20	300
5	115	25	100



PSEUDOSTATIC STABILITY ANDLYSIS OF
RIVER BLUFF FOR DESIGN BASIS
EARTHQUAKE ASSUMING EXTENDED
LOOSE ZONE SOIL 4 AND REEVALUATED
STRENGTH FOR SOIL 2

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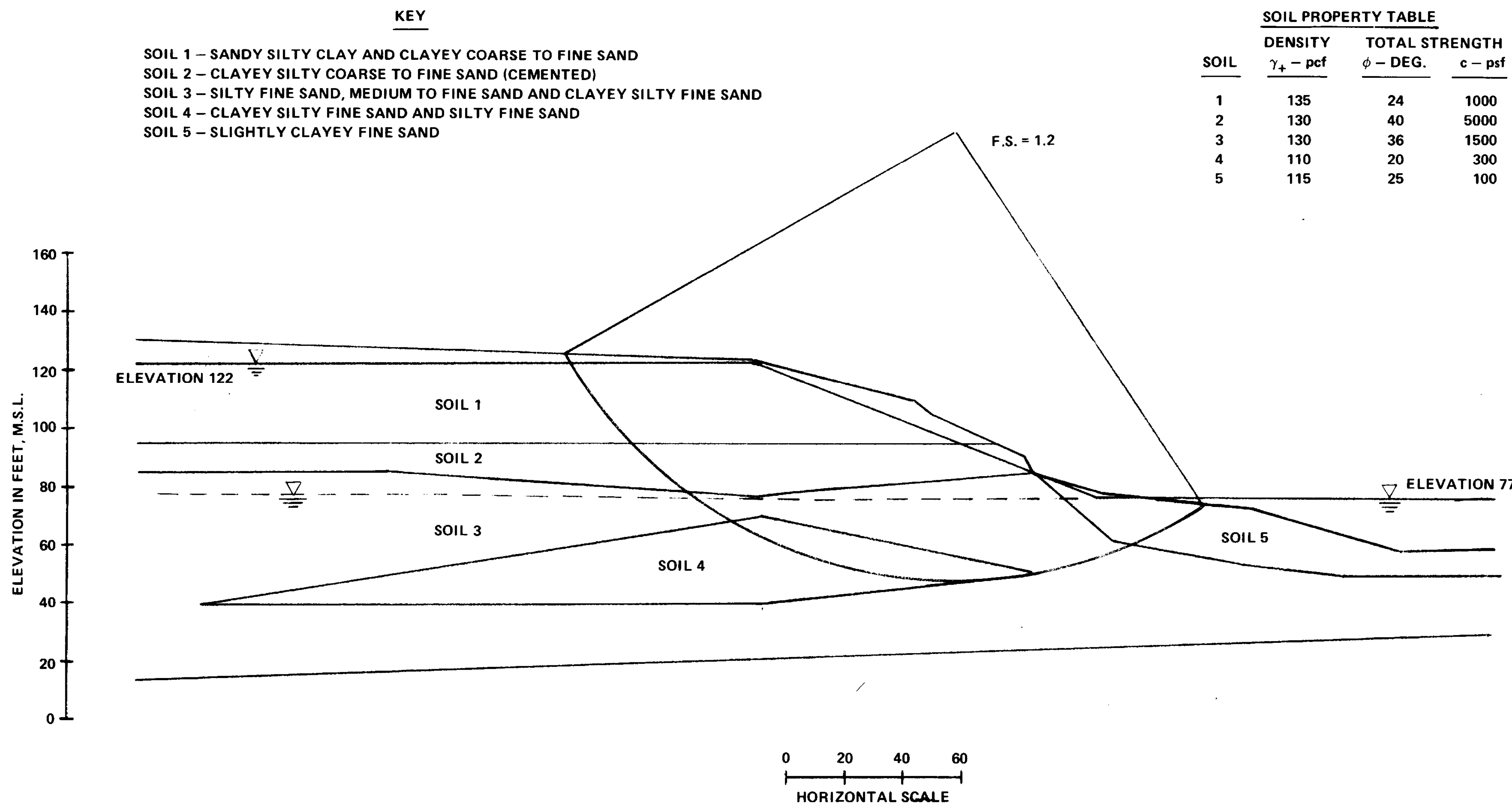
STABILITY ANALYSIS FOR DBE
EXTENDED LOOSE ZONE OF SOIL 4 AND
REEVALUATED STRENGTH FOR SOIL 2

FIGURE 2A-34



SOIL	DENSITY γ_+ – pcf	TOTAL STRENGTH ϕ – DEG.	c – psf
1	135	24	1000
2	130	40	5000
3	130	36	1500
4	110	20	300
5	115	25	100

PSEUDOSTATIC STABILITY ANALYSIS OF RIVER BLUFF FOR DESIGN BASIS EARTHQUAKE ASSUMING WATER IN SLOPE AT el 122 ft

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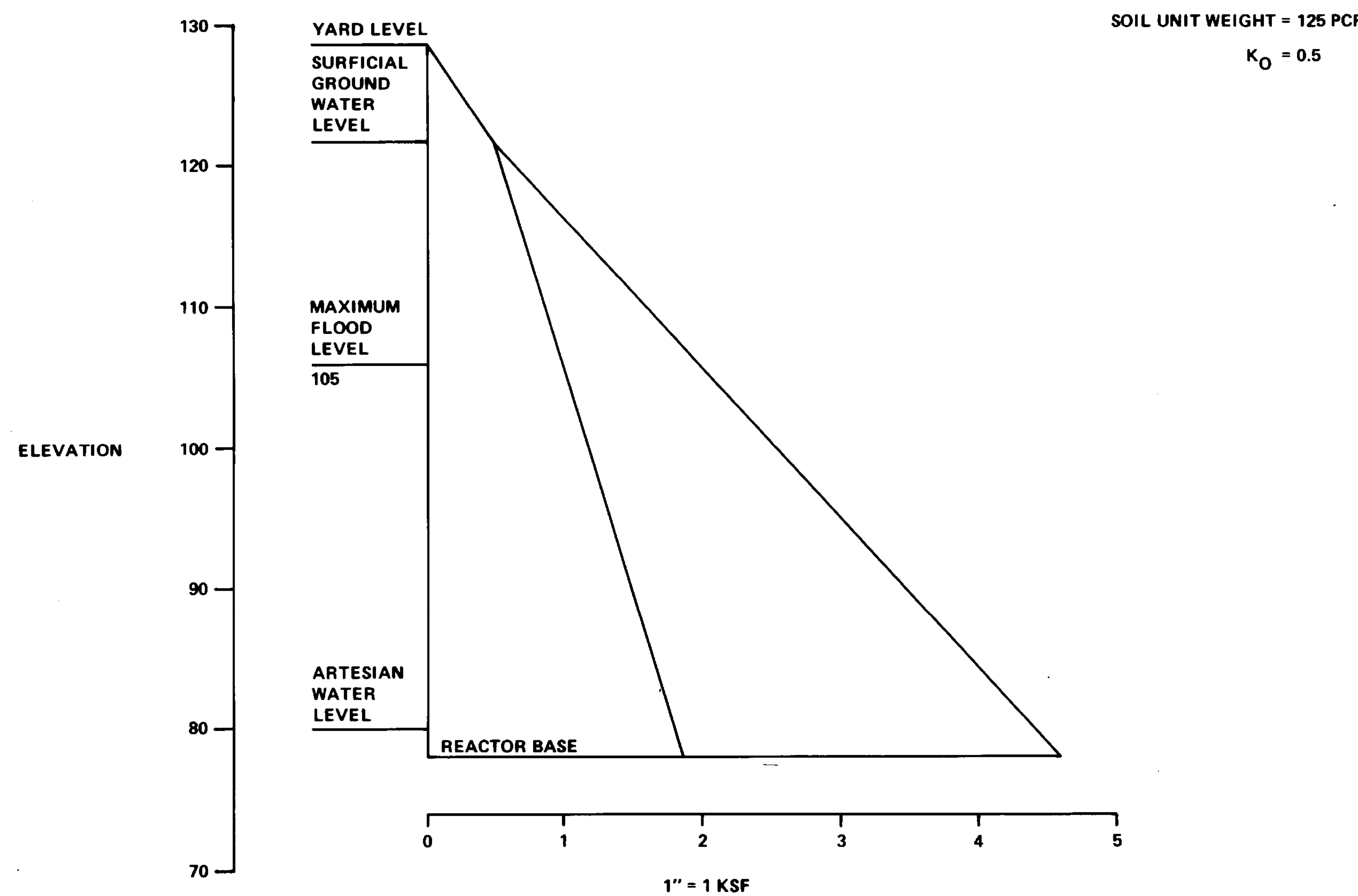
STABILITY ANALYSIS FOR DBE ASSUMING WATER IN SLOPE AT el 122 ft

FIGURE 2A-35



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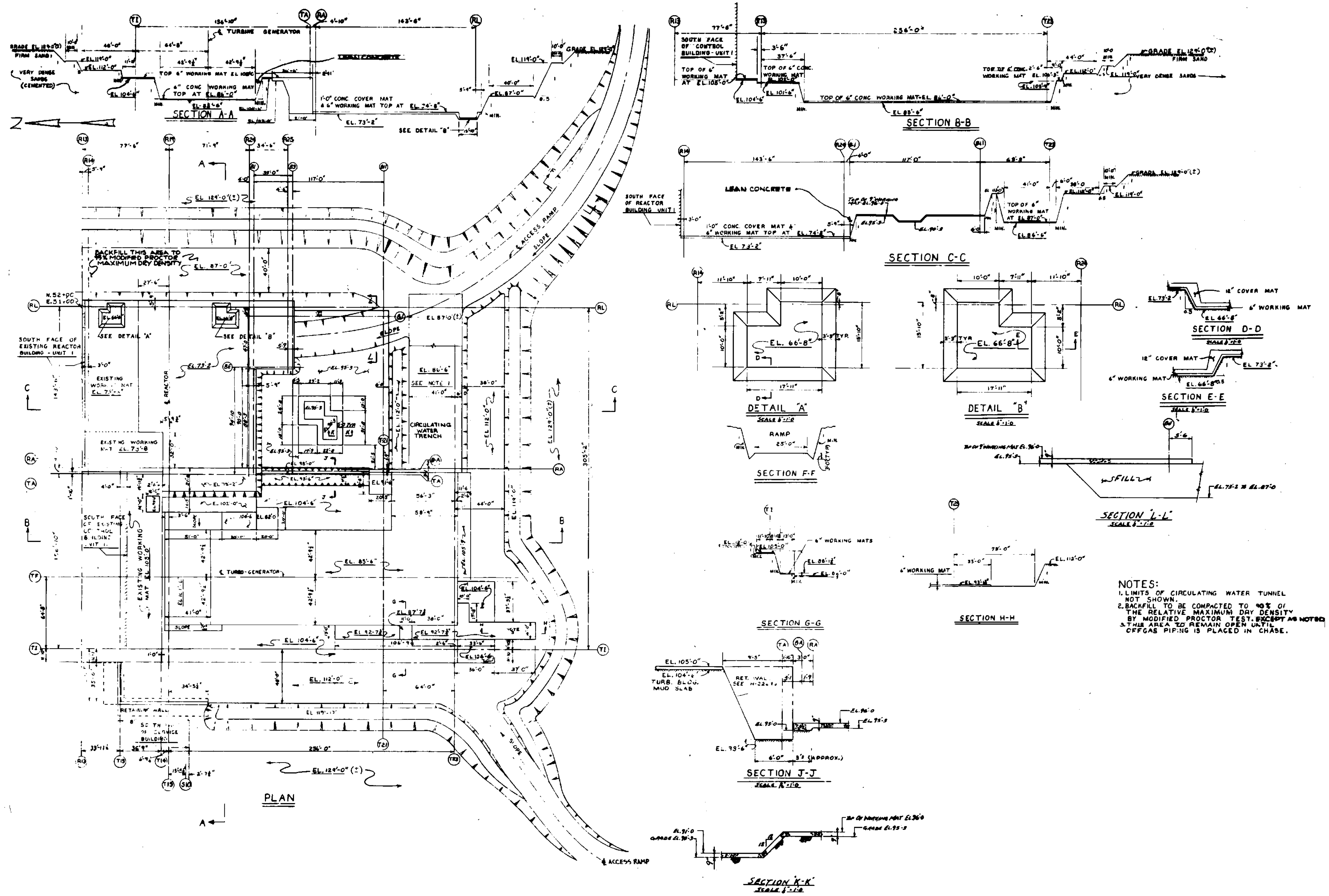
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	EARTH PRESSURE DIAGRAM FIGURE 2A-36



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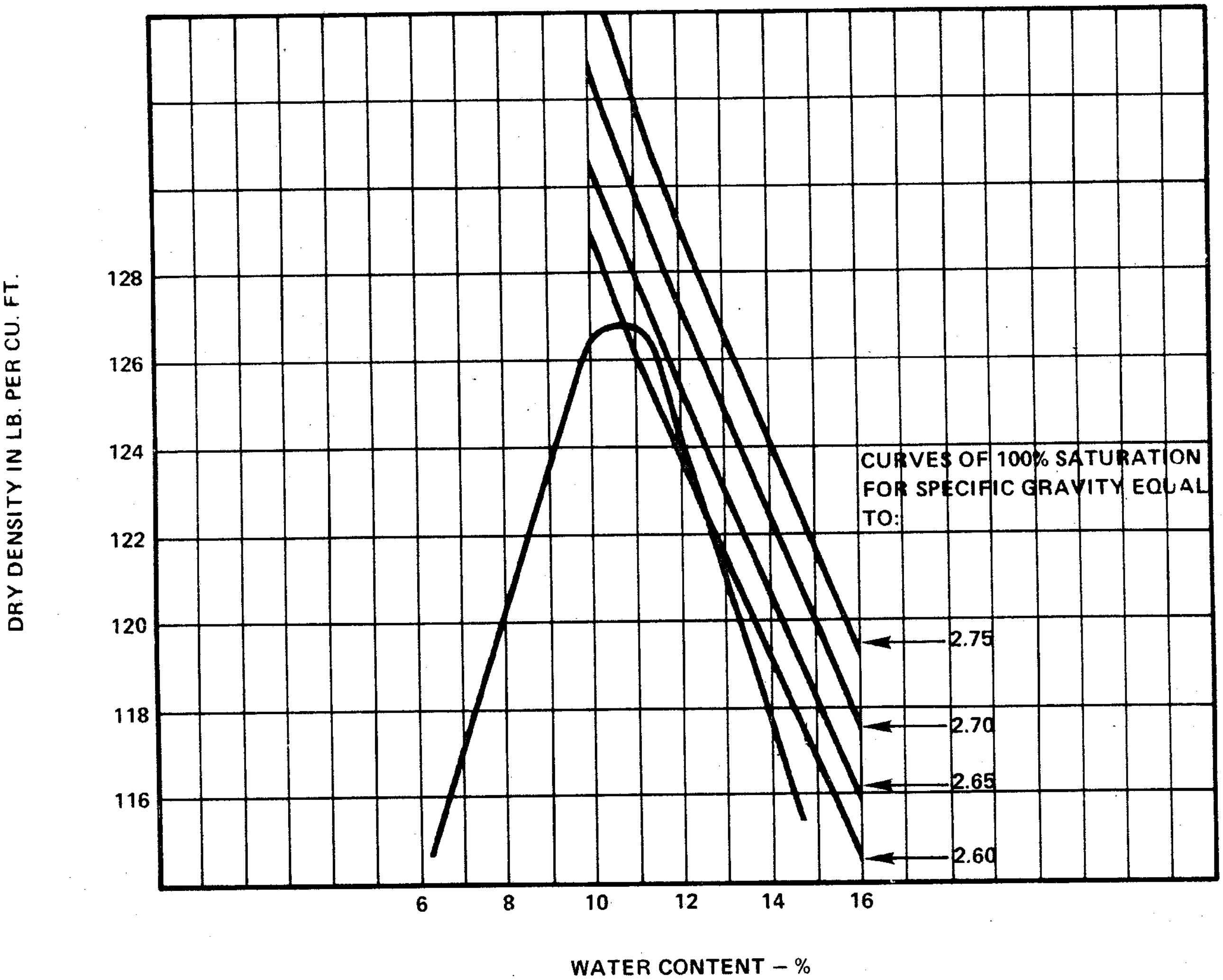
EXCAVATION PLAN AND SECTIONS

FIGURE 2A-37



SAMPLE NO. 1

MAXIMUM DRY DENSITY 126.8 LB. PER CU. FT.

OPTIMUM MOISTURE 10.6 %

METHOD OF TEST – ASTM 1557, METHOD C

TYPE MATERIAL – BROWN SILTY CLAYEY MEDIUM TO FINE SAND

SAMPLE FROM – B-411, EL. 139 1/2 - 144

FIELD MOISTURE –

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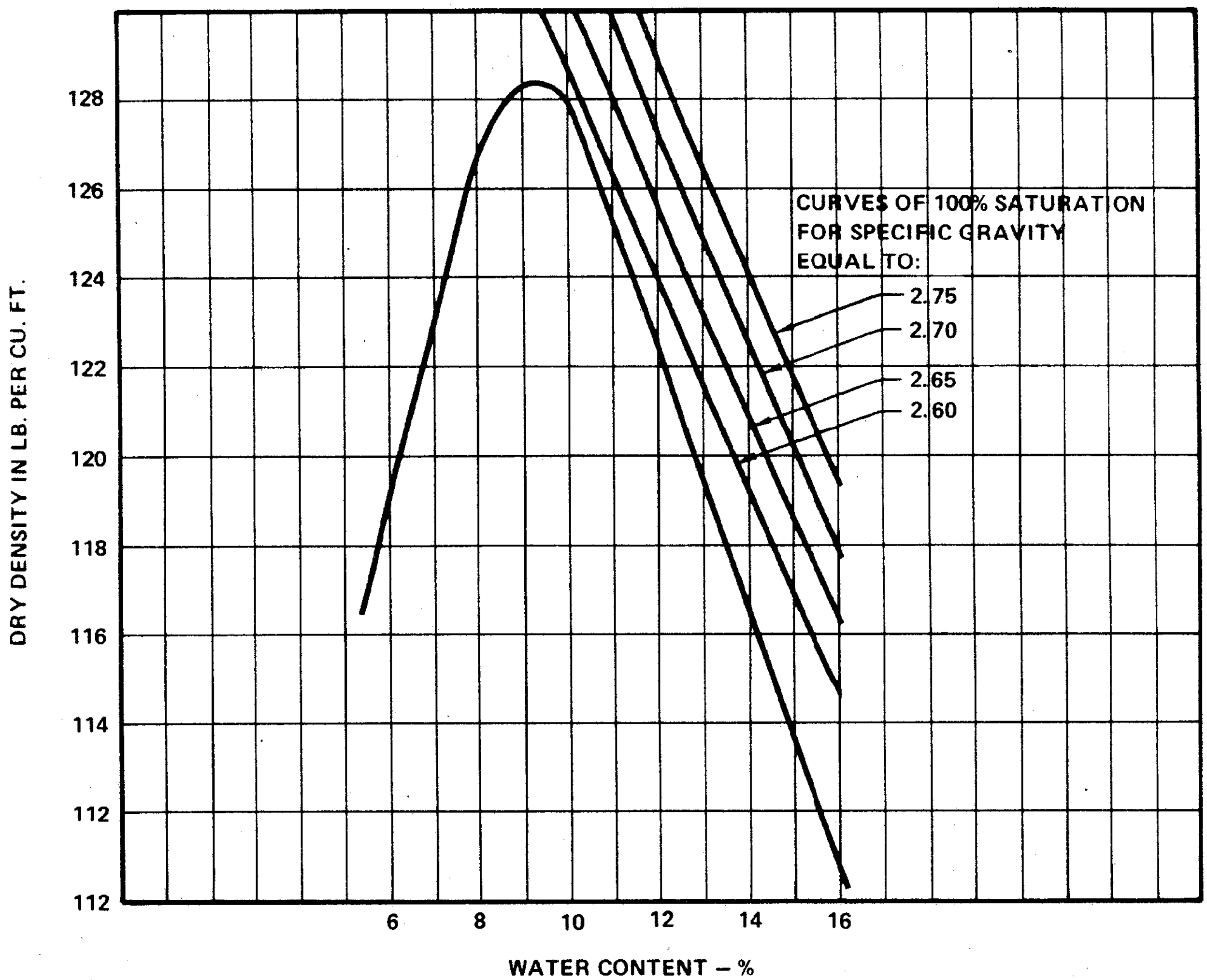
UNIT 2

COMPACTION TEST

FIGURE 2A-38 (SHEET 1 OF 9)



SAMPLE NO. 2

MAXIMUM DRY DENSITY 128.3 LB. PER CU. FT.

OPTIMUM MOISTURE 9.3 %

METHOD OF TEST – ASTM, D 1557, METHOD C

TYPE MATERIAL – BROWN SILTY CLAYEY MEDIUM TO FINE SAND

SAMPLE FROM – B-411, EL. 139 1/2 - 144

FIELD MOISTURE –

ACAD

HISTORICAL

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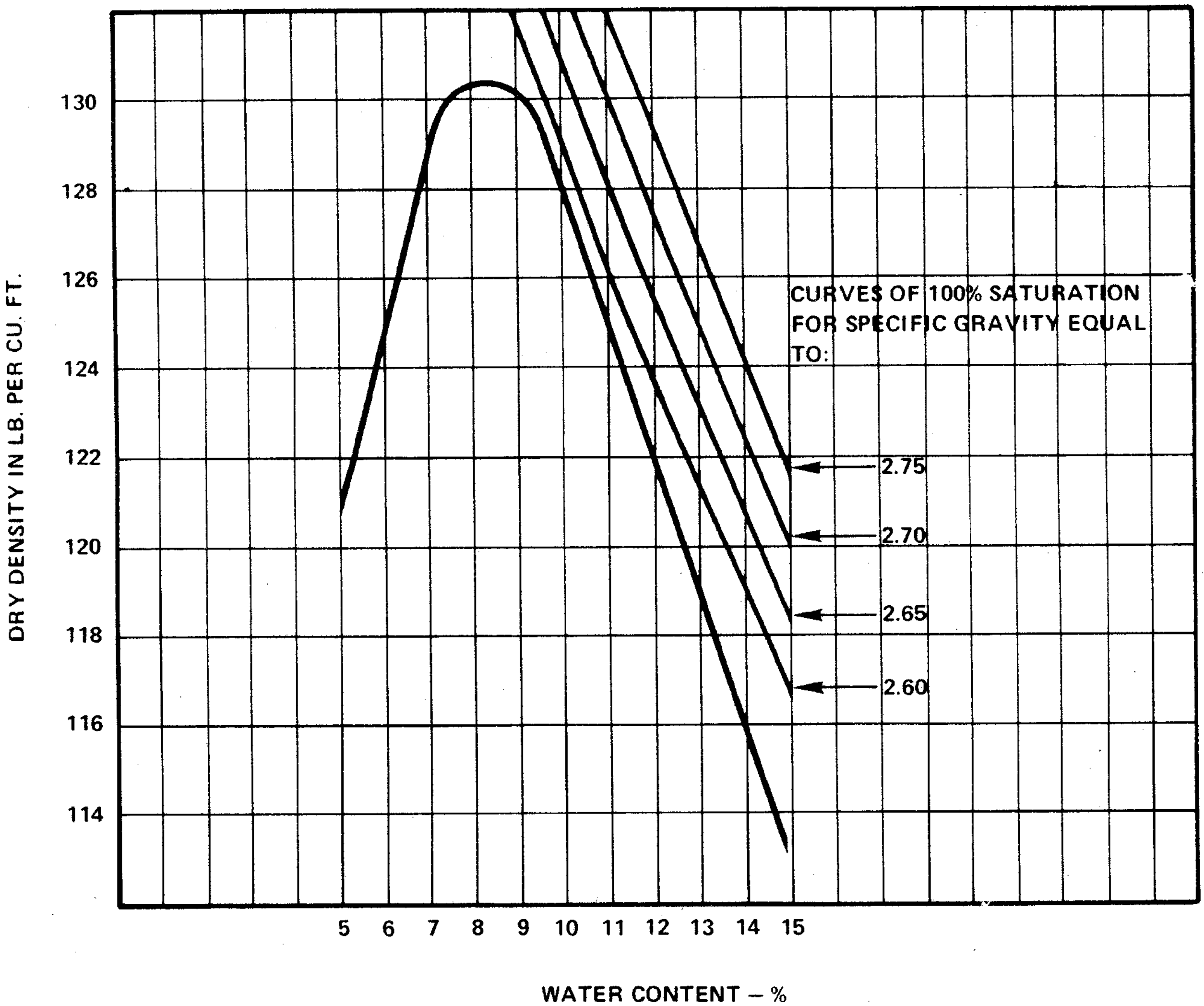
UNIT 2

COMPACTION TEST

FIGURE 2A-38 (SHEET 2 OF 9)



SAMPLE NO. 3

MAXIMUM DRY DENSITY 130.4 LB. PER CU. FT.

OPTIMUM MOISTURE 8.2 %

METHOD OF TEST – ASTM D 1557, METHOD C

TYPE MATERIAL – RED BROWN SILTY CLAYEY MEDIUM TO FINE SAND

SAMPLE FROM – B-411, EL. 123 - 128

FIELD MOISTURE –

ACAD

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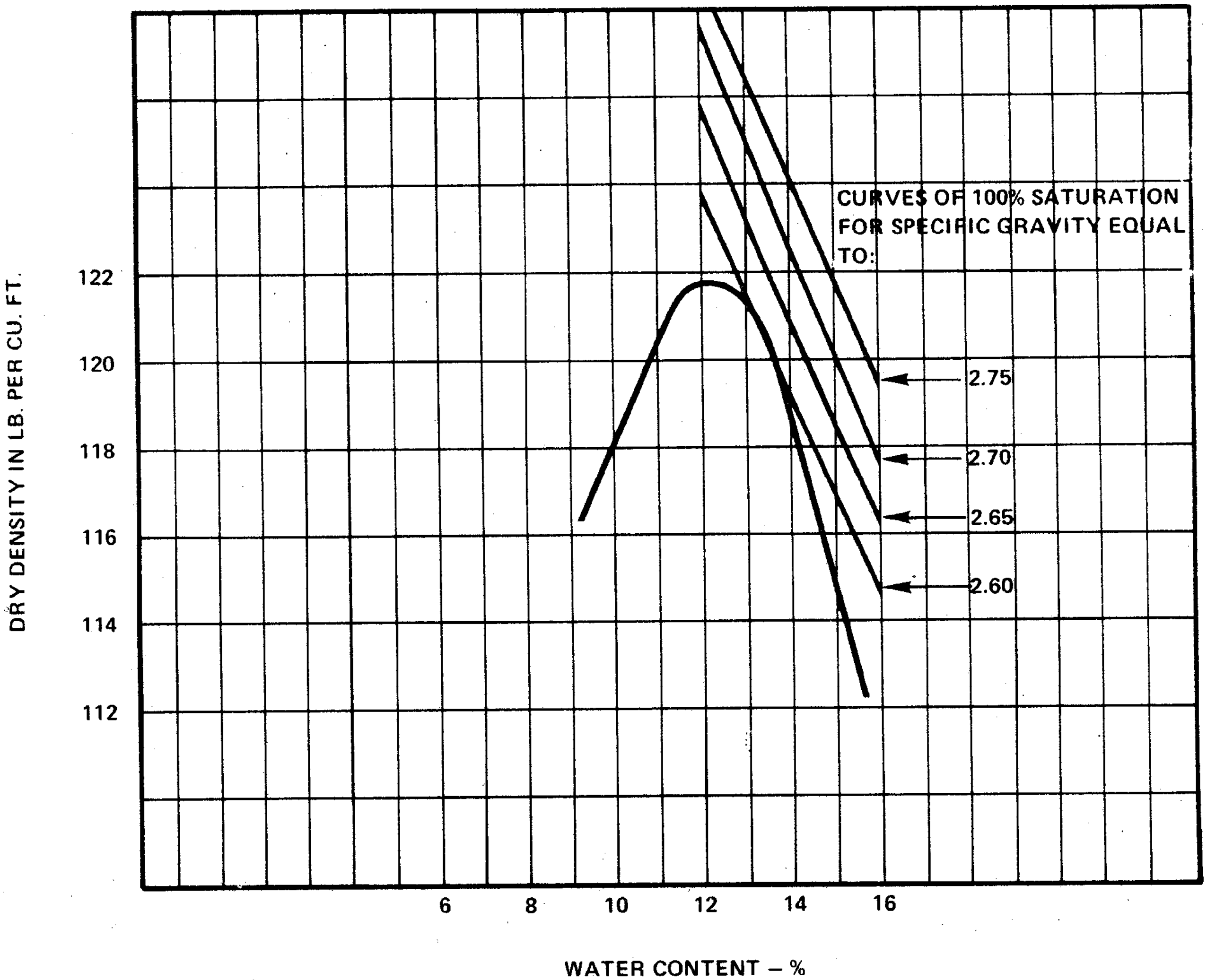
UNIT 2

COMPACTION TEST

FIGURE 2A-38 (SHEET 3 OF 9)



SAMPLE NO. 4

MAXIMUM DRY DENSITY 121.8 LB. PER CU. FT.

OPTIMUM MOISTURE 12.3 %

METHOD OF TEST – ASTM D 1557, METHOD C

TYPE MATERIAL – LIGHT BROWN MEDIUM TO FINE SANDY CLAY

SAMPLE FROM – B-411, EL. 128 - 132

FIELD MOISTURE –

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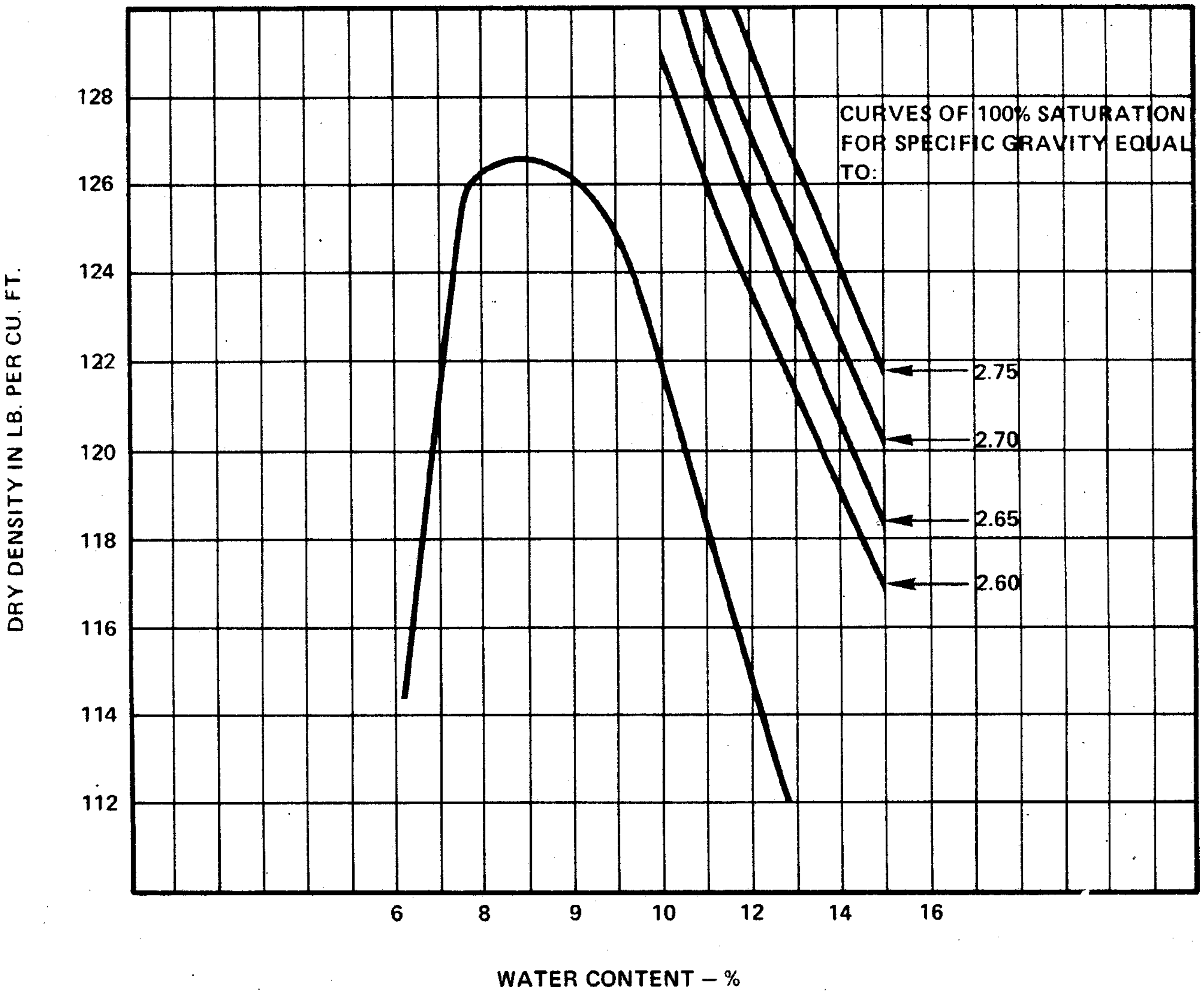
UNIT 2

COMPACTION TEST

FIGURE 2A-38 (SHEET 4 OF 9)



SAMPLE NO. B-405

MAXIMUM DRY DENSITY 126.6 LB. PER CU. FT.

OPTIMUM MOISTURE 8.8 %

METHOD OF TEST – ASTM D 1557, METHOD C

TYPE MATERIAL – RED BROWN SILTY CLAYEY MEDIUM TO FINE SAND

SAMPLE FROM – B-405, EL. 138 - 15 1 1/2

FIELD MOISTURE –

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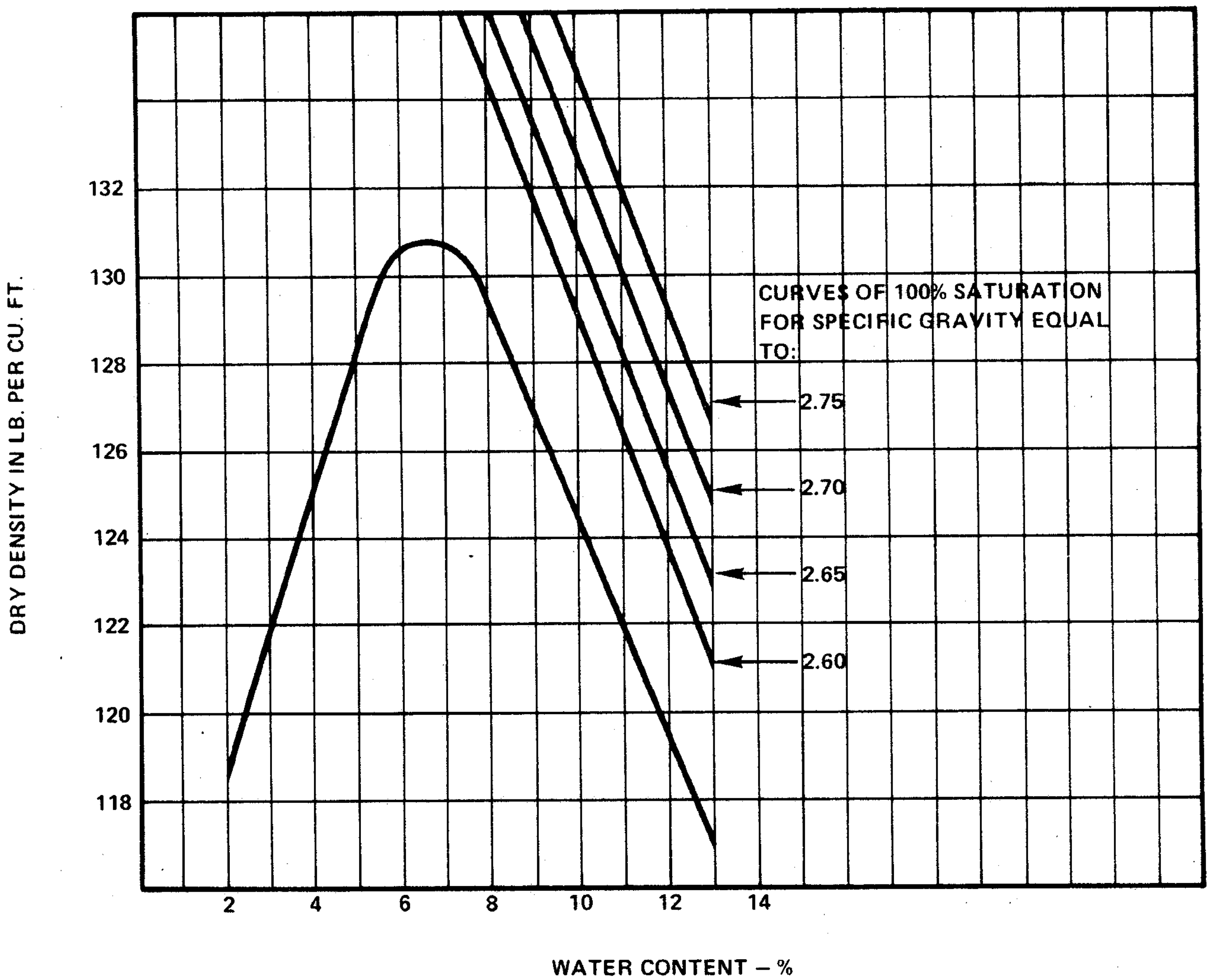
UNIT 2

COMPACTION TEST

FIGURE 2A-38 (SHEET 5 OF 9)



SAMPLE NO. ______________

MAXIMUM DRY DENSITY 130.7 LB. PER CU. FT.

OPTIMUM MOISTURE 6.7 %

METHOD OF TEST – ASTM D 1557, METHOD C

TYPE MATERIAL – BROWN CLAYEY MEDIUM TO FINE SAND

SAMPLE FROM – B-418, EL. 136 - 142

FIELD MOISTURE –

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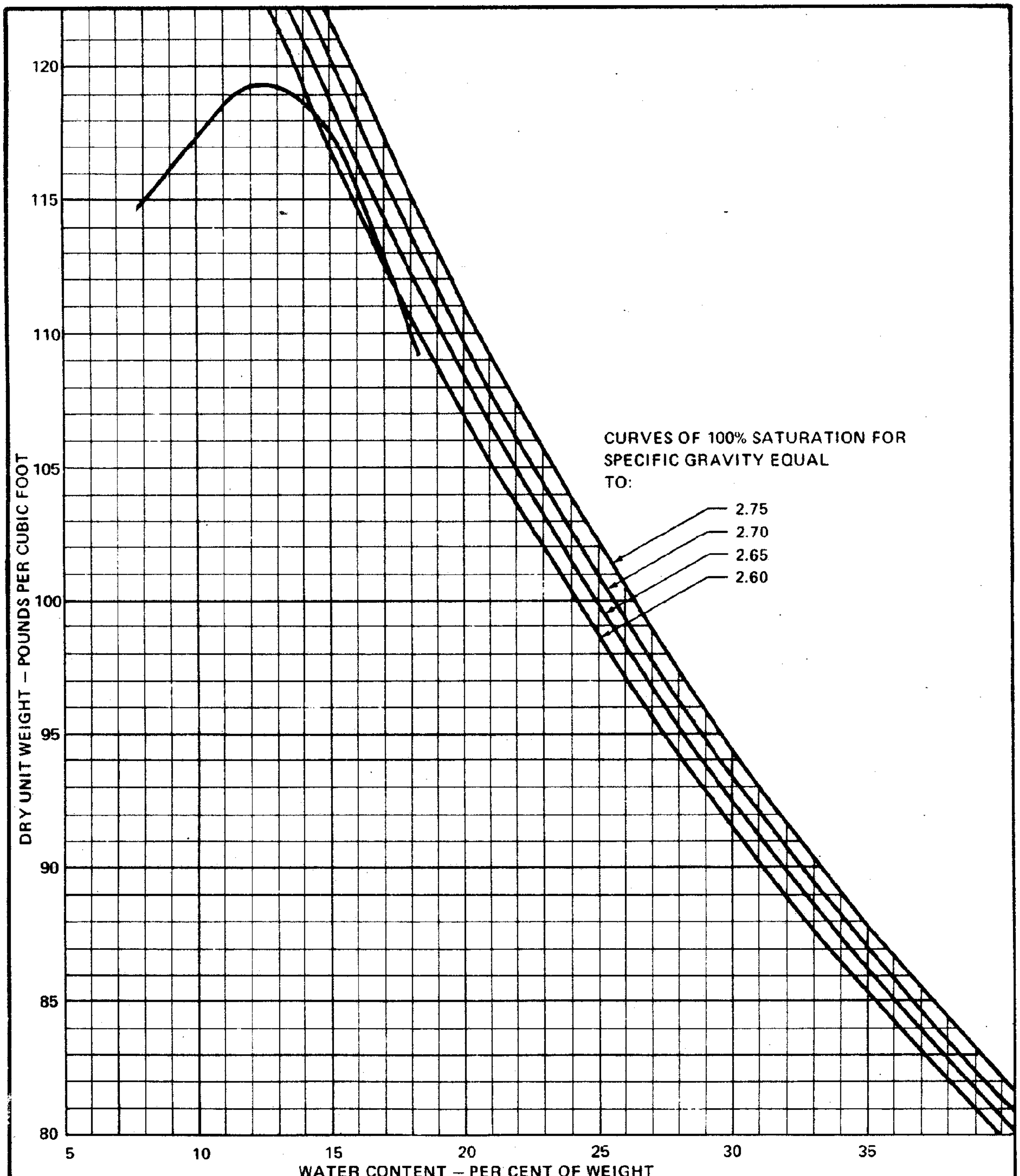
UNIT 2

COMPACTION TEST

FIGURE 2A-38 (SHEET 6 OF 9)



MOISTURE DENSITY RELATION	METHOD OF TEST	MAX. DRY DENSITY PCF	OPTIMUM MOISTURE CONTENT %	SOIL DESCRIPTION OR CLASSIFICATION AND SAMPLE LOCATION
	ASTM D 1557 METHOD C	119.4	12.5	RED BROWN FINE SANDY CLAYEY SILT B-418, EL. 123 - 136

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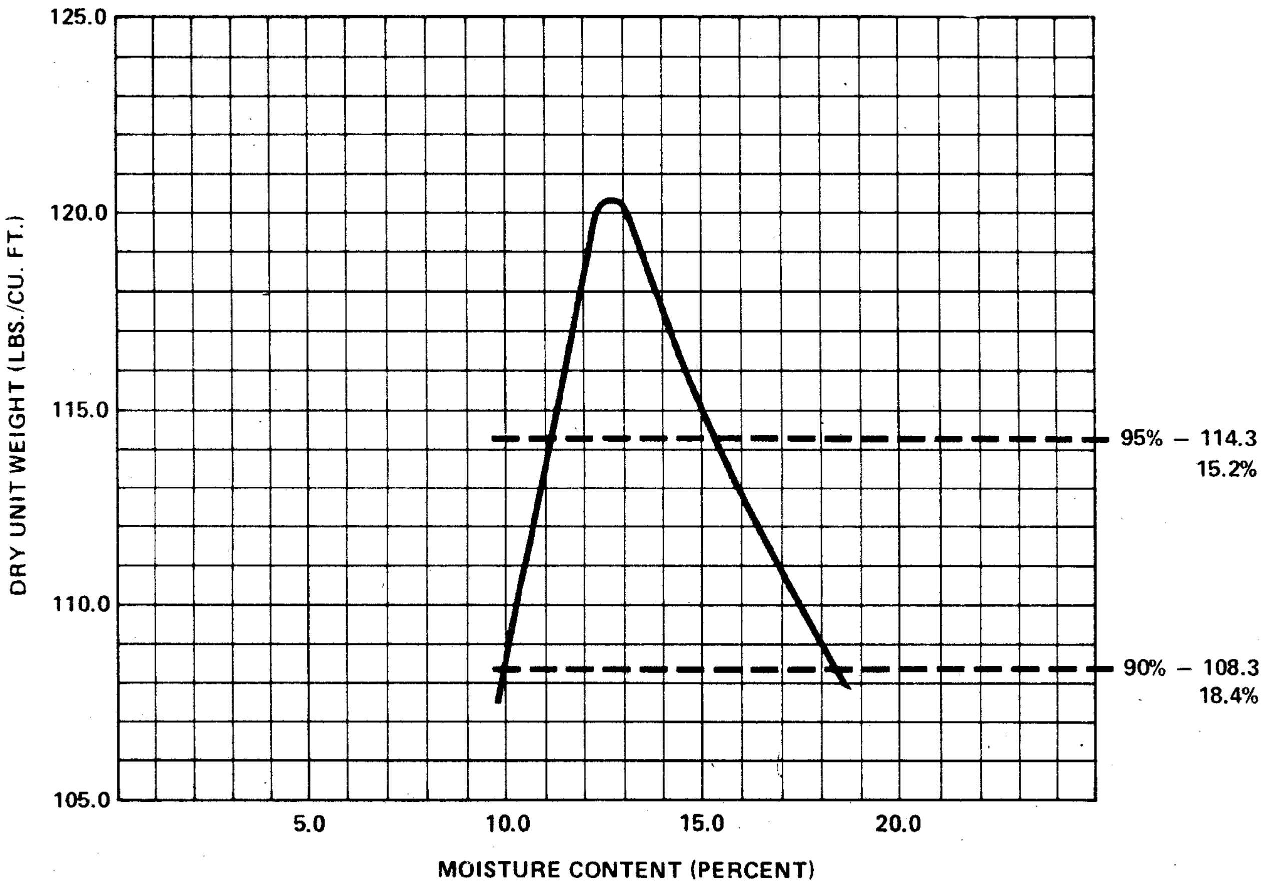
UNIT 2

COMPACTION TEST

FIGURE 2A-38 (SHEET 7 OF 9)



SAMPLE IDENTIFICATION: # 1C + # 2, @ 3' , COMBINED
MAXIMUM DRY DENSITY: 120.3 LB./CU. FT.
OPTIMUM MOISTURE CONTENT: 12.7 PERCENT
METHOD OF TEST: ASTM D 1557 (MODIFIED)

% – 200 = 40.9
LL = 34
PL = 21
PI = 13

TYPE MATERIAL: TAN BROWN VERY FINE SANDY SILTY CLAY
LOCATION: OFF SITE BORROW AREA NO. 3
IN SITU MOISTURE: # 1C = 19.6 PERCENT
2 = 19.1 PERCENT

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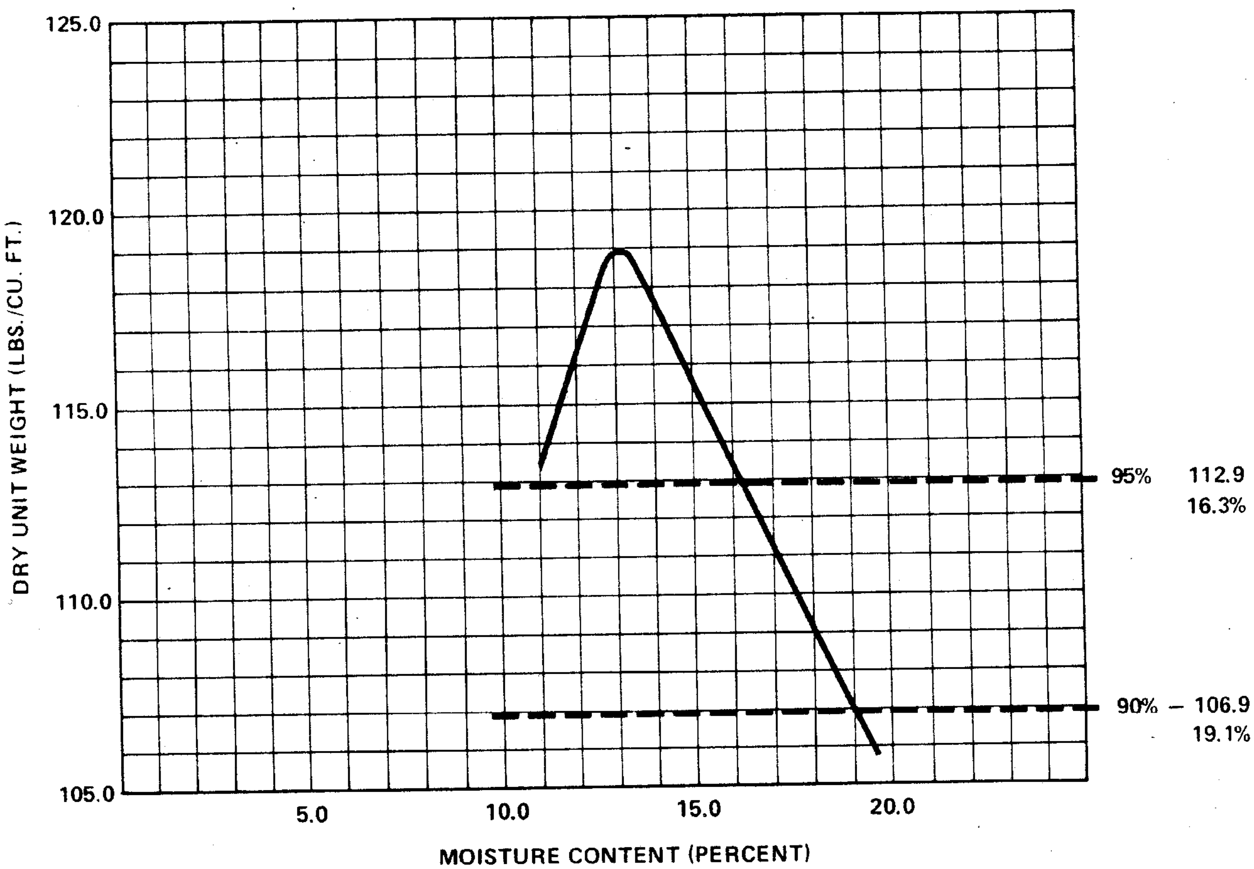
UNIT 2

COMPACTION TEST

FIGURE 2A-38 (SHEET 8 OF 9)



SAMPLE IDENTIFICATION: # 2A + # 2B, @ 7', COMBINED
MAXIMUM DRY DENSITY: 118.8 LB./CU. FT.
OPTIMUM MOISTURE CONTENT: 13.2 PERCENT
METHOD OF TEST: ASTM D 1557

% – 200 = 34.7
LL = 36
PL = 19
PI = 17

TYPE MATERIAL: BROWN SILTY SLIGHTLY CLAYEY MEDIUM–FINE
LOCATION: OFF SITE BORROW AREA NO. 3
IN SITU MOISTURE: # 2A = 16.2 PERCENT
2B = 16.9 PERCENT

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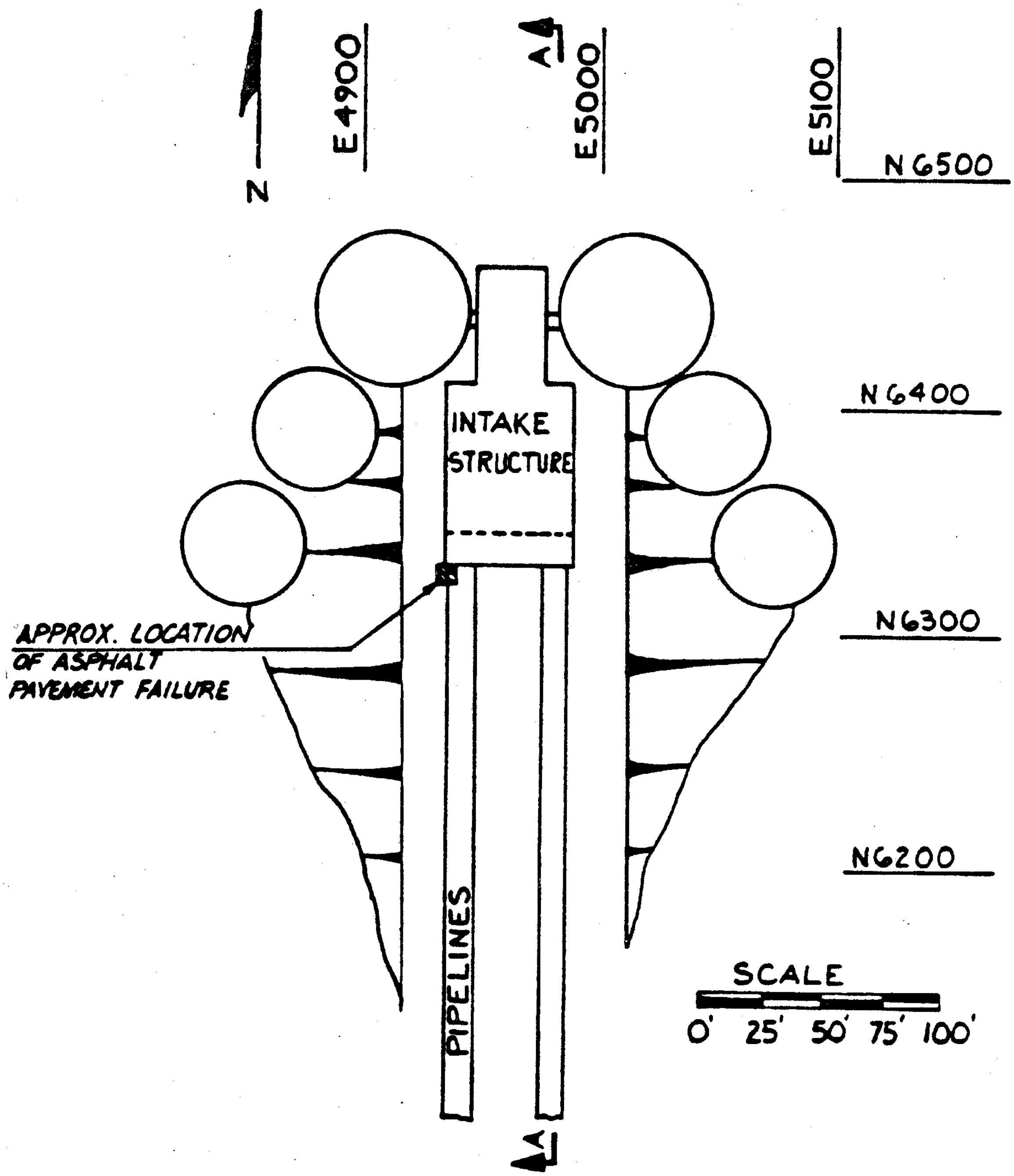
UNIT 2

COMPACTION TEST

FIGURE 2A-38 (SHEET 9 OF 9)



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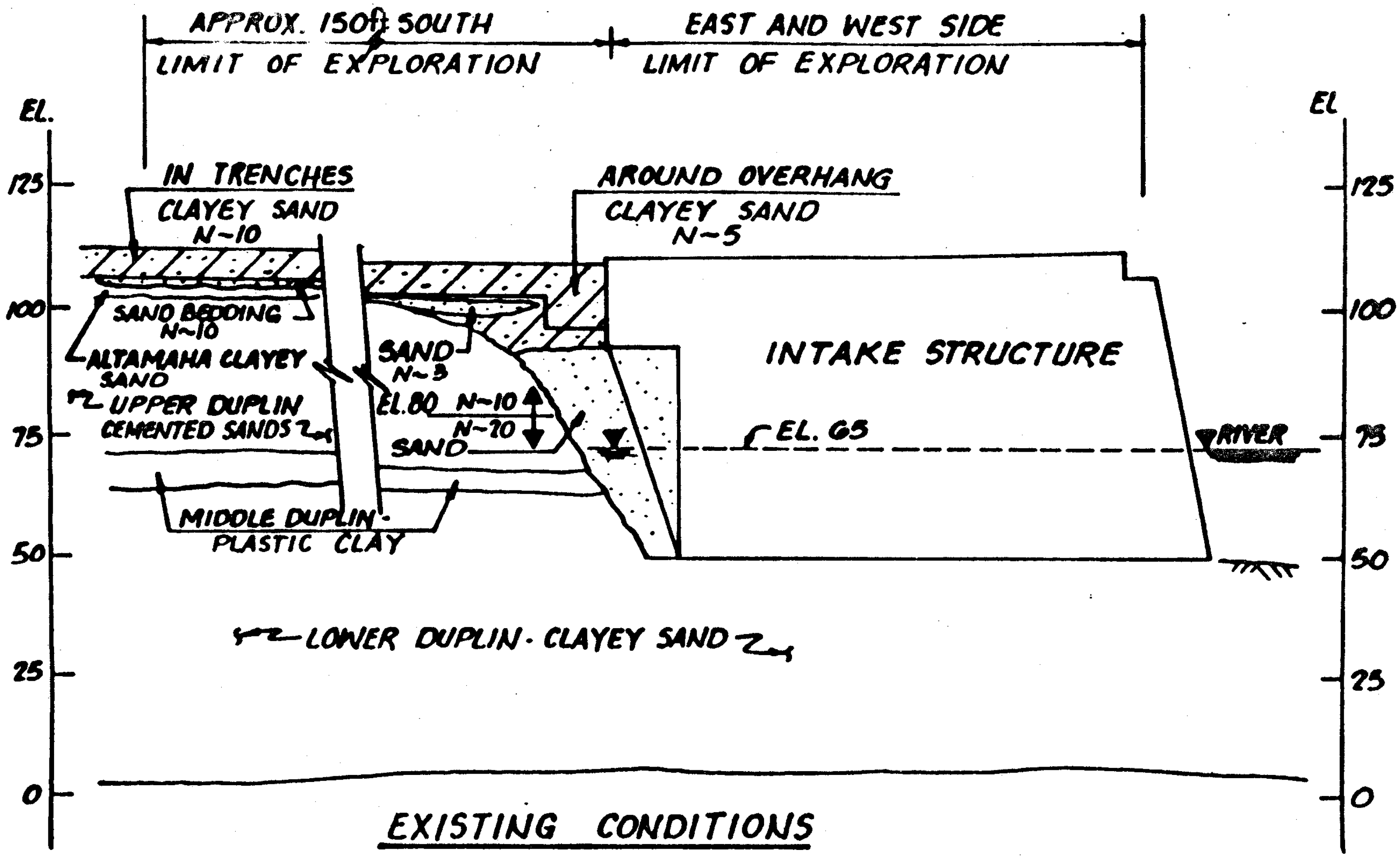
LOCATION OF ASPHALT
PAVEMENT FAILURE

FIGURE 2A-39



SECTION A-A (FROM FIG. 2A35)

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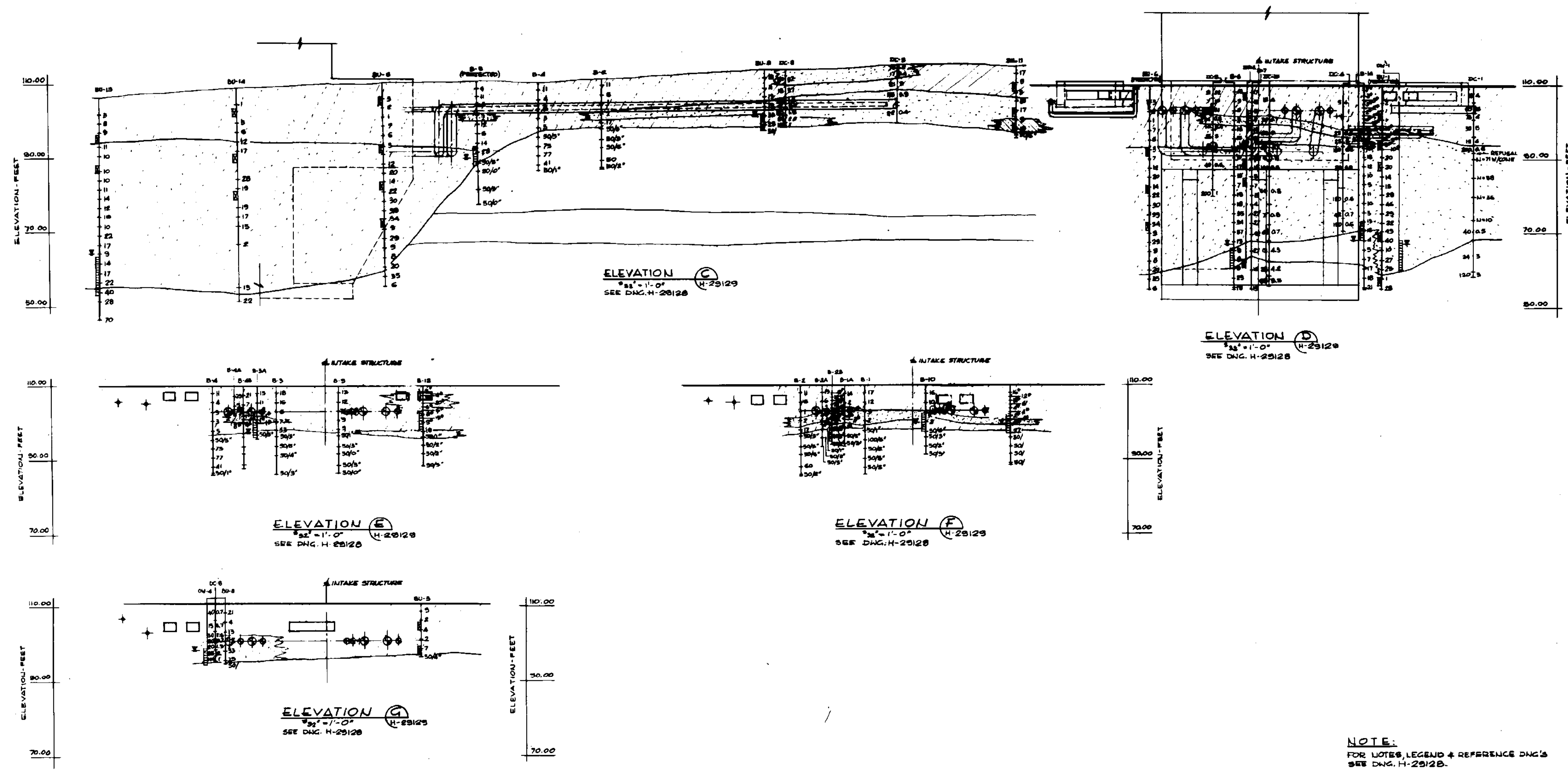
EXISTING CONDITIONS SECTION A-A (FROM FIGURE 2A-39) SUMMARY OF PHASES I AND II FIELD INVESTIGATIONS

FIGURE 2A-40



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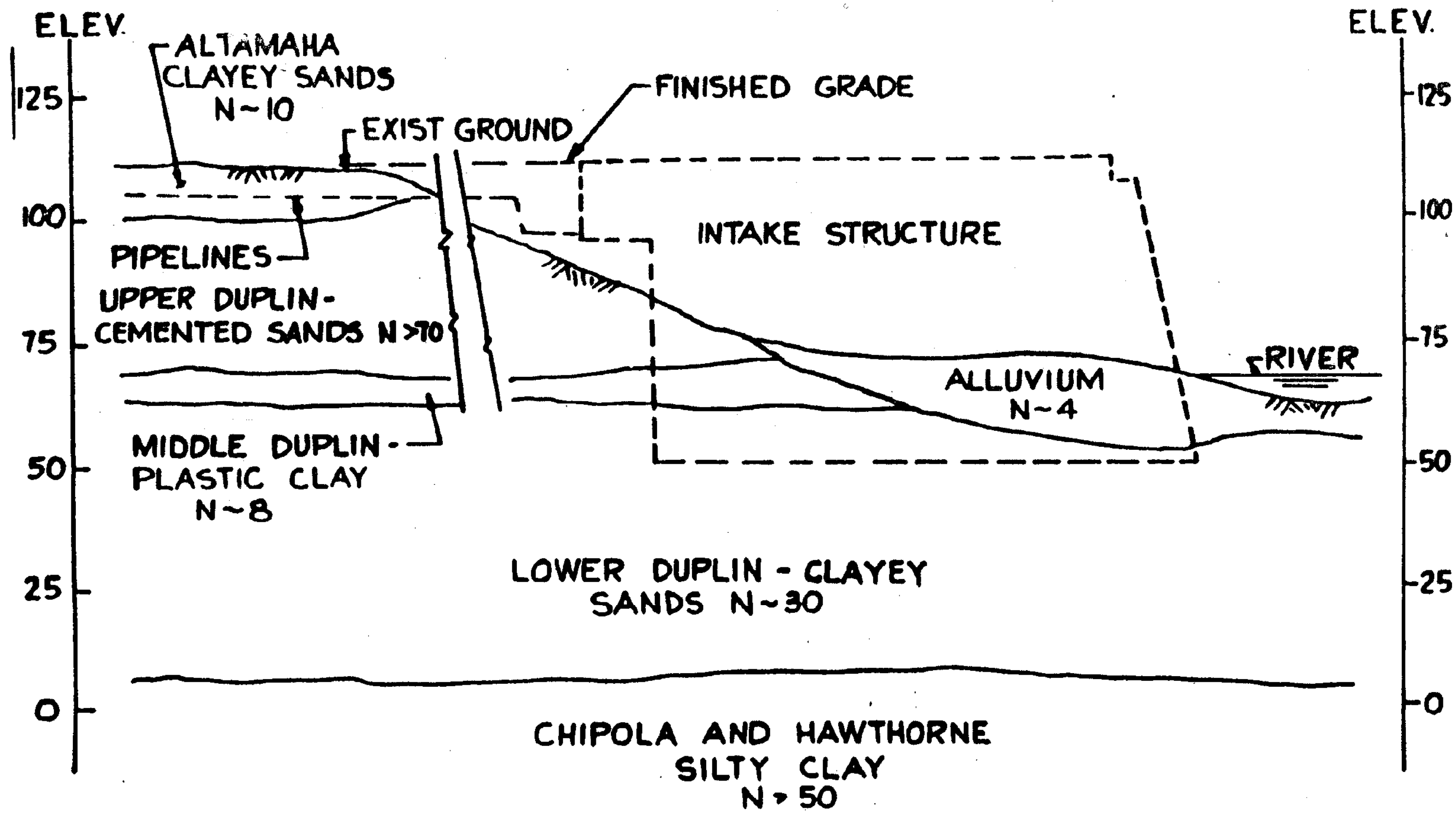
SUBSURFACE PROFILES

FIGURE 2A-41



PRECONSTRUCTION CONDITIONS

SECTION A-A

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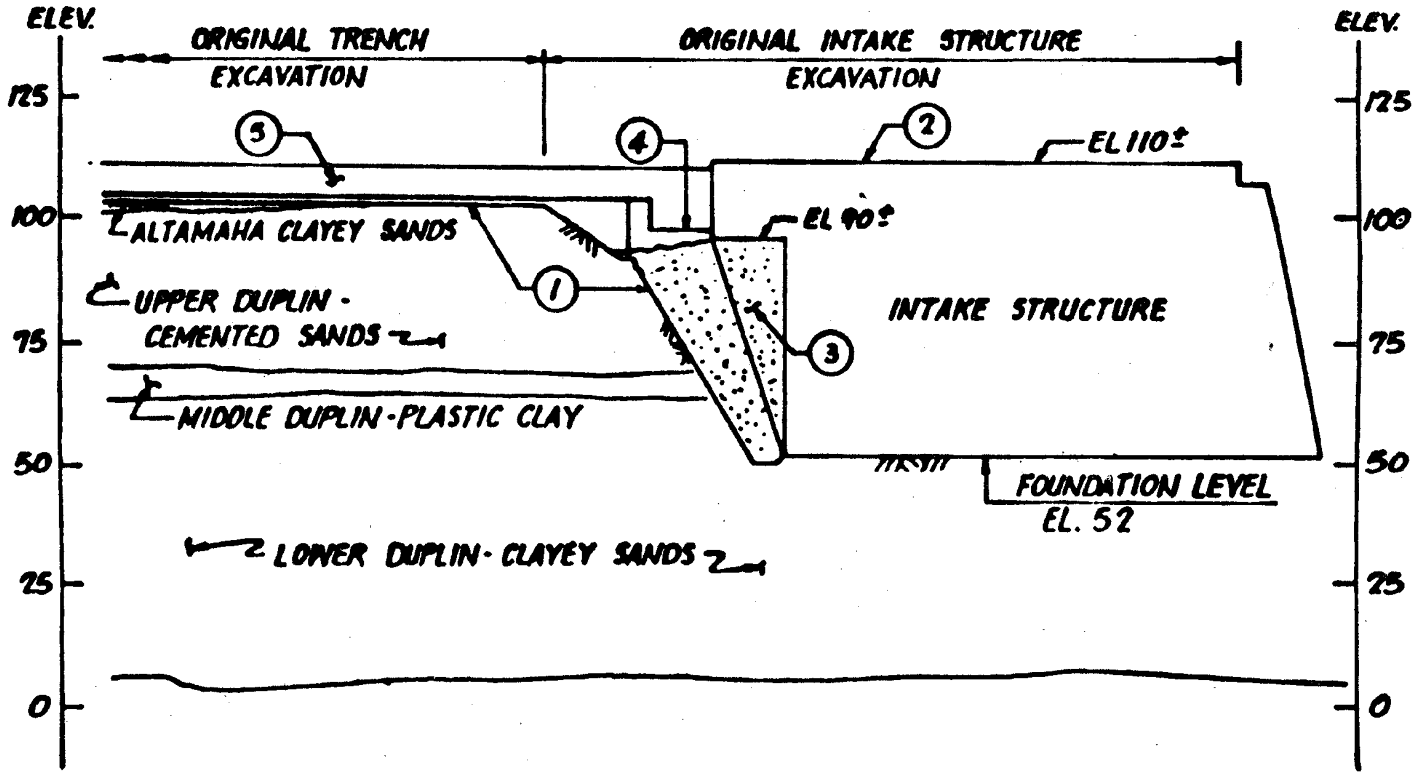
PRECONSTRUCTION CONDITIONS
SECTION A-A (FROM FIGURE 2A-39)

FIGURE 2A-42

CONSTRUCTION SEQUENCE:

1. EXCAVATION MADE IN THE DRY WITH CELLULAR COFFERDAM.
2. STRUCTURE BUILT INCLUDING SOUTHERN OVERHANG WITH COUNTERFORTS.
3. RIVER SAND BACKFILL PLACED TO EL. 90±. (90% ASTM D-1557)
4. PIPES INSTALLED WITH SUPPORTS NEAR INTAKE AND IN TRENCH
5. CLAYEY SAND BACKFILL PLACED TO GRADE. (90% ASTM D-1557)



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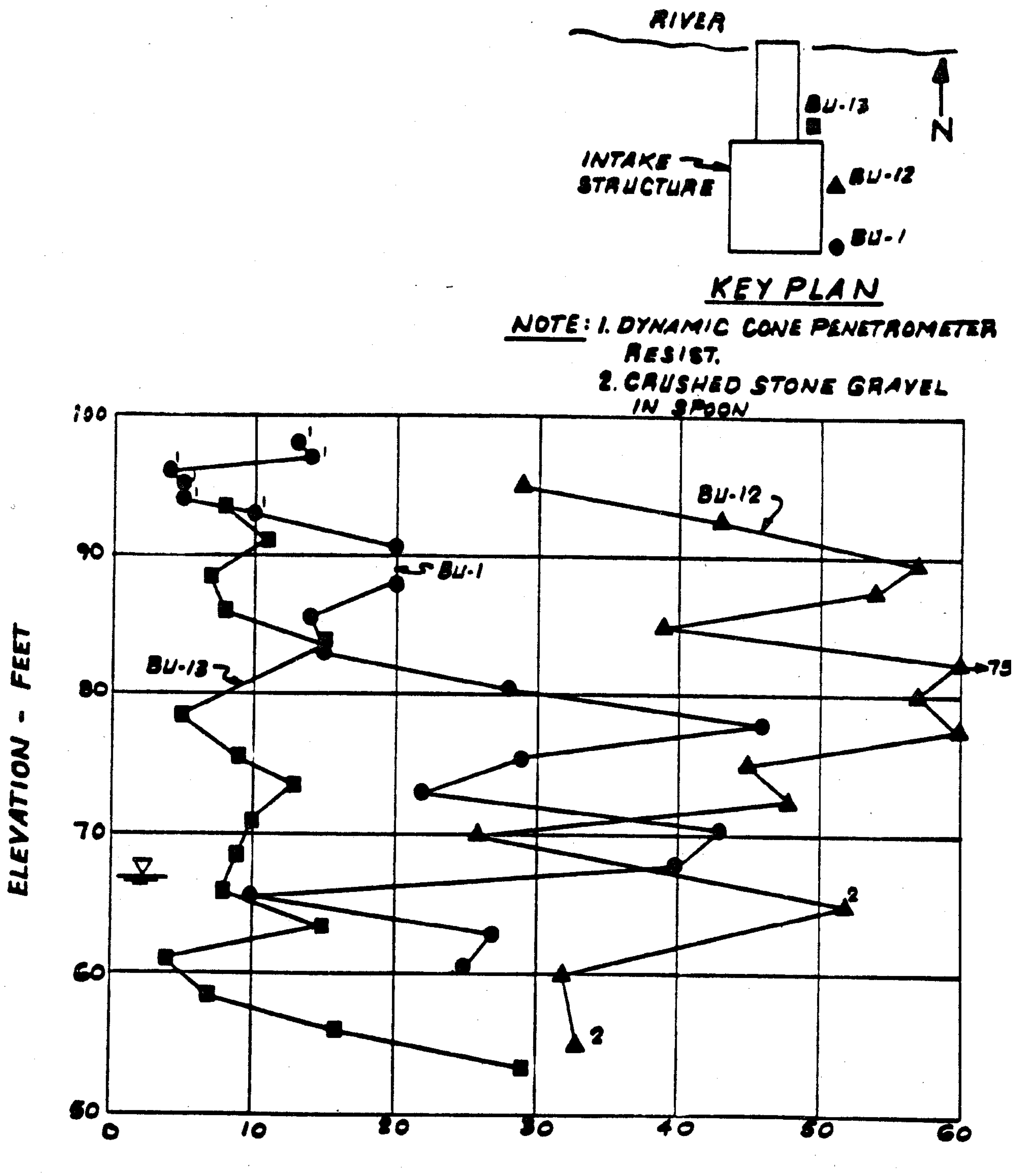
CONSTRUCTION PROCEDURE
SECTION A-A (FROM FIGURE 2A-39)

FIGURE 2A-43



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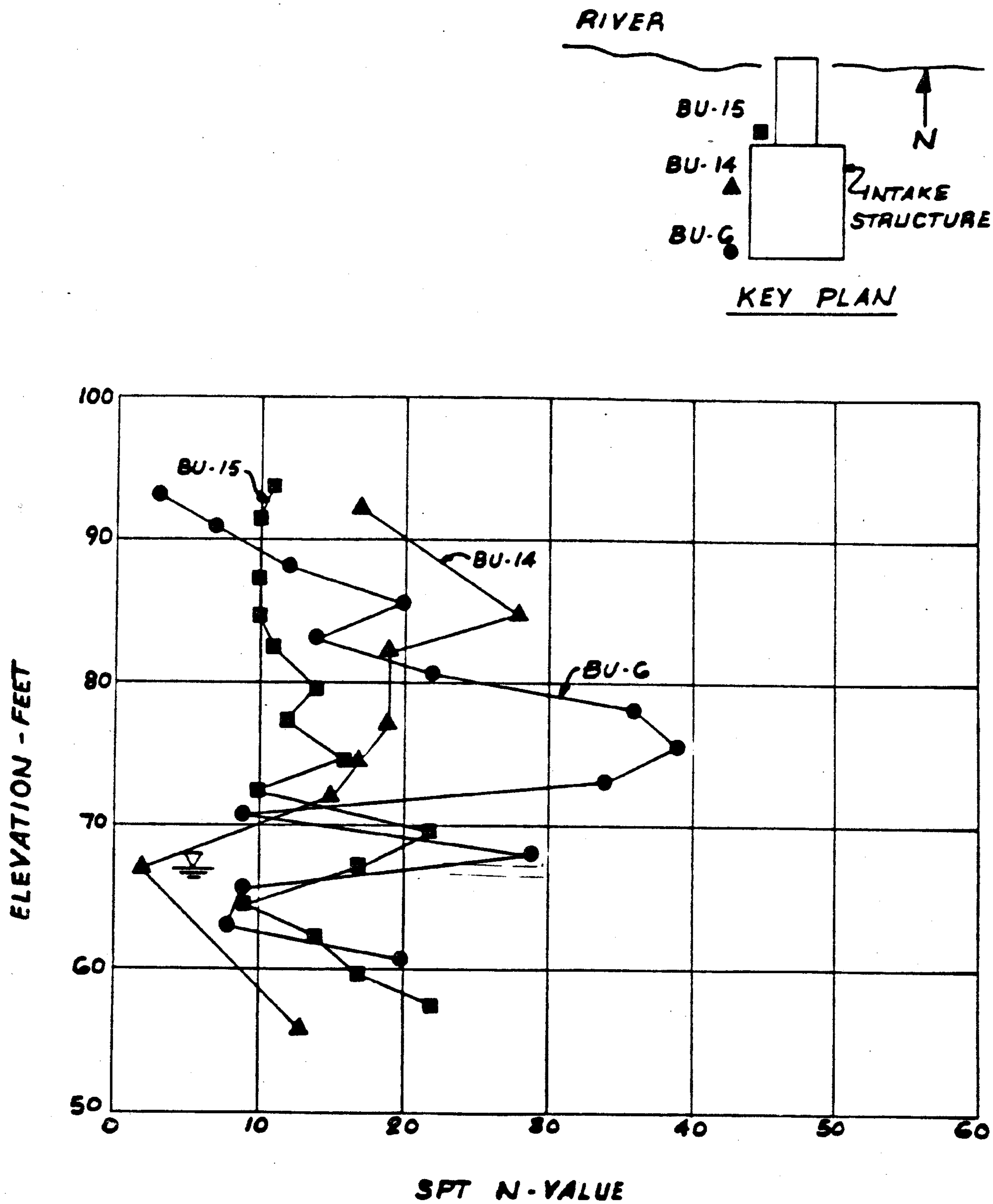
UNIT 2

SPT RESULTS FOR BACKFILL IN EAST SIDE OF INTAKE STRUCTURE

FIGURE 2A-44



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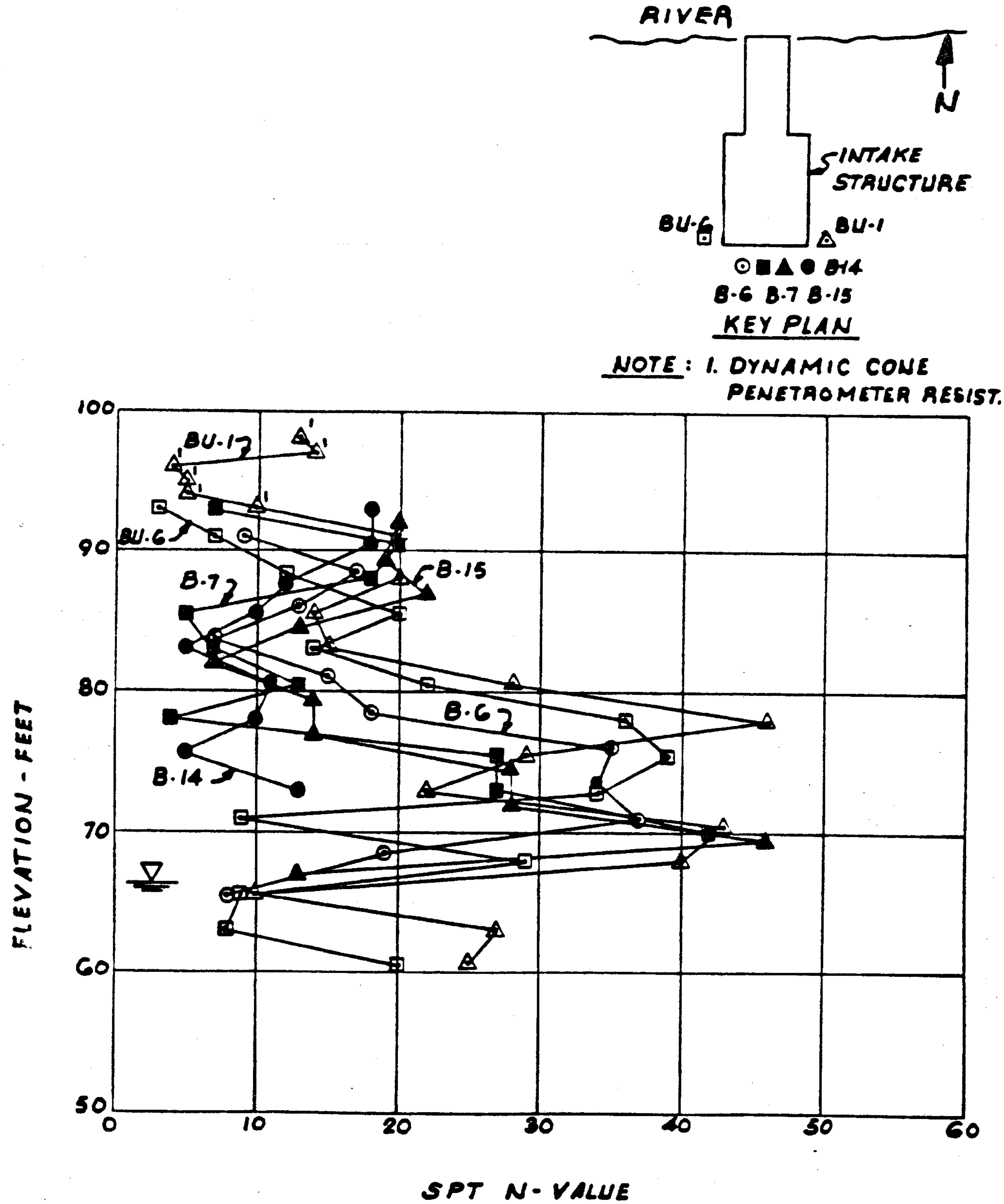
UNIT 2

SPT RESULTS FOR BACKFILL IN WEST SIDE OF INTAKE STRUCTURE

FIGURE 2A-45



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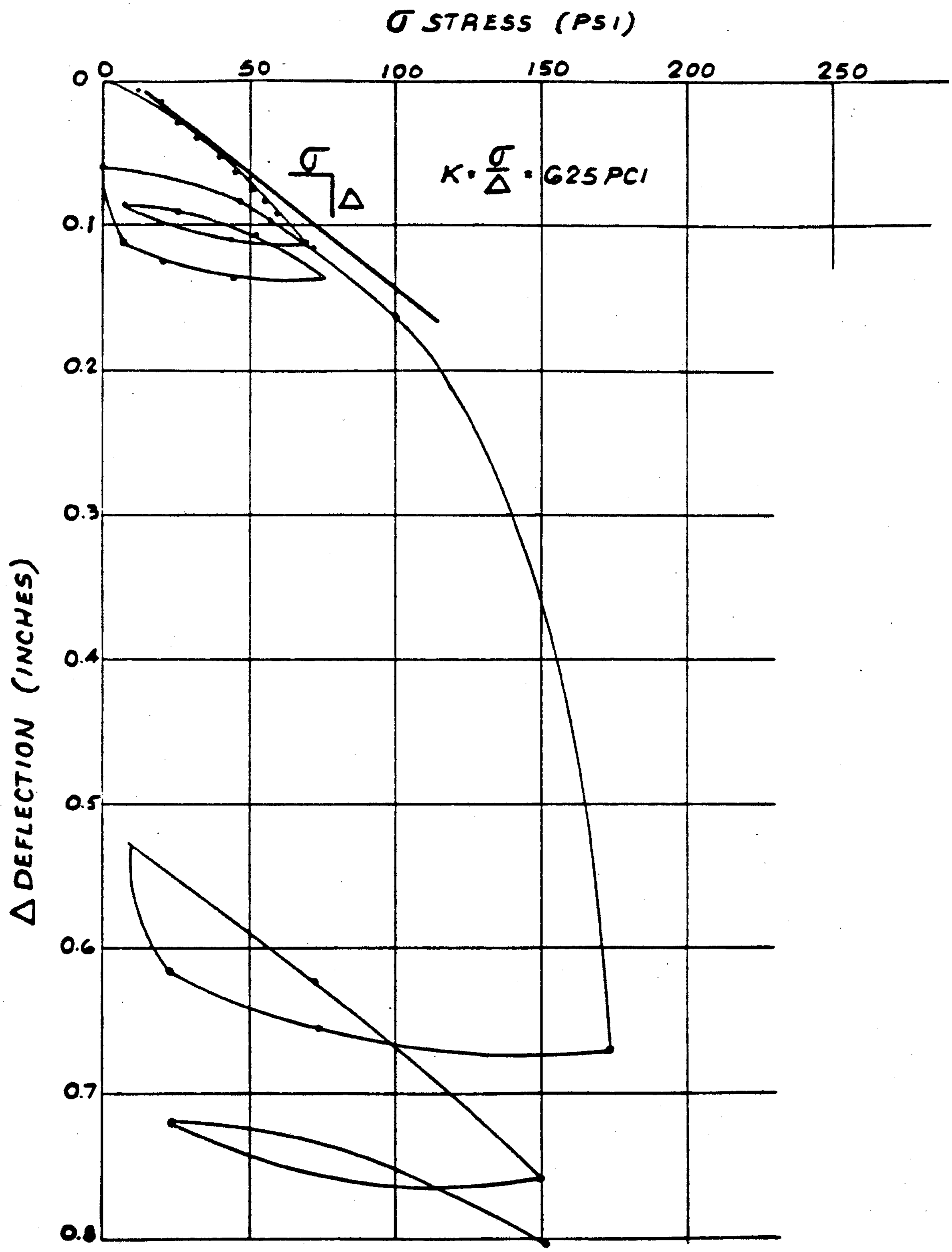
SPT RESULTS FOR BACKFILL IN SOUTH SIDE OF INTAKE STRUCTURE

FIGURE 2A-46



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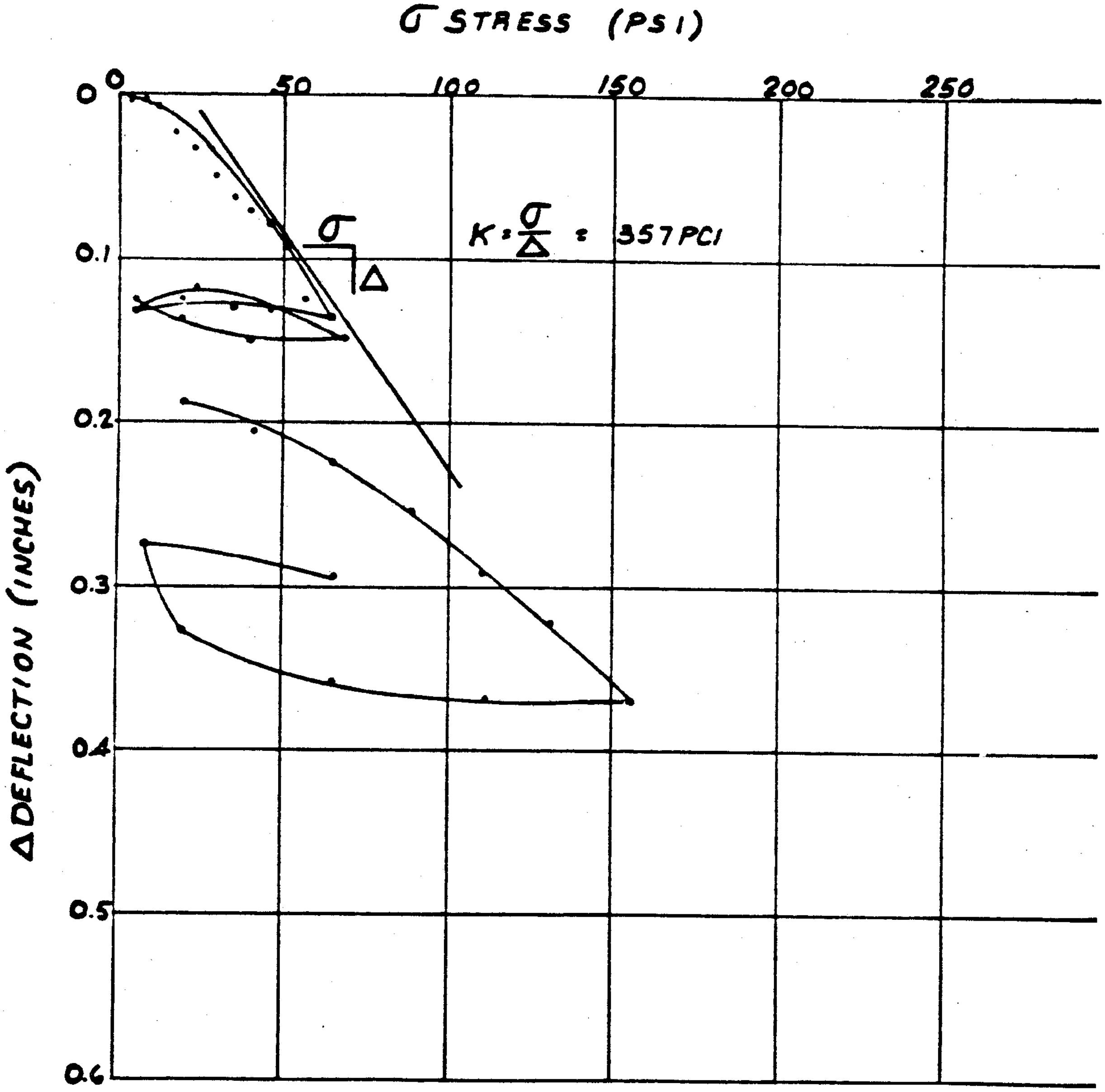
HORIZONTAL PLATE LOAD TEST ON SAND BACKFILL. TP-1 18-in.-DIAMETER PLATE AT DEPTH OF 10 ft ON WEST WALL

FIGURE 2A-47



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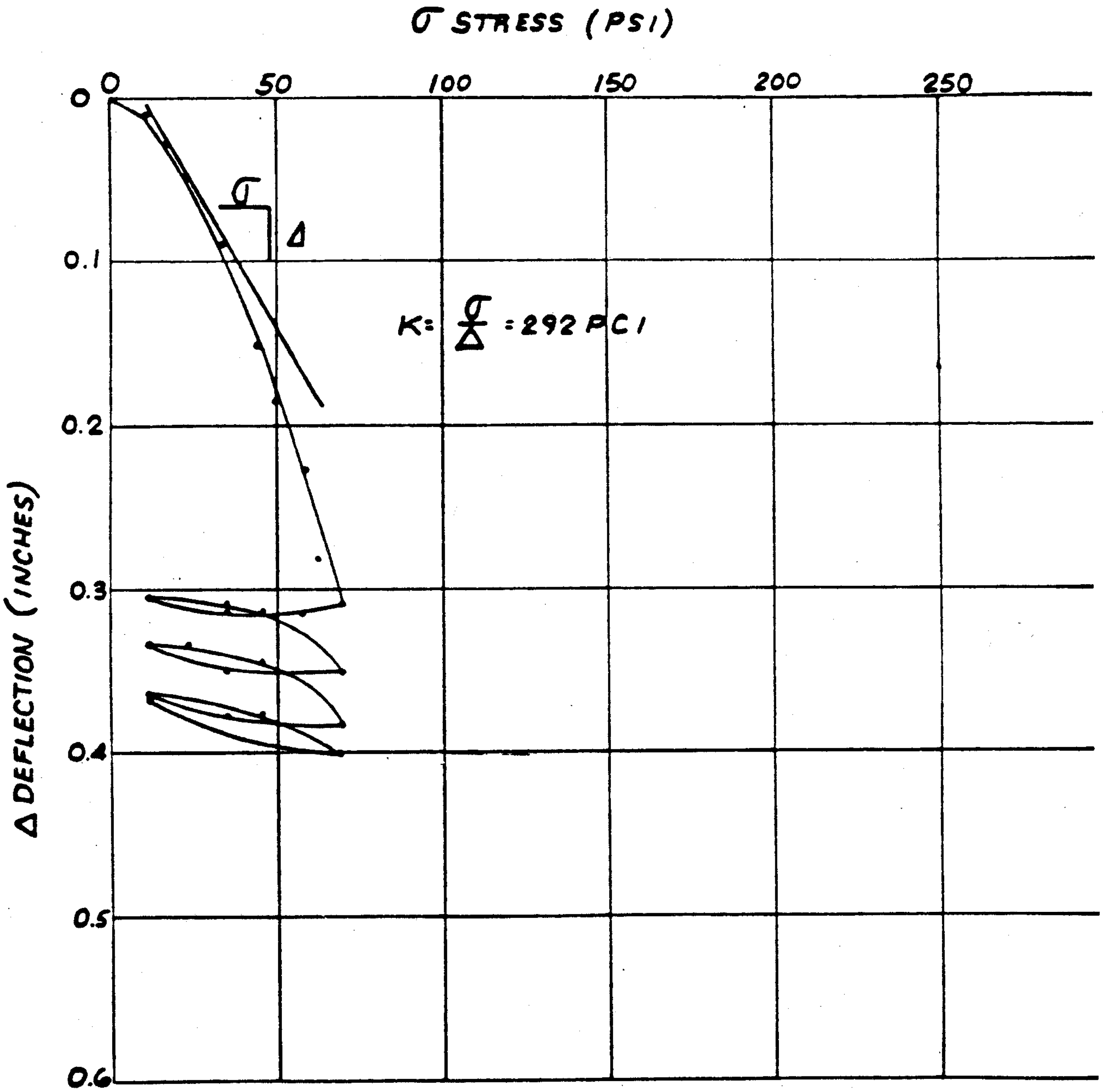
HORIZONTAL PLATE LOAD TEST ON SAND BACKFILL, TP-1 30-in.-DIAMETER PLATE AT DEPTH OF 12 ft ON EAST WALL

FIGURE 2A-48



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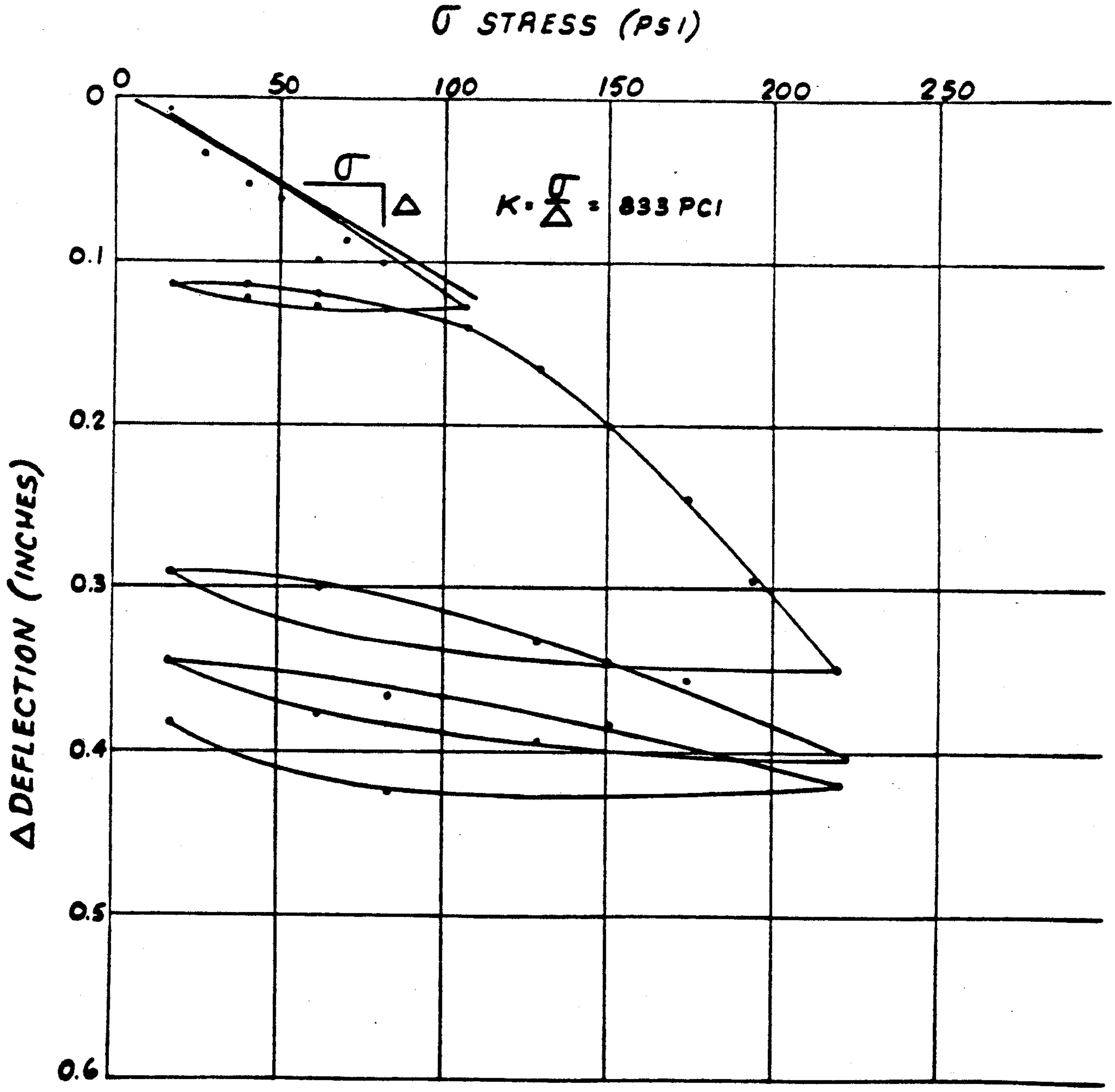
UNIT 2

VERTICAL PLATE LOAD TEST ON SAND BACKFILL, TP-2 12-in. PLATE AT DEPTH OF 10 ft

FIGURE 2A-49



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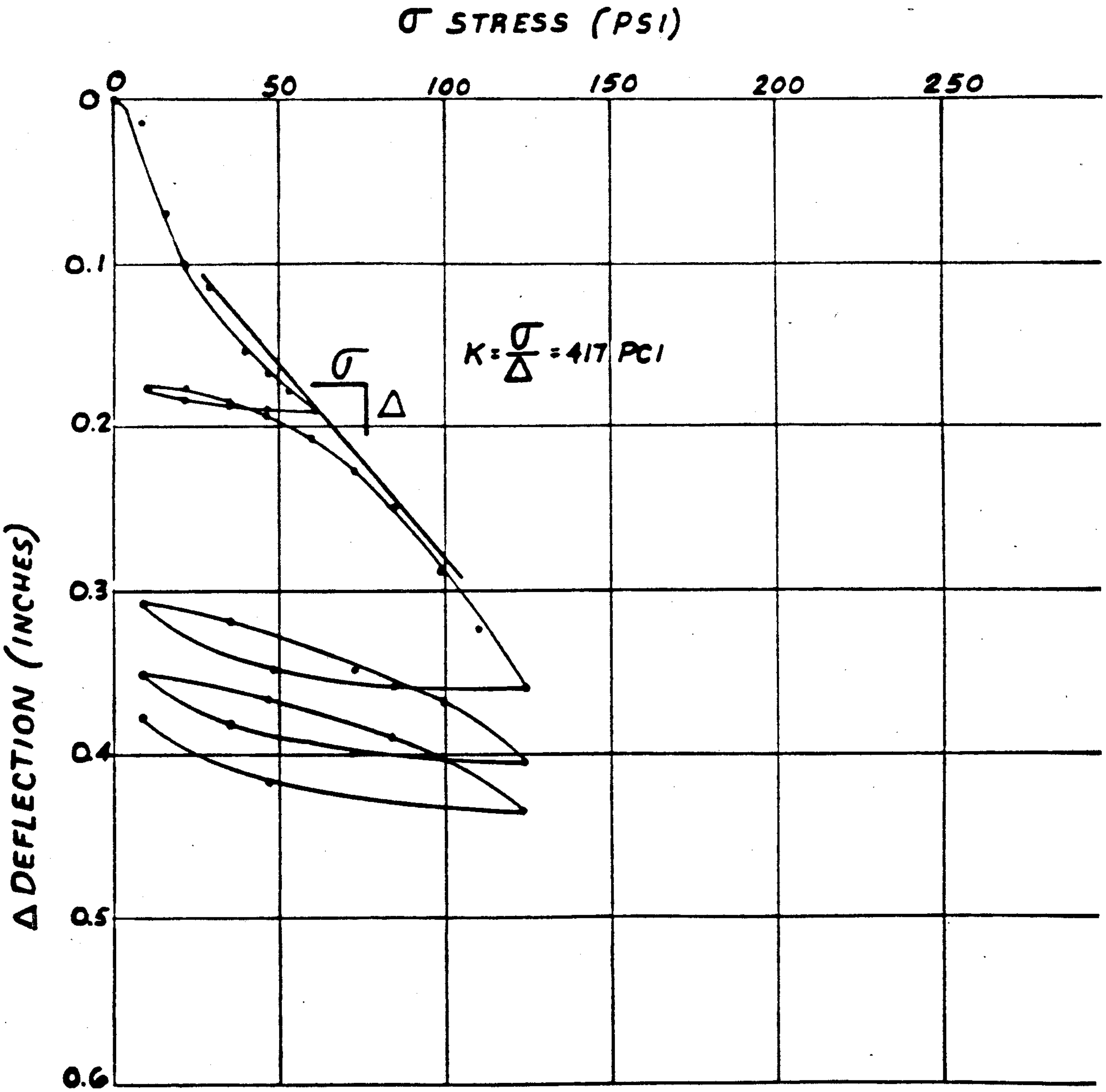
HORIZONTAL PLATE LOAD TEST ON SAND BACKFILL, TP-5 12-in. SQUARE PLATE AT DEPTH OF 10 ft WEST WALL

FIGURE 2A-50



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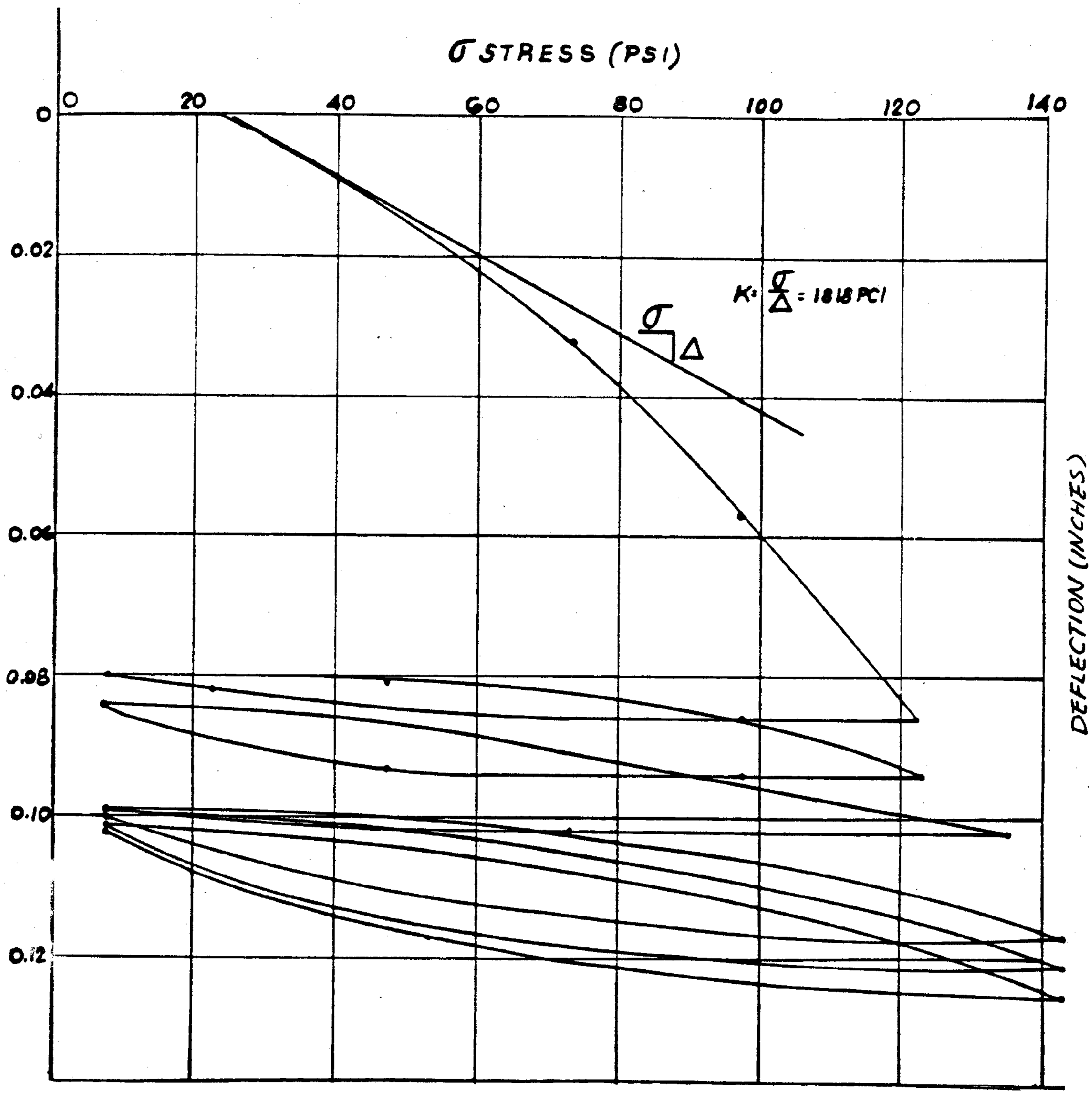
HORIZONTAL PLATE LOAD TEST ON SAND BACKFILL, TP-5 18-in.-DIAMETER PLATE AT DEPTH OF 10 ft ON EAST WALL

FIGURE 2A-51



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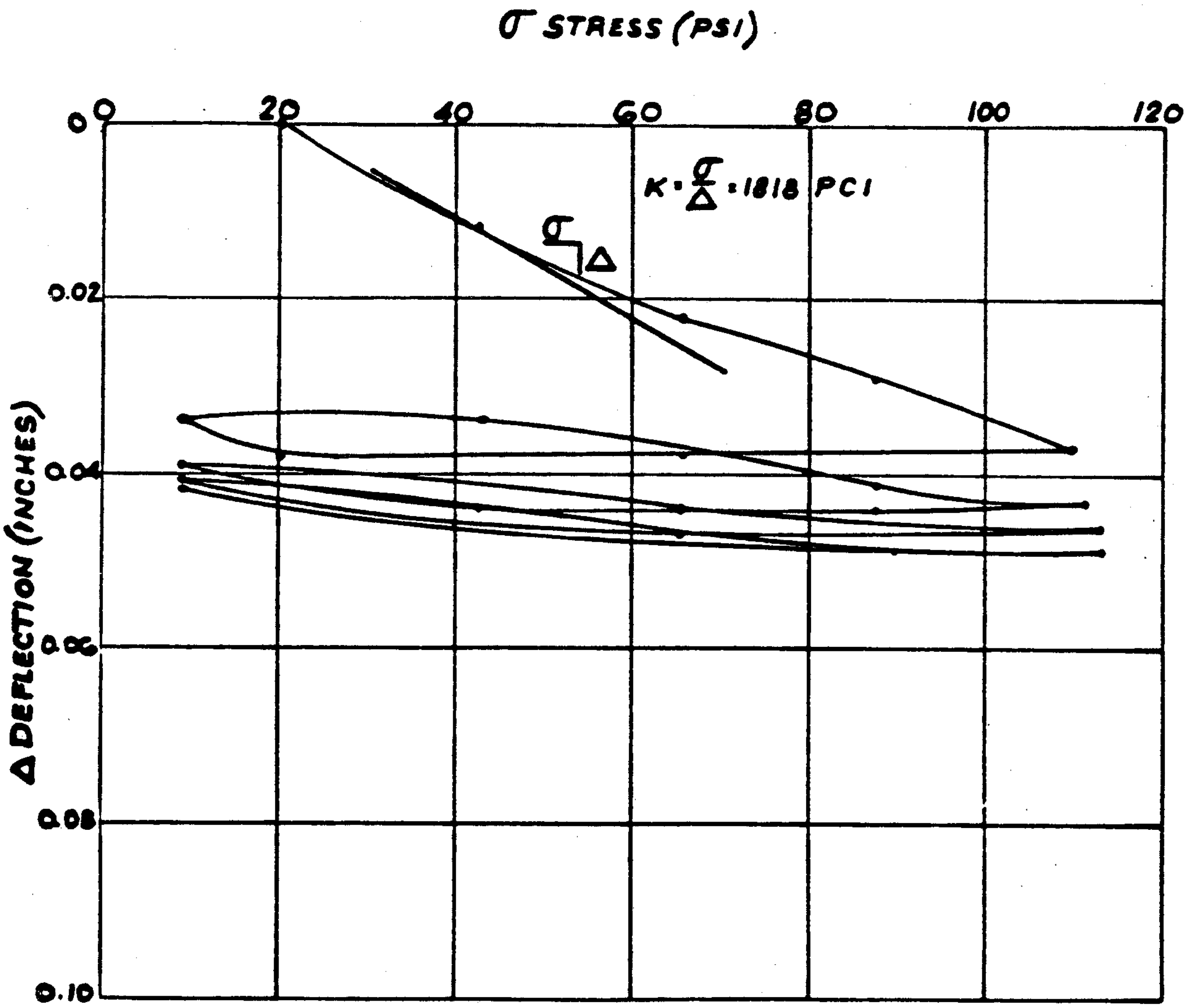
7-DAY VERTICAL PLATE LOAD TEST ON K-KRETE 18-in.-DIAMETER PLATE SOUTH END OF TEST SLAB

FIGURE 2A-52



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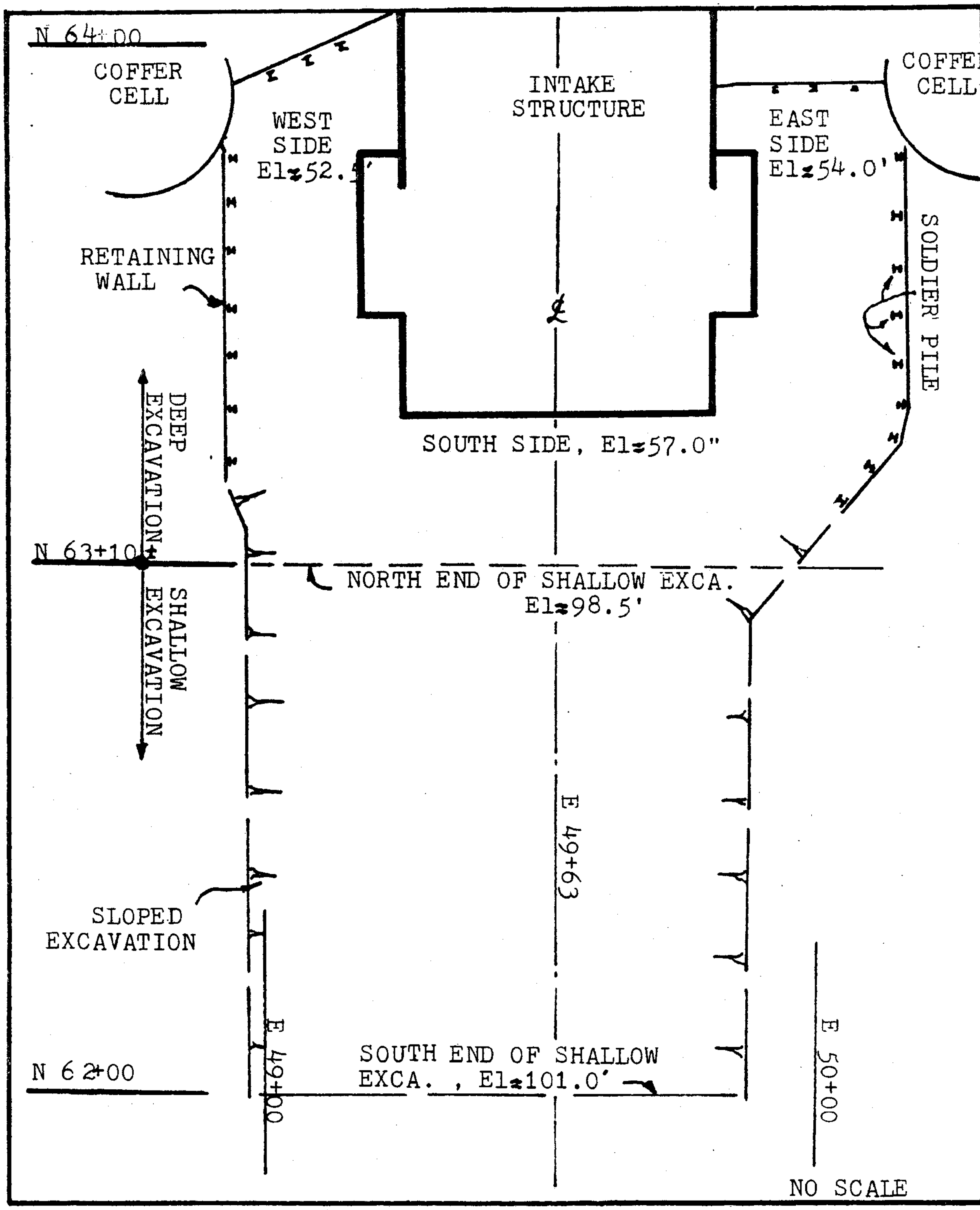
UNIT 2

7-DAY VERTICAL PLATE LOAD TEST ON K-KRETE 30-in.-DIAMETER PLATE SOUTH END OF TEST SLAB

FIGURE 2A-53



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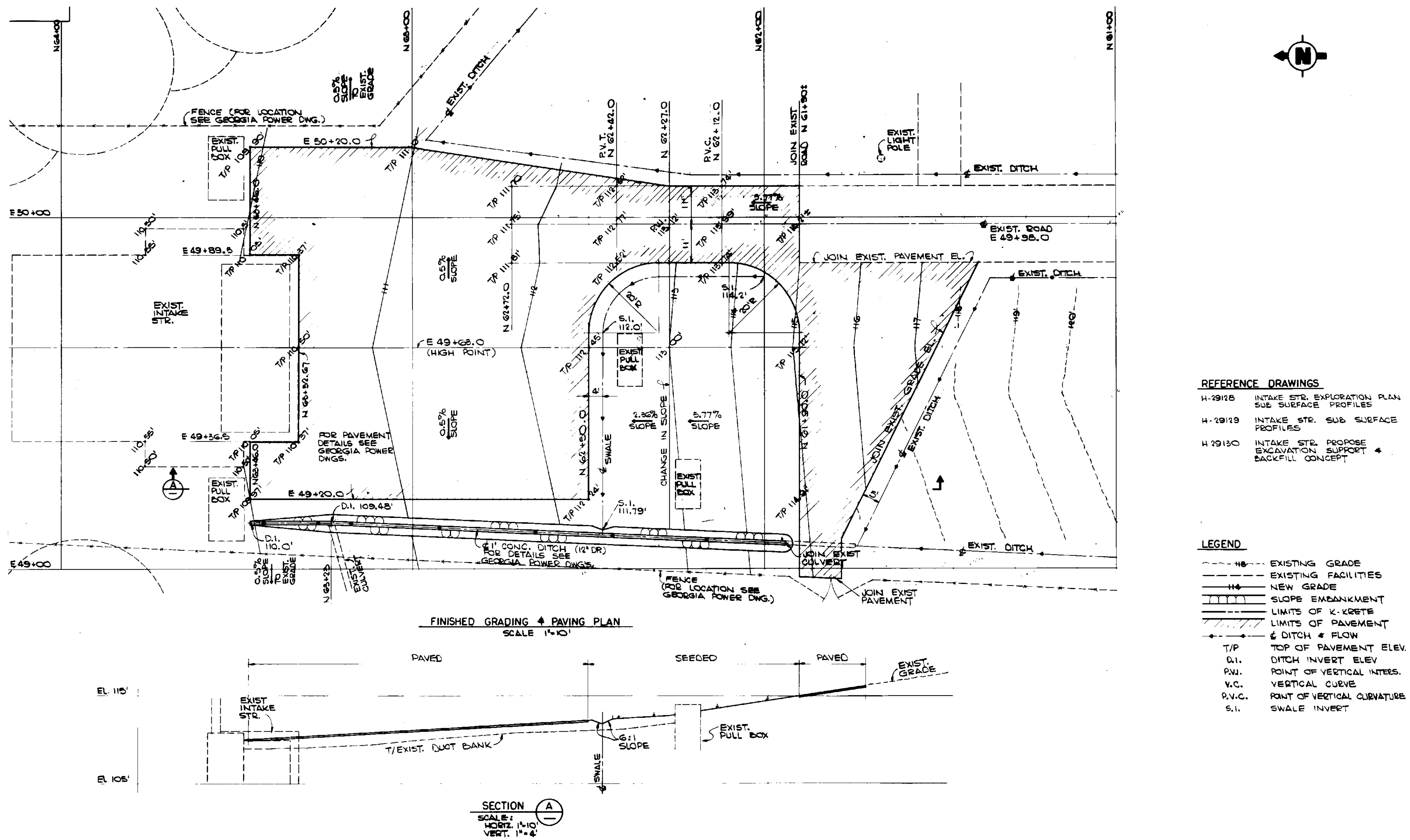
UNIT 2

APPROXIMATE ELEVATIONS OF VIRGIN GROUND BEFORE K-KRETE BACKFILL

FIGURE 2A-54



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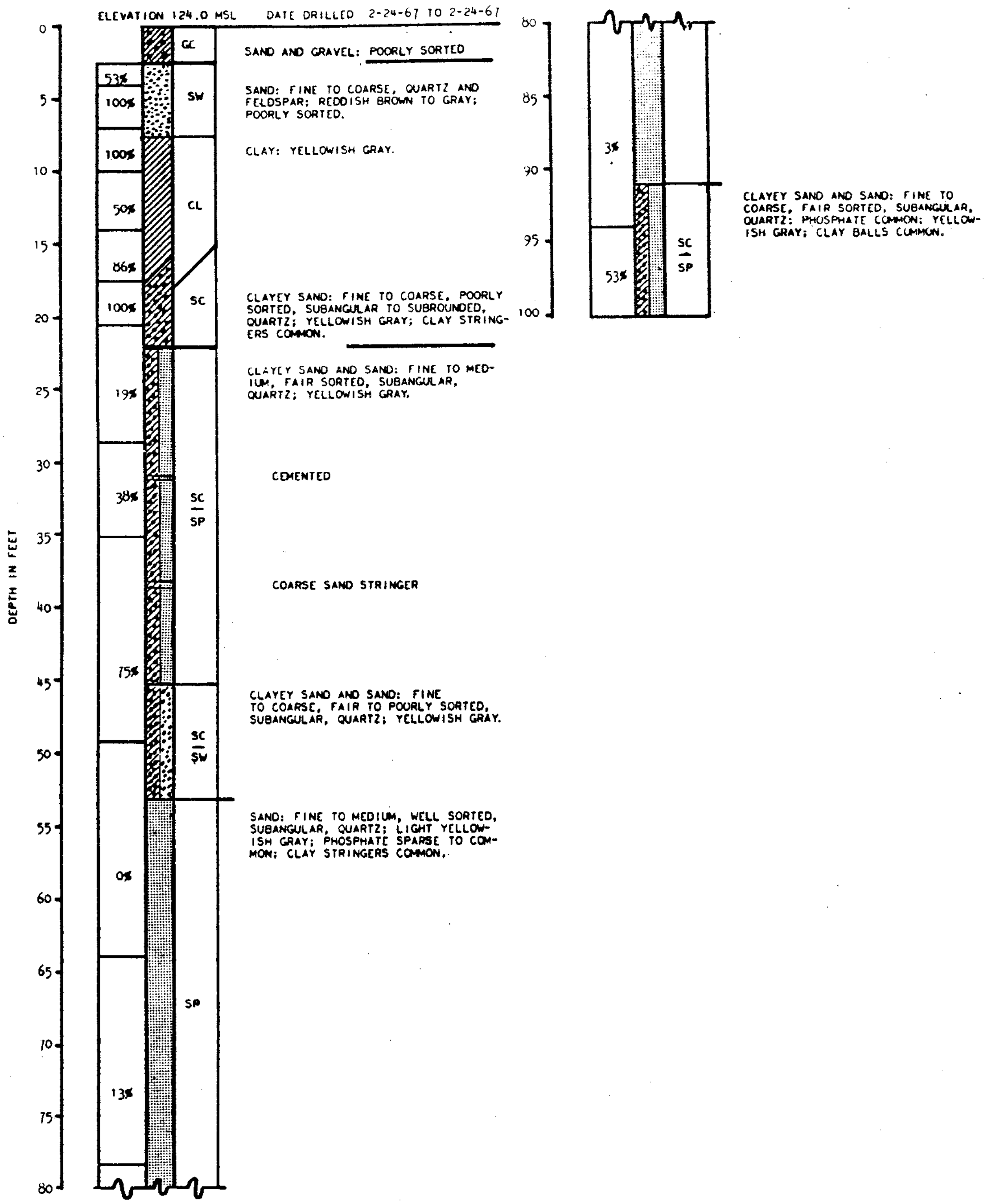
UNIT 2

INTAKE STRUCTURE
FINISHED GRAVELING AND PAVING PLAN

FIGURE 2A-55

BORING 102



87% — % CORE RECOVERY. (4" VACUUM CORE BARREL). BALANCE OF BORING DRILLED WITH 4" HAWTHORNE BIT.

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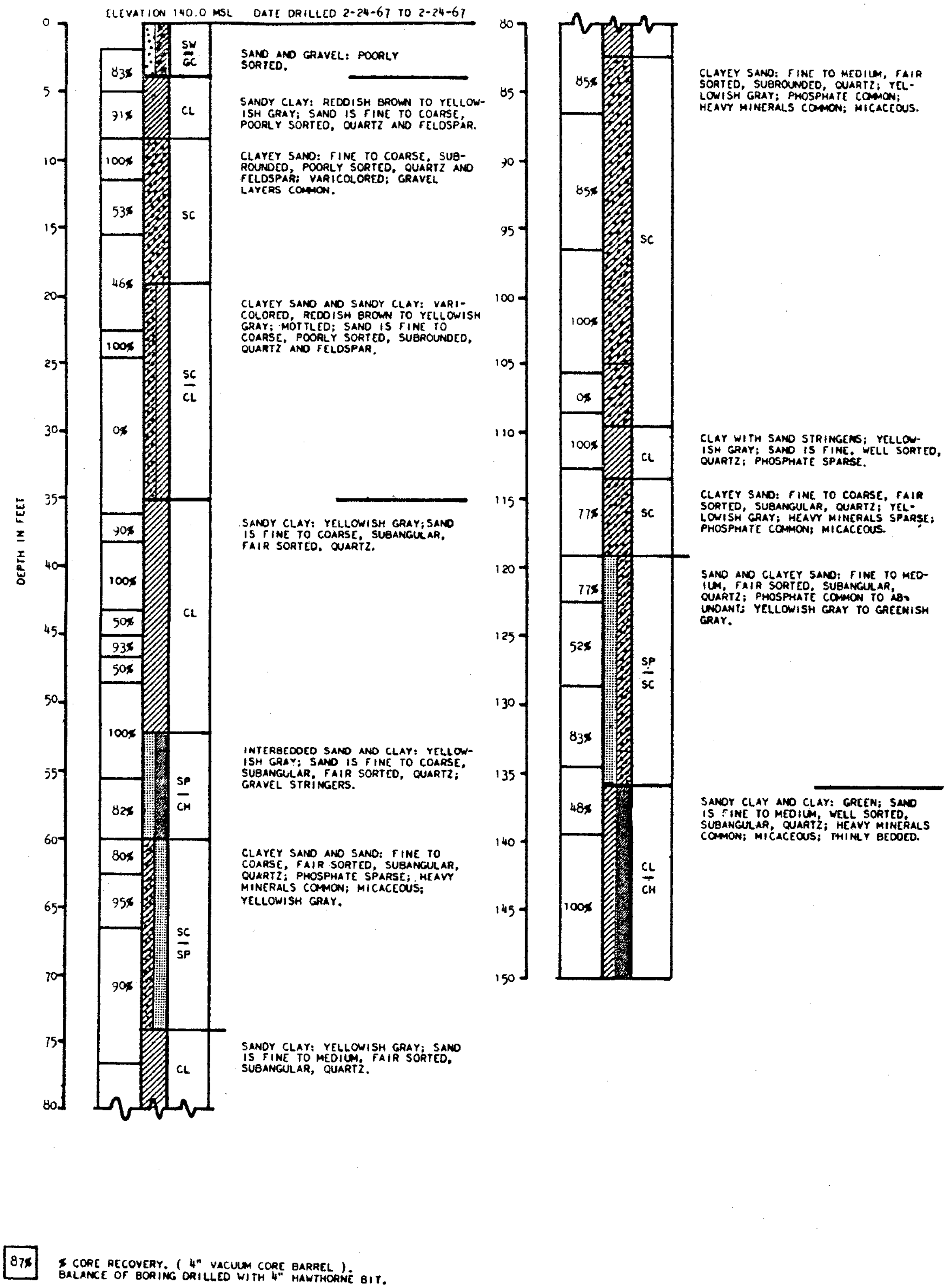
UNIT 2

TEST BORING RECORD
BORING NO. 102

FIGURE 2B-1

BORING 103



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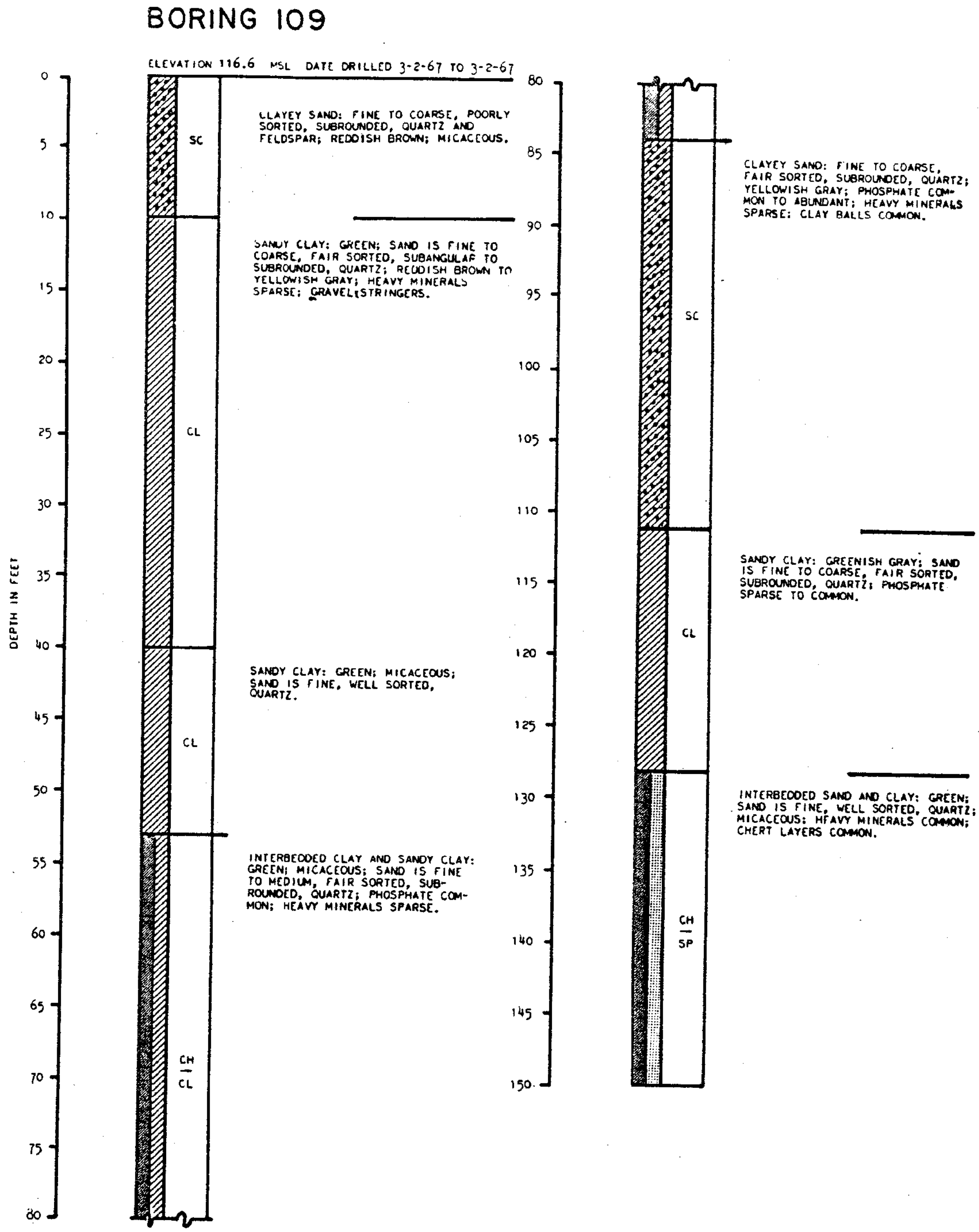
UNIT 2

TEST BORING RECORD
BORING NO. 103

FIGURE 2B-2



BORING DRILLED WITH 4" HAWTHORNE BIT.

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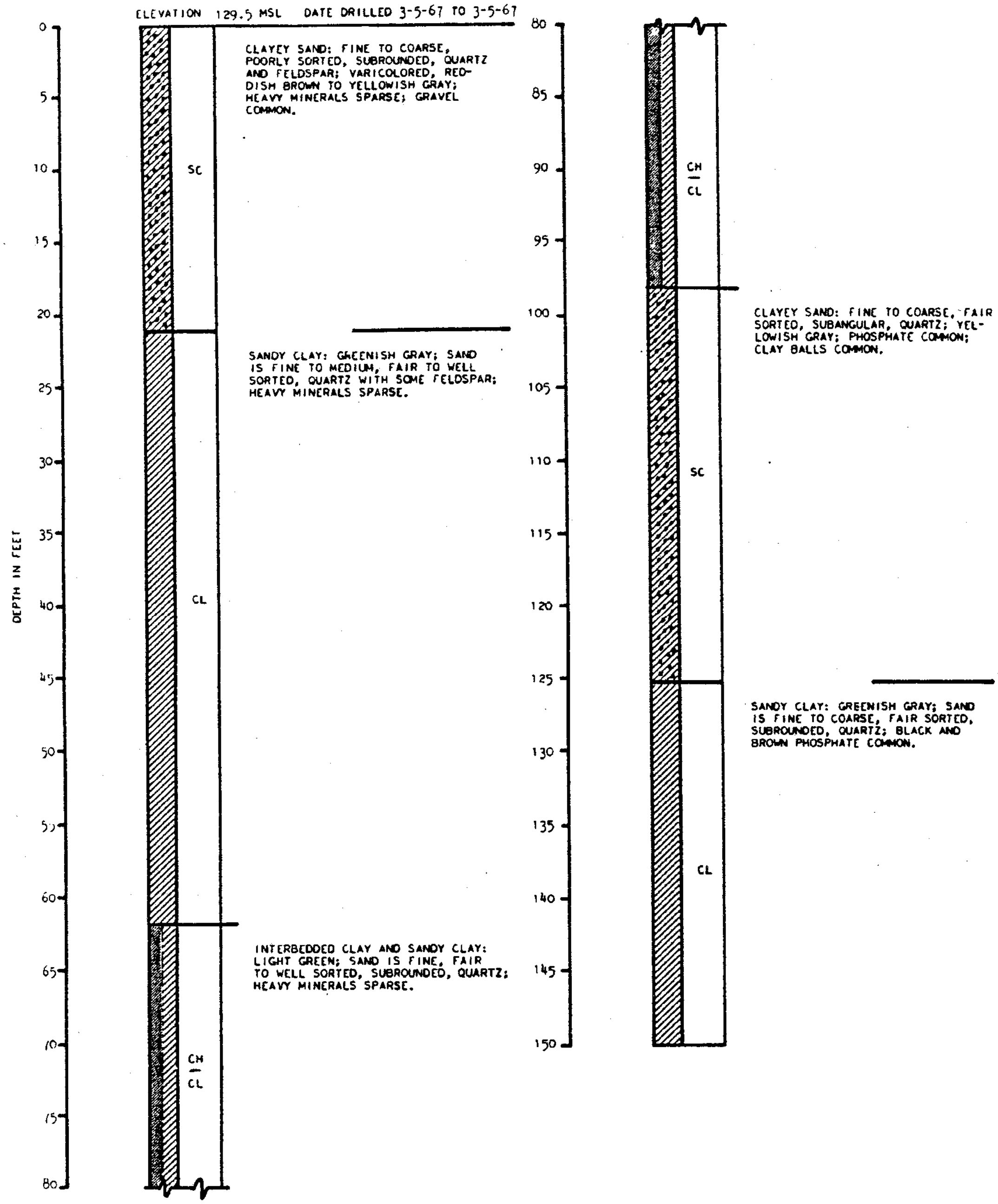
UNIT 2

TEST BORING RECORD
BORING NO. 109

FIGURE 2B-3

BORING 110



BORING DRILLED WITH 4" HAWTHORNE BIT.

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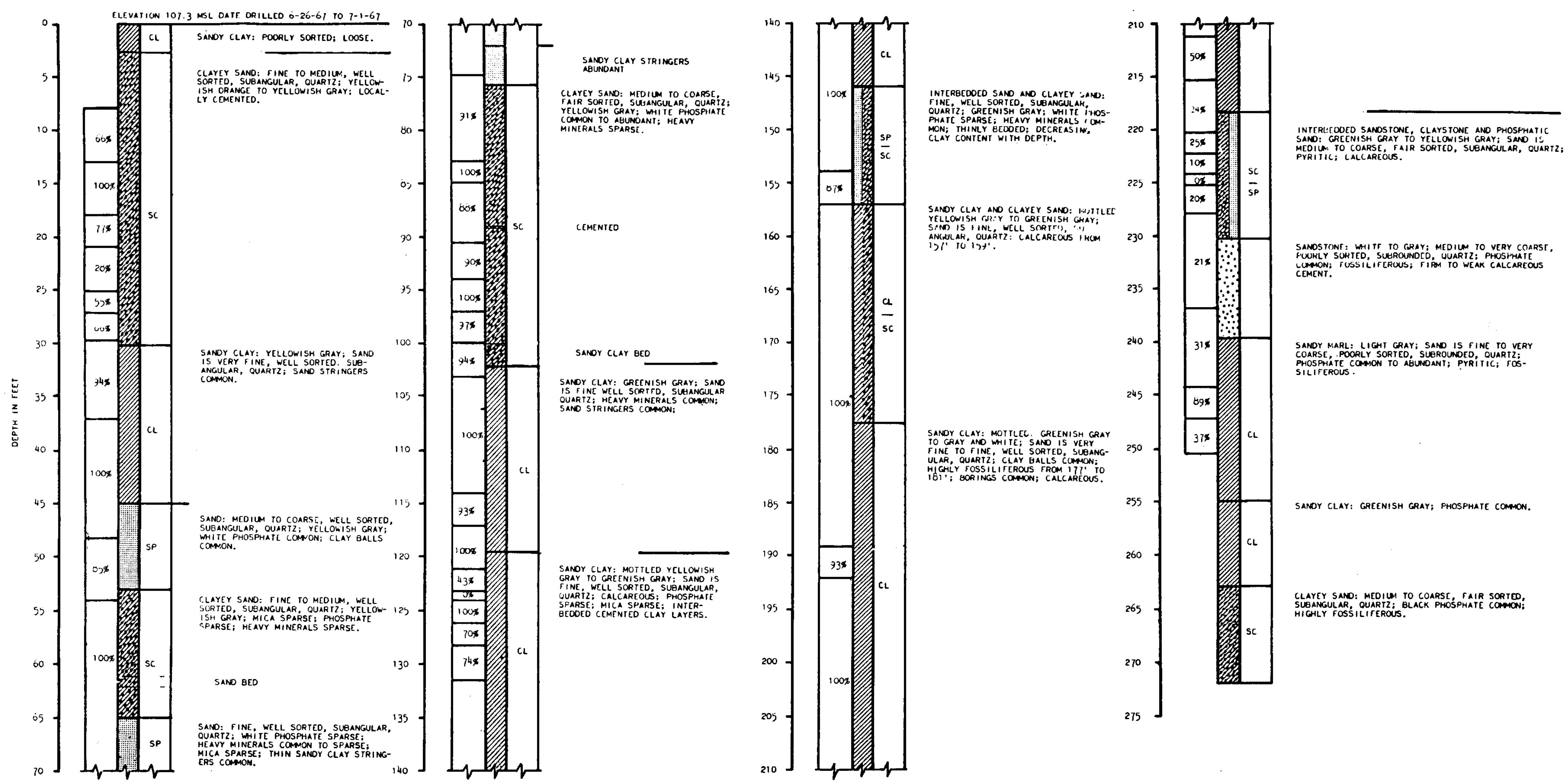
UNIT 2

TEST BORING RECORD
BORING NO. 110

FIGURE 2B-4

BORING 300



81% % CORE RECOVERY. (4" VACUUM CORE BARREL). BALANCE OF BORING DRILLED WITH 4" HAWTHORNE BIT.

LOG OF BORING

HISTORICAL
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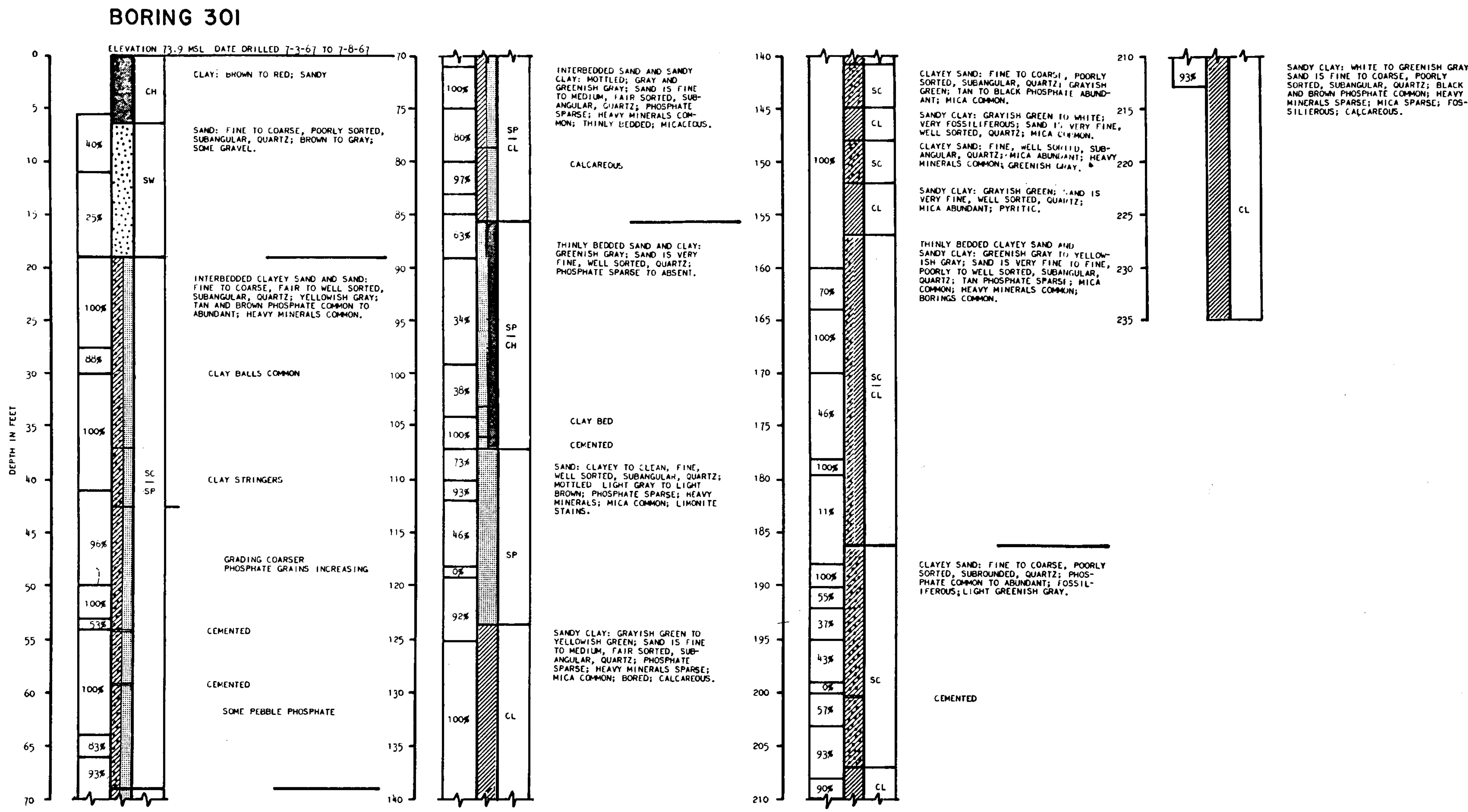
		FIGURE 2B-5



LOG OF BORING

87% % CORE RECOVERY. (4" VACUUM CORE BARREL).
BALANCE OF BORING DRILLED WITH 4" HAWTHORNE BIT.

HISTORICAL

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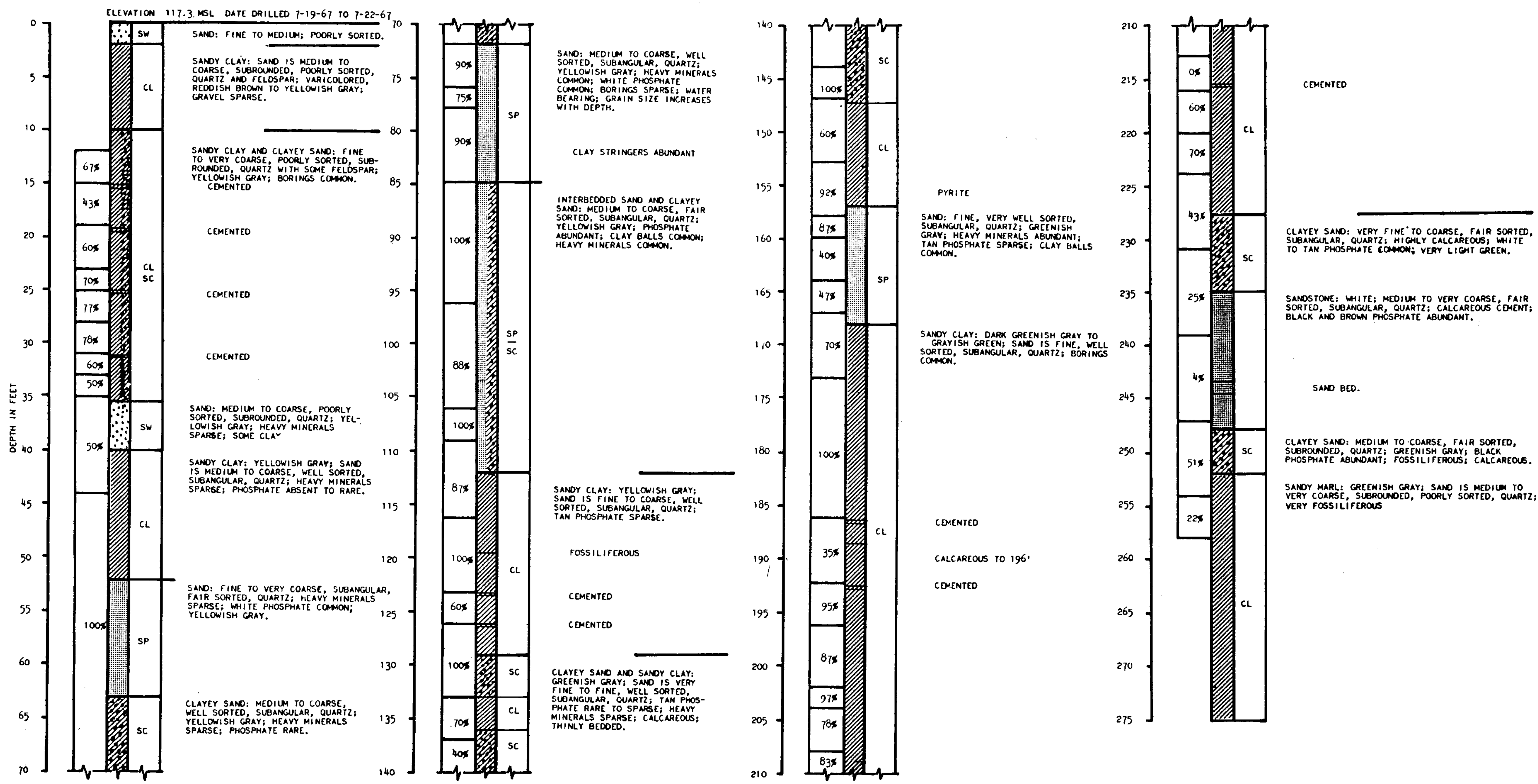
		FIGURE 2B-6

BORING 306



LOG OF BORING

87% — % CORE RECOVERY. (4" VACUUM CORE BARREL).
BALANCE OF BORING DRILLED WITH 4" HAWTHORNE BIT.

HISTORICAL
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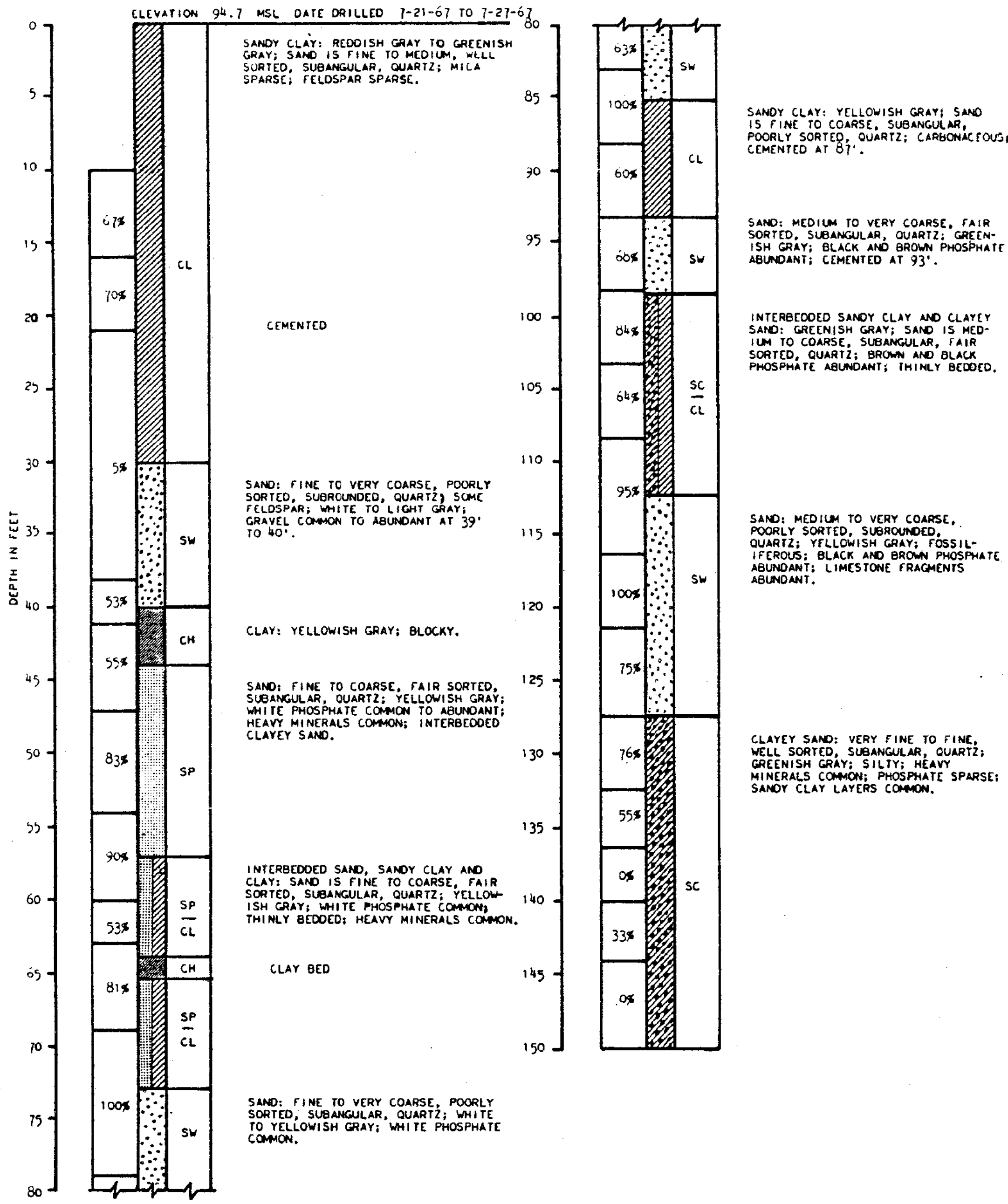
		FIGURE 2B-7

BORING 307



87% — % CORE RECOVERY. (4" VACUUM CORE BARREL).
BALANCE OF BORING DRILLED WITH 4" HAWTHORNE BIT.

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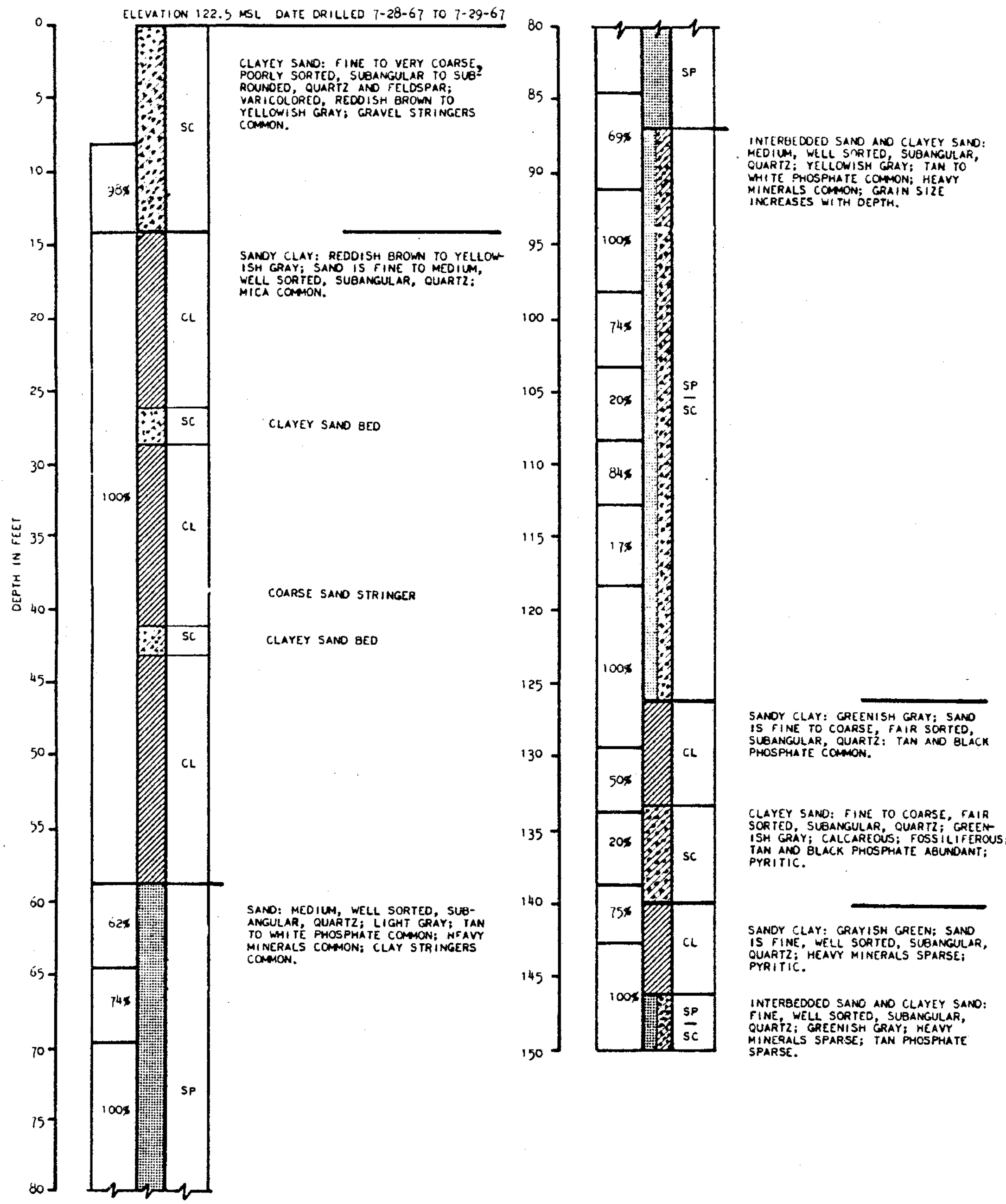
UNIT 2

TEST BORING RECORD
BORING NO. 307

FIGURE 2B-8

BORING 309



87% % CORE RECOVERY. (4" VACUUM CORE BARREL).
BALANCE OF BORING DRILLED WITH 4" HAWTHORNE BIT.

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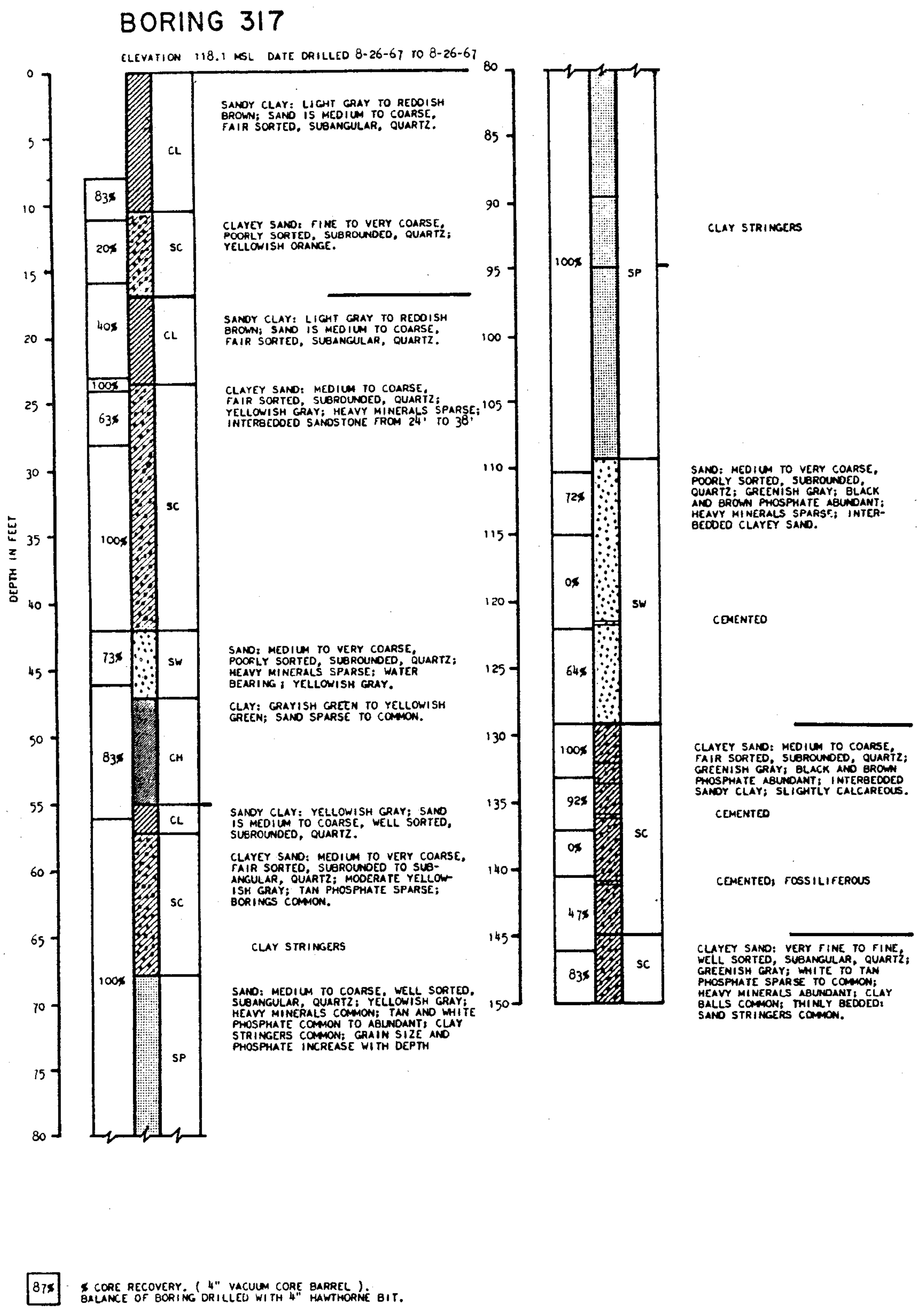
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
RECORD NO. 309

FIGURE 2B-9



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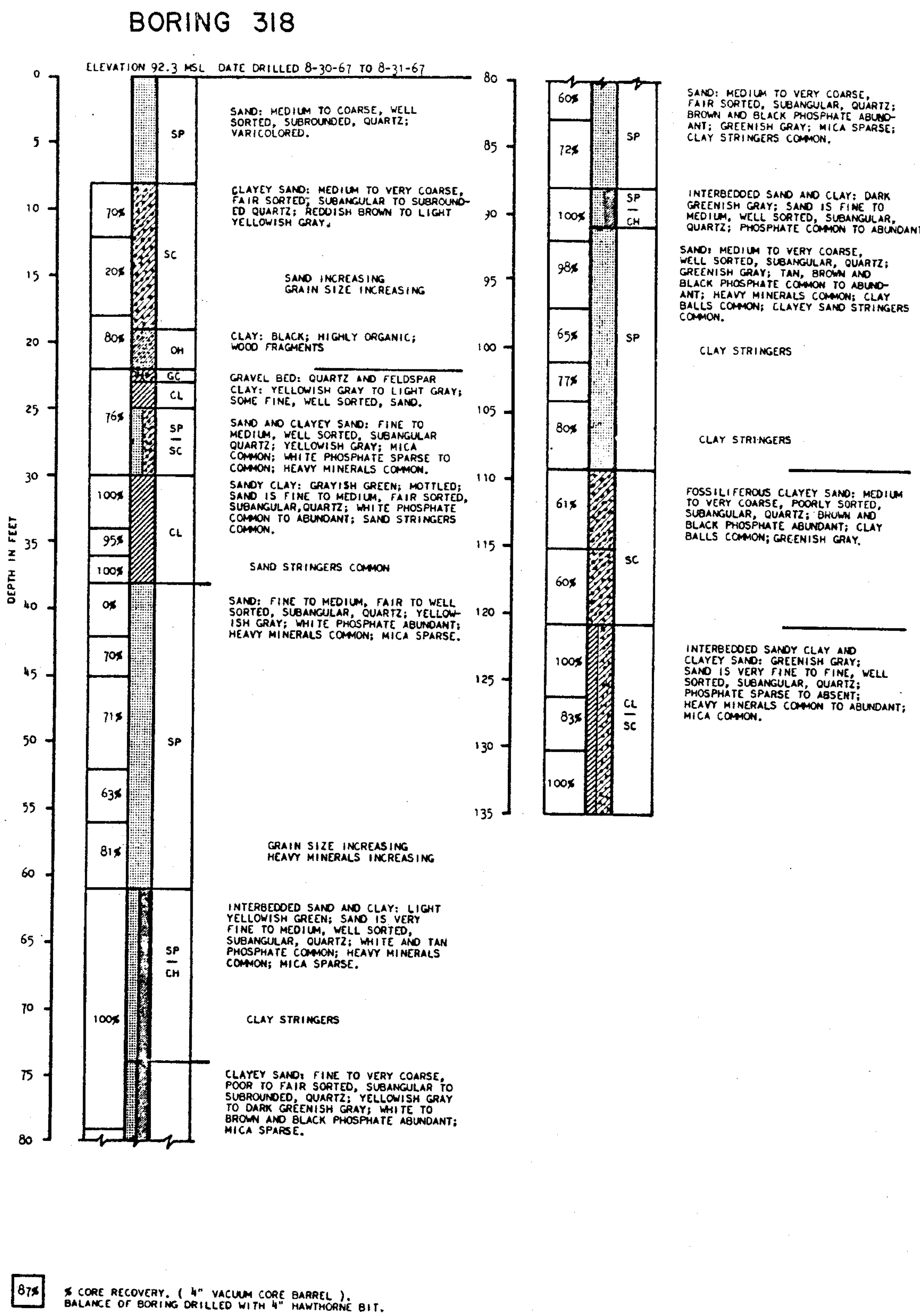
UNIT 2

TEST BORING RECORD
RECORD NO. 317

FIGURE 2B-10



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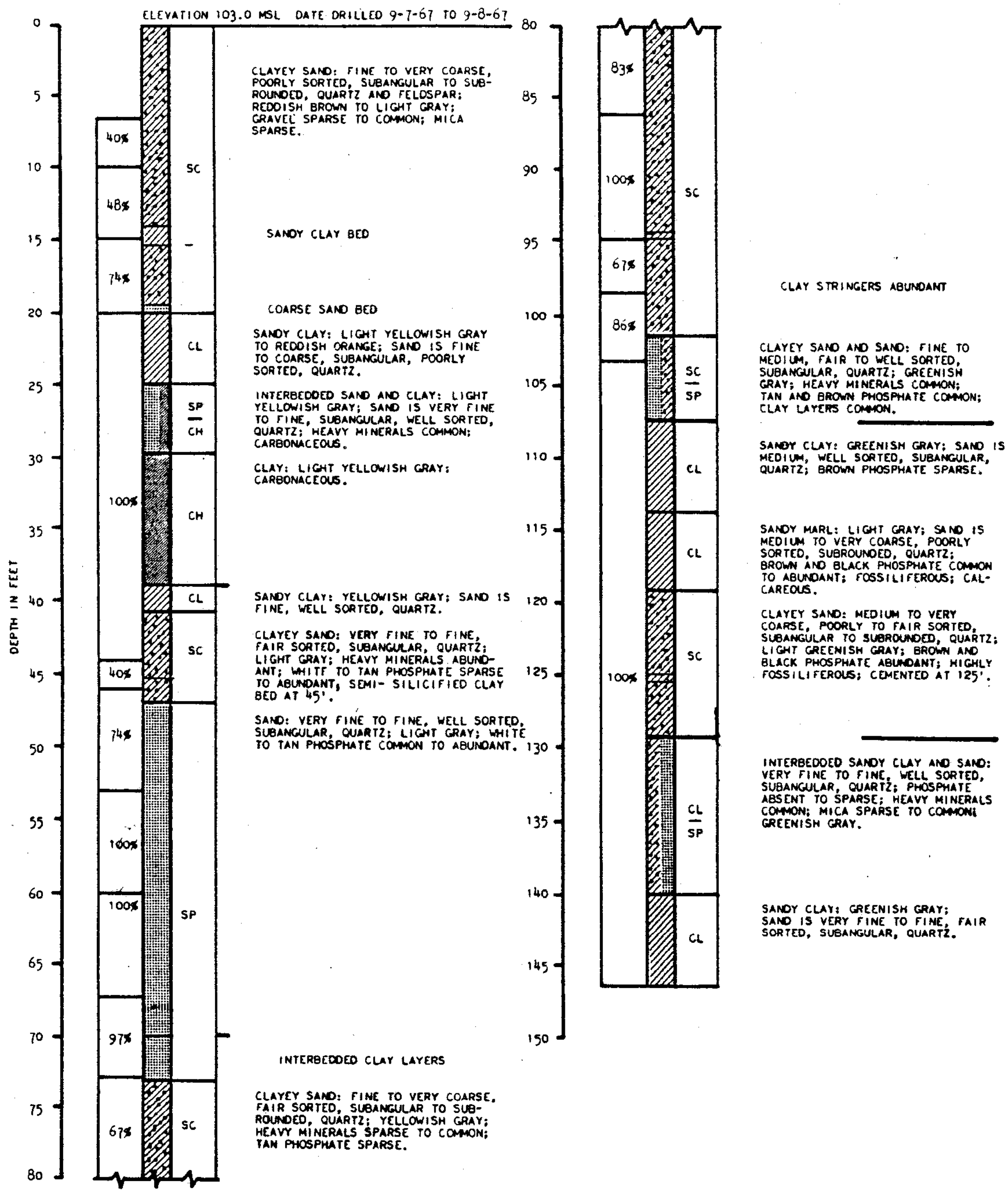
UNIT 2

TEST BORING RECORD
RECORD NO. 318

FIGURE 2B-11

BORING 320



87% — % CORE RECOVERY. (4" VACUUM CORE BARREL)
BALANCE OF BORING DRILLED WITH 4" HAWTHORNE BIT.

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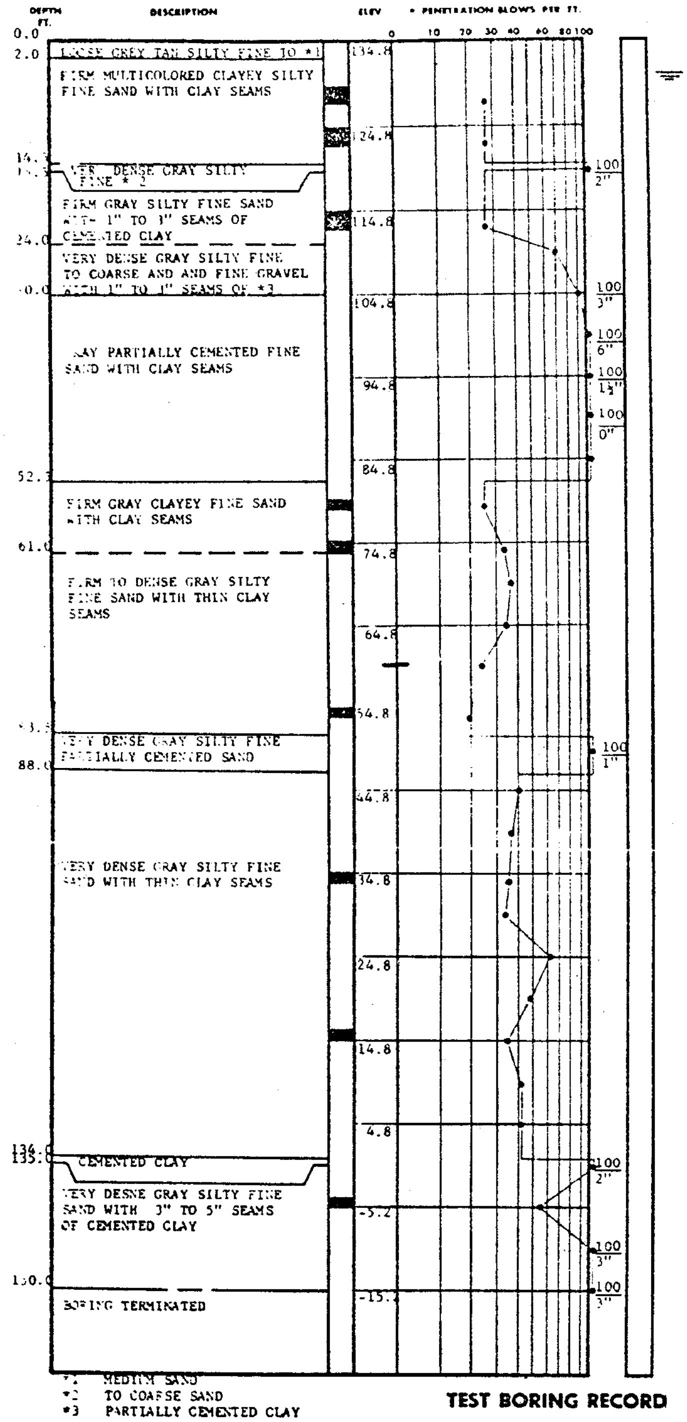
UNIT 2

TEST BORING RECORD
RECORD NO. 320

FIGURE 2B-12



TEST BORING RECORD

BORING NO. B-401
DATE DRILLED 3/14-16/67
JOB NO. 5056-B

LAW ENGINEERING TESTING CO.

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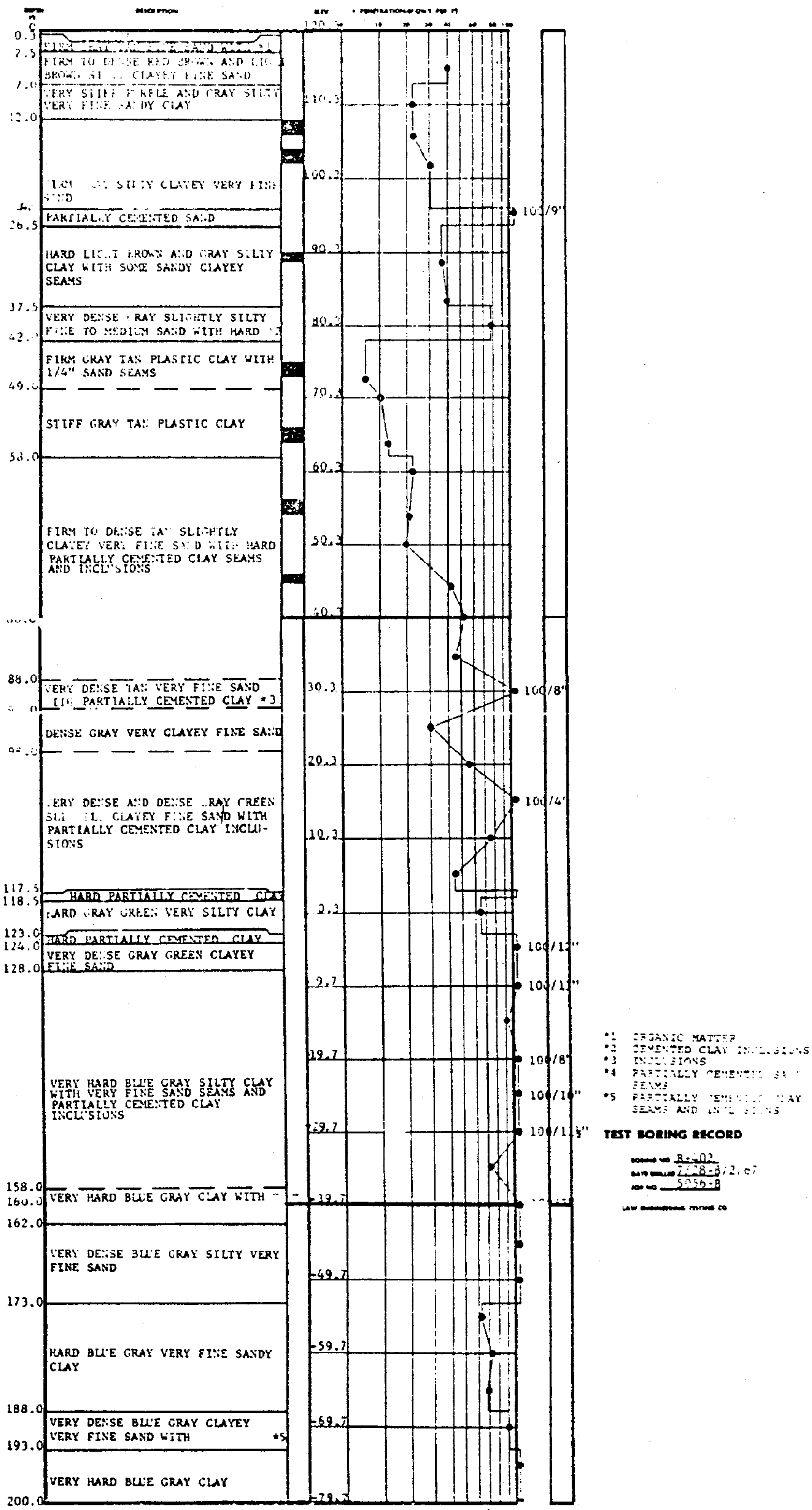
UNIT 2

TEST BORING RECORD
RECORD NO. 401

FIGURE 2B-13



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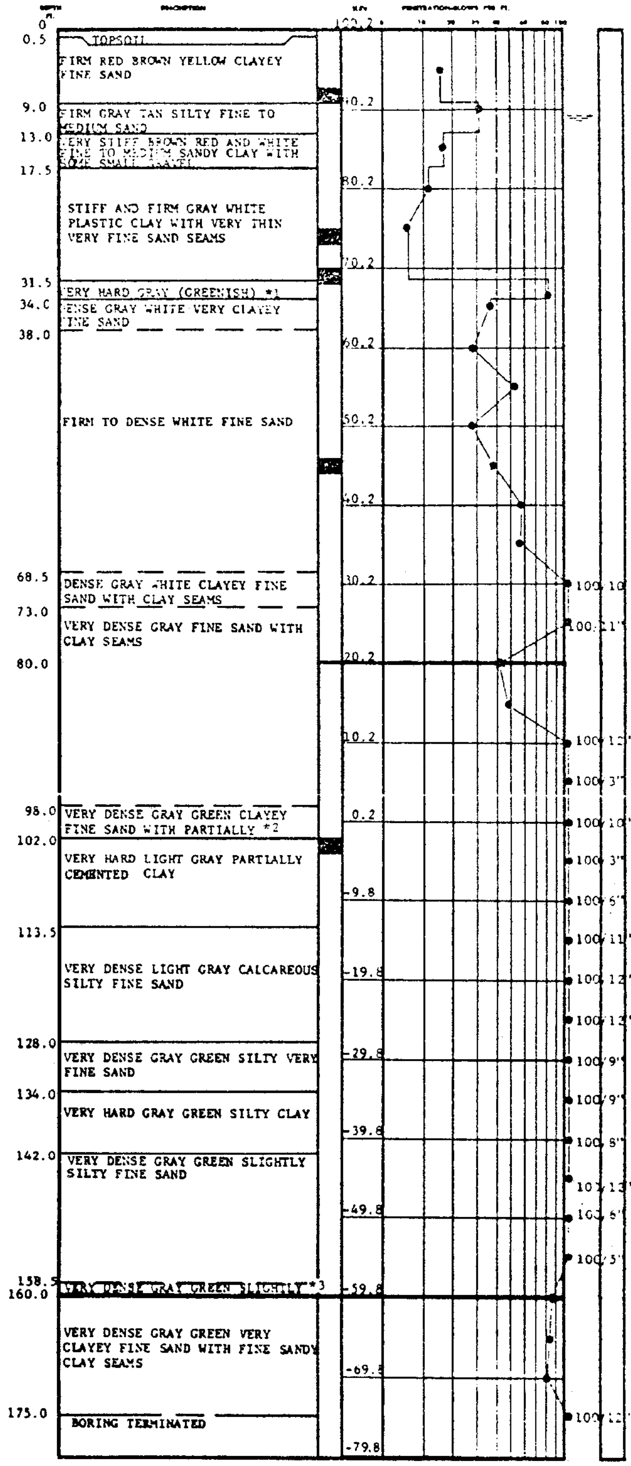
UNIT 2

TEST BORING RECORD
RECORD NO. 402

FIGURE 2B-14



*1 PARTIALLY CEMENTED CLAY
*2 CEMENTED CLAY SEAMS AND CLAY INCLUSIONS
*3 SILTY FINE SAND

TEST BORING RECORD

BORING NO. B-403
DATE DRILLED 8/16
JOB NO. 5056-B

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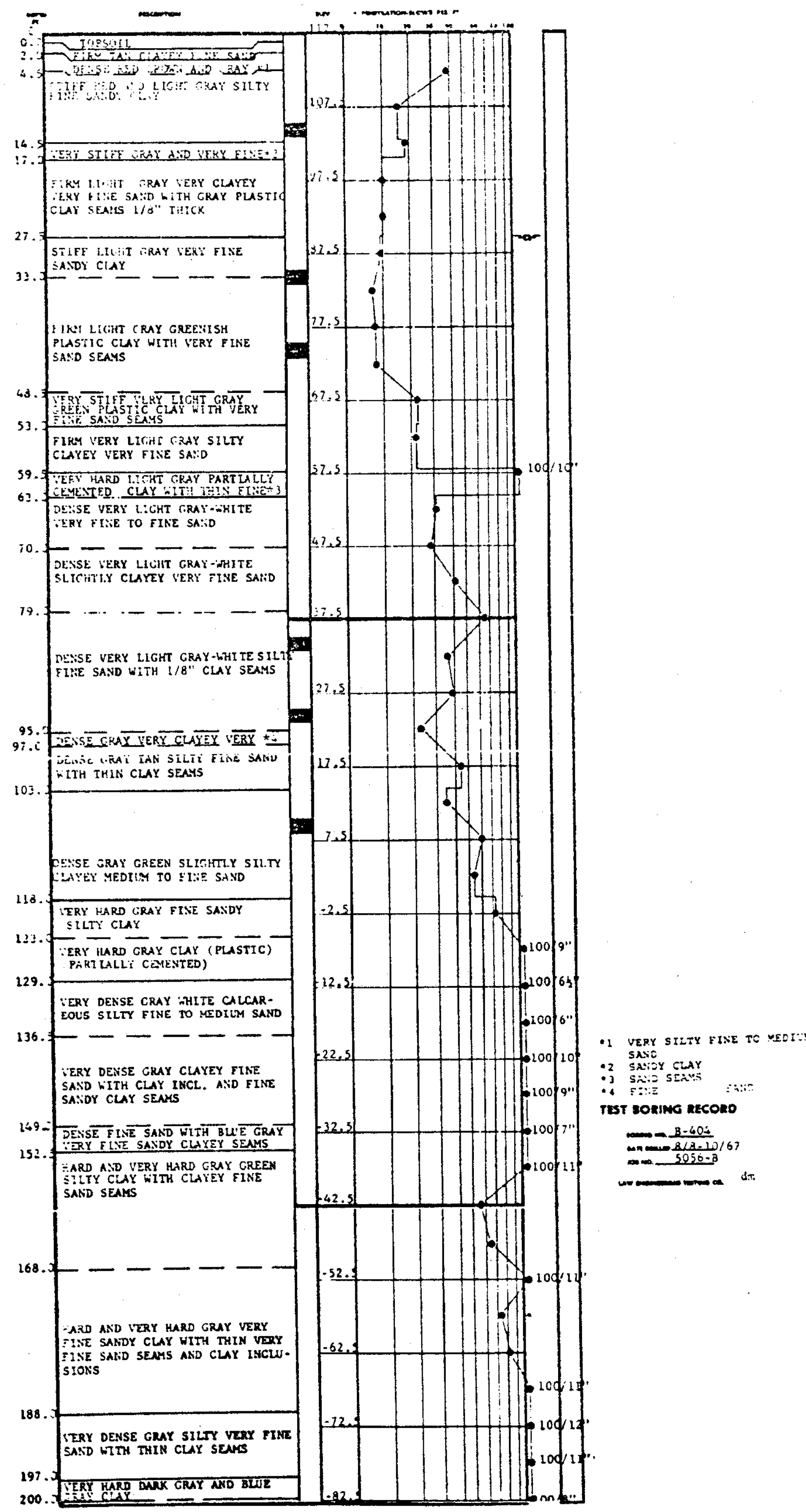
UNIT 2

TEST BORING RECORD
RECORD NO. 403

FIGURE 2B-15



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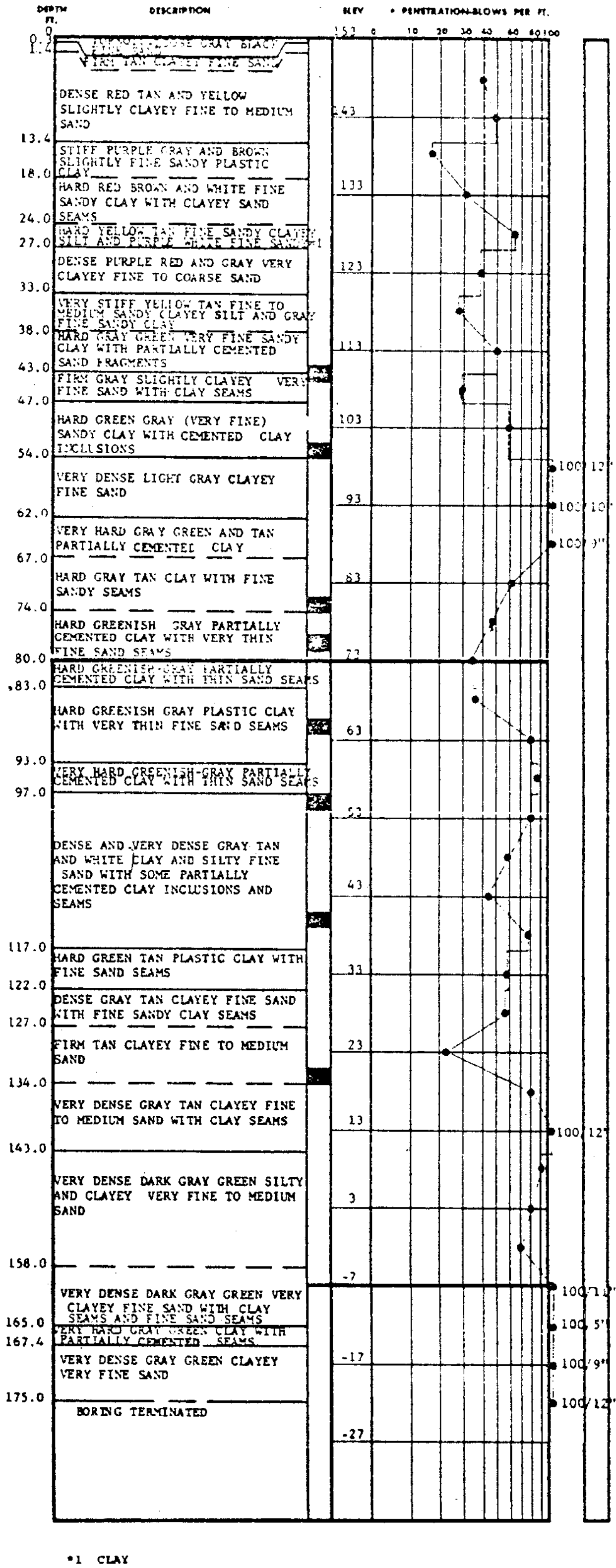
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
RECORD NO. 404

FIGURE 2B-16



TEST BORING RECORD

BORING NO. B-405

DATE DRILLED 8/27-28/'67

JOB NO. 5056-B

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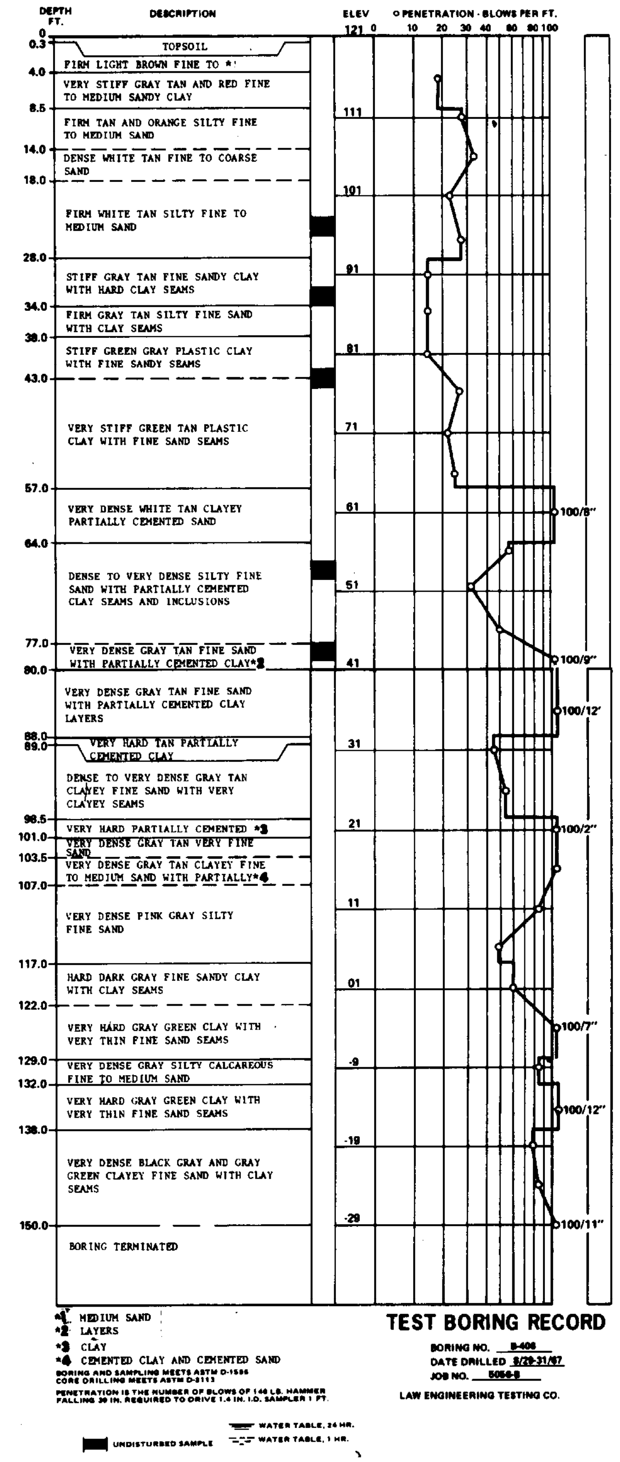
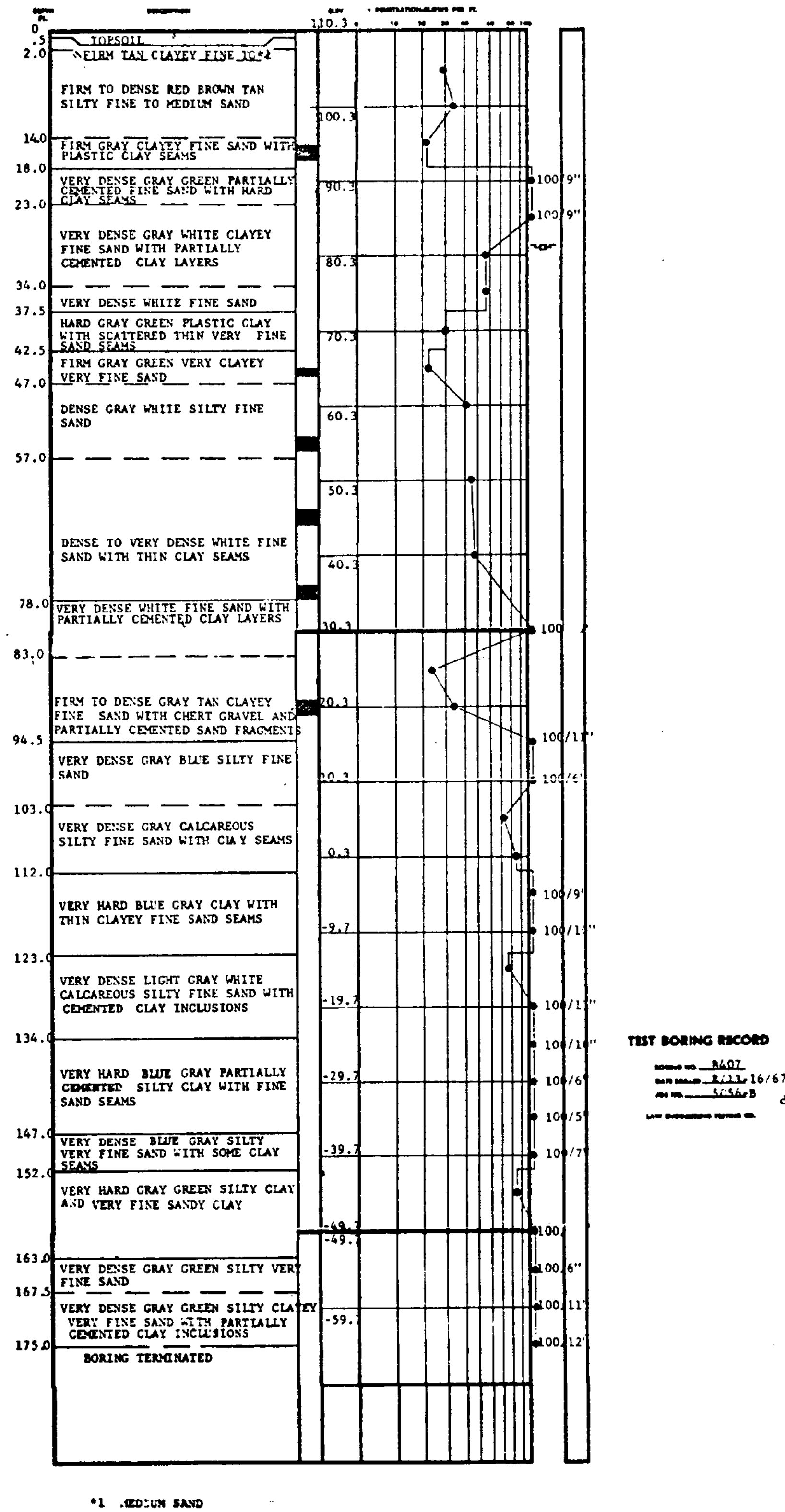
UNIT 2

TEST BORING RECORD
BORING No. 405

FIGURE 2B-17





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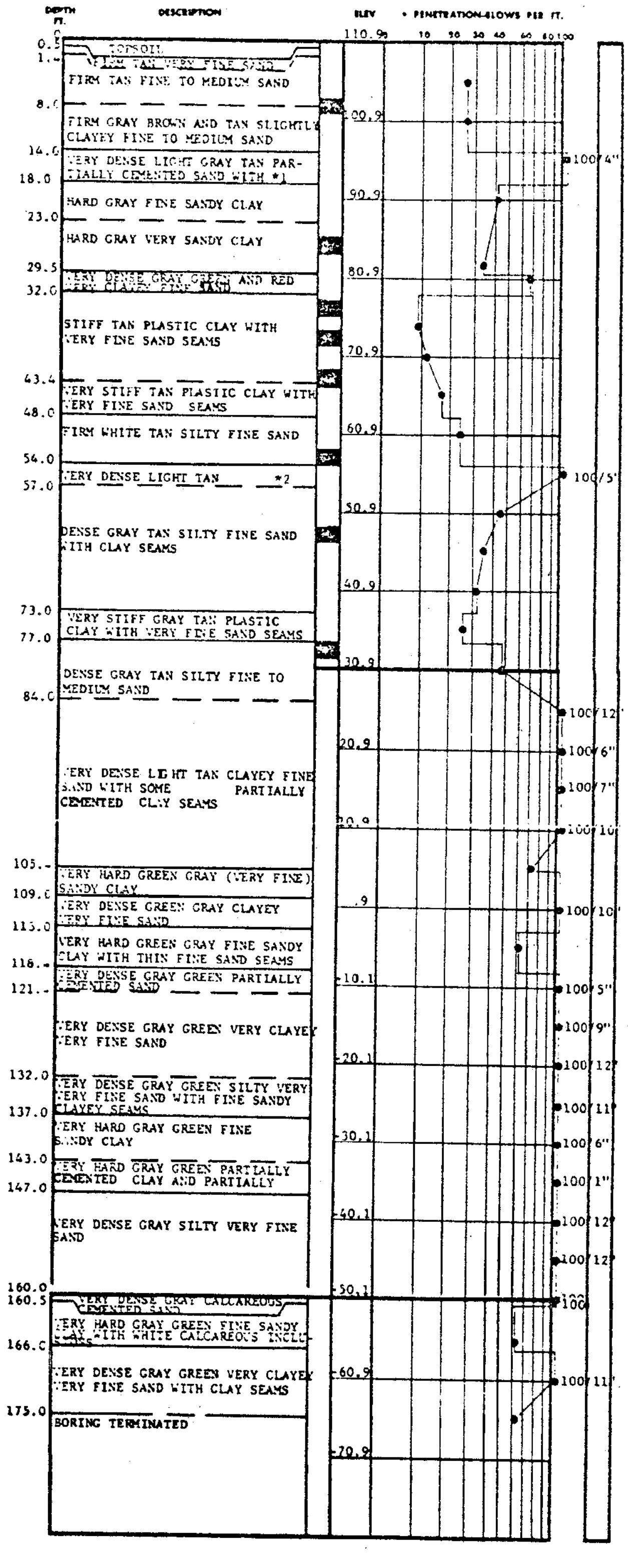
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TEST BORING RECORD BORING NOS. 406 AND 407
		FIGURE 2B-18



*1 CLAY SEAMS
*2 CLAYEY SAND WITH CEMENTED FINE SAND SEAMS

TEST BORING RECORD

BORING NO. B-408
DATE DRILLED 8/24-25/67
JOB NO. 5036-B

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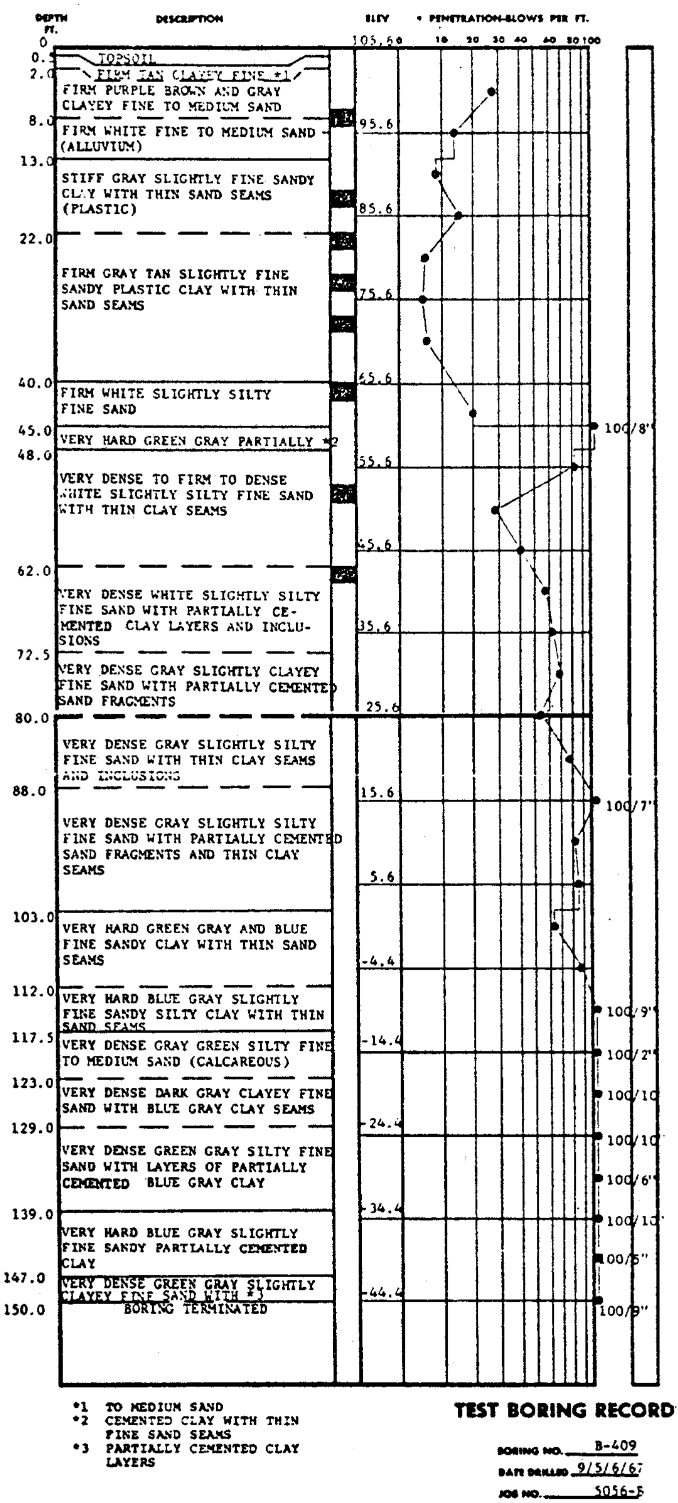
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 408

FIGURE 2B-19



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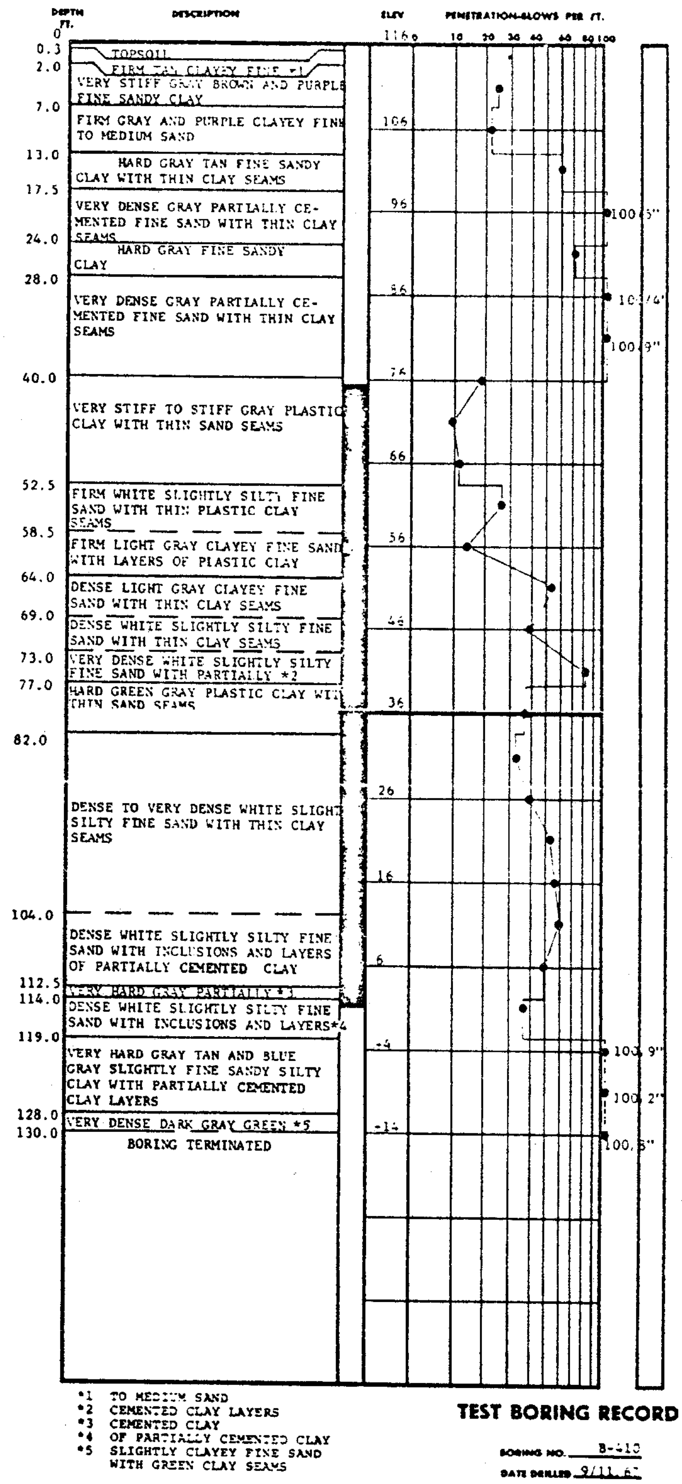
UNIT 2

TEST BORING RECORD
BORING NO. 409

FIGURE 2B-20



TEST BORING RECORD

BORING NO. B-410
DATE DRILLED 9/11/67
JOB NO. 5056-B

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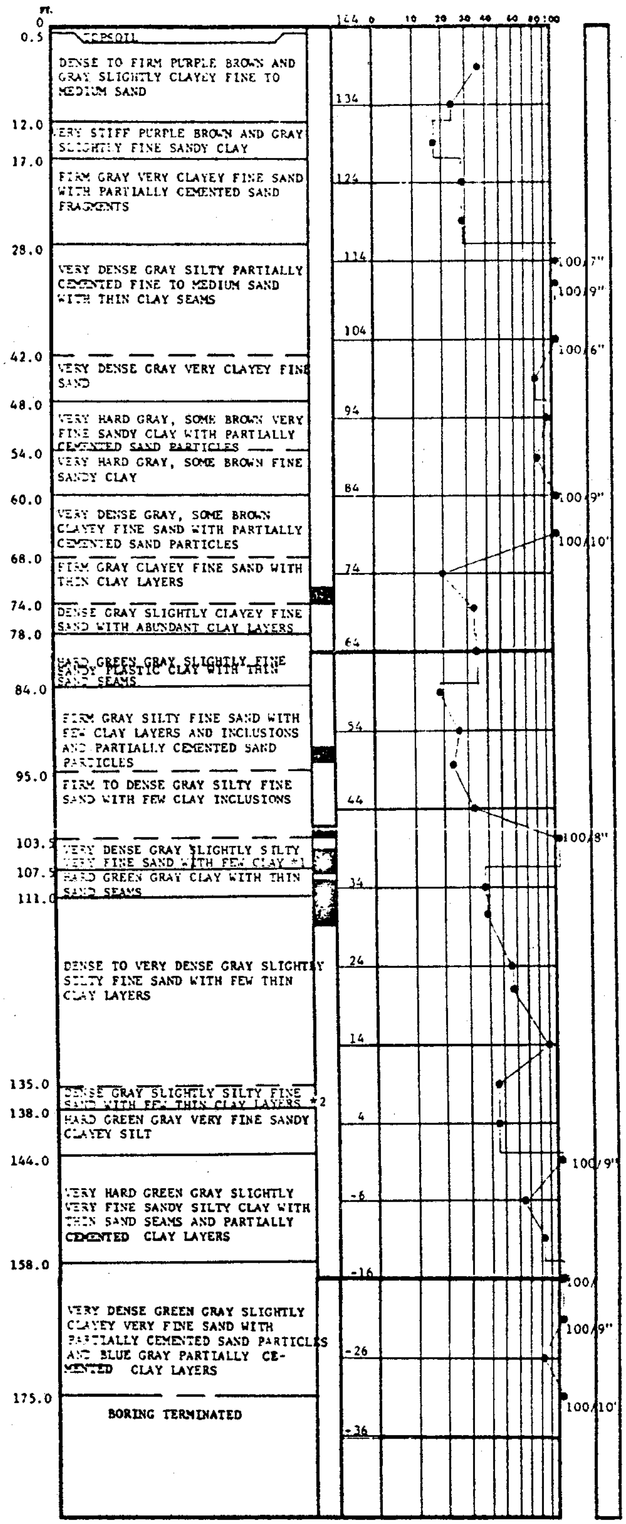
UNIT 2

TEST BORING RECORD
BORING NO. 410

FIGURE 2B-21



*1 LAYERS
*2 AND PARTIALLY INDURATED CLAY INCLUSIONS

TEST BORING RECORD

BORING NO. B-411
DATE DRILLED 9/9/11/67
JOB NO. 5056-B

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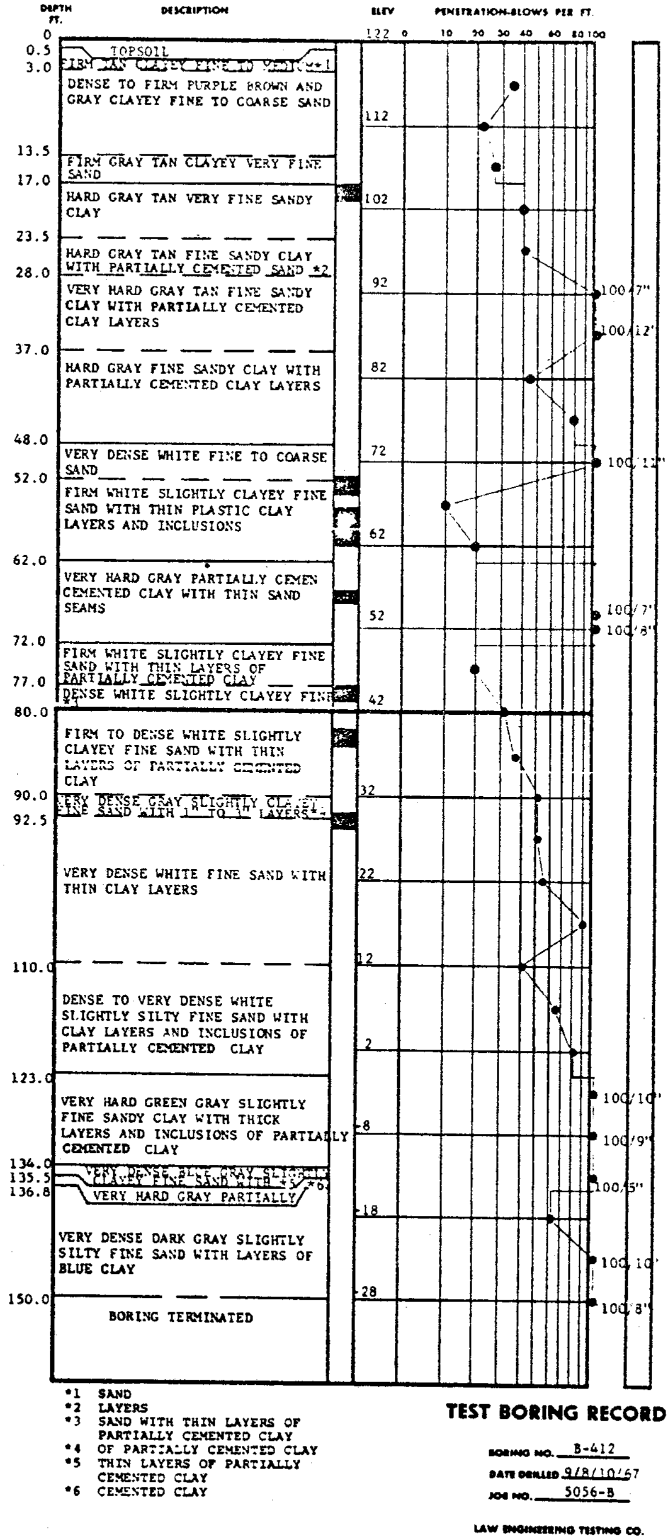
UNIT 2

TEST BORING RECORD
BORING NO. 411

FIGURE 2B-22



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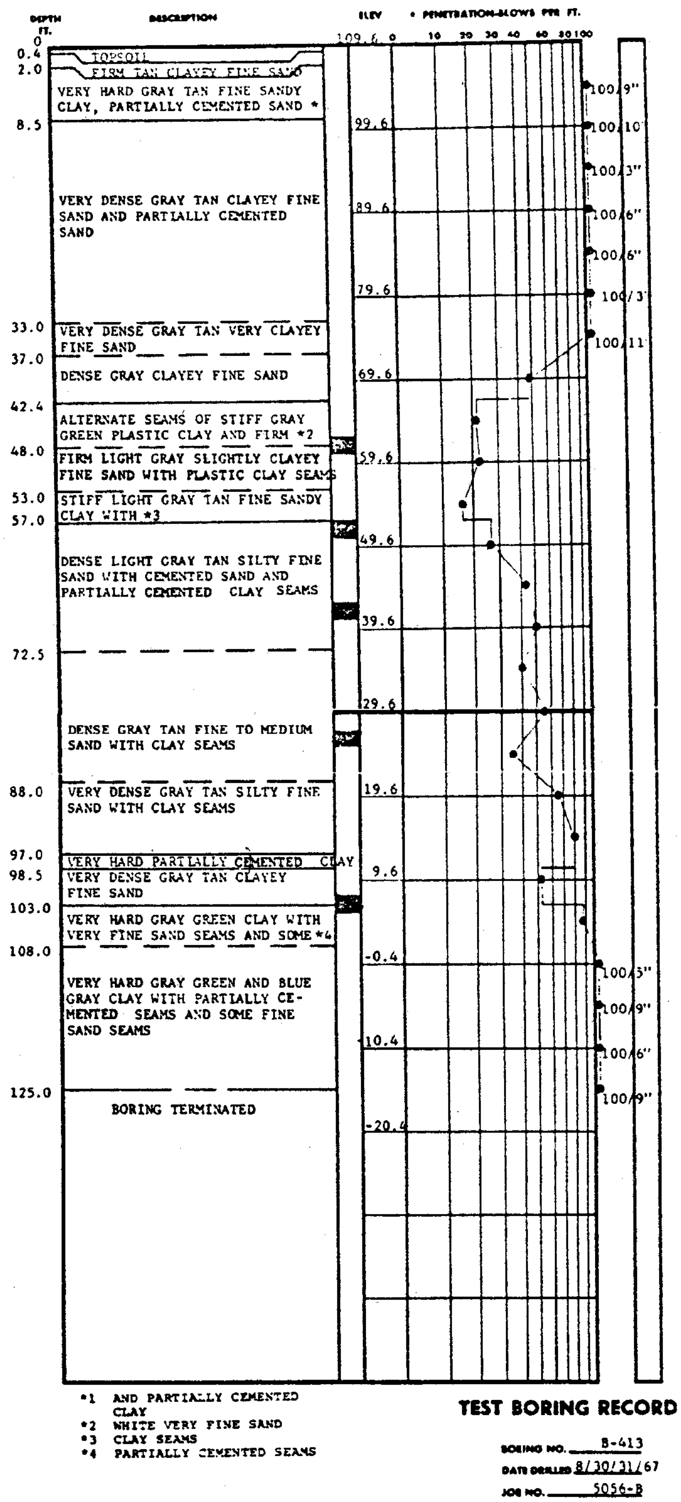
UNIT 2

TEST BORING RECORD
BORING NO. 412

FIGURE 2B-23



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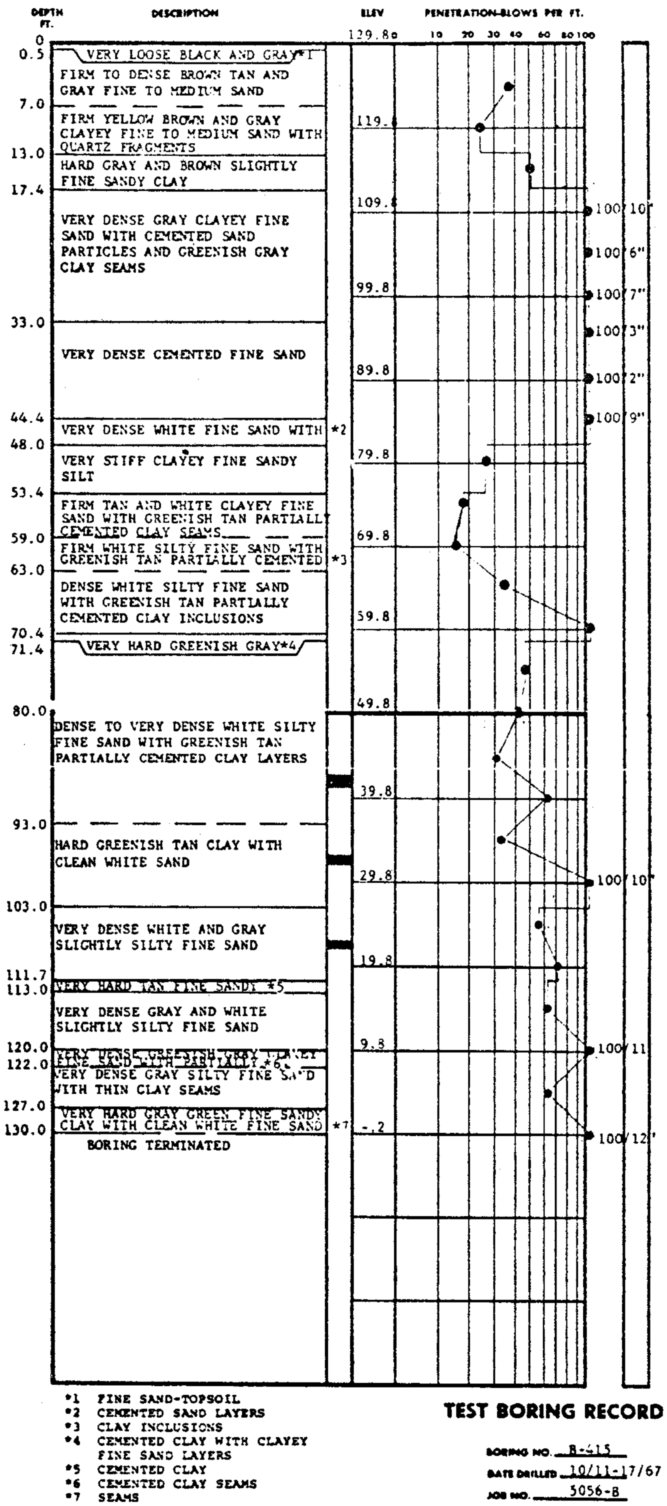
UNIT 2

TEST BORING RECORD
BORING NO. 413

FIGURE 2B-24



HISTORICAL

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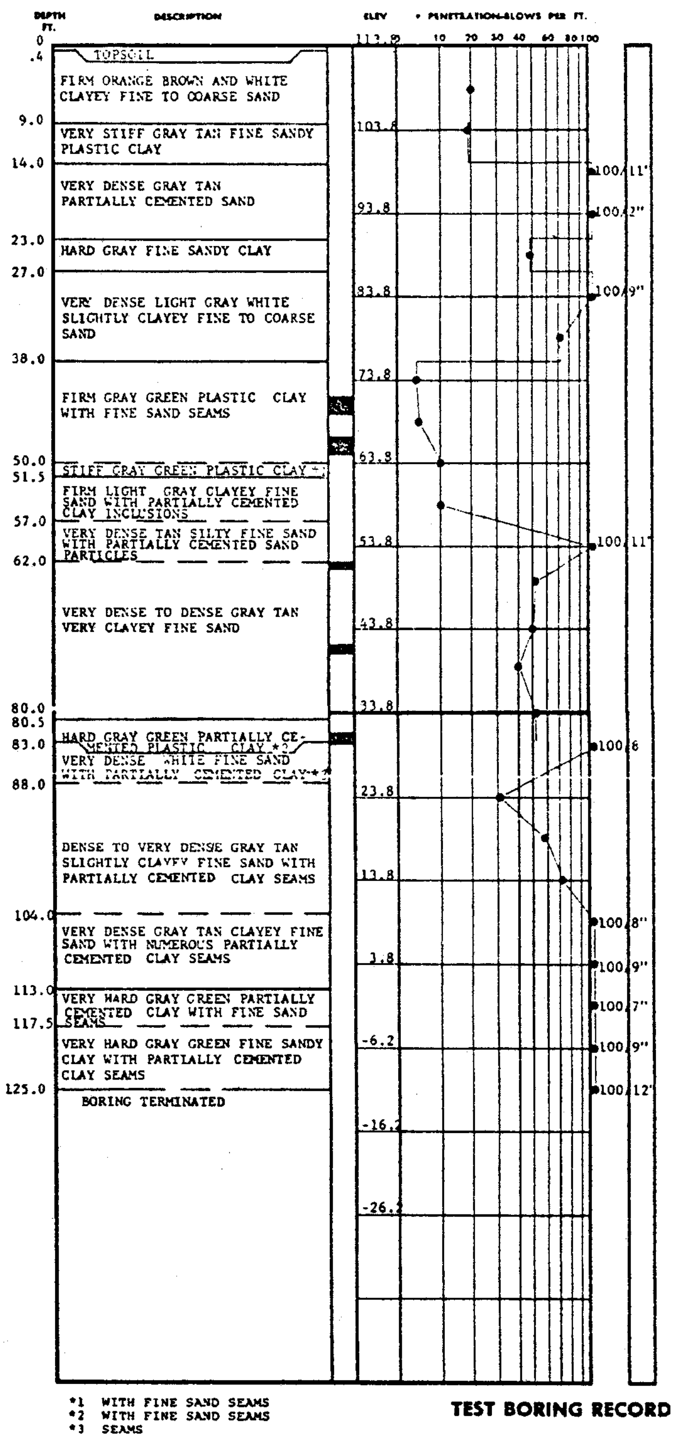
UNIT 2

TEST BORING RECORD
RECORD NO. 415

FIGURE 2B-25



TEST BORING RECORD

BORING NO. B-416
DATE DRILLED 9-23-24-67
JOB NO. 5056-B

HISTORICAL

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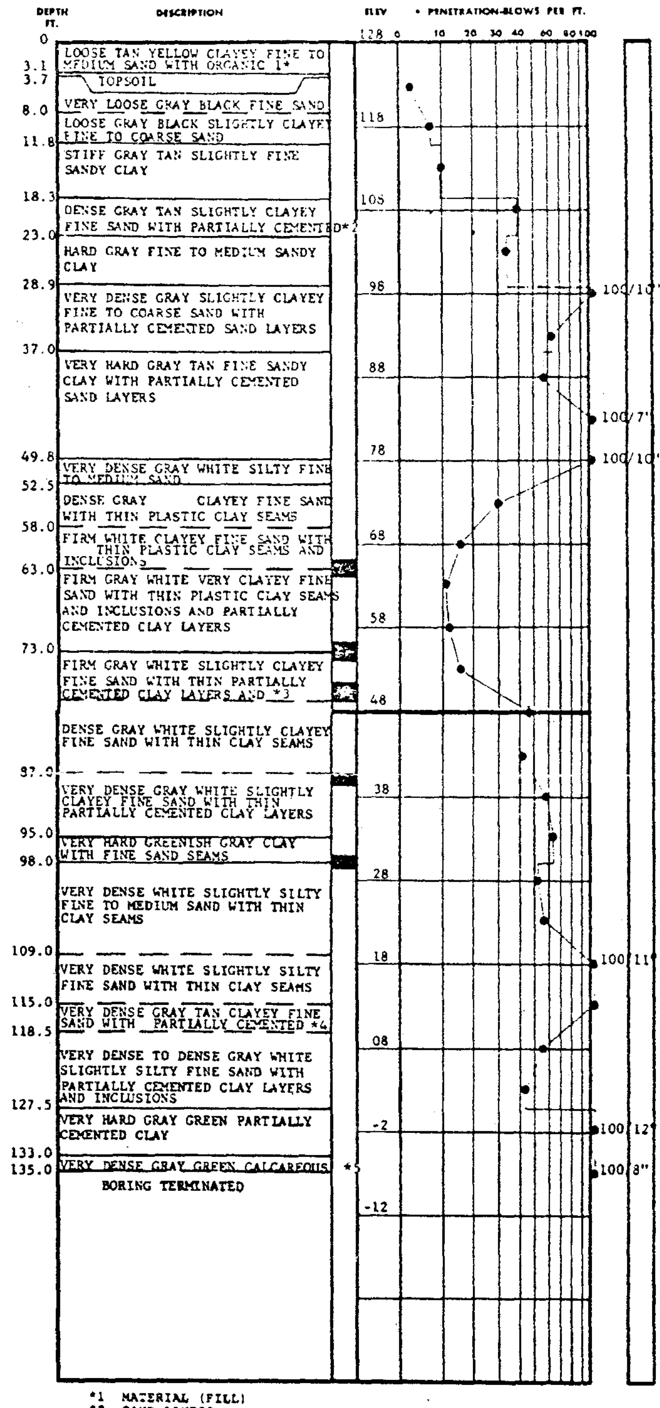
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
RECORD NO. 416

FIGURE 2B-26



*1 MATERIAL (FILL)
*2 SAND LAYERS
*3 INCLUSIONS
*4 CLAY LAYERS AND INCLUSIONS
*5 SLIGHTLY CLAYEY FINE TO MEDIUM SAND

TEST BORING RECORD

BORING NO. B-417
DATE DRILLED 9/24-25/67
JOB NO. 5056-B

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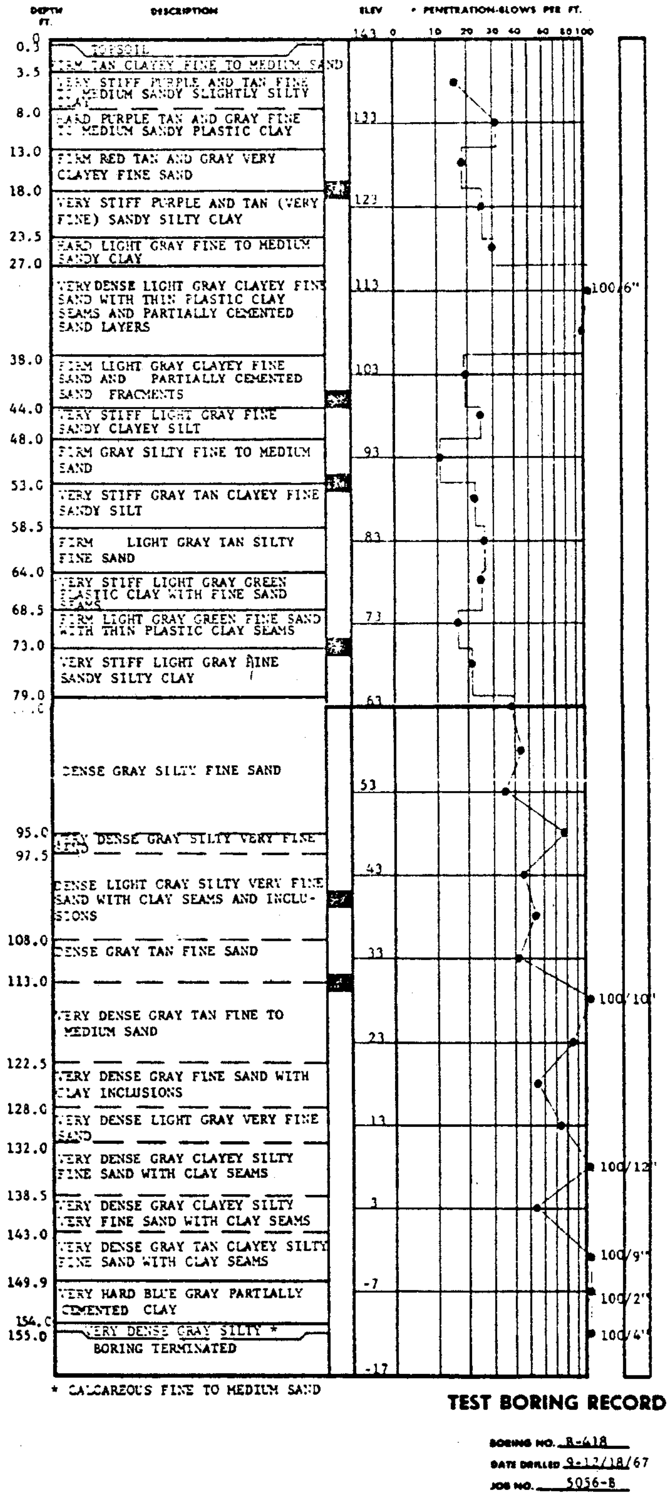
UNIT 2

TEST BORING RECORD
BORING NO. 417

FIGURE 2B-27



HISTORICAL

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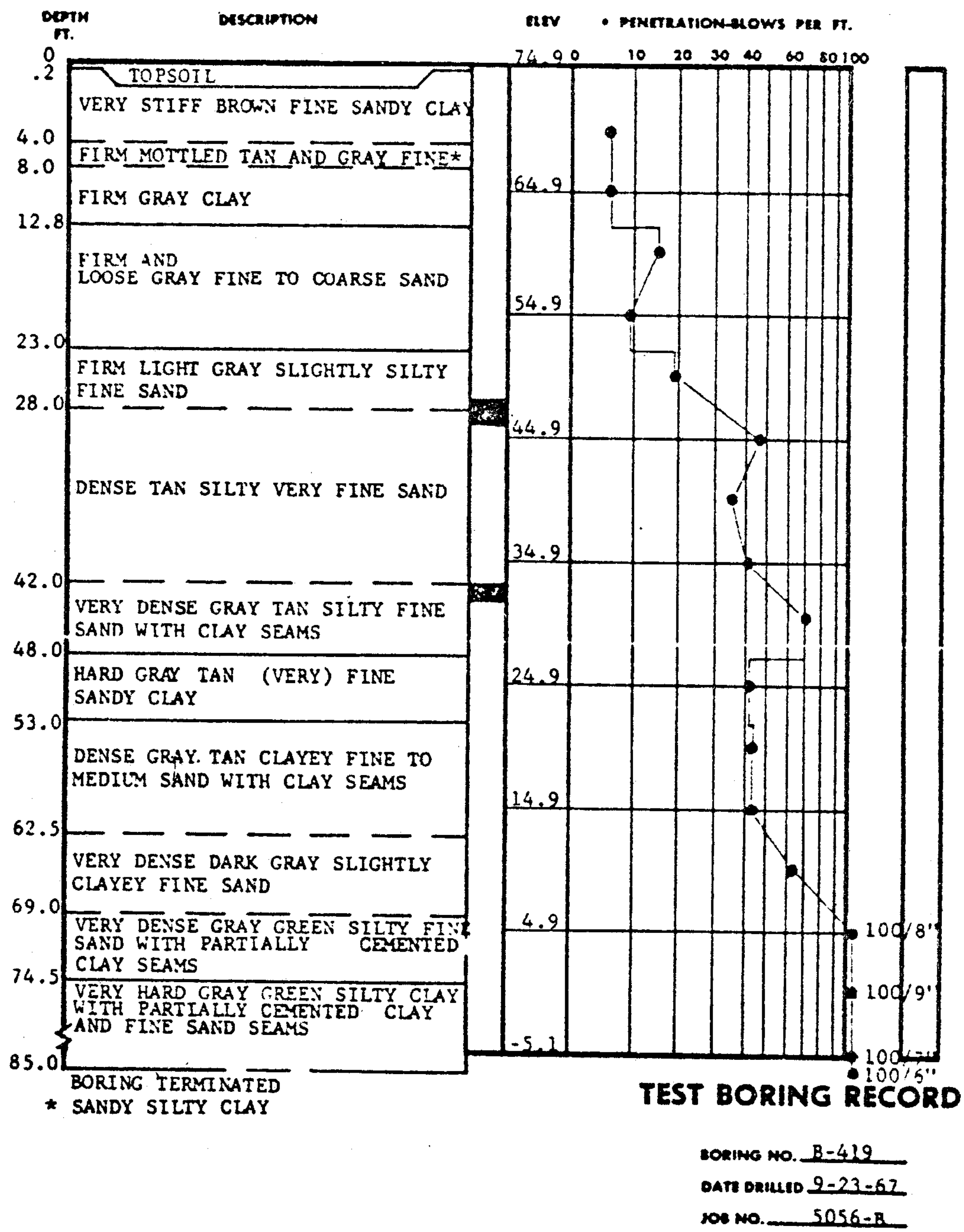
UNIT 2

TEST BORING RECORD
BORING NO. 418

FIGURE 2B-28



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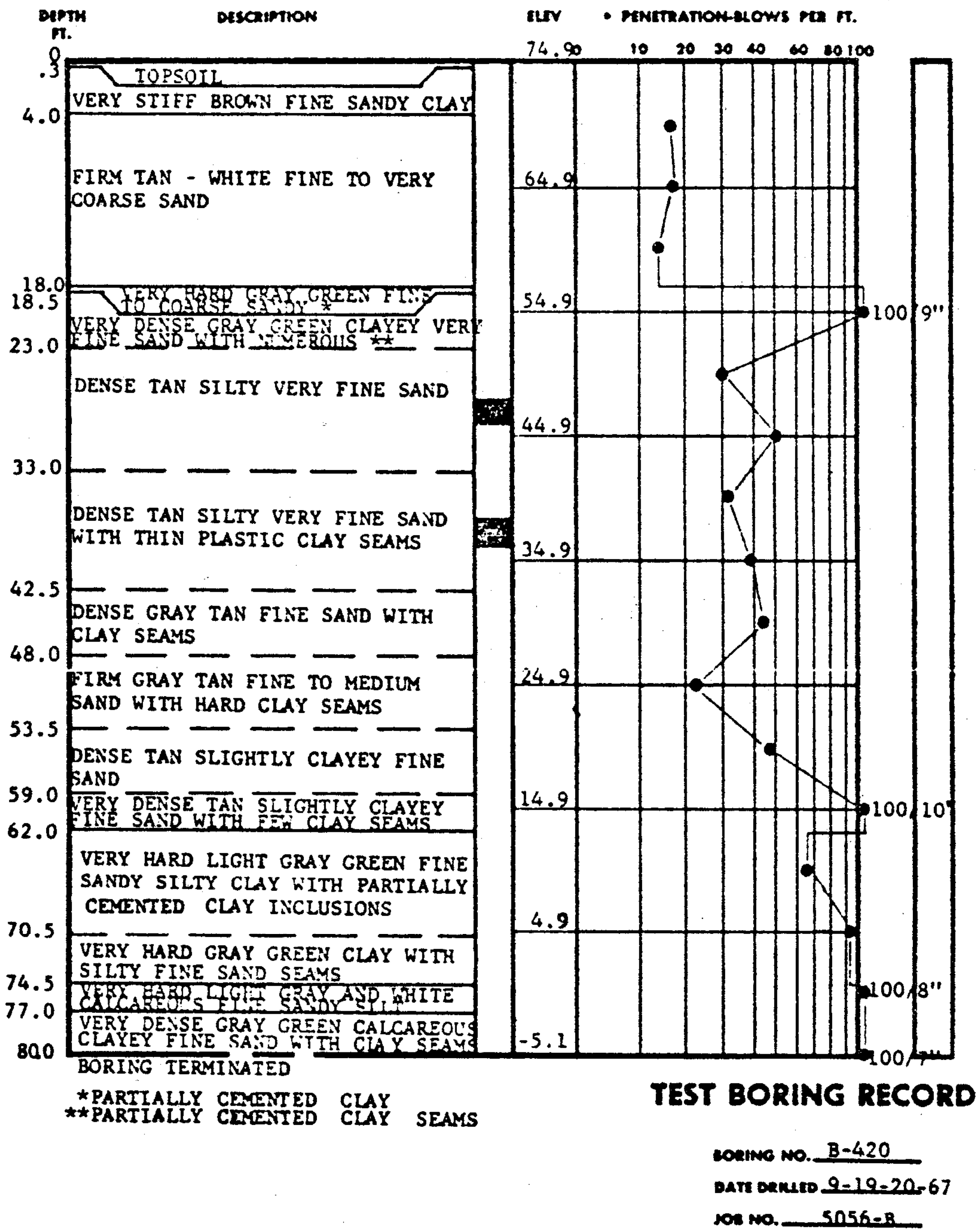
UNIT 2

TEST BORING RECORD
BORING NO. 419

FIGURE 2B-29



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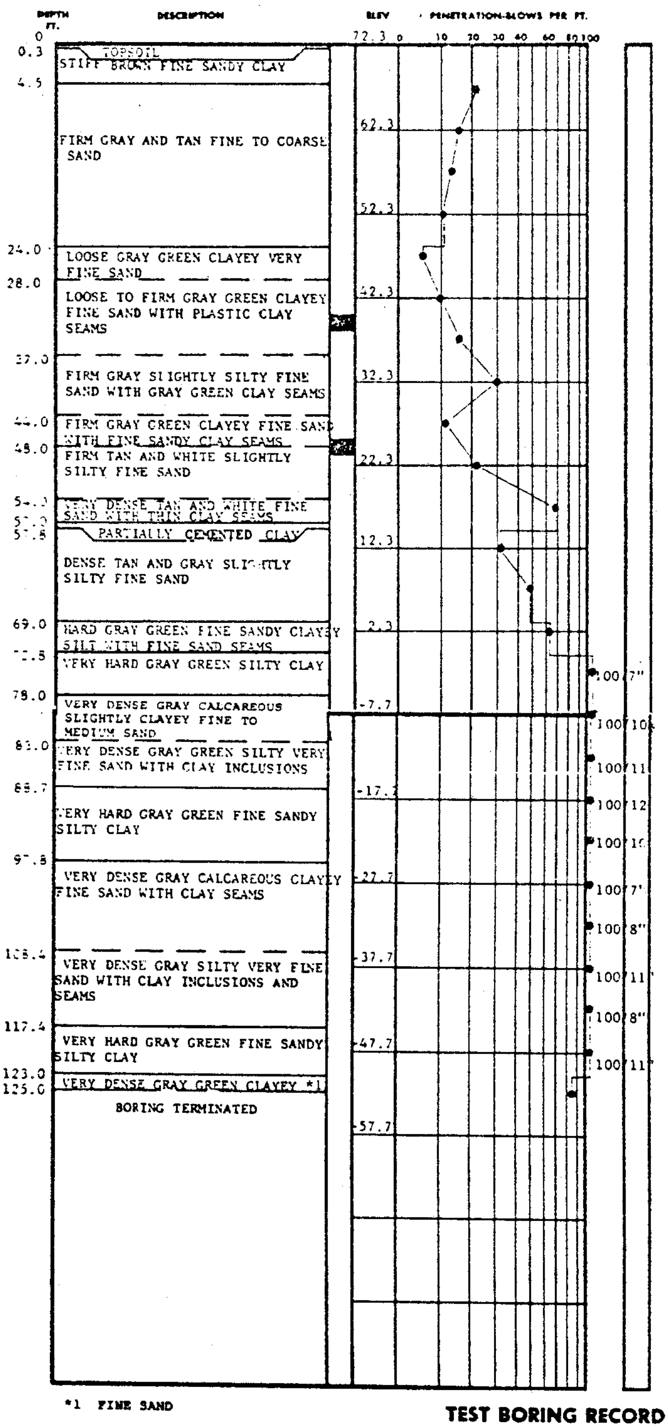
UNIT 2

TEST BORING RECORD
BORING NO. 420

FIGURE 2B-30



TEST BORING RECORD

BORING NO. B-421
DATE DRILLED 9/21/22/67
JOB NO. 5056-B

HISTORICAL

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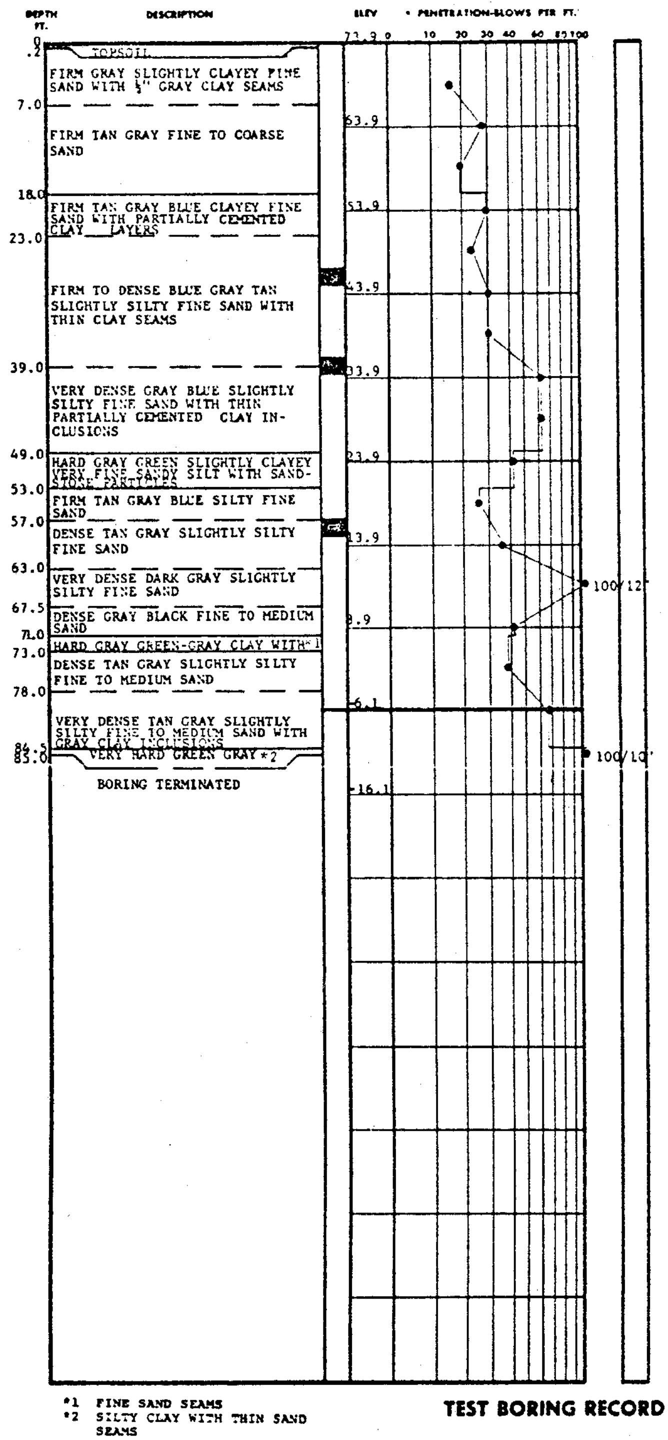
UNIT 2

TEST BORING RECORD
BORING NO. 421

FIGURE 2B-31



TEST BORING RECORD

BORING NO. B-422
DATE DRILLED 9-20/21-67
JOB NO. 5056-B

HISTORICAL

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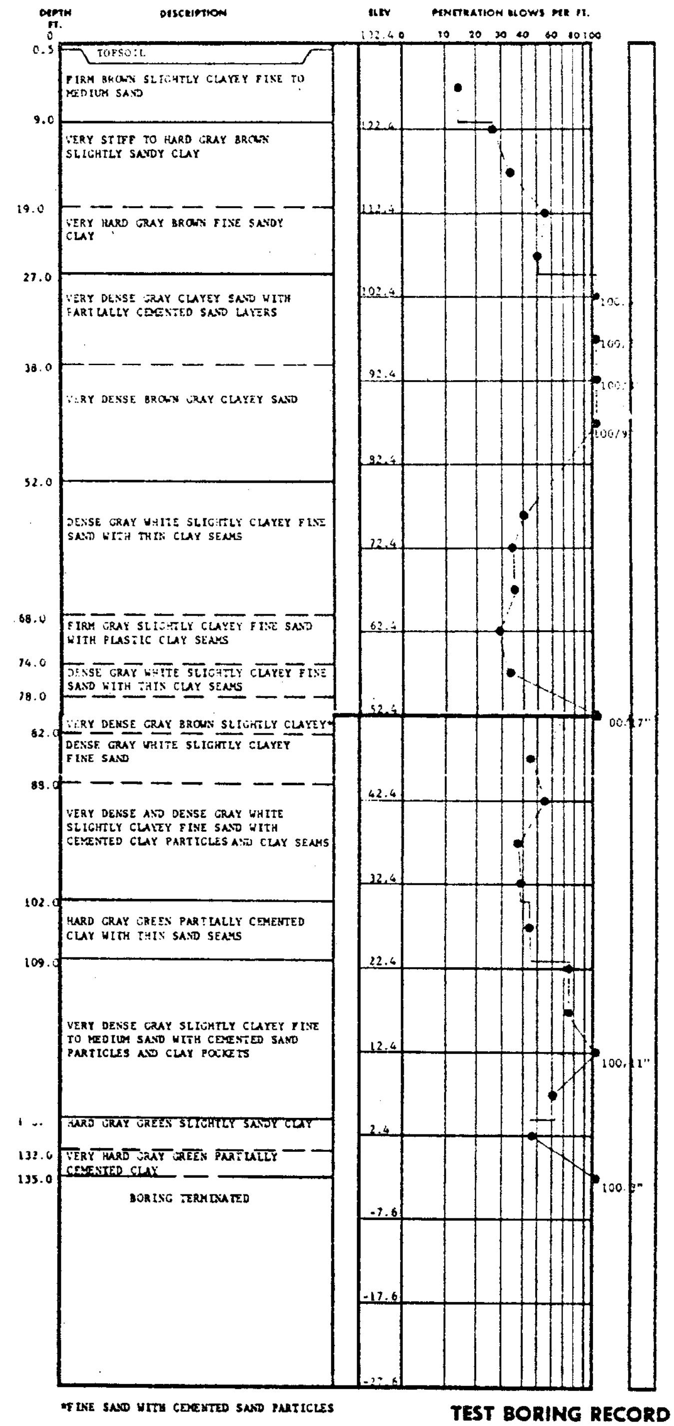
UNIT 2

TEST BORING RECORD
BORING NO. 422

FIGURE 2B-32



TEST BORING RECORD

BORING NO. B-423
DATE DRILLED 11/30/67
JOB NO. 5056-B

HISTORICAL

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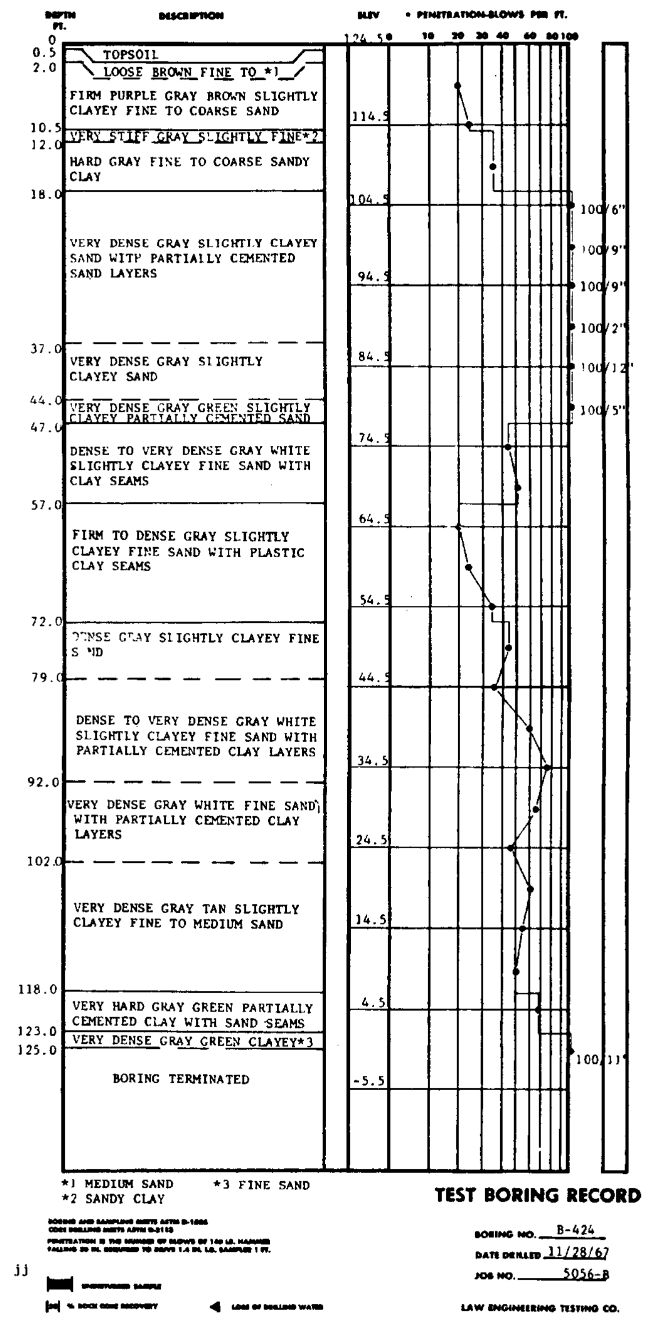
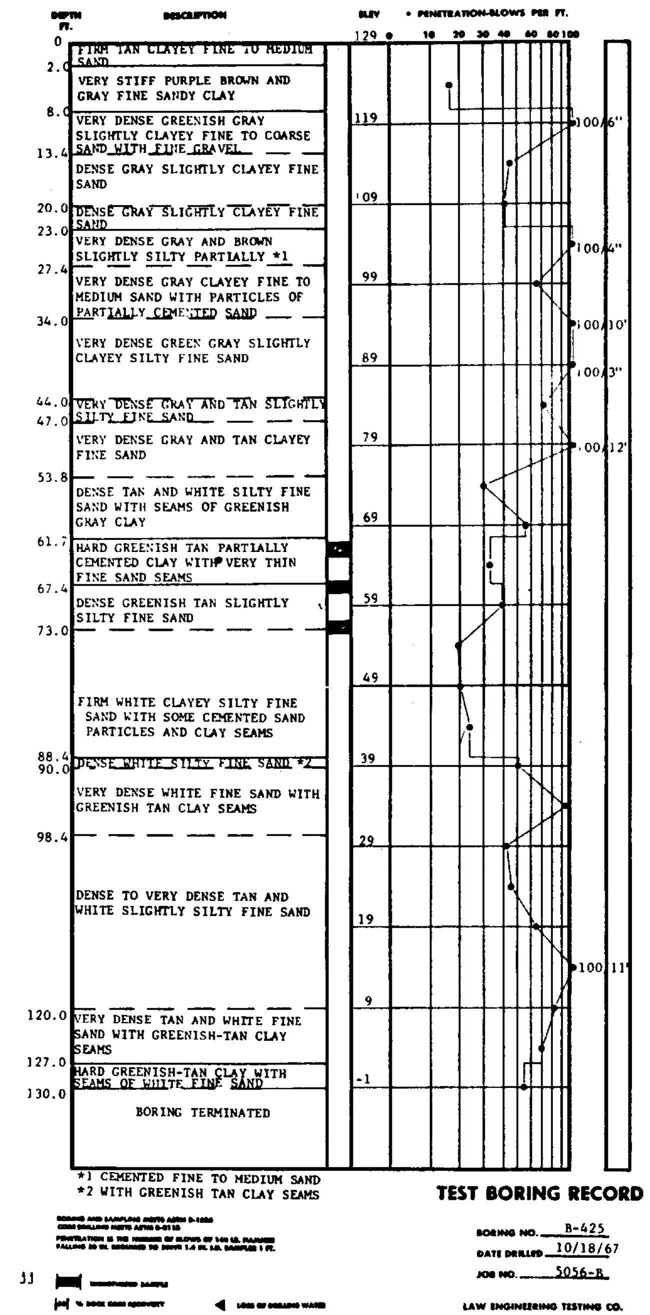
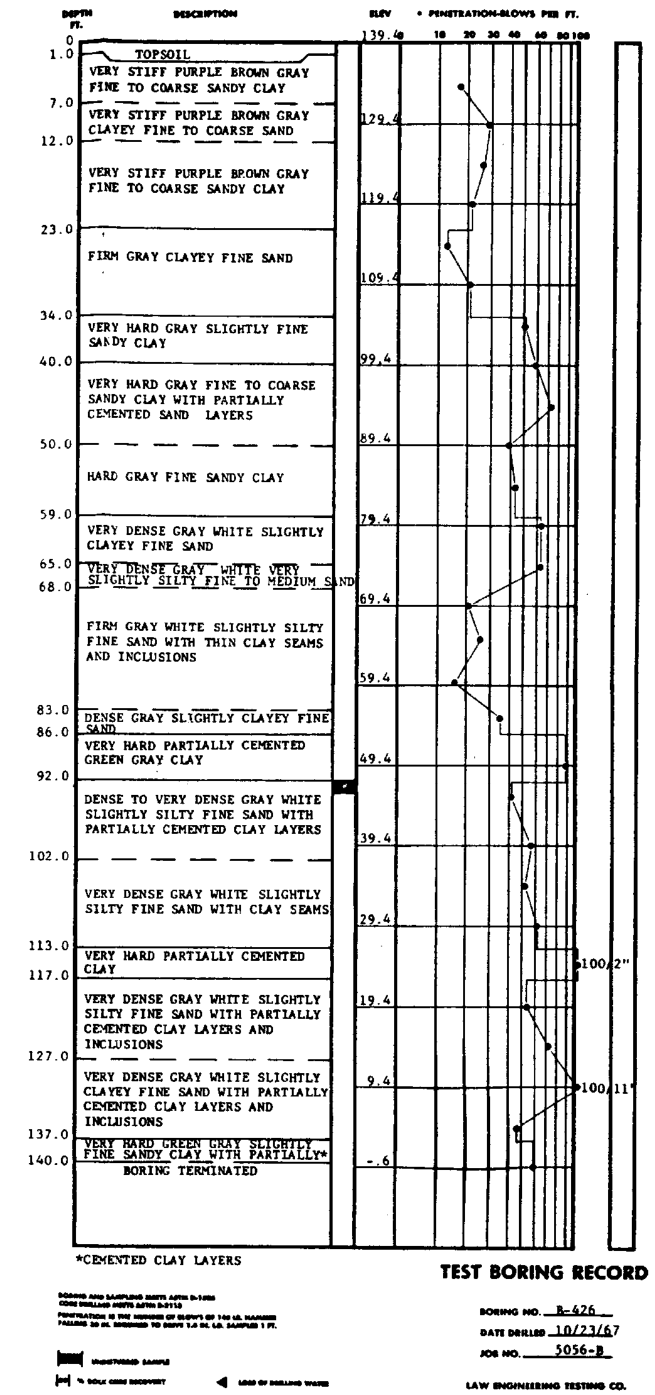
UNIT 2

TEST BORING RECORD
BORING NO. 423

FIGURE 2B-33



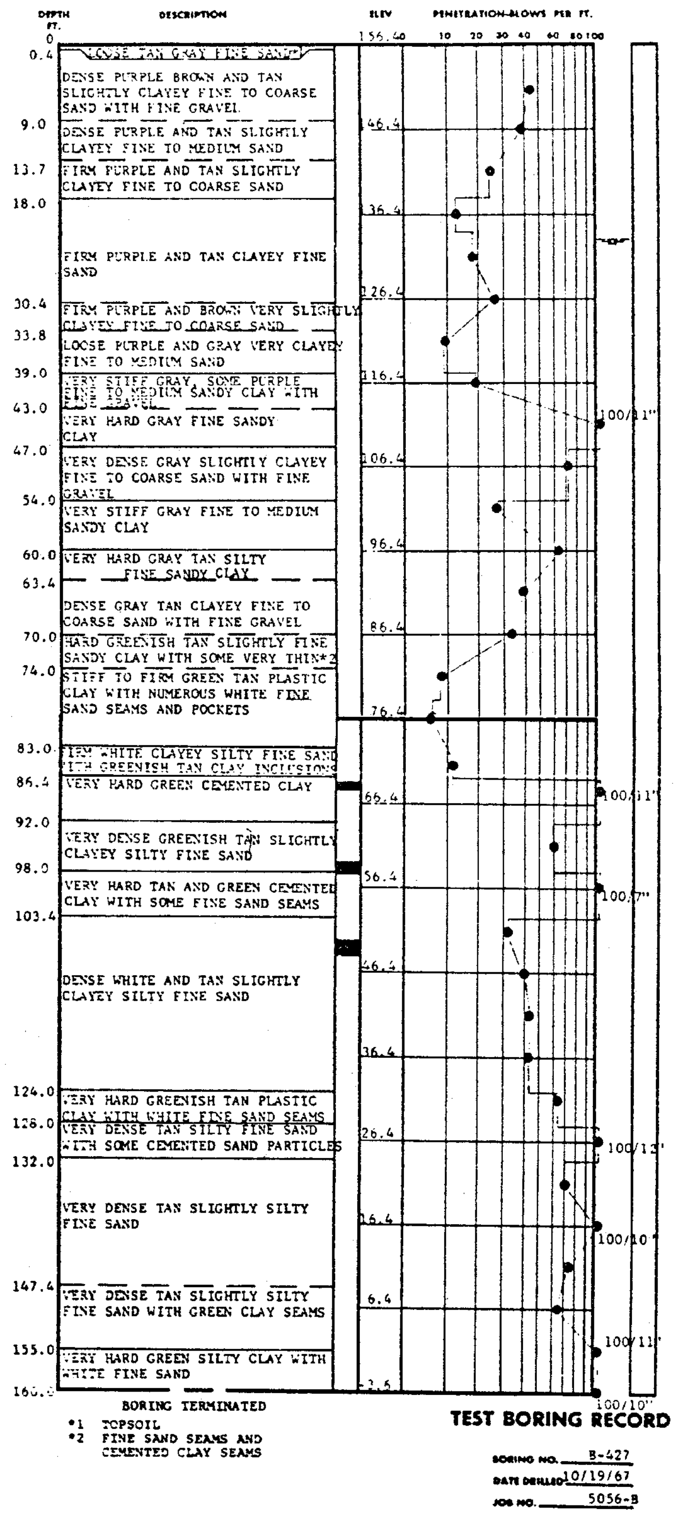




HISTORICAL

REV 19 7/01

SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TEST BORING RECORD BORING NOS. 424, 425, AND 426
		FIGURE 2B-34



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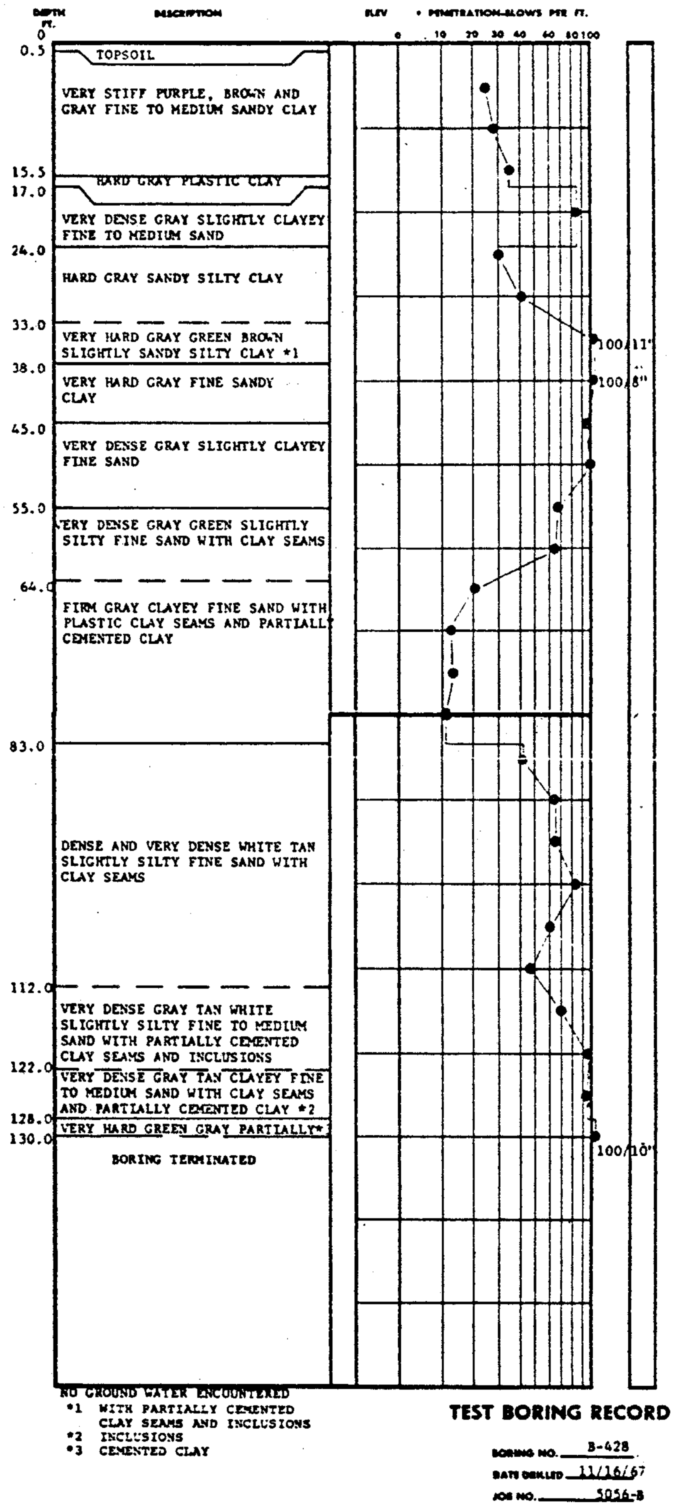
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 427

FIGURE 2B-35



HISTORICAL

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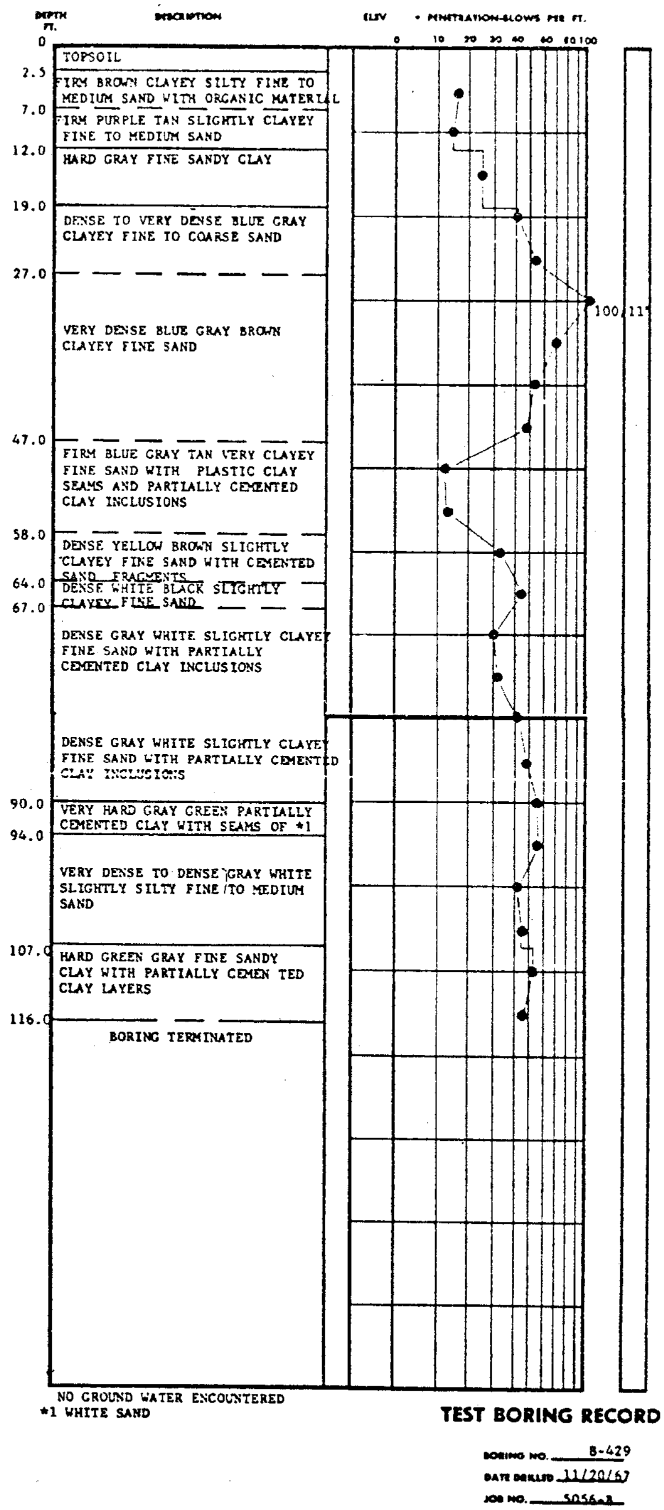
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 428

FIGURE 2B-36



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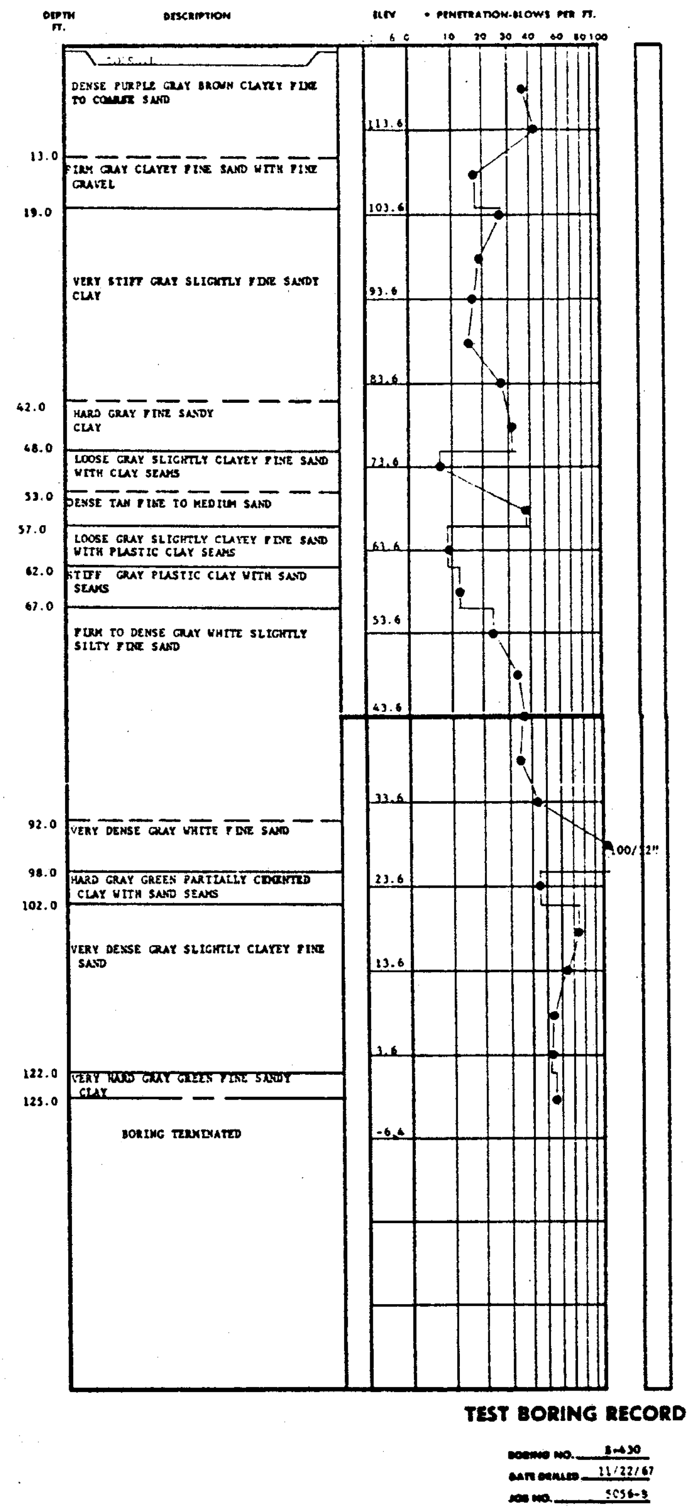
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 429

FIGURE 2B-37



HISTORICAL

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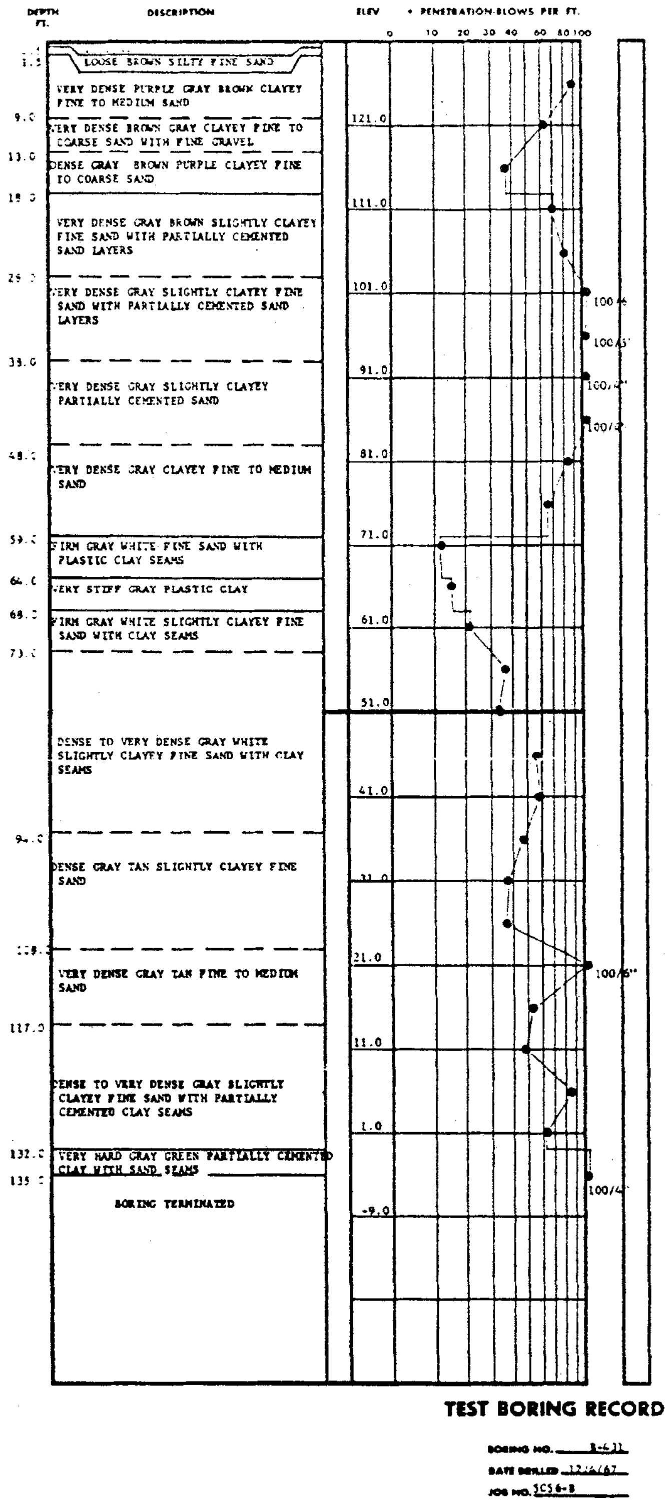
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 430

FIGURE 2B-38



HISTORICAL

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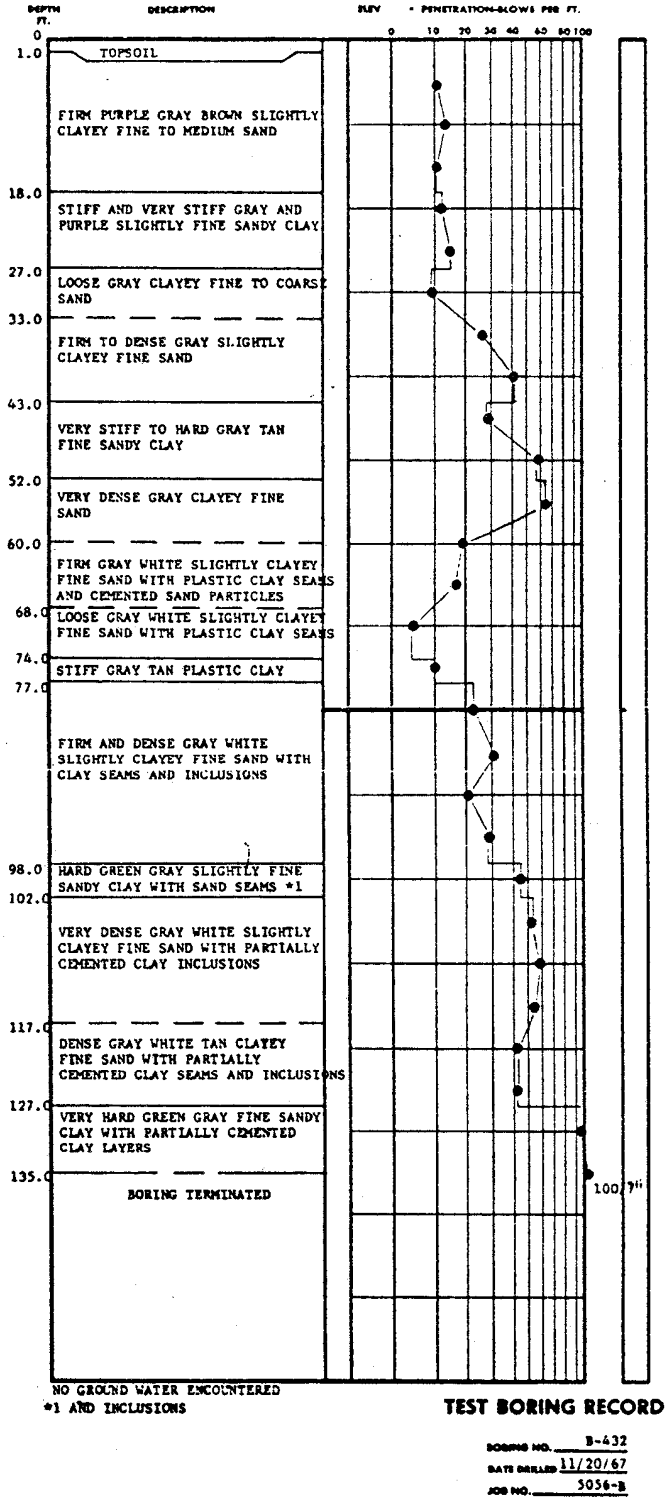
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 431

FIGURE 2B-39



HISTORICAL

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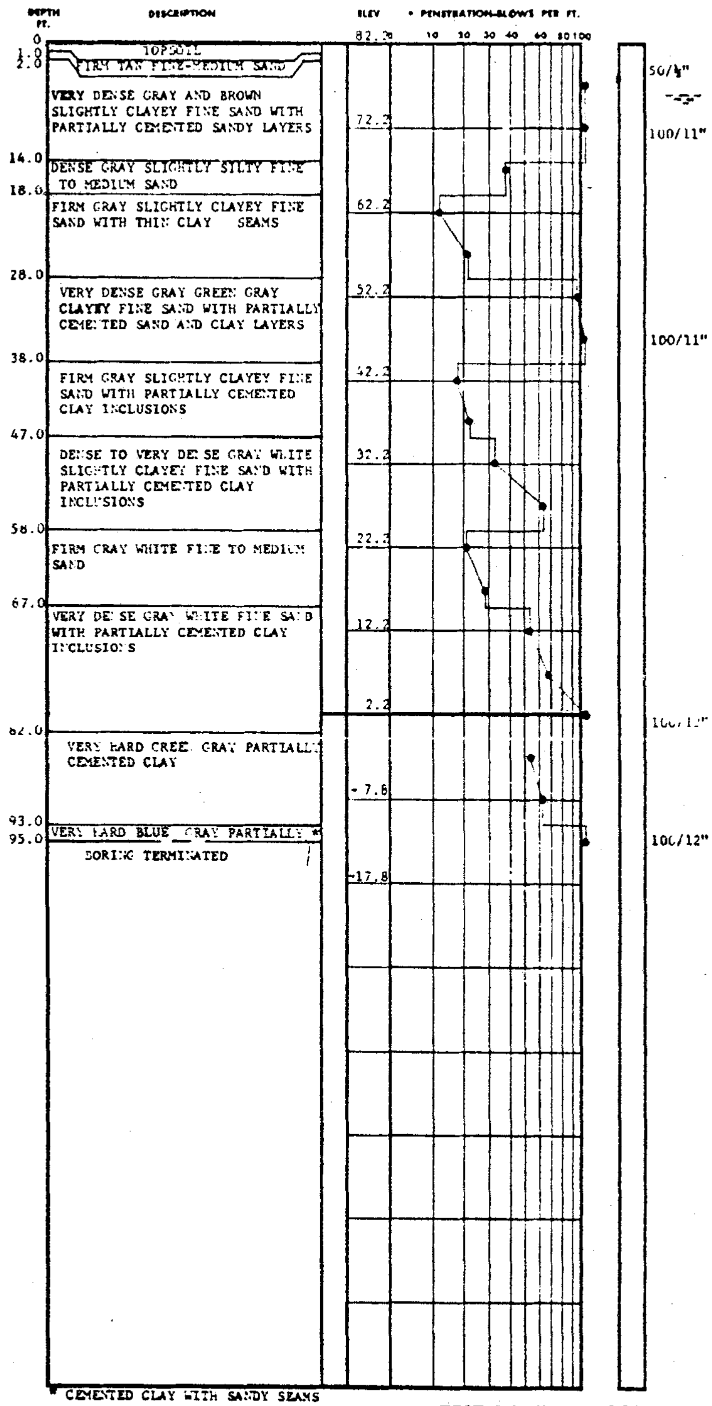
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 432

FIGURE 2B-40



TEST BORING RECORD

BORING NO. B-433
DATE DRILLED 1-15-68
JOB NO. 5056-B

HISTORICAL

REV 19 7/01

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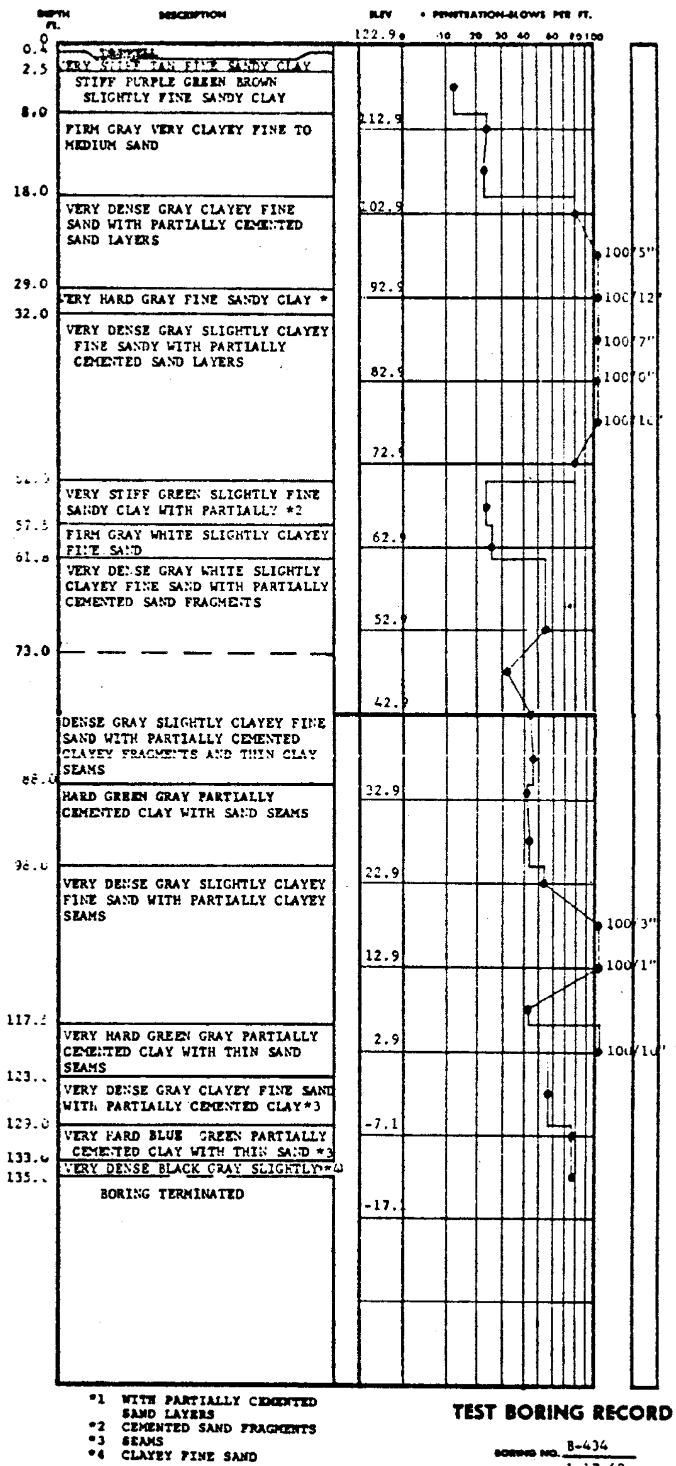
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 433

FIGURE 2B-41



TEST BORING RECORD

BORING NO. B-434
DATE DRILLED 1-17-68
JOB NO. 5056-B

HISTORICAL

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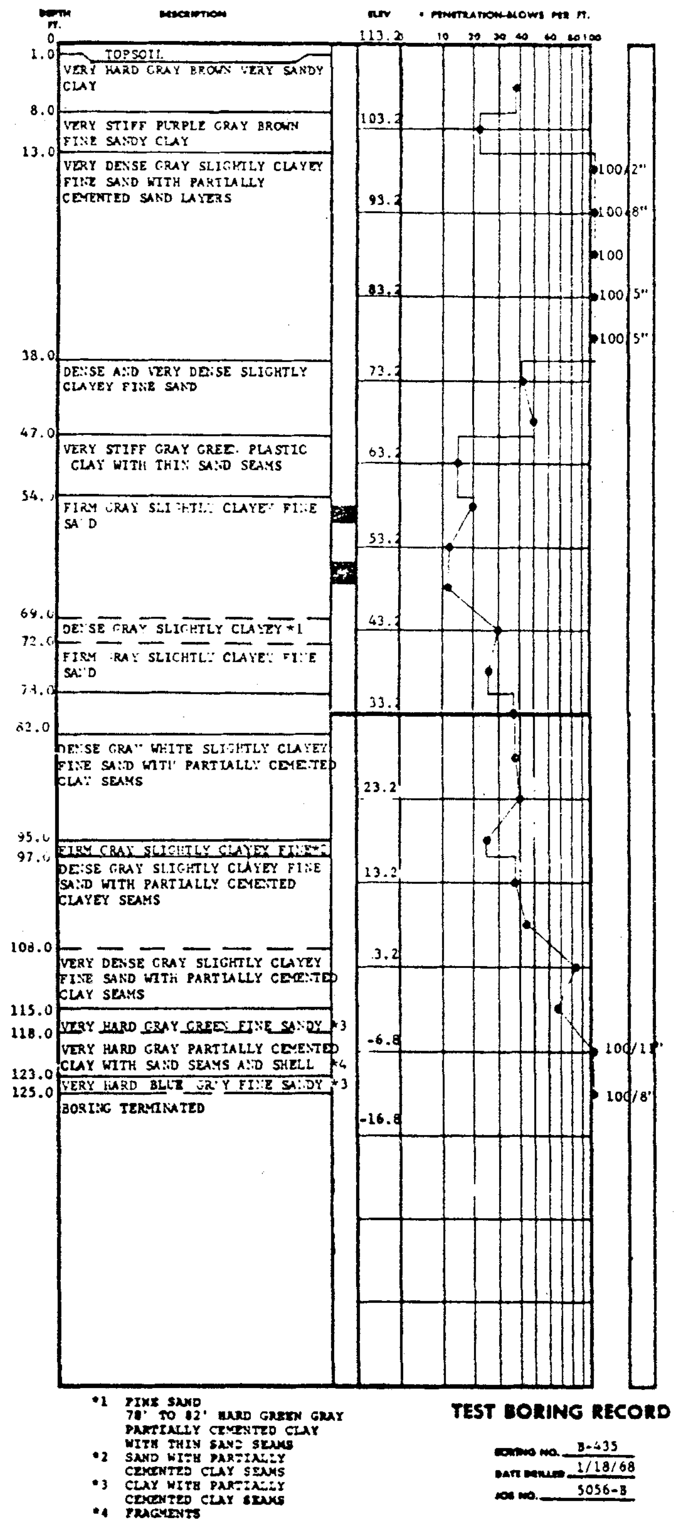
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 434

FIGURE 2B-42



HISTORICAL

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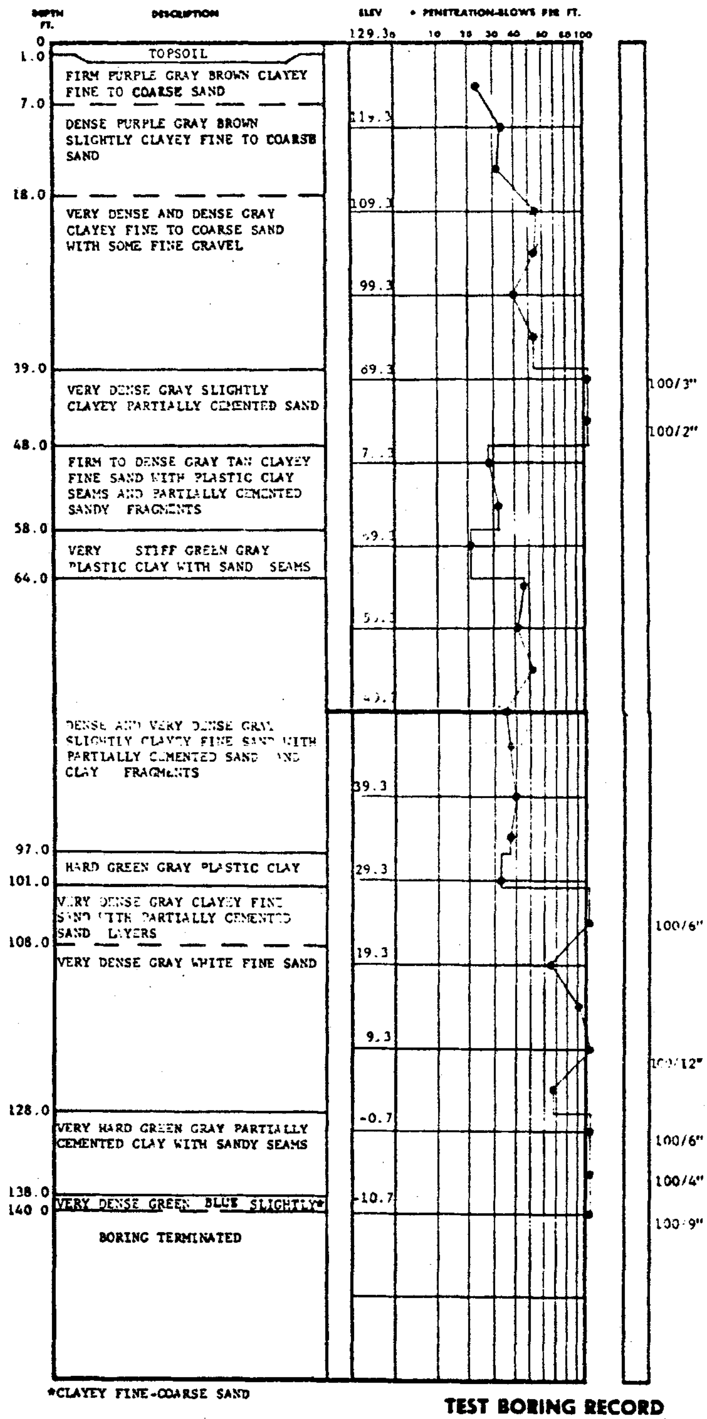
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 435

FIGURE 2B-43



TEST BORING RECORD

BORING NO. B-436

DATE DRILLED 1-21-68

JOB NO. 5056-B

HISTORICAL

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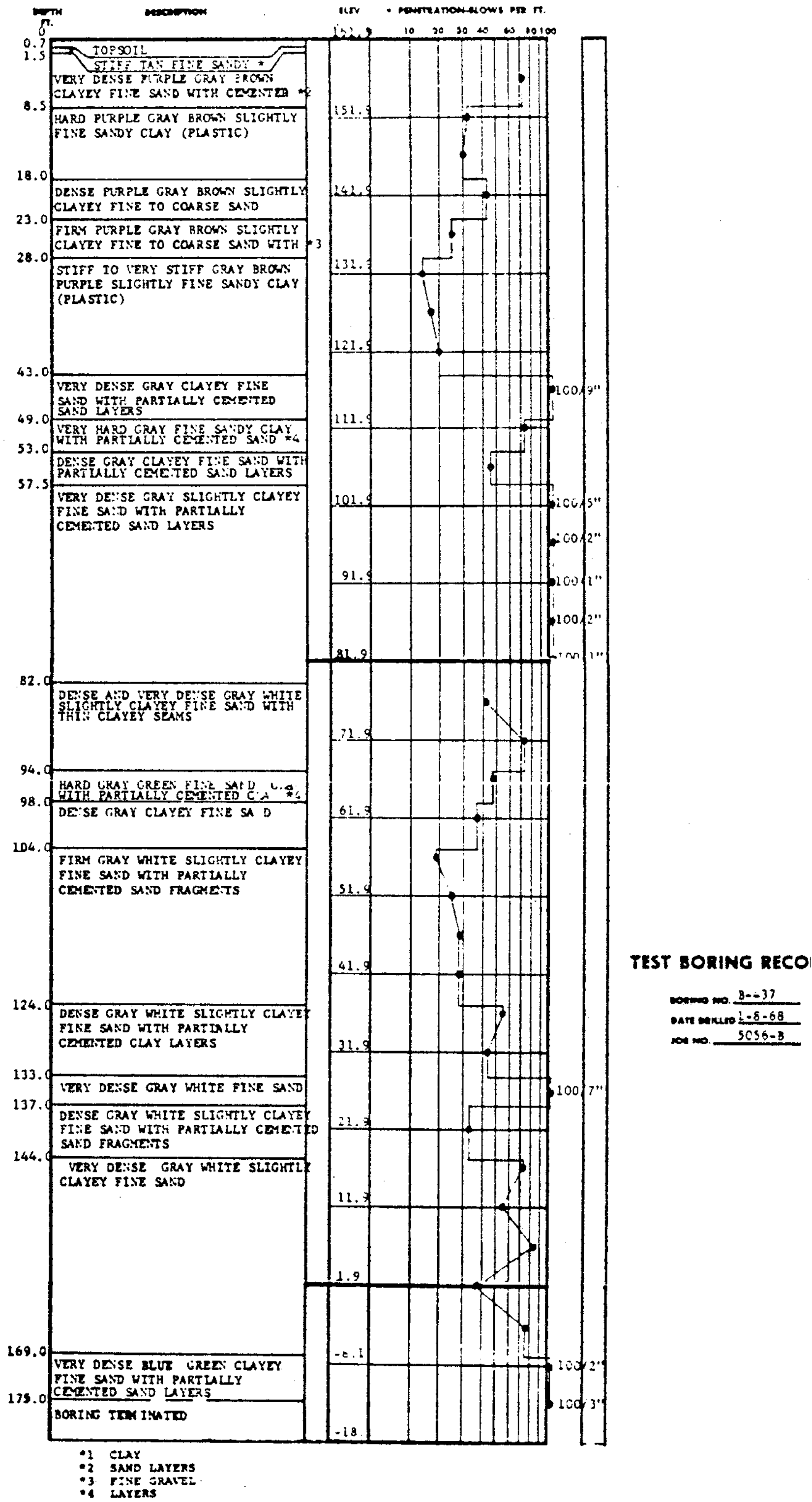
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 436

FIGURE 2B-44



HISTORICAL

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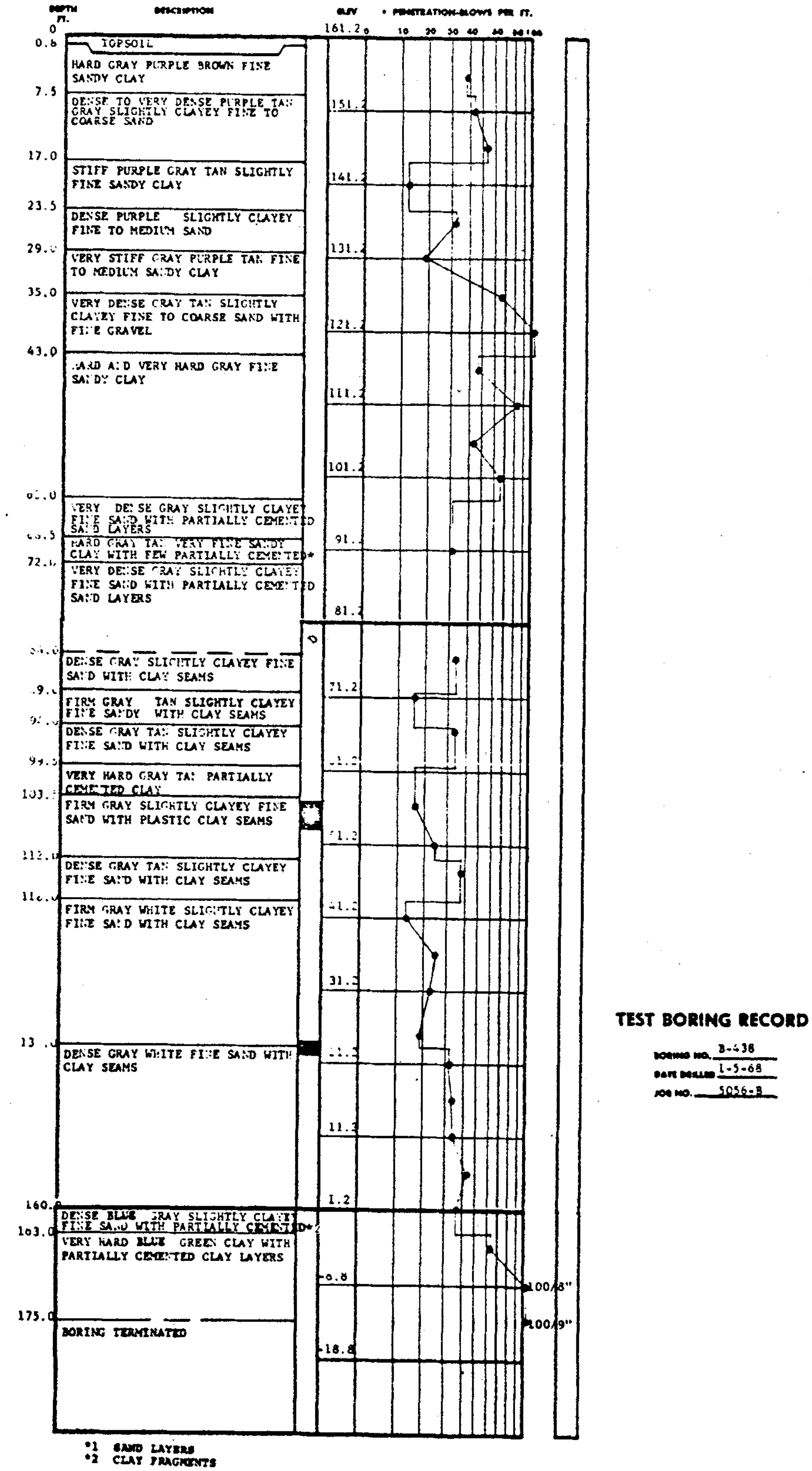
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 437

FIGURE 2B-45



HISTORICAL

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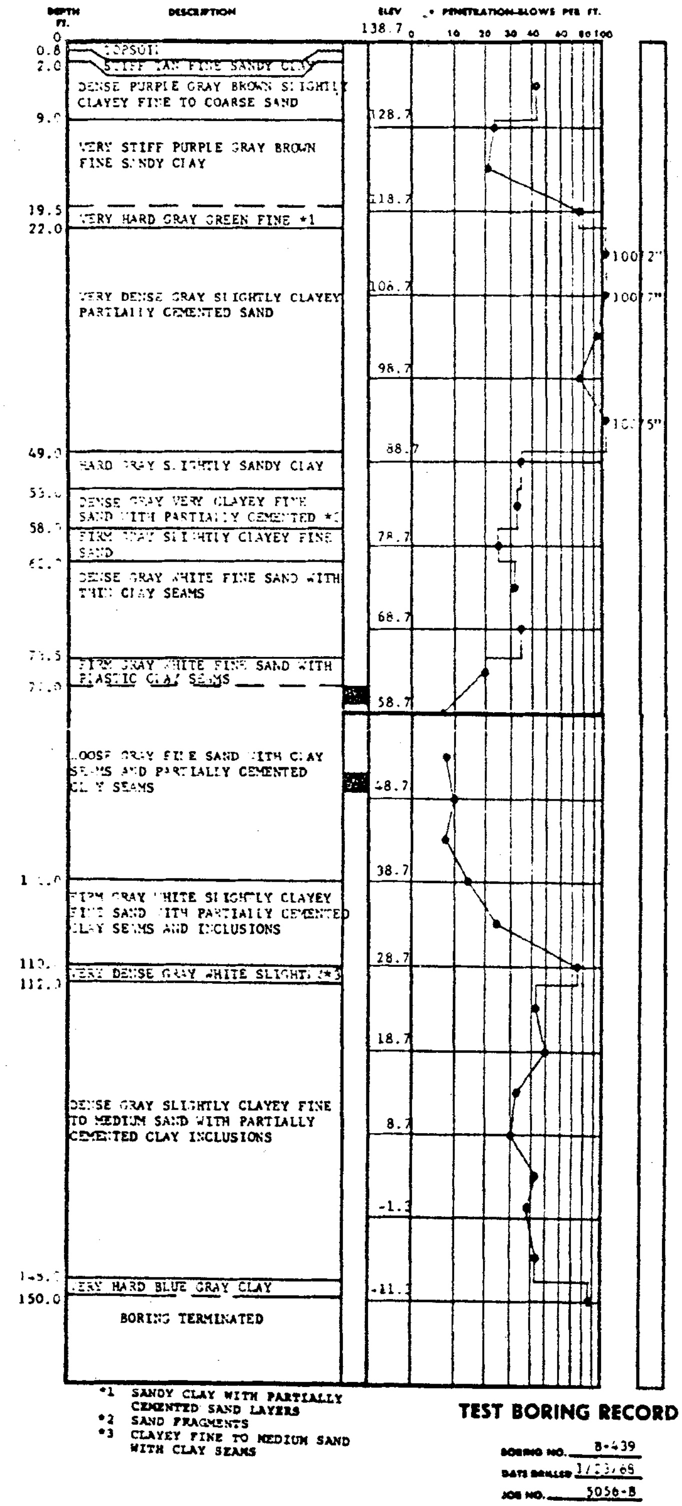
SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 438

FIGURE 2B-46



HISTORICAL

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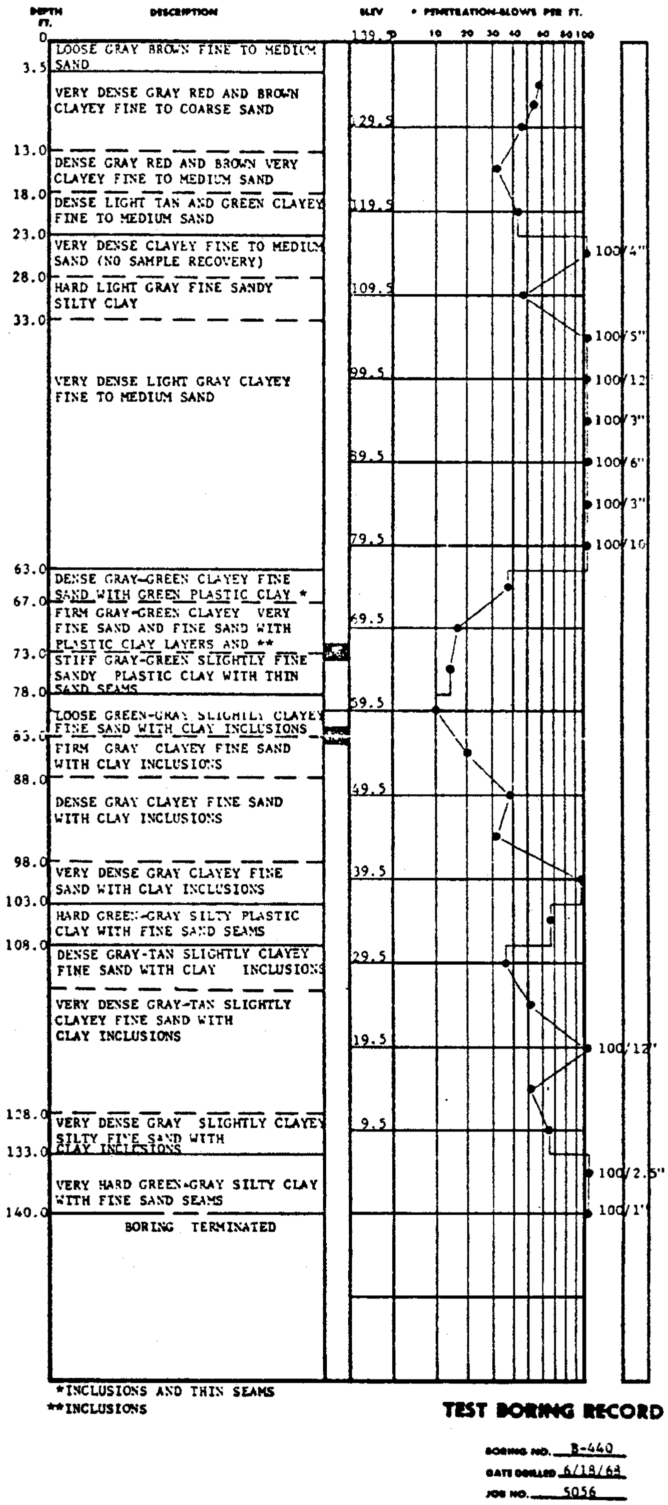
SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 439

FIGURE 2B-47



HISTORICAL

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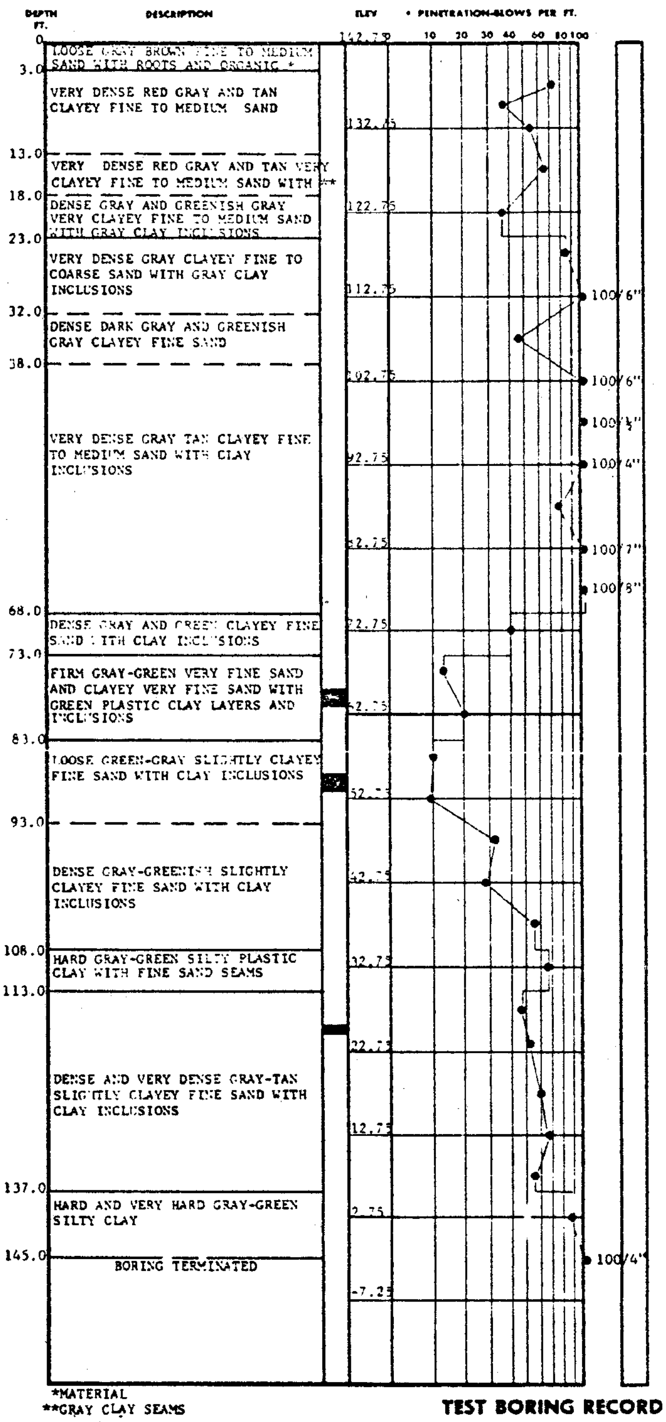
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 440

FIGURE 2B-48



TEST BORING RECORD

BORING NO. B-441
DATE DRILLED 2-19&25/68
JOB NO. 5036

HISTORICAL

REV 19 7/01

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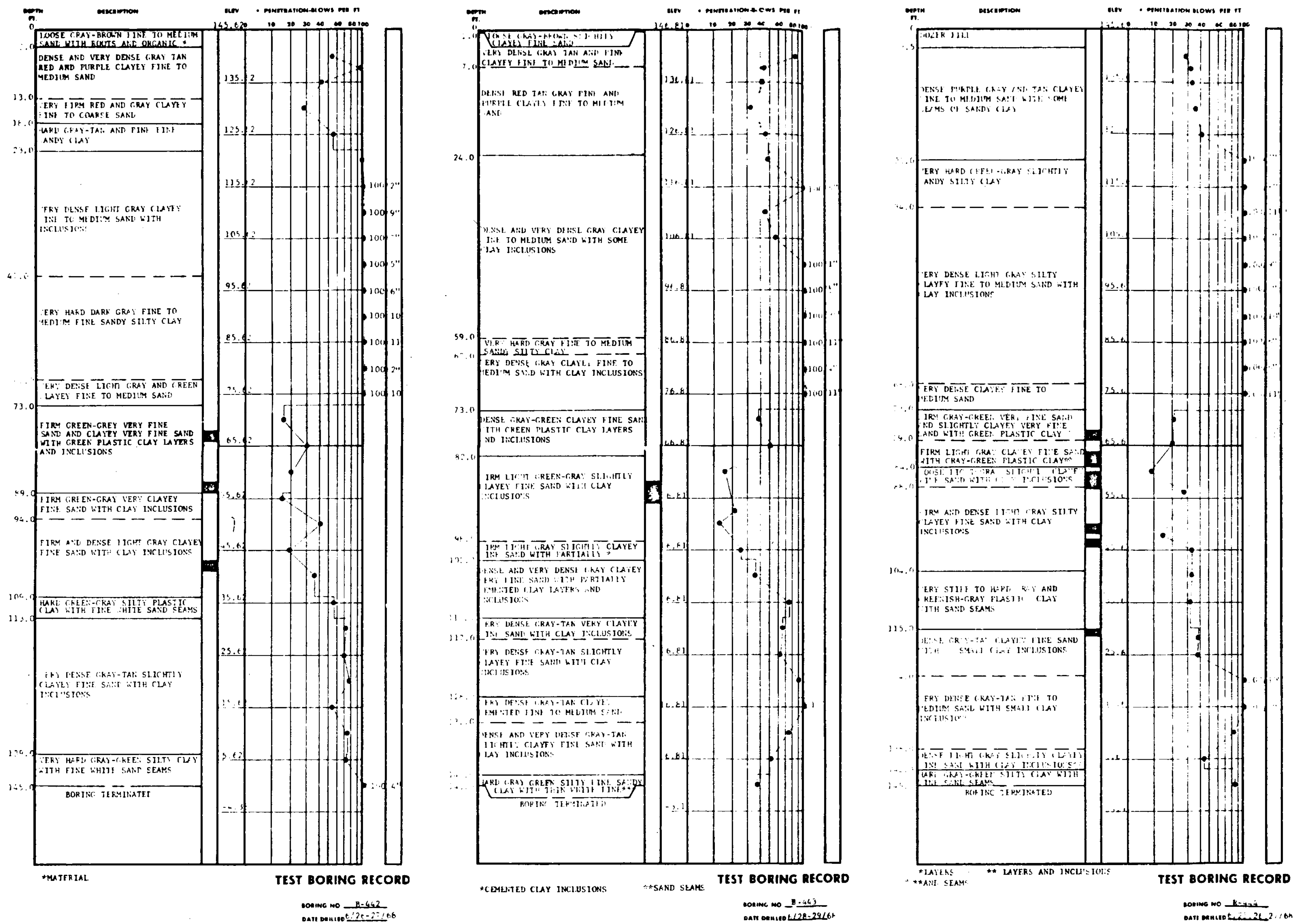
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 441

FIGURE 2B-49



HISTORICAL

REV 19 7/01

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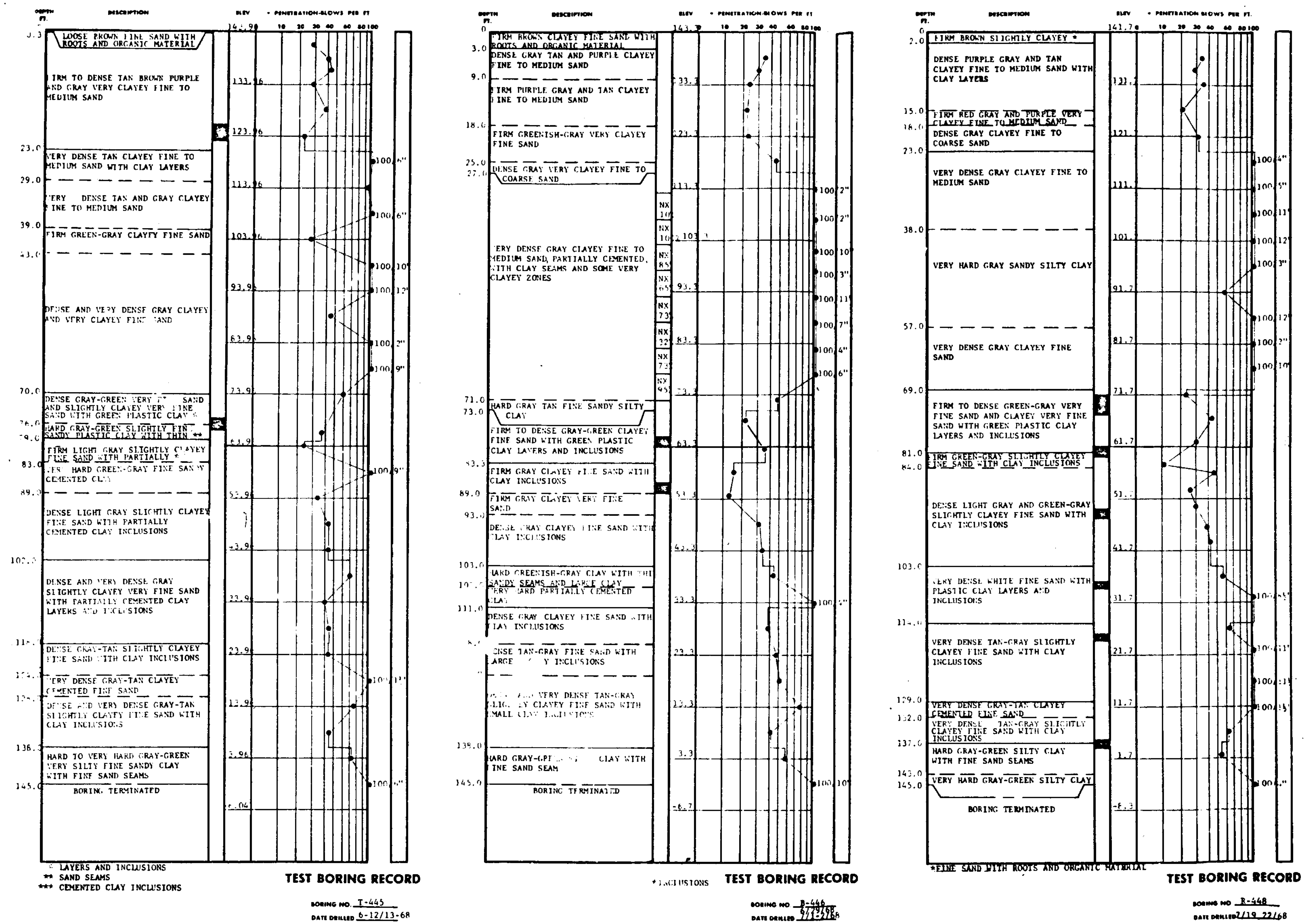
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 442, 443, AND 444

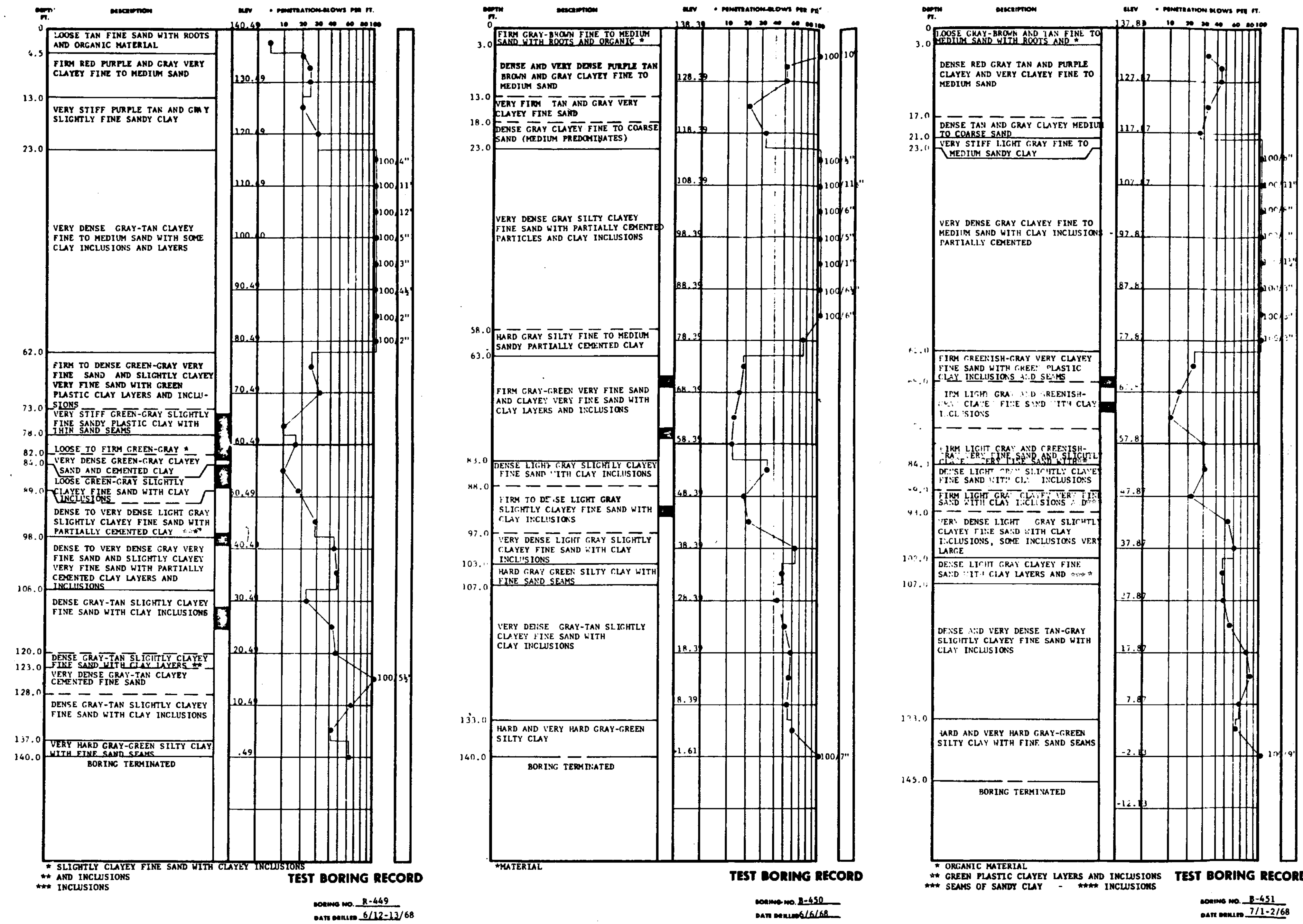
FIGURE 2B-50



HISTORICAL

REV 19 7/01

SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TEST BORING RECORD BORING NOS. 445, 446, AND 448
		FIGURE 2B-51



HISTORICAL

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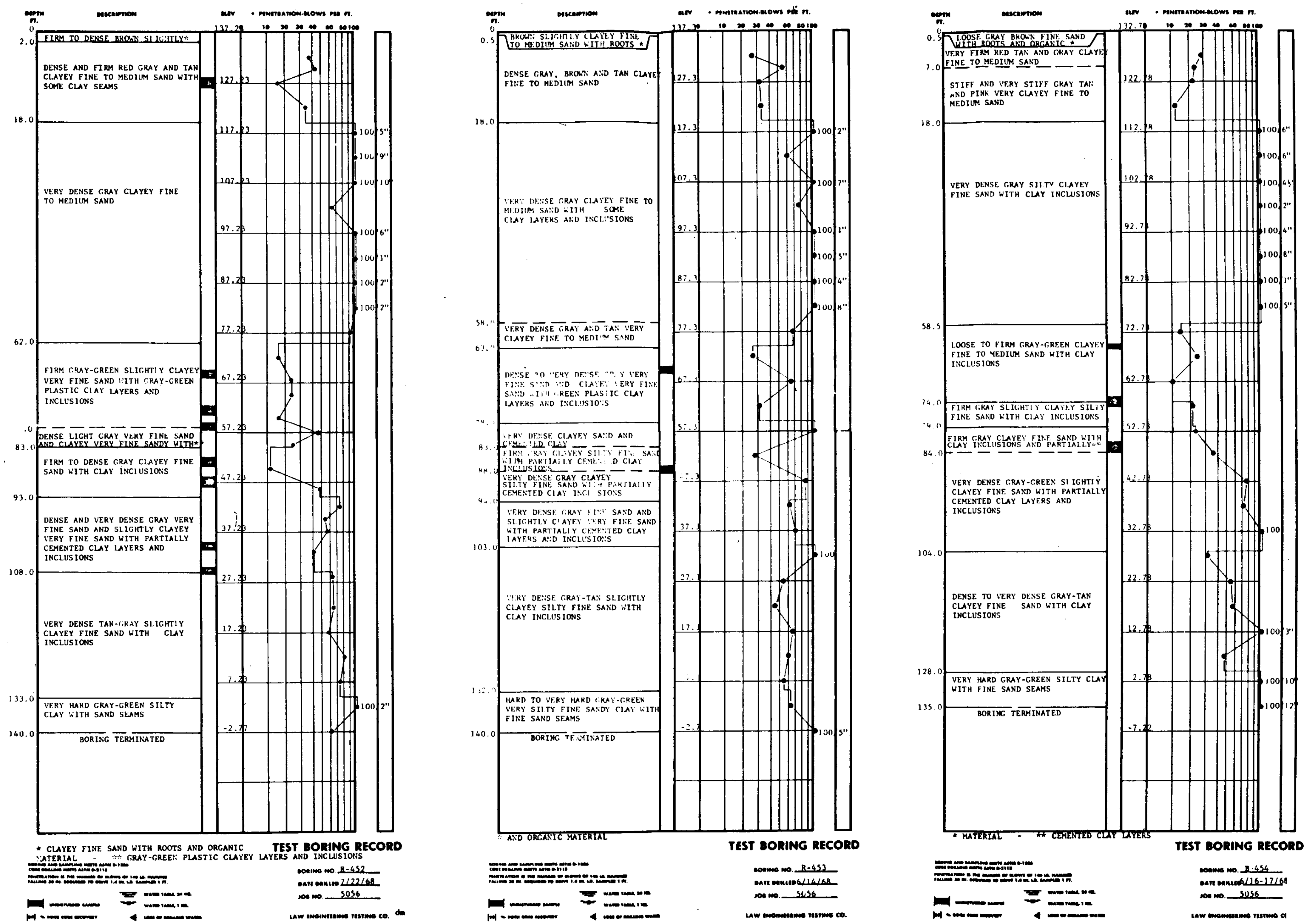
		FIGURE 2B-52



HISTORICAL

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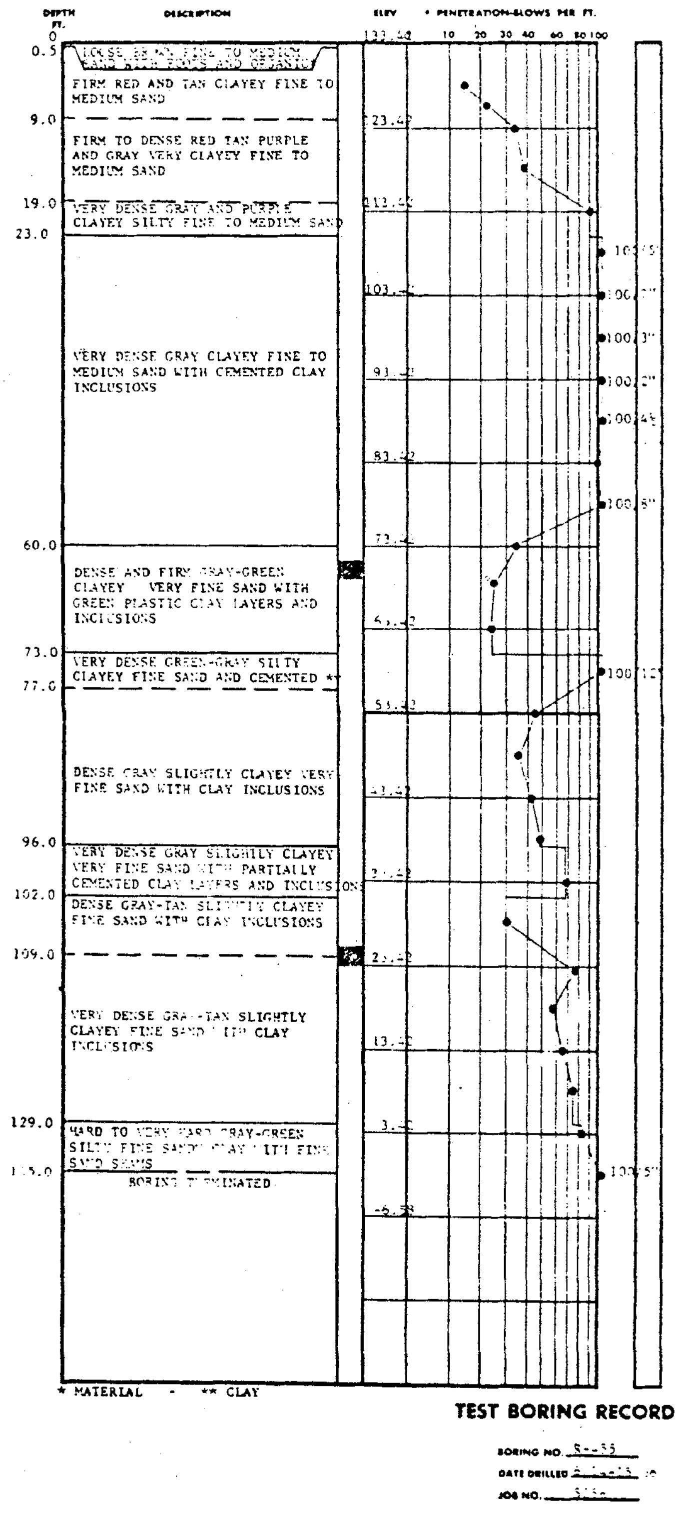
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 452, 453, AND 454

FIGURE 2B-53



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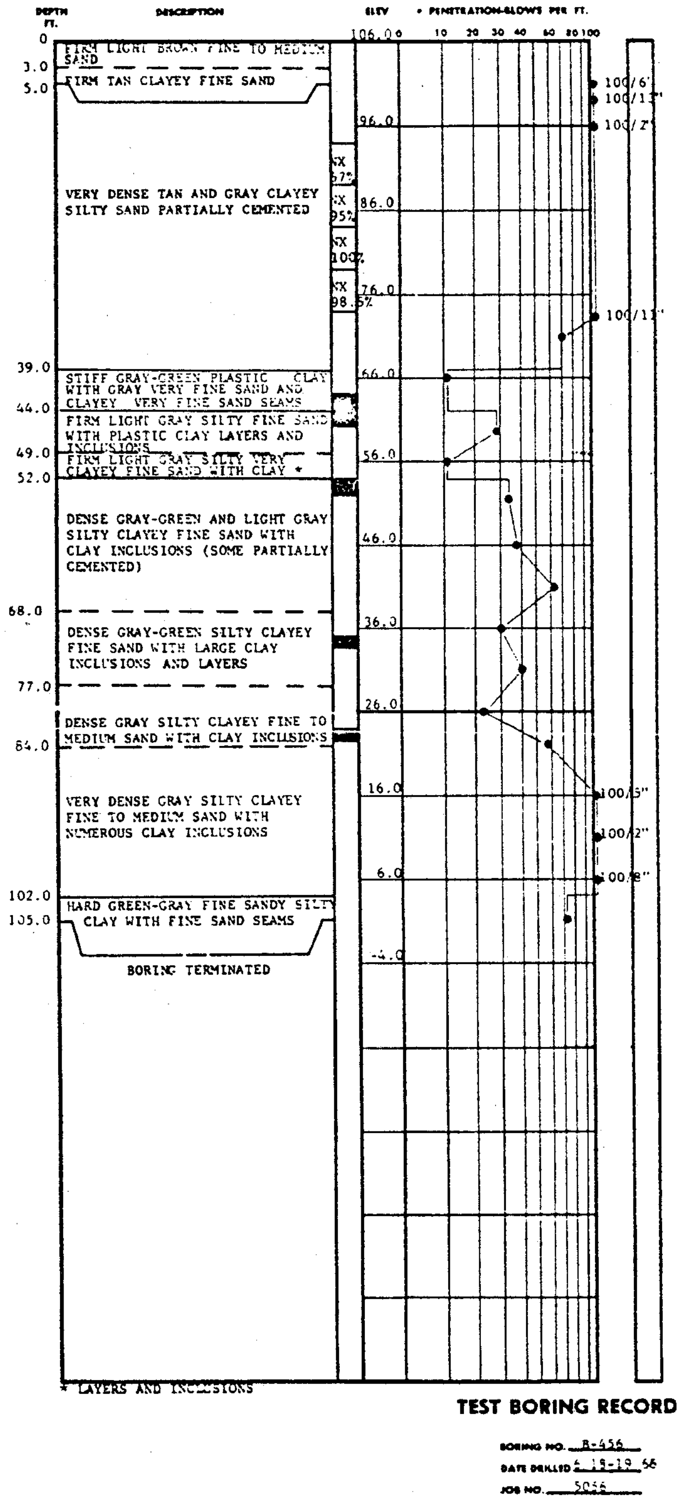
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 455

FIGURE 2B-54



HISTORICAL

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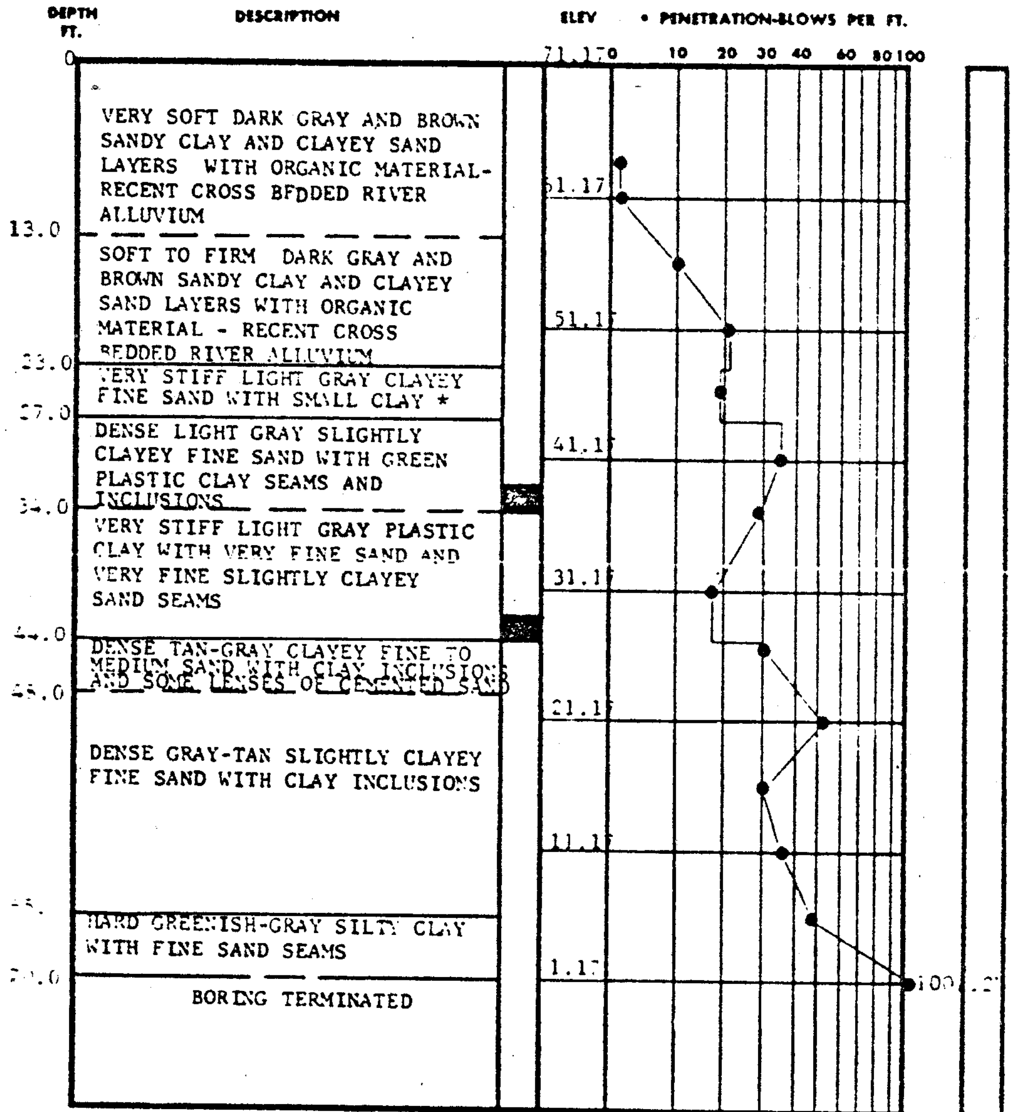
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 456

FIGURE 2B-55



*INCLUSIONS

TEST BORING RECORD

BORING NO. B-457

DATE DRILLED [illegible]

JOB NO. 5056

ACAD

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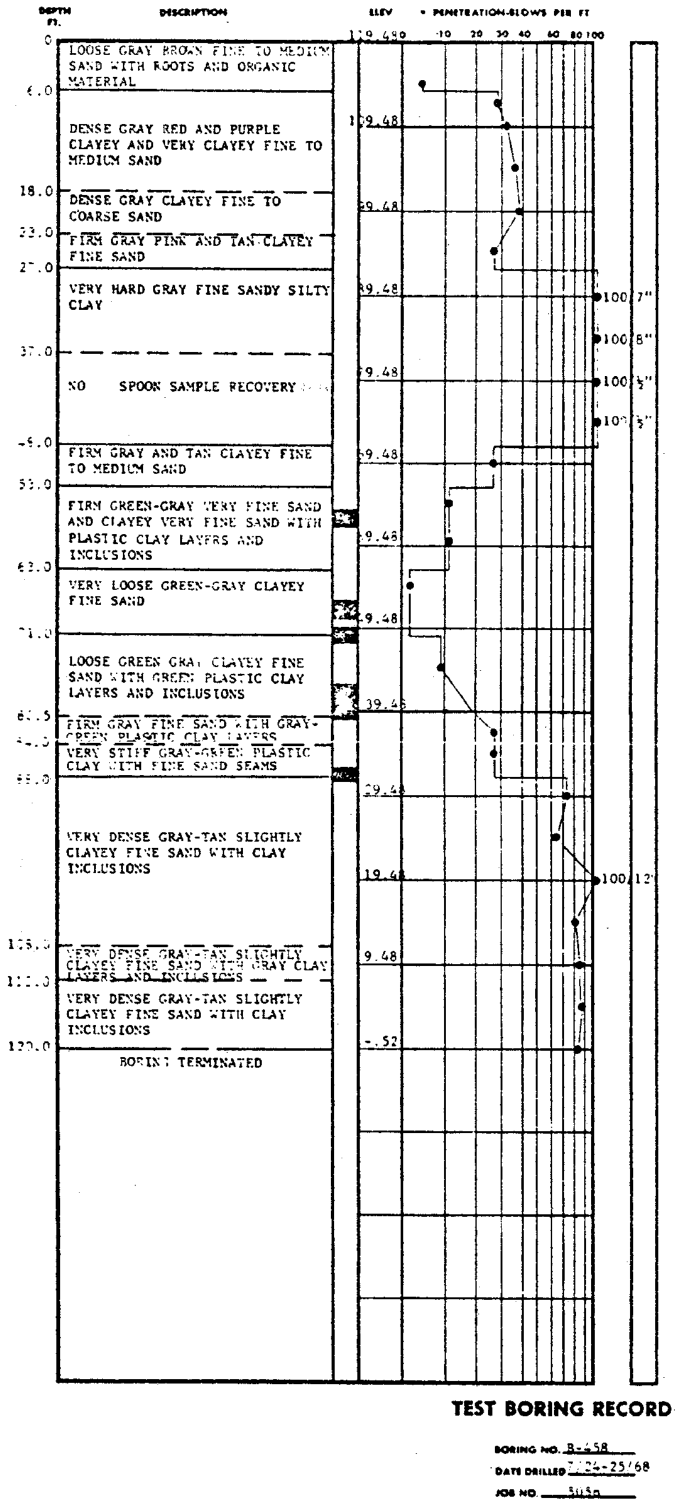
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 457

FIGURE 2B-56



HISTORICAL

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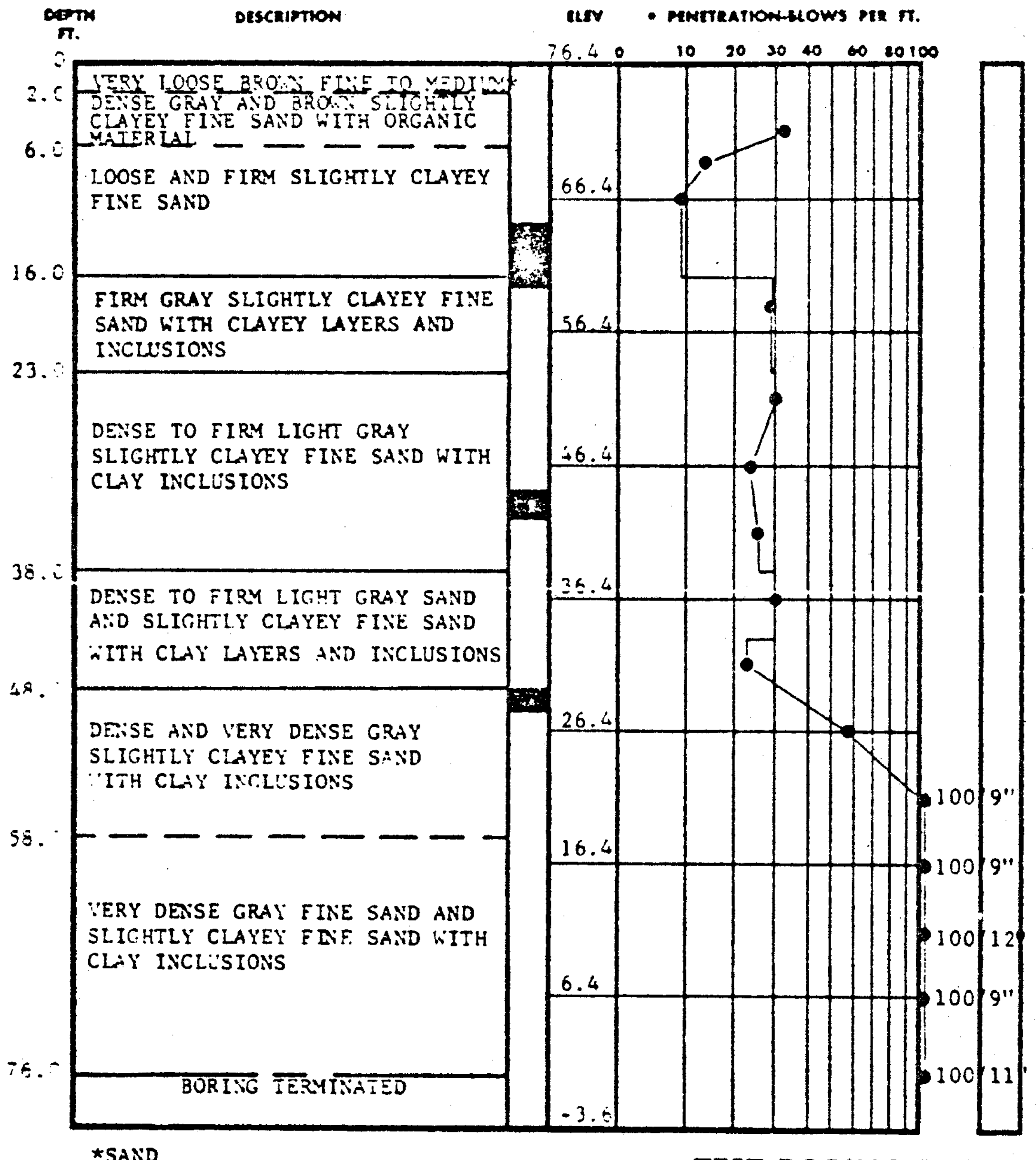
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 458

FIGURE 2B-57



TEST BORING RECORD

BORING NO. B-459

DATE DRILLED 2.26.68

JOB NO. 5056

HISTORICAL

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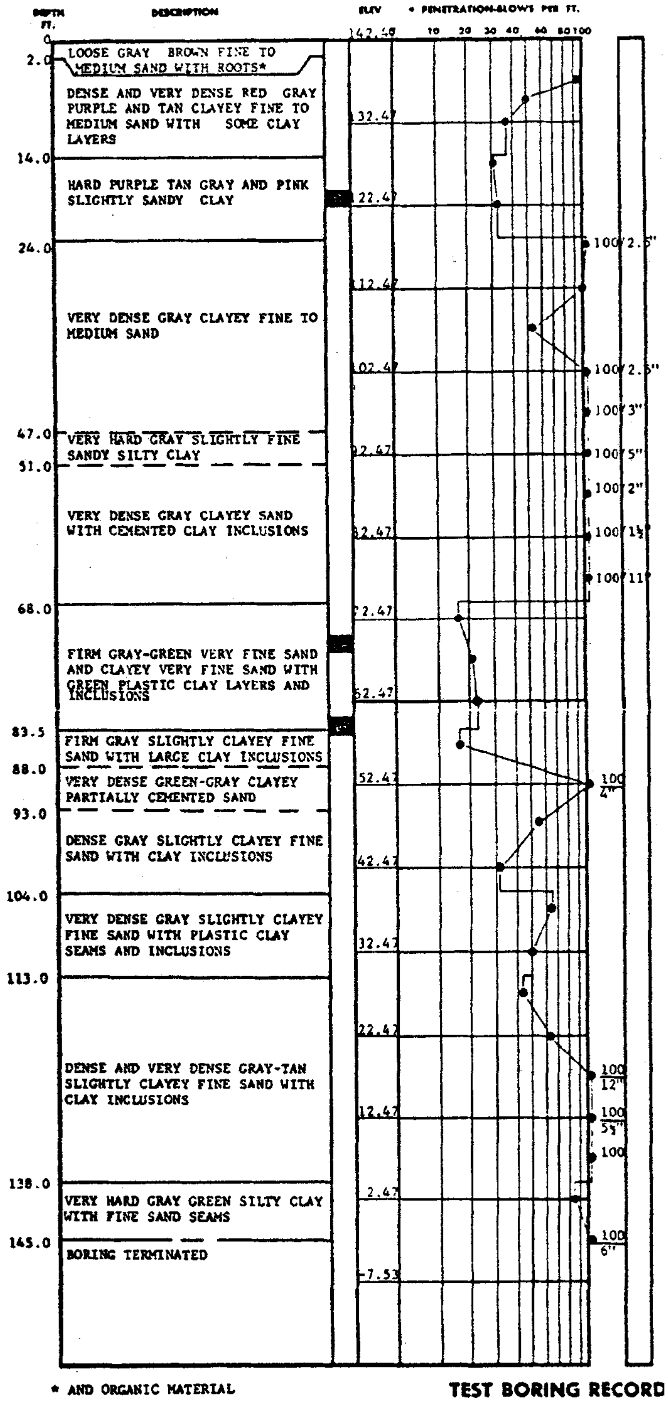
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 459

FIGURE 2B-58



* AND ORGANIC MATERIAL

TEST BORING RECORD

BORING NO. T-460
DATE DRILLED 7-16/17-68
JOB NO. 5056

HISTORICAL

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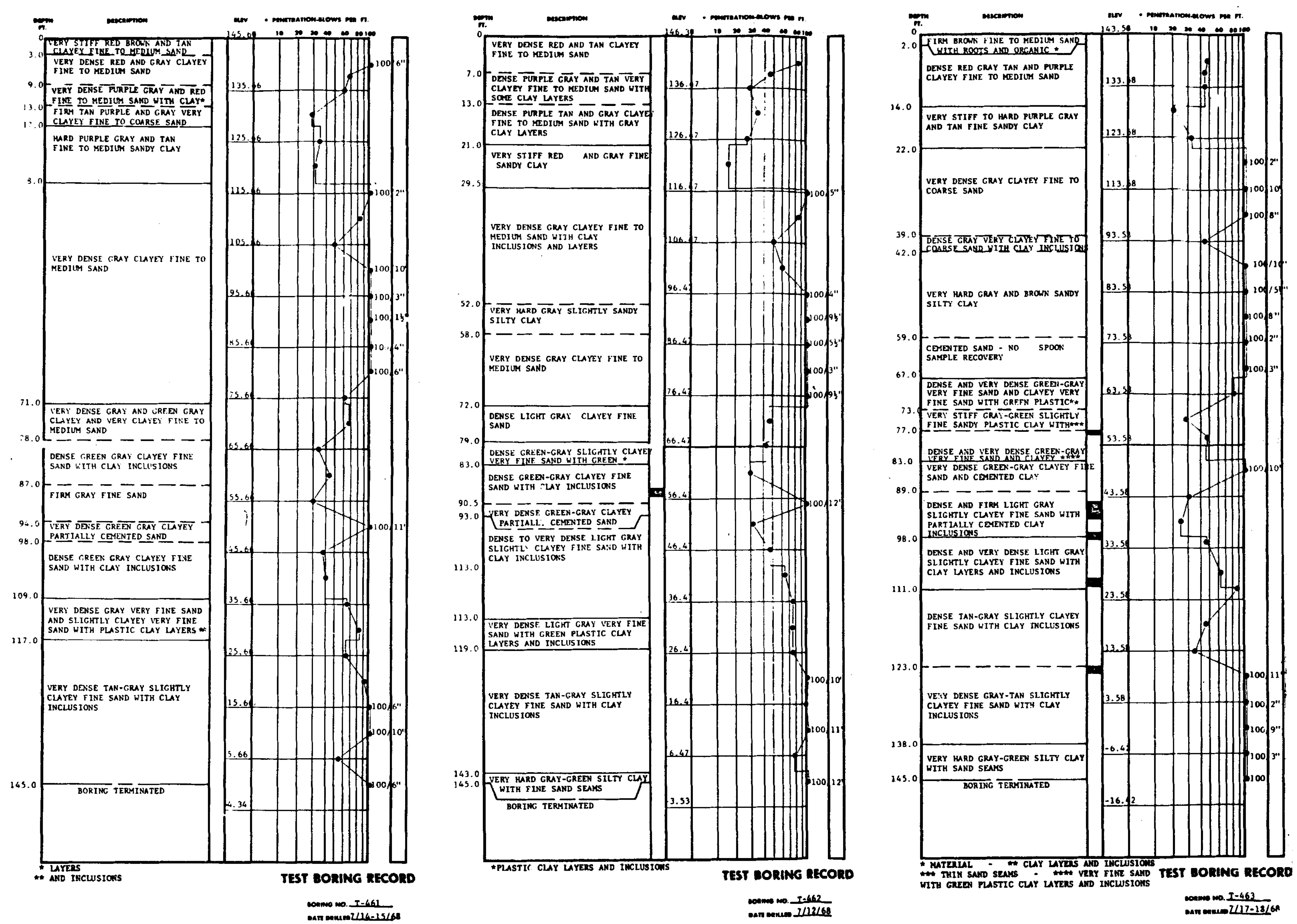
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 460

FIGURE 2B-59



HISTORICAL

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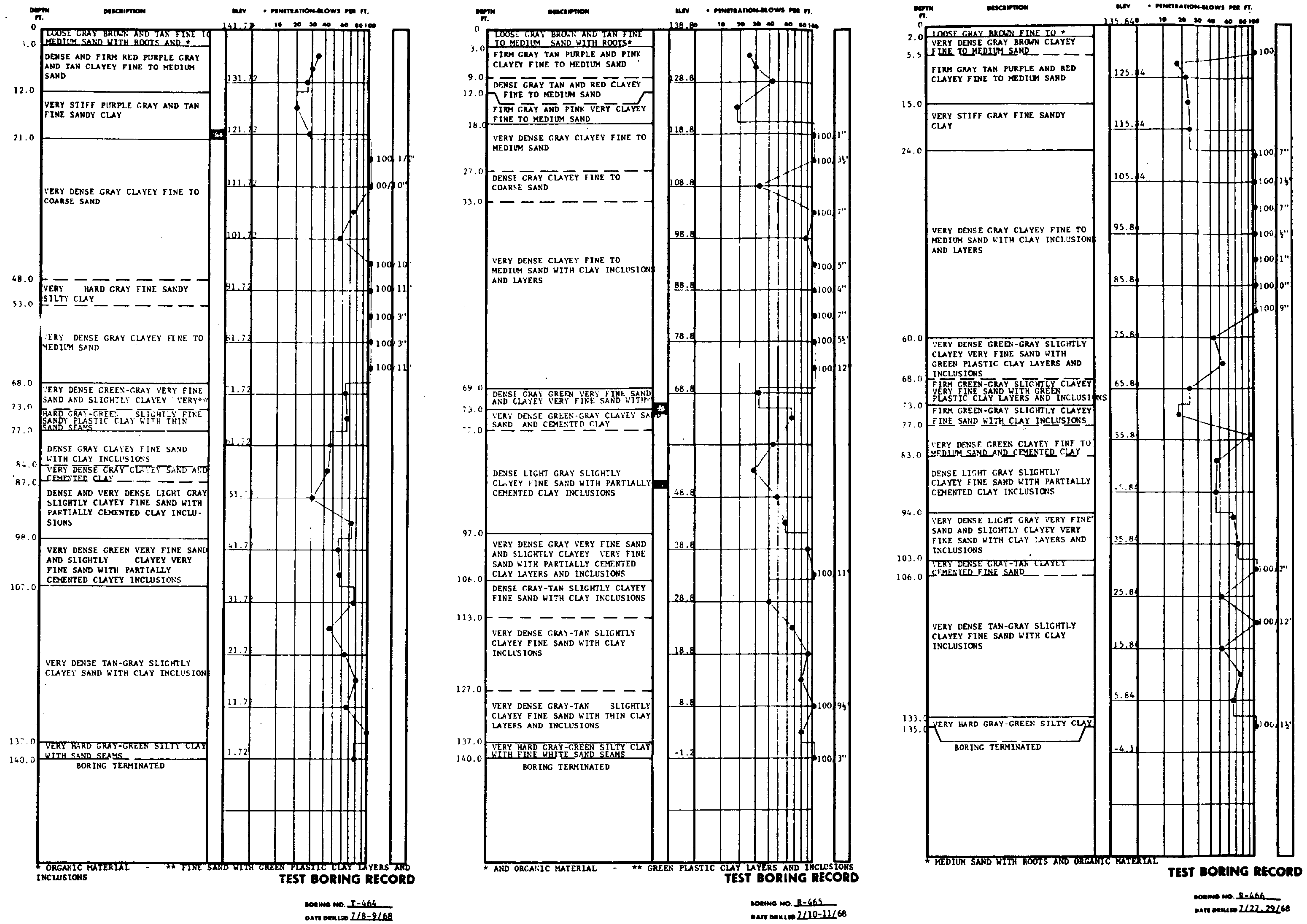
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TEST BORING RECORD BORING NOS. 461, 462, AND 463
		FIGURE 2B-60



HISTORICAL

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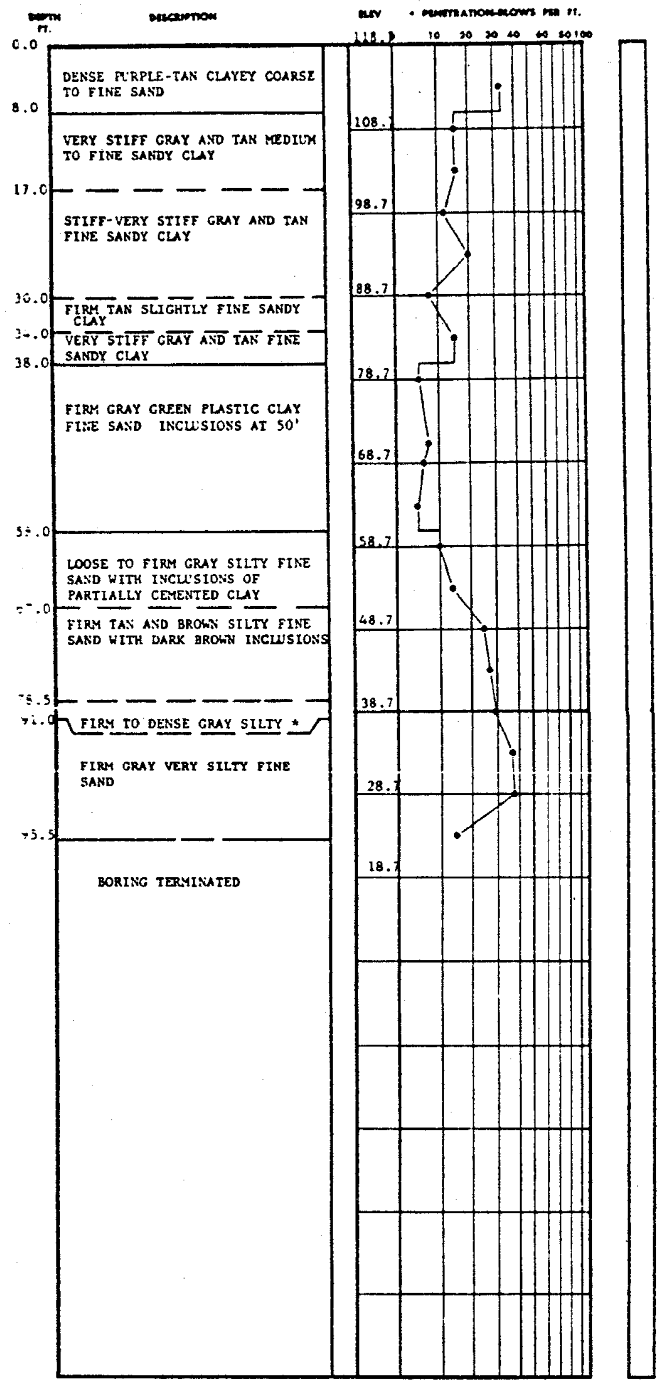
SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 464, 465, AND 466

FIGURE 2B-61



*FINE SAND, CLAY INCLUSIONS AT 80'

TEST BORING RECORD

BORING NO. B-475
DATE DRILLED 2-6-69
JOB NO. 5056

HISTORICAL

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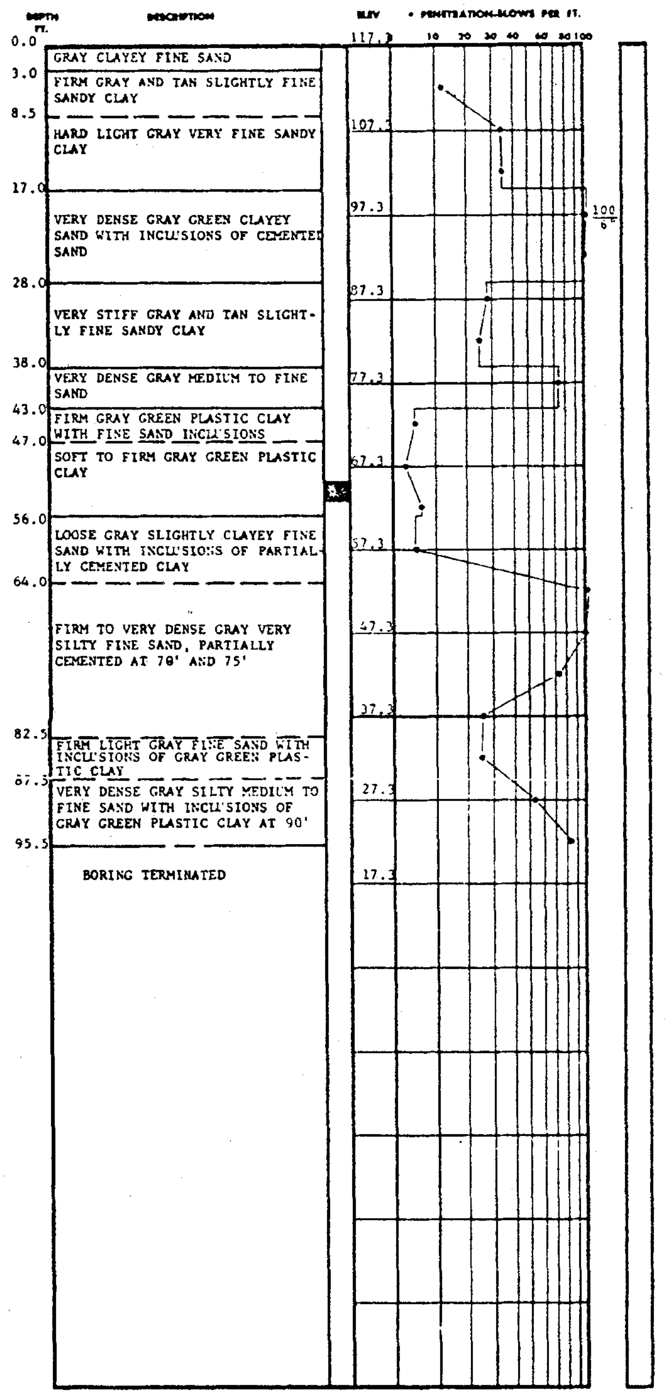
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 475

FIGURE 2B-62



TEST BORING RECORD

BORING NO. B-477

DATE DRILLED

JOB NO. 505-

HISTORICAL

REV 19 7/01

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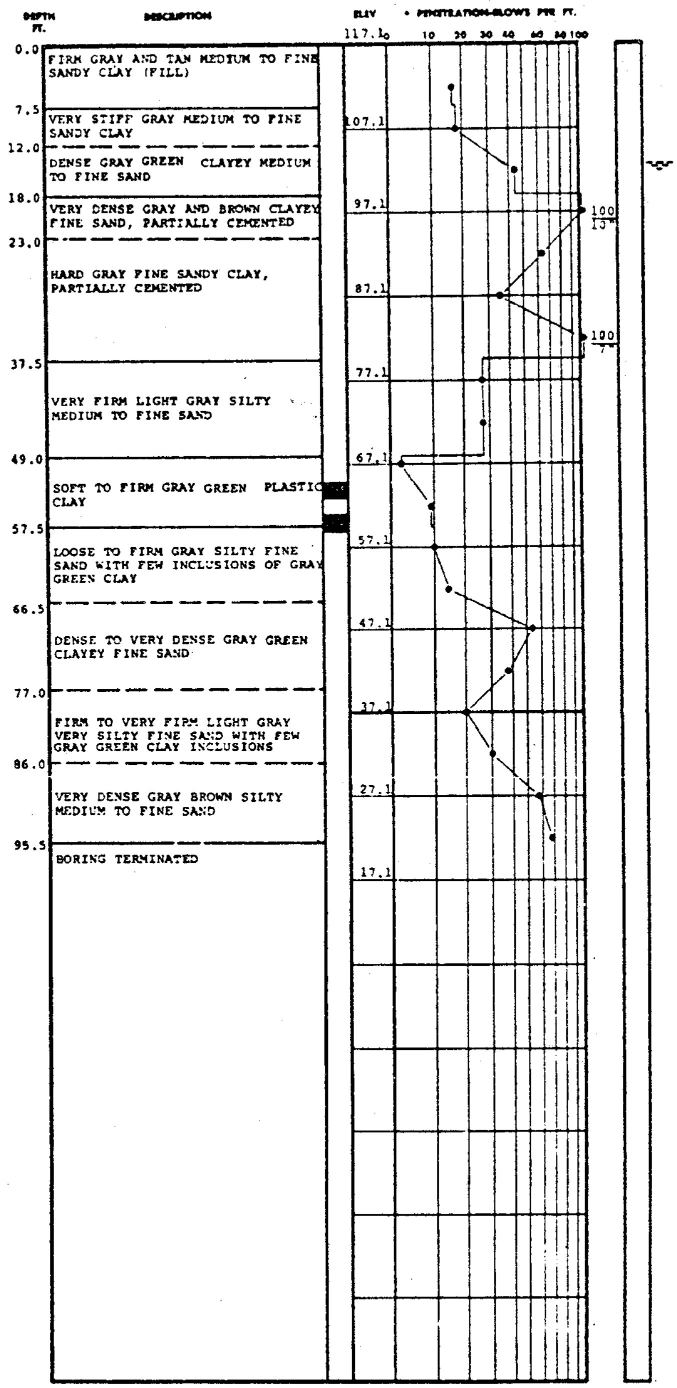
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 477

FIGURE 2B-63



TEST BORING RECORD

BORING NO. B-478
DATE DRILLED 2-11-69
JOB NO. 5256

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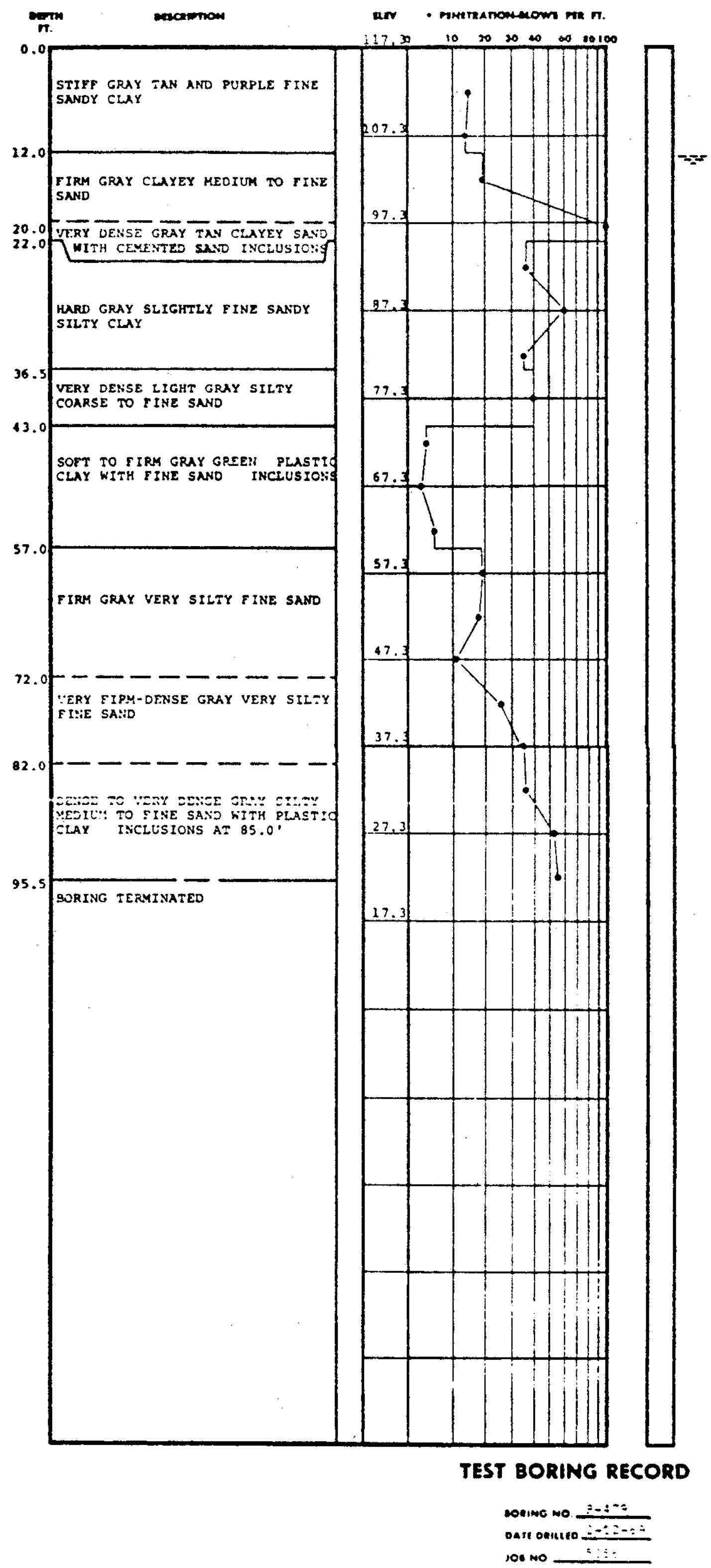
TEST BORING RECORD
BORING NO. 478

FIGURE 2B-64



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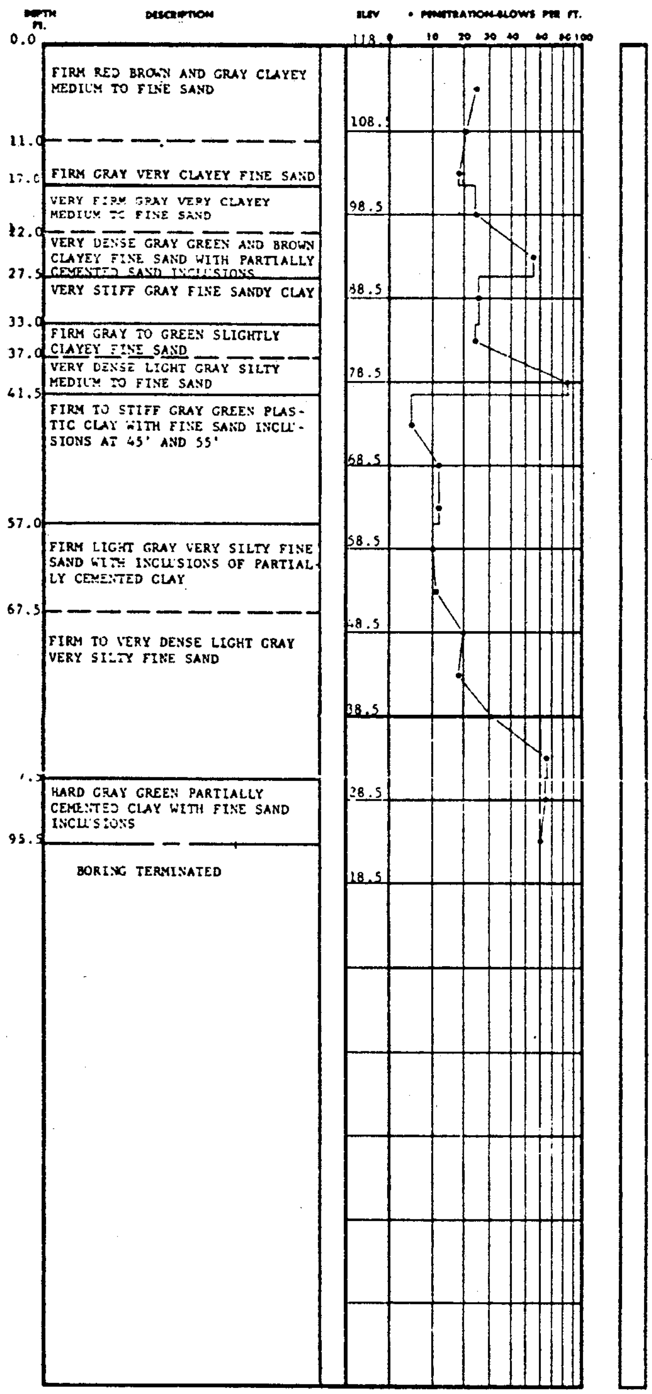
TEST BORING RECORD
BORING NO. 479

FIGURE 2B-65



TEST BORING RECORD

BORING NO. B-480

DATE DRILLED

JOB NO. 5036

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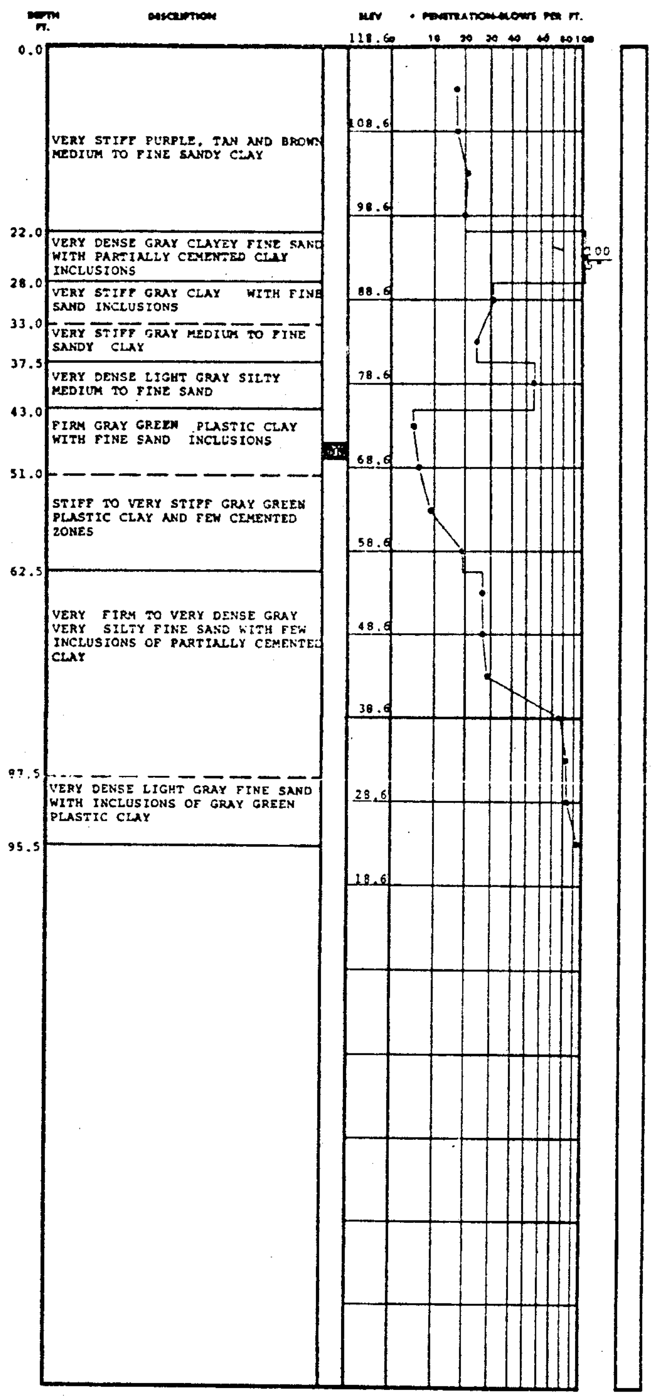
TEST BORING RECORD
BORING NO. 480

FIGURE 2B-66



TEST BORING RECORD

BORING NO. B-481

DATE DRILLED 2-13-69

JOB NO. 5156

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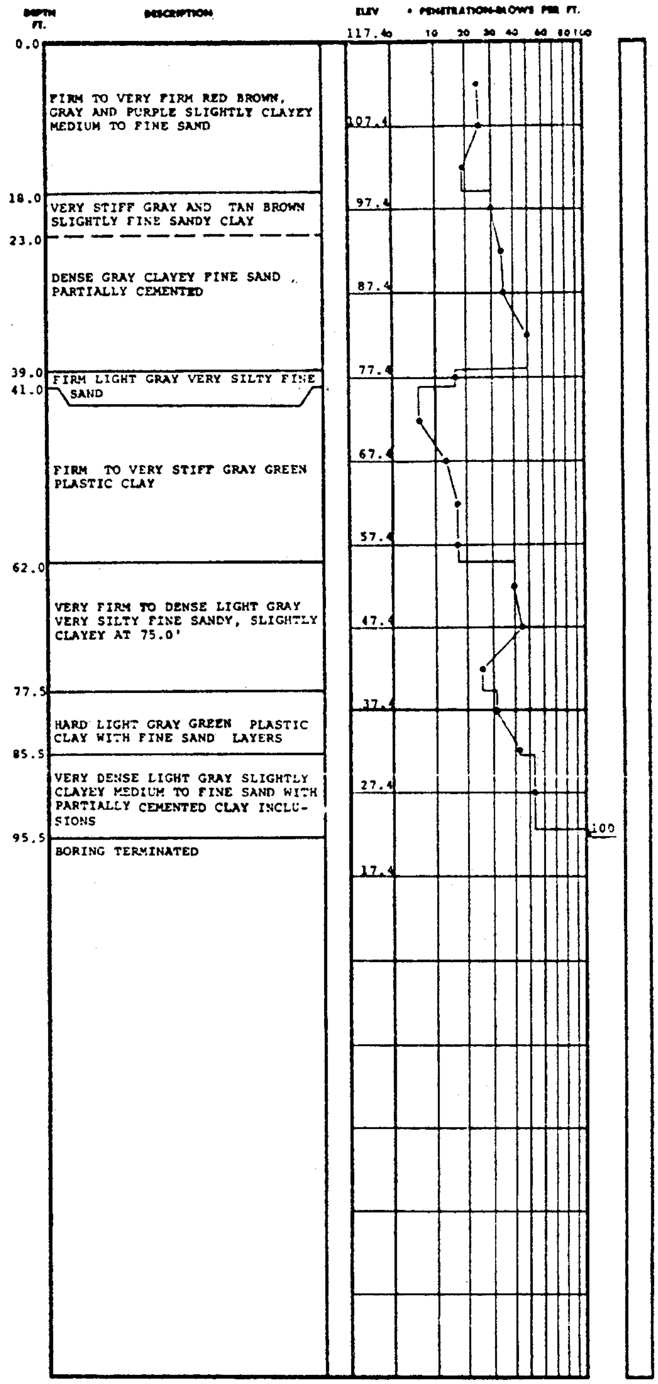
TEST BORING RECORD
BORING NO. 481

FIGURE 2B-67



TEST BORING RECORD

BORING NO. ______

DATE DRILLED ______

JOB NO. ______

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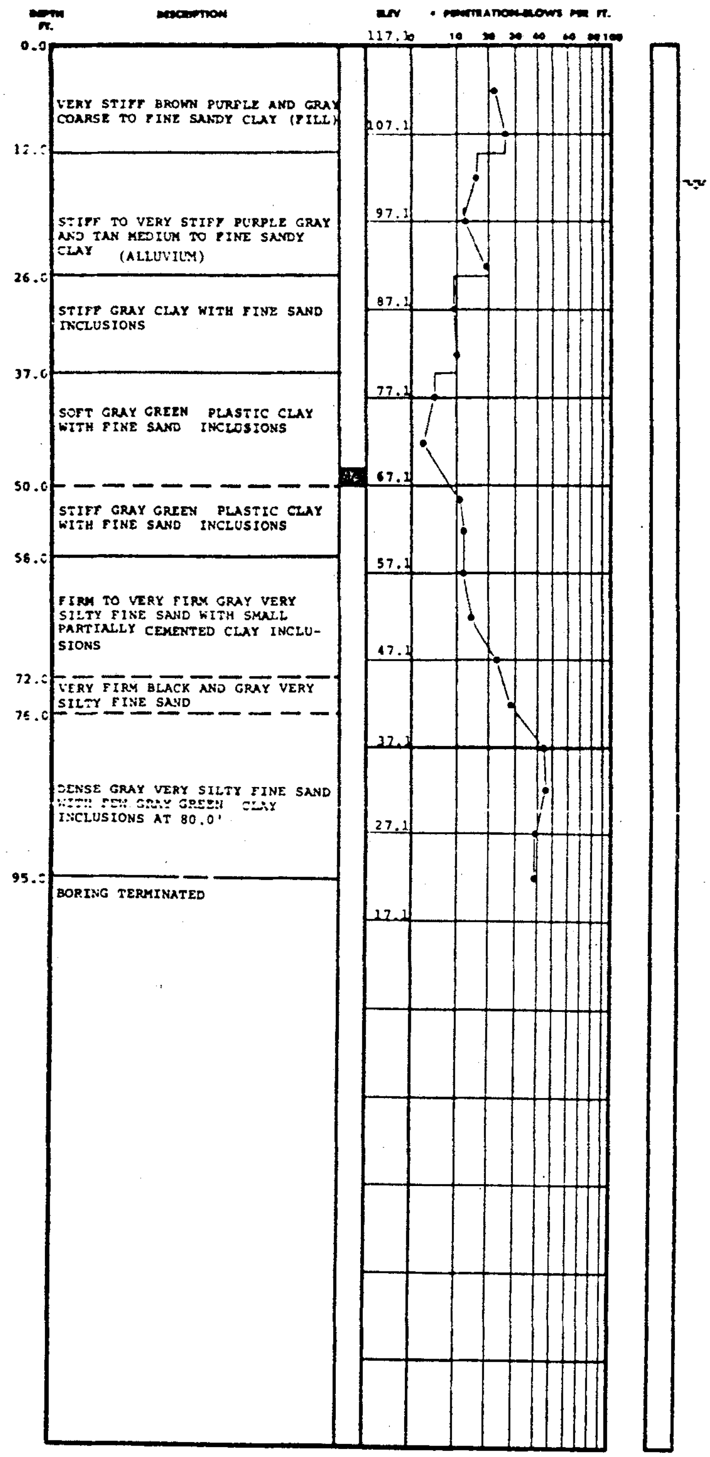
TEST BORING RECORD
BORING NO. 482

FIGURE 2B-68



TEST BORING RECORD

BORING NO. B-483
DATE DRILLED 2-14-
JOB NO. 5056

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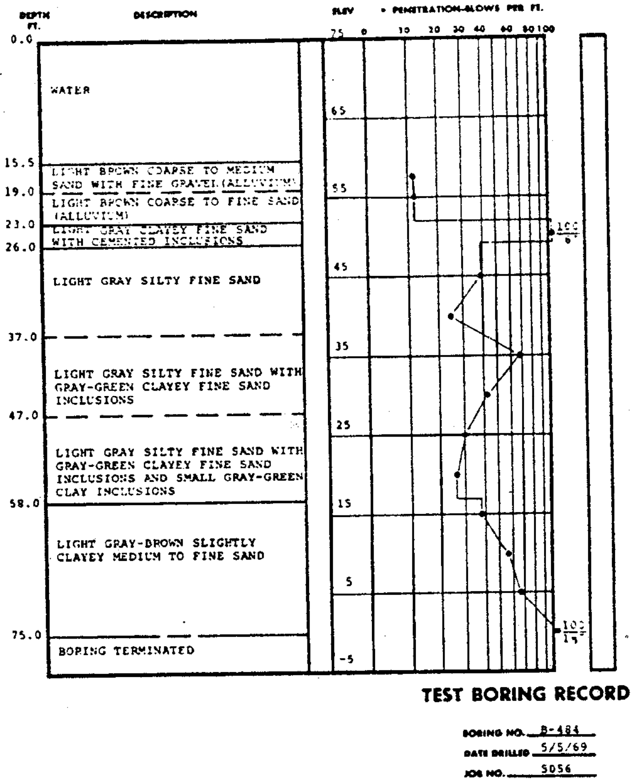
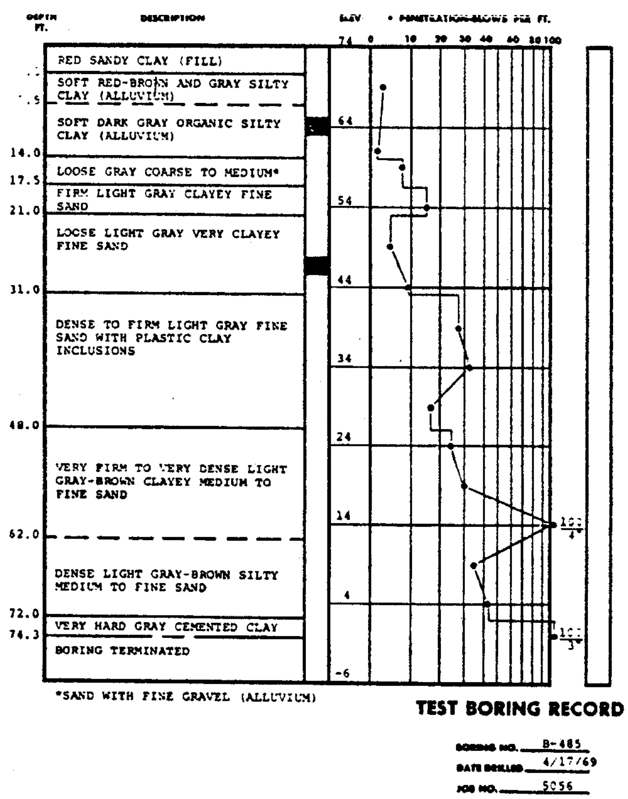
TEST BORING RECORD
BORING NO. 483

FIGURE 2B-69





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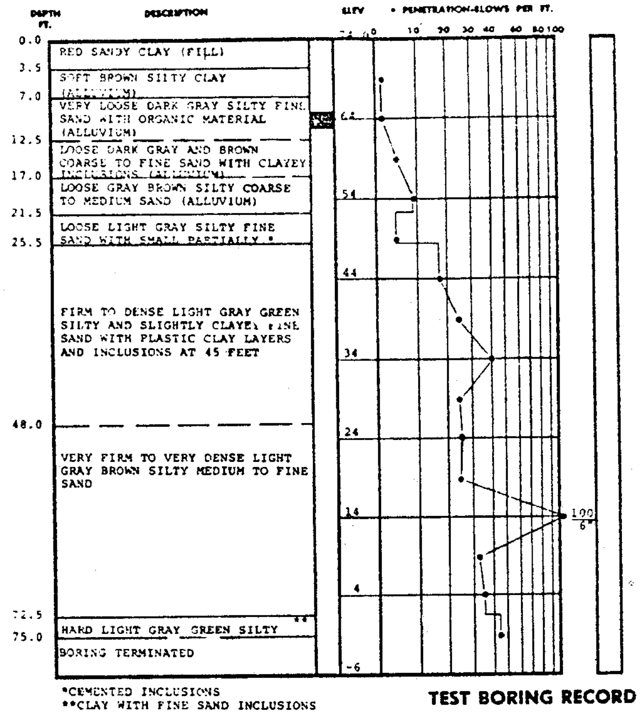
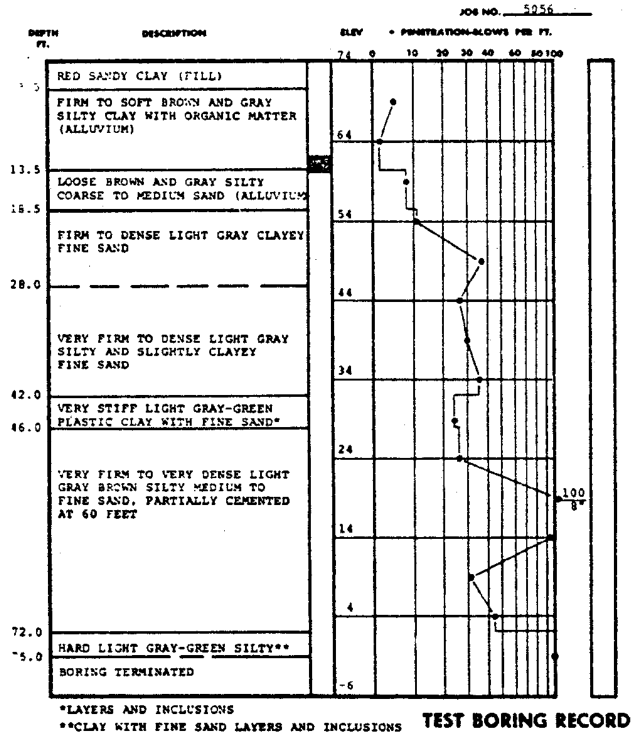
TEST BORING RECORD
RECORD NOS. 484 AND 485

FIGURE 2B-70



TEST BORING RECORD

BORING NO. B-486
DATE DRILLED 4/16/69
JOB NO. 5056



TEST BORING RECORD

BORING NO. B-487
DATE DRILLED 4/15/69
JOB NO. 5056

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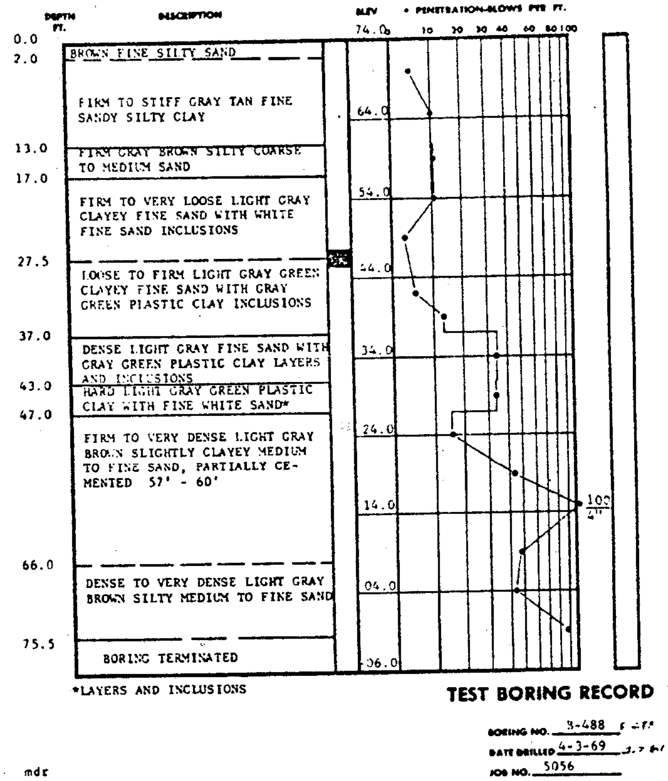
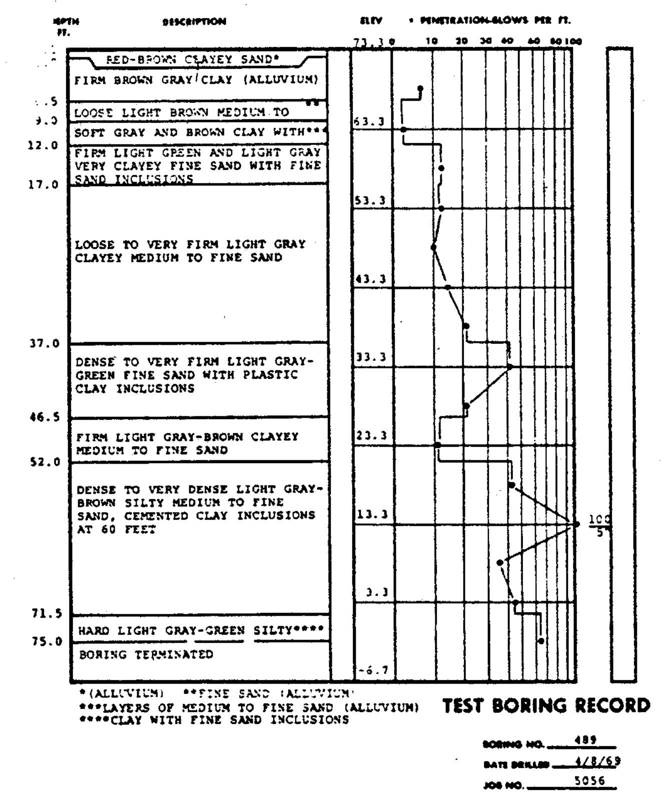
TEST BORING RECORD
BORING NOS. 486 AND 487

FIGURE 2B-71





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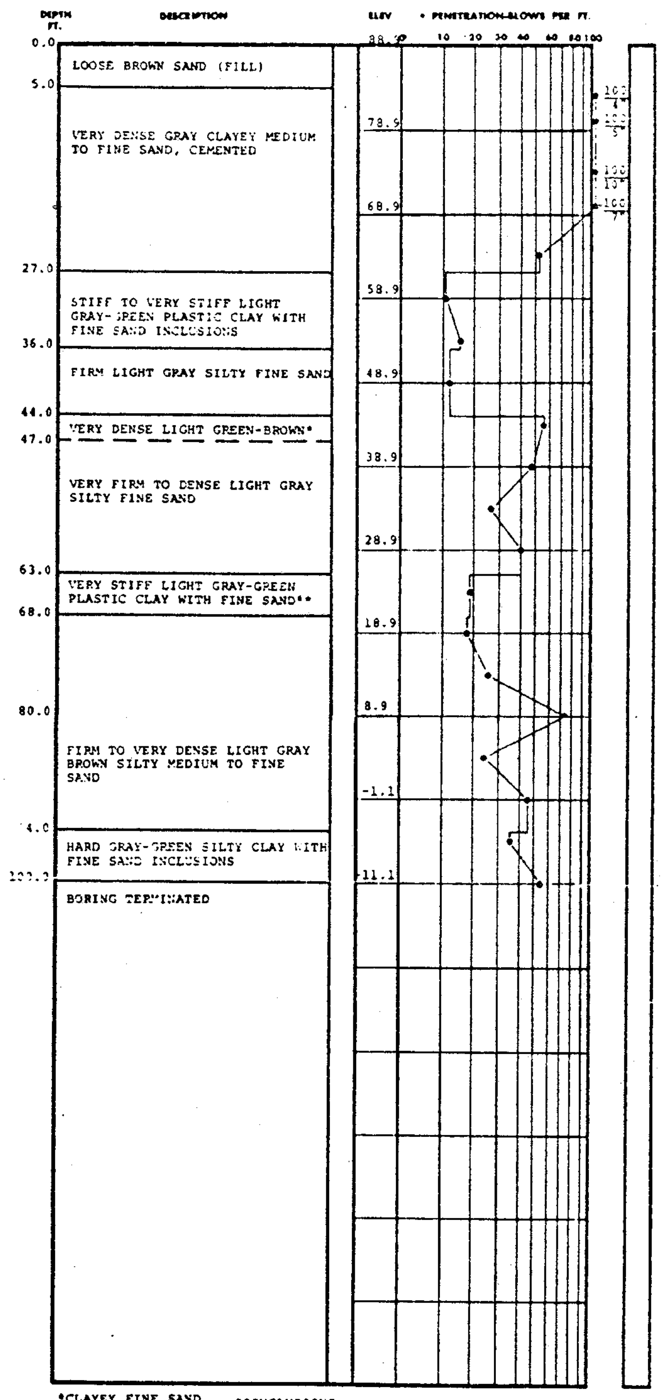
TEST BORING RECORD
BORING NO.S 488 AND 489

FIGURE 2B-72



TEST BORING RECORD

BORING NO. B-490
DATE DRILLED 4/27/69
JOB NO. 5756

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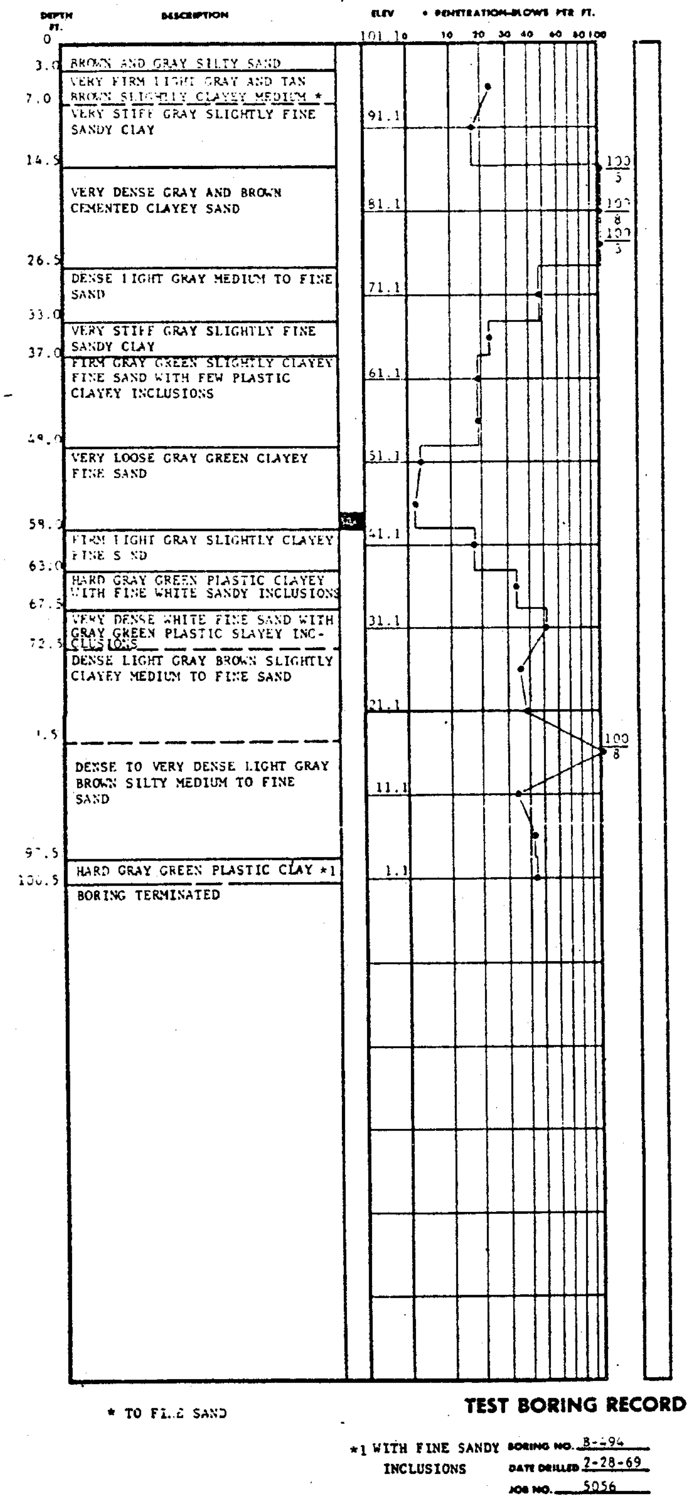
TEST BORING RECORD
BORING NO. 490

FIGURE 2B-73



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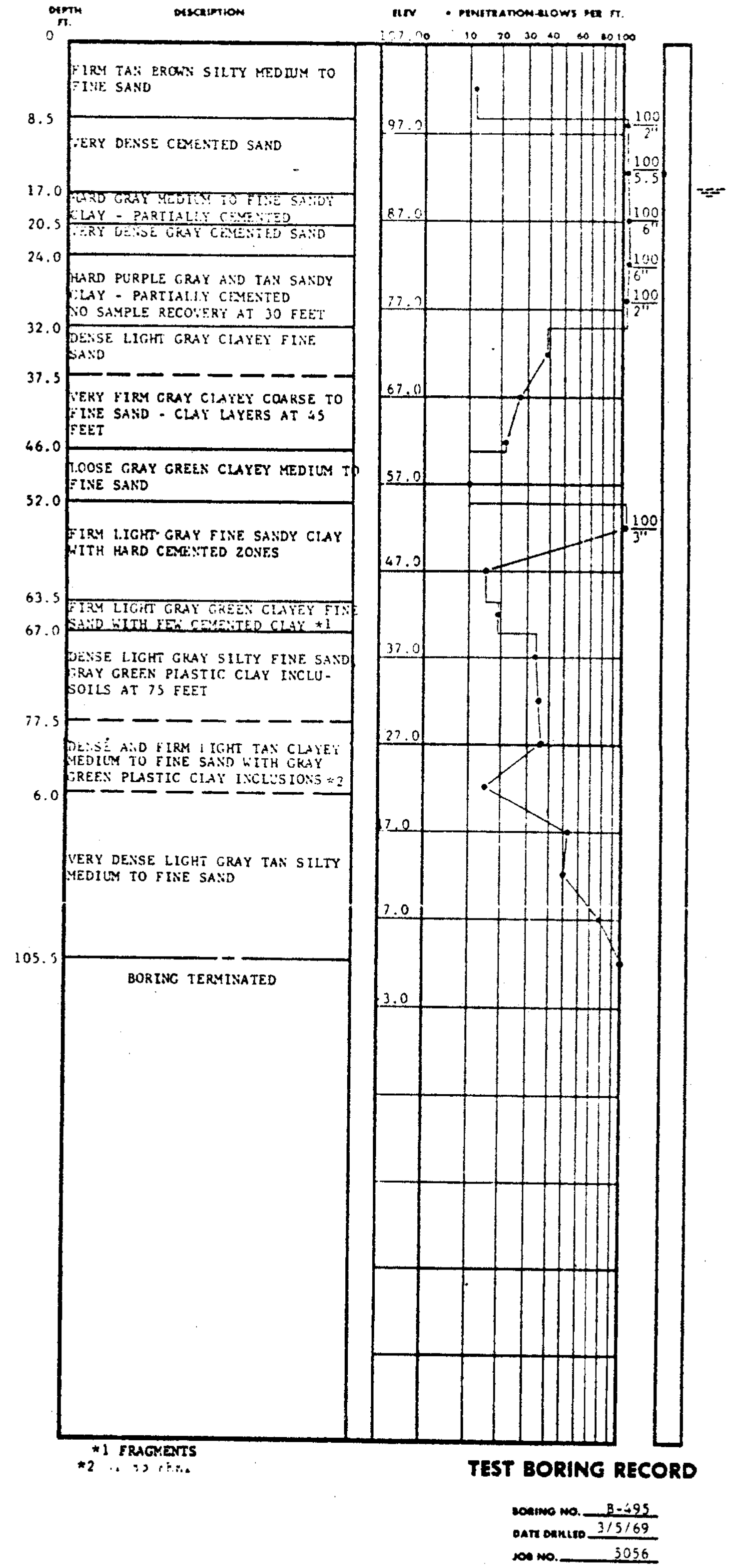
TEST BORING RECORD
BORING NO. 494

FIGURE 2B-74



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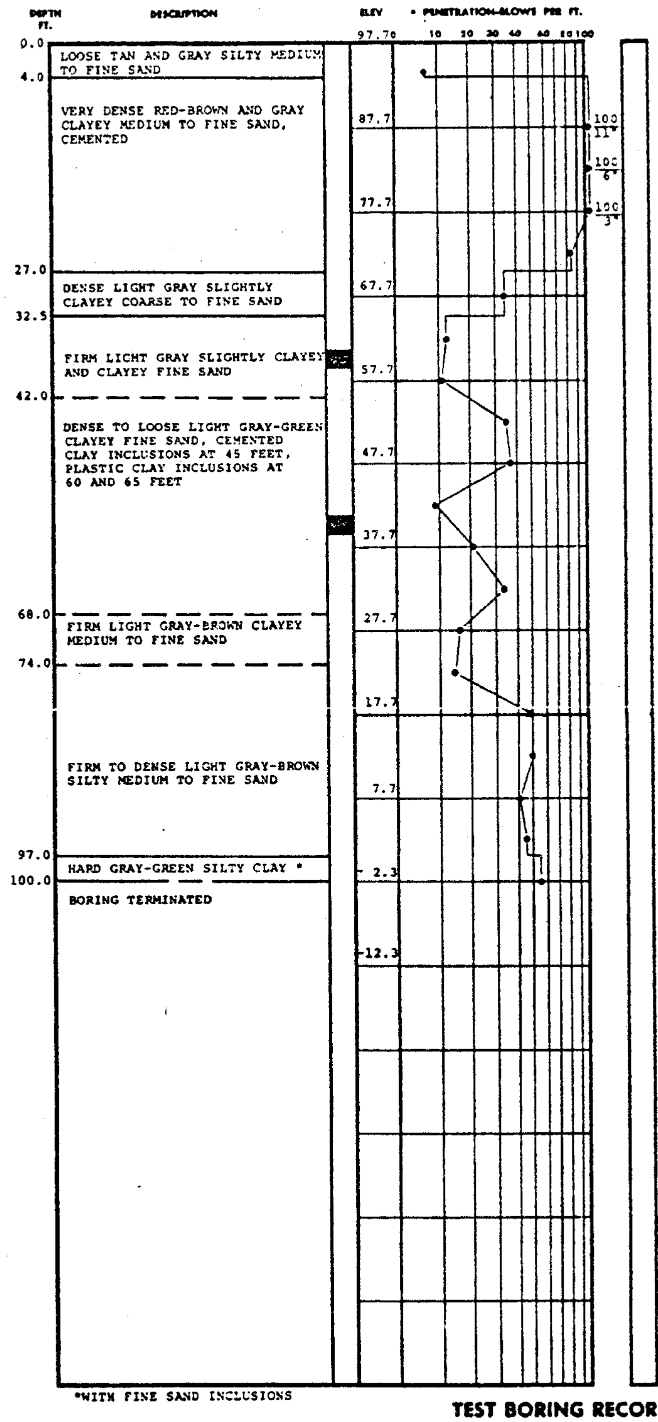
TEST BORING RECORDS
BORING NO. 495

FIGURE 2B-75



TEST BORING RECORD

BORING NO. B-511
DATE DRILLED 4/1/69
JOB NO. 5056

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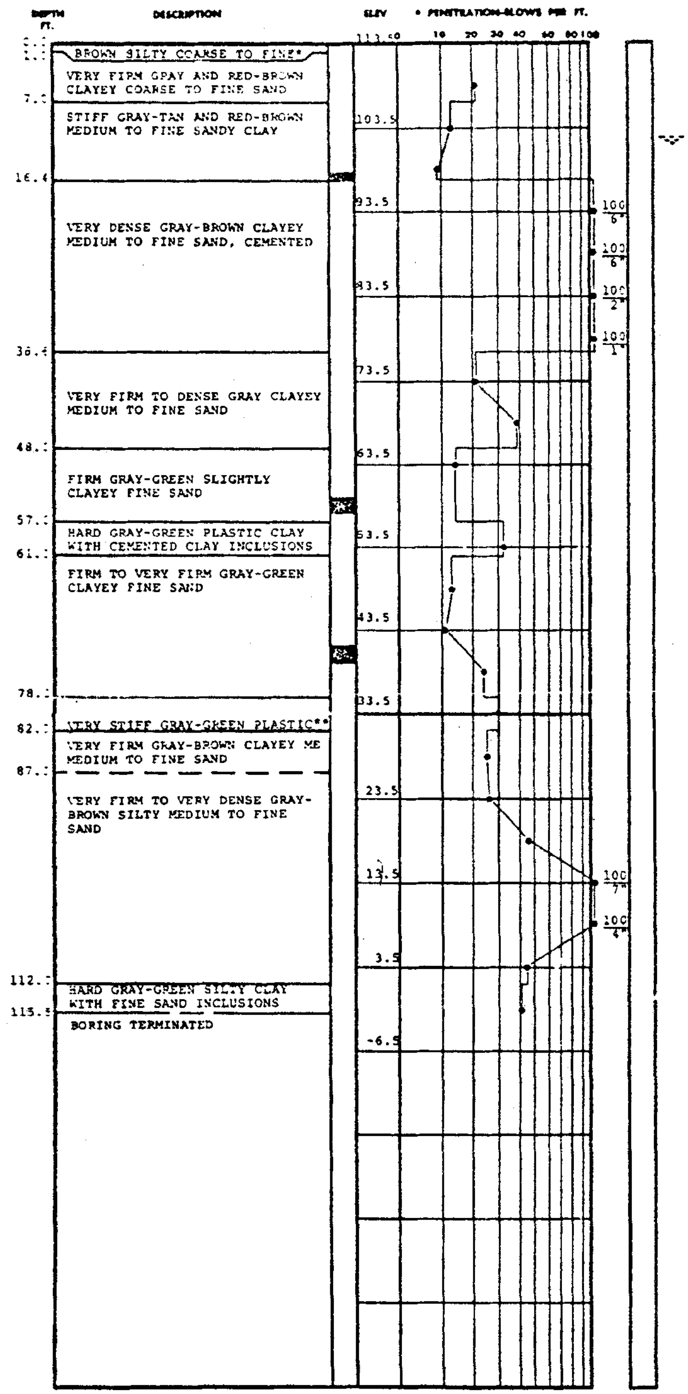
TEST BORING RECORDS
BORING NO. 511

FIGURE 2B-76



*SAND **CLAY WITH FINE SAND INCLUSIONS

TEST BORING RECORD

BORING NO. B-512
DATE DRILLED 4/2/69
JOB NO. 5056

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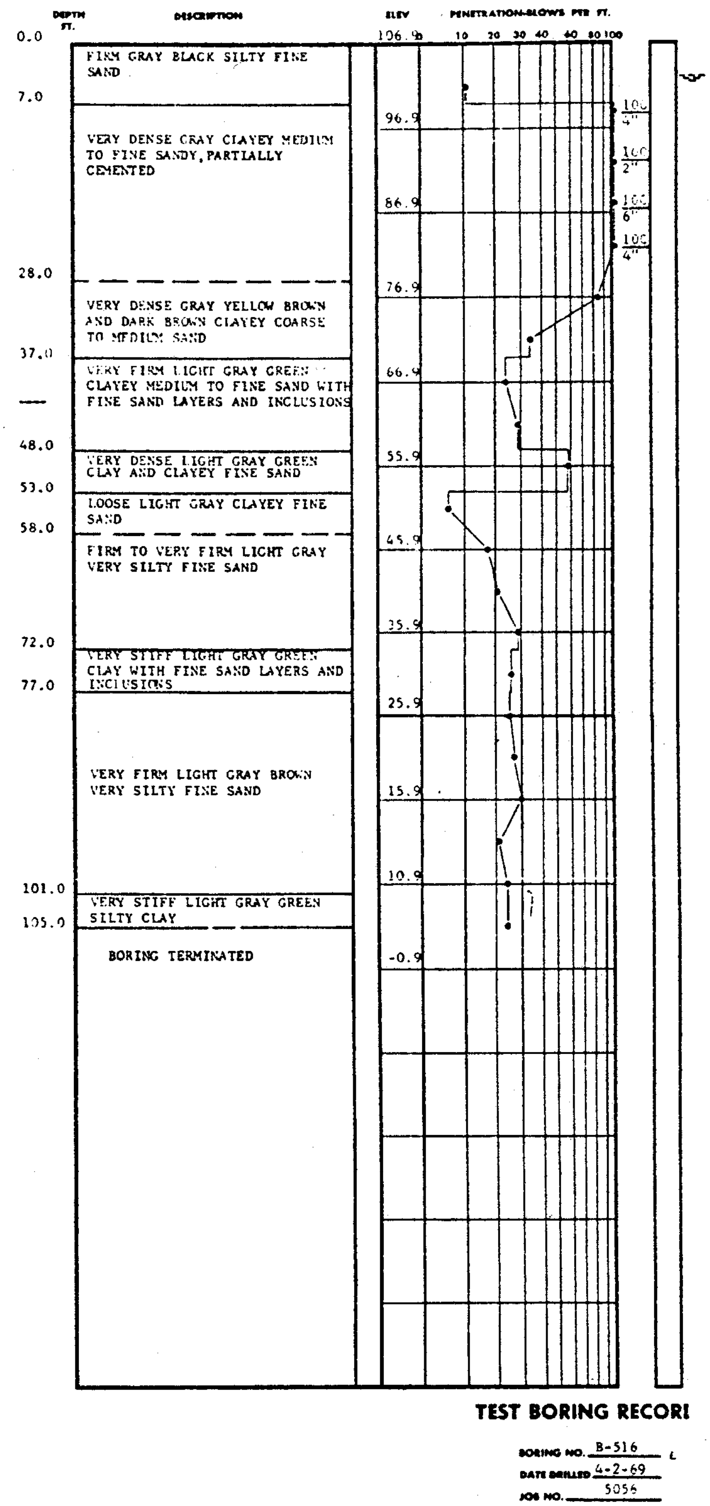
TEST BORING RECORD
BORING NO. 512

FIGURE 2B-77



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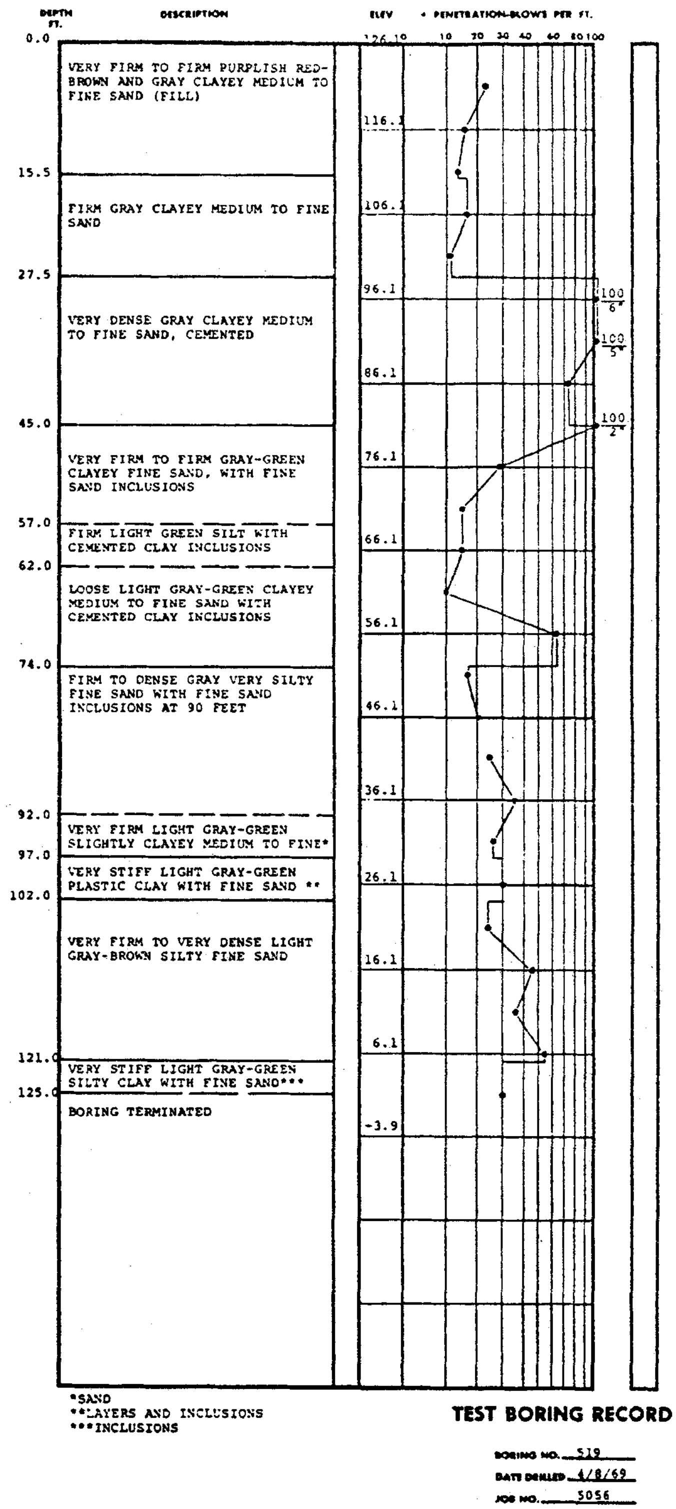
TEST BORING RECORD
BORING NO. 516

FIGURE 2B-78



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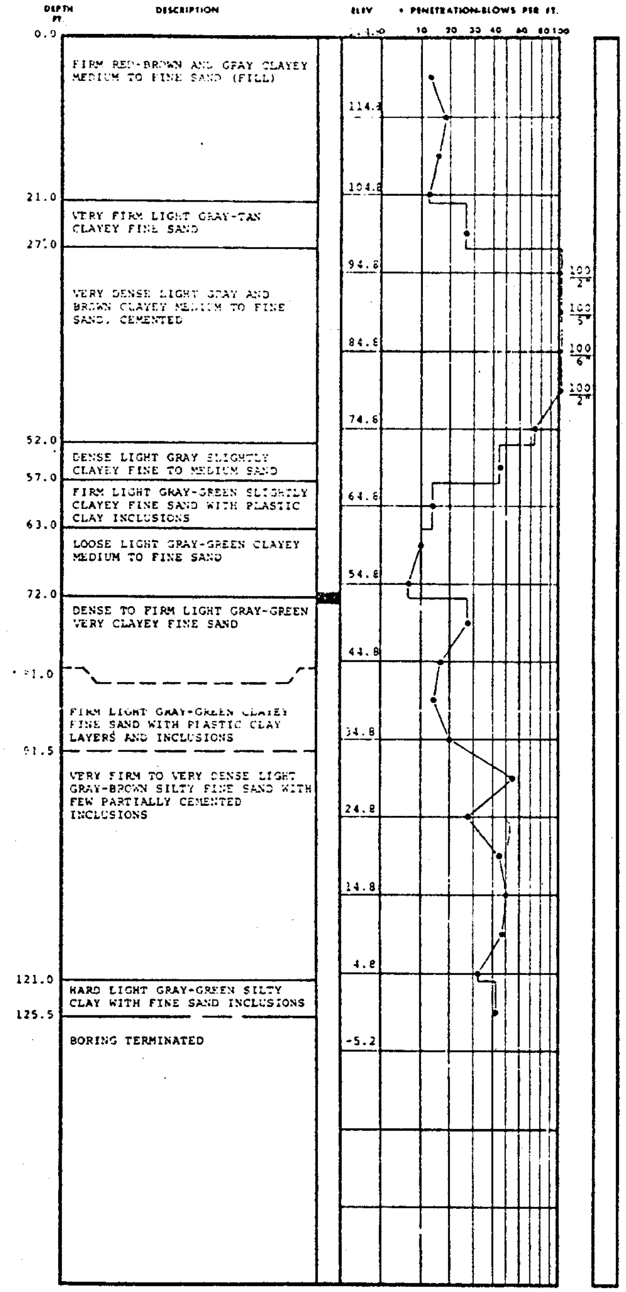
TEST BORING RECORD
BORING NO. 519

FIGURE 2B-79



TEST BORING RECORD

BORING NO. 521
DATE DRILLED 4/7/69
JOB NO. 5056

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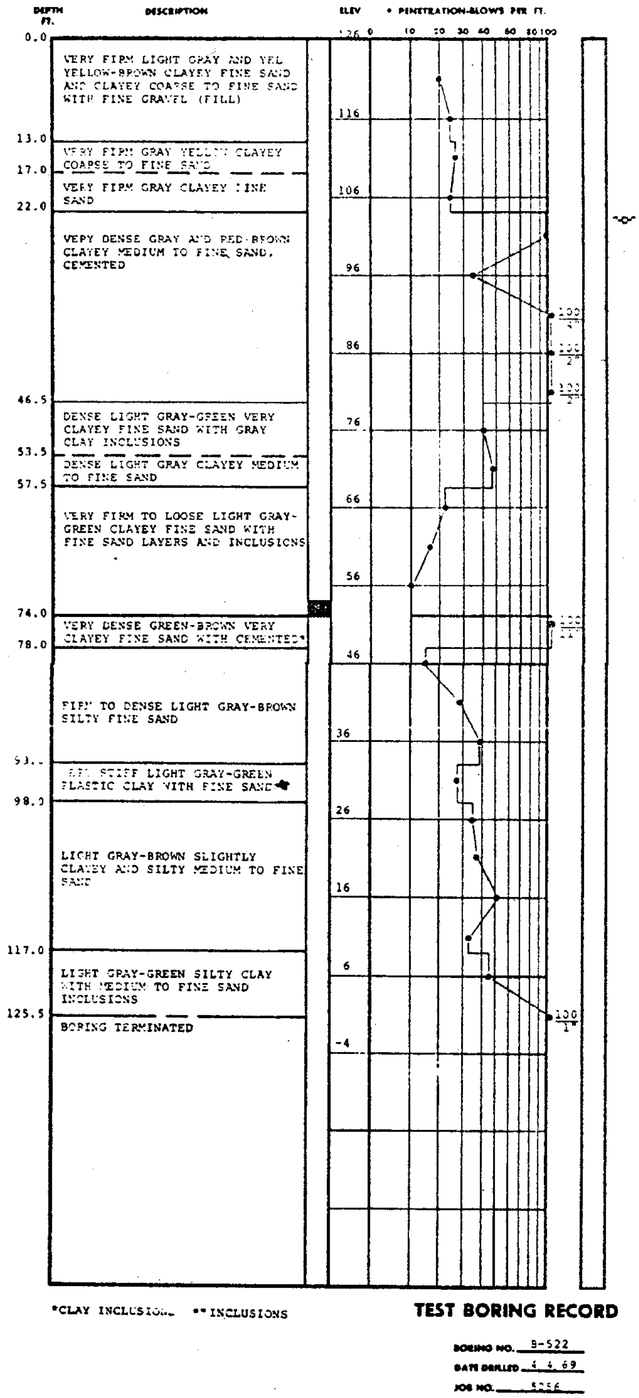
TEST BORING RECORD
BORING NO. 521

FIGURE 2B-80



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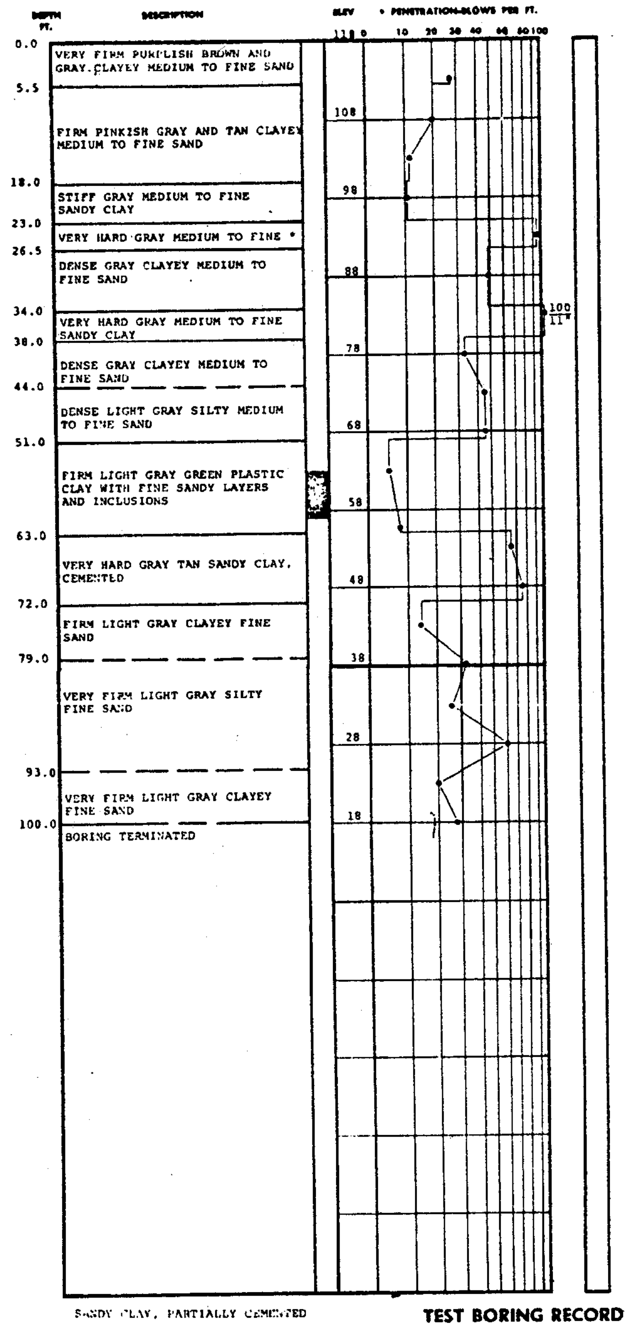
TEST BORING RECORD
BORING NO. 522

FIGURE 2B-81



TEST BORING RECORD

BORING NO. B-529

DATE DRILLED 4/20/69

JOB NO. 5056

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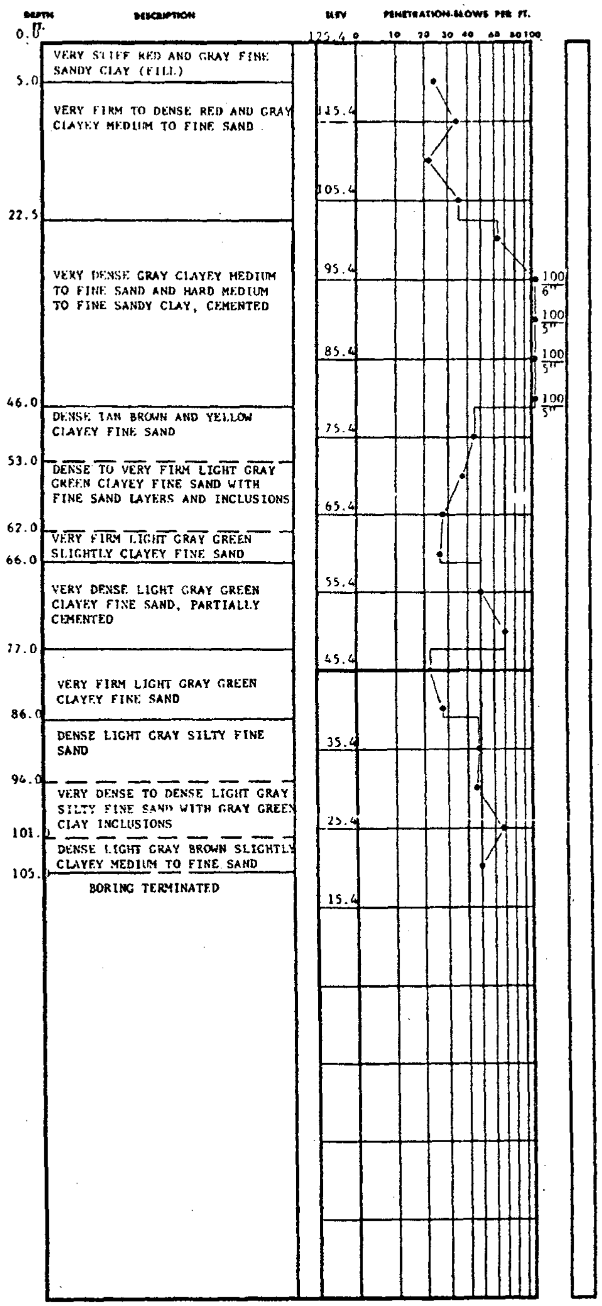
TEST BORING RECORD
BORING NO. 529

FIGURE 2B-82



TEST BORING RECORD

BORING NO. B-548
DATE DRILLED 4-30-60
JOB NO. 5056

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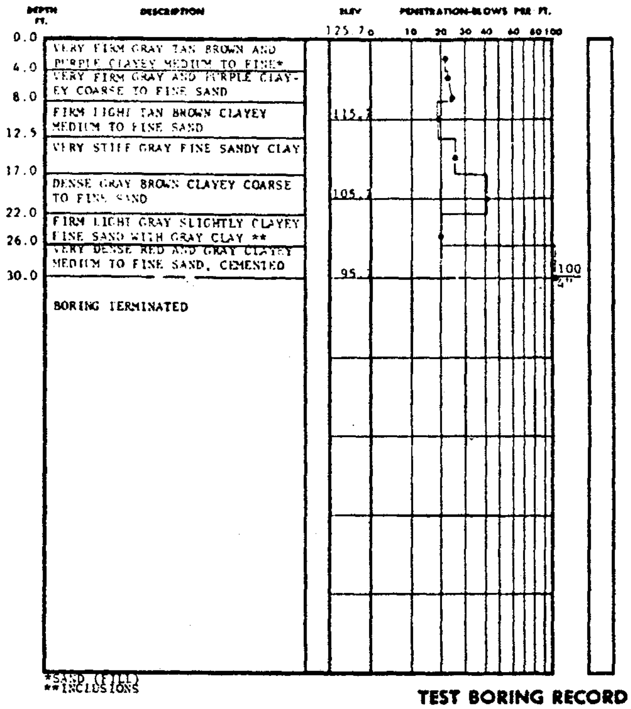
TEST BORING RECORD
BORING NO. 548

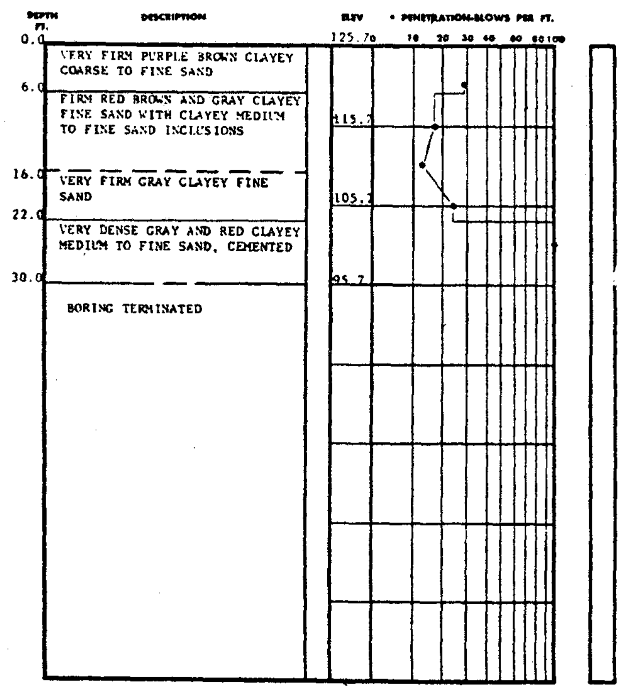
FIGURE 2B-83



TEST BORING RECORD

BORING NO. B-549
DATE DRILLED 5-5-69
JOB NO. 5056

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

BORING NO. B-550
DATE DRILLED 5-1-69
JOB NO. 5056

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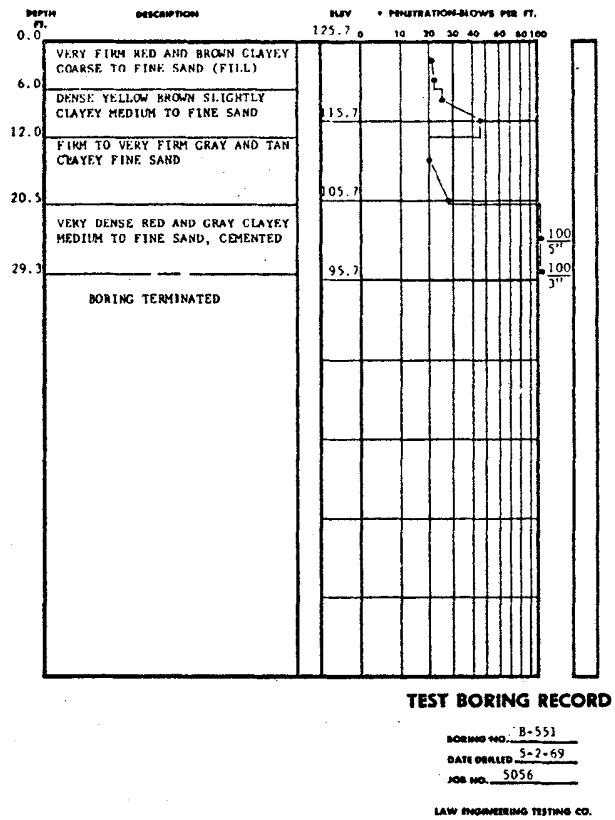
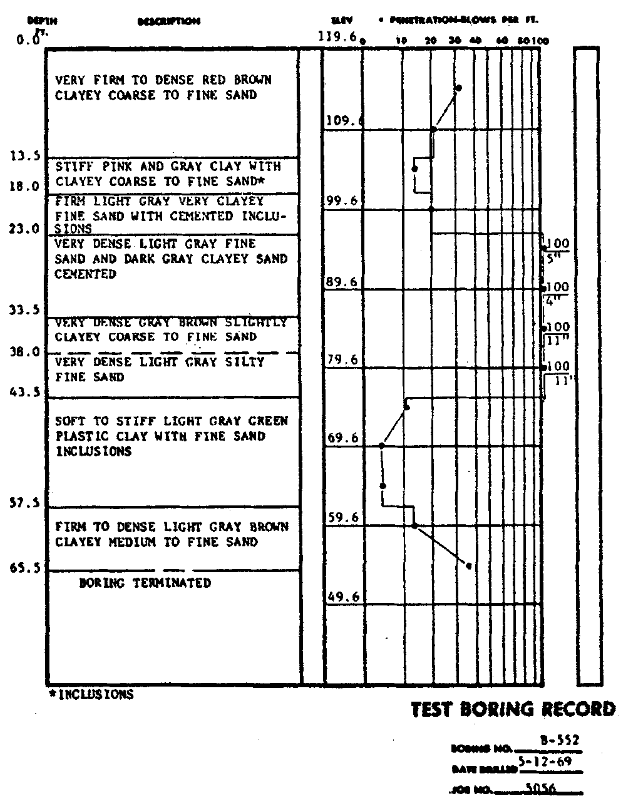
TEST BORING RECORD
BORING NOS. 549 AND 550

FIGURE 2B-84





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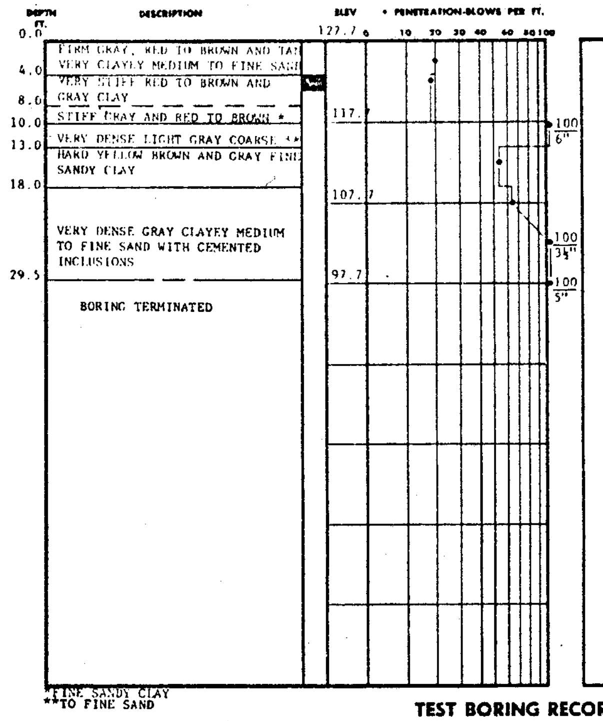
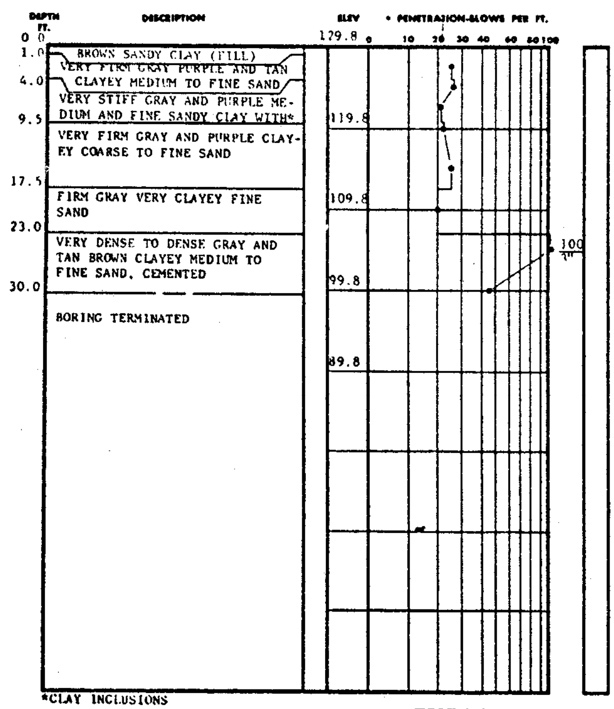
TEST BORING RECORD
BORING NOS. 551 AND 552

FIGURE 2B-85



TEST BORING RECORD

BORING NO. B-556
DATE DRILLED 5-8-69
JOB NO. 5056



TEST BORING RECORD

BORING NO. B-557
DATE DRILLED 5-9-69

HISTORICAL

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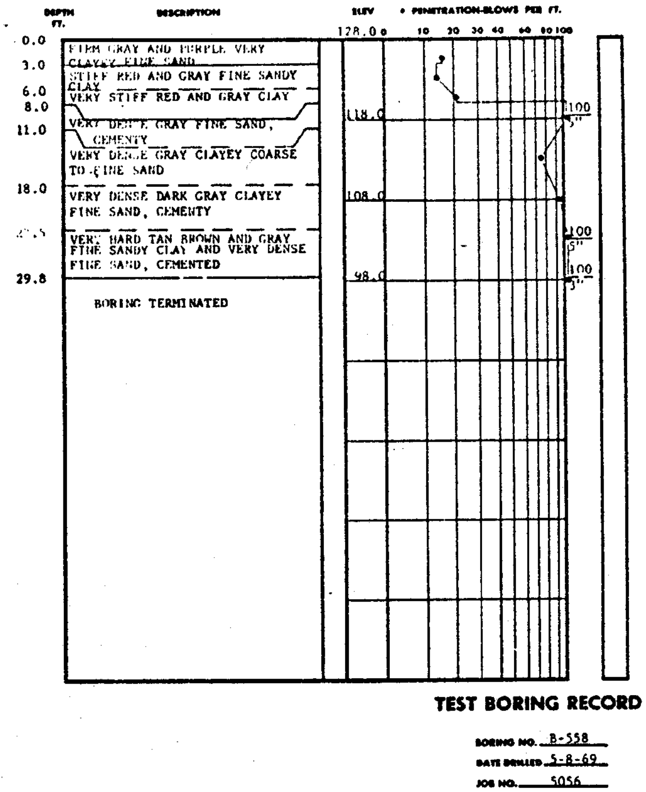
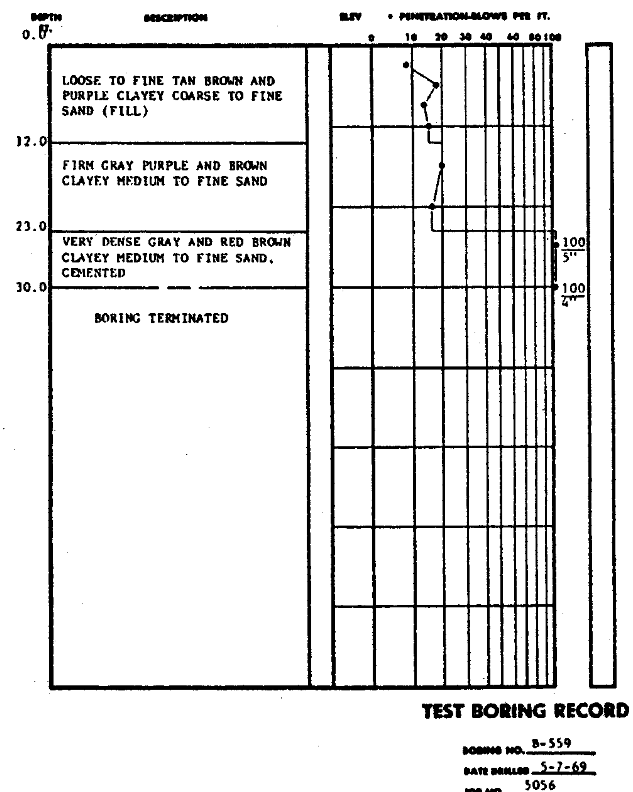
TEST BORING RECORD
BORING NOS. 556 AND 557

FIGURE 2B-86





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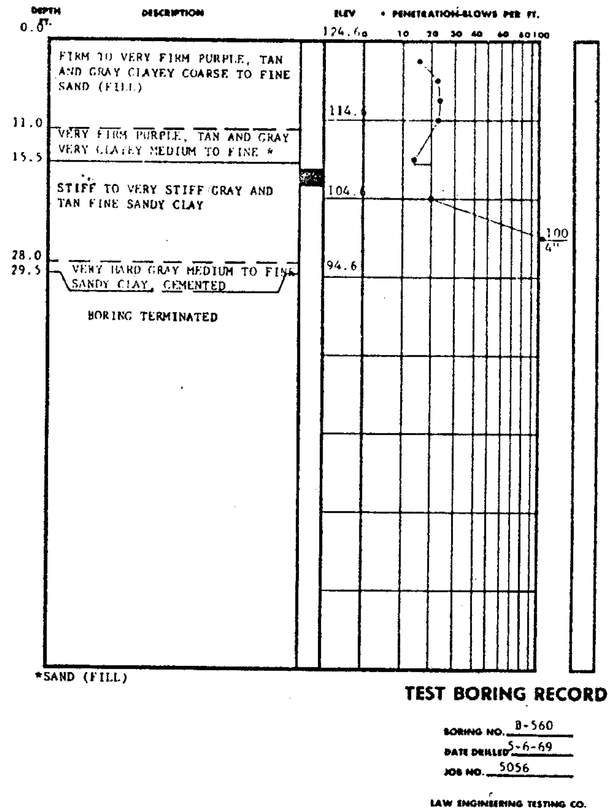
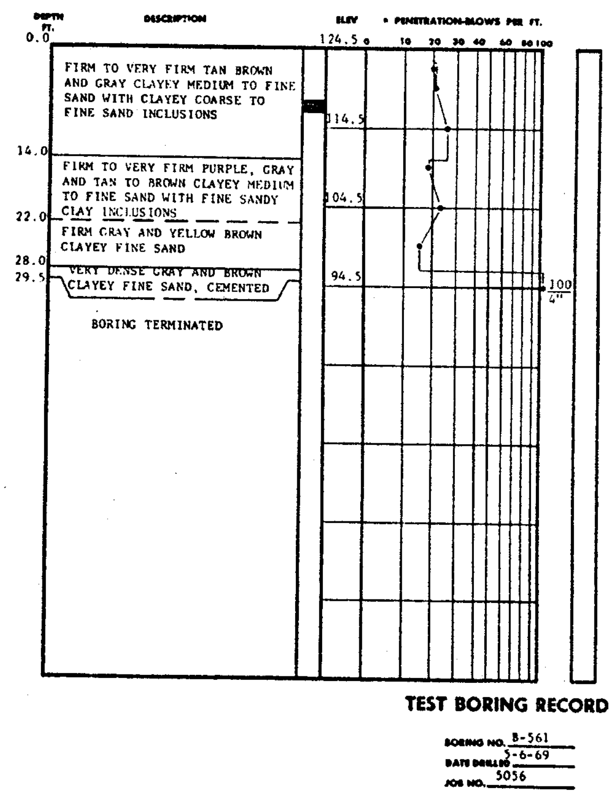
TEST BORING RECORD
BORING NOS. 558 AND 559

FIGURE 2B-87





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TEST BORING RECORD
BORING NOS. 560 AND 561

FIGURE 2B-88

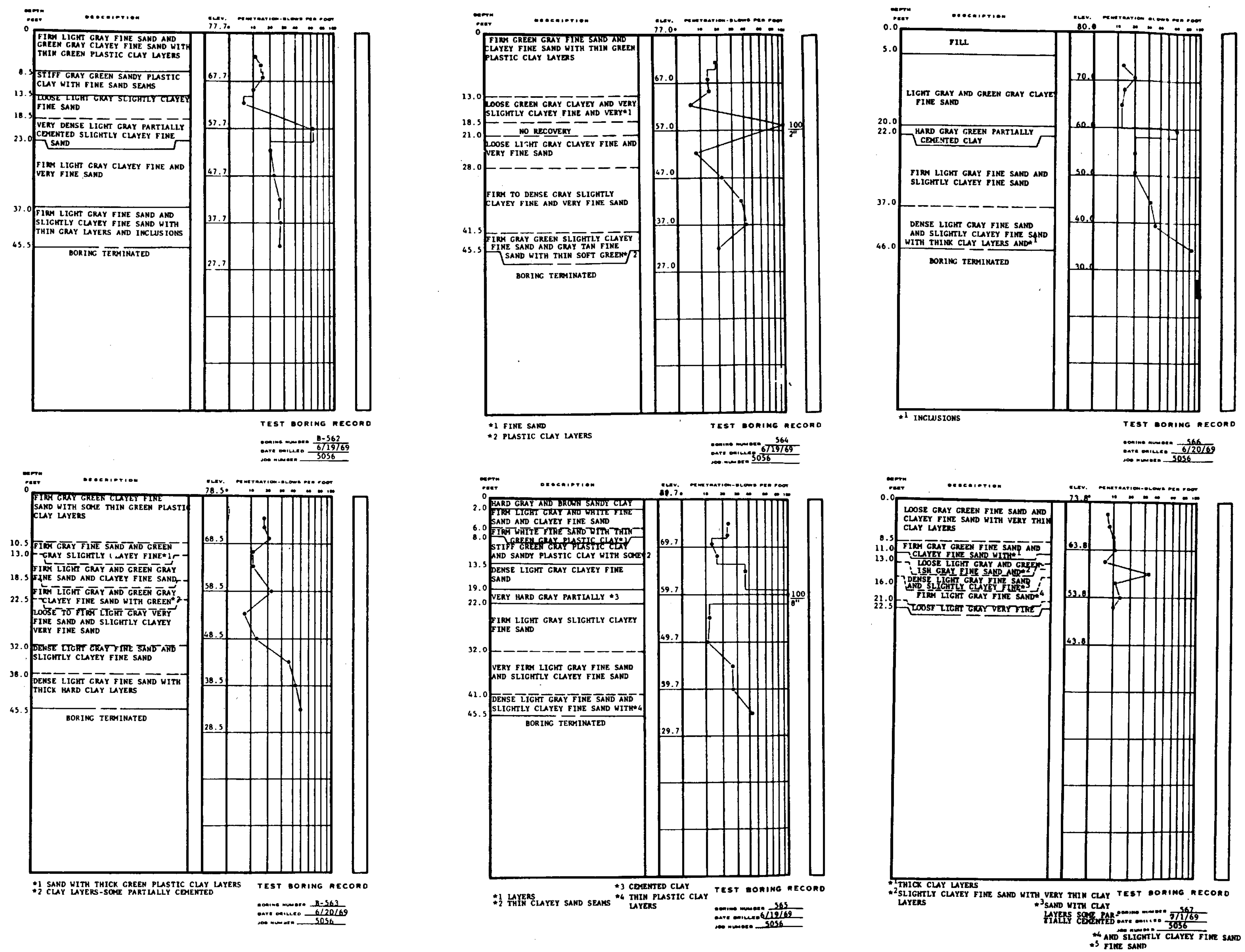

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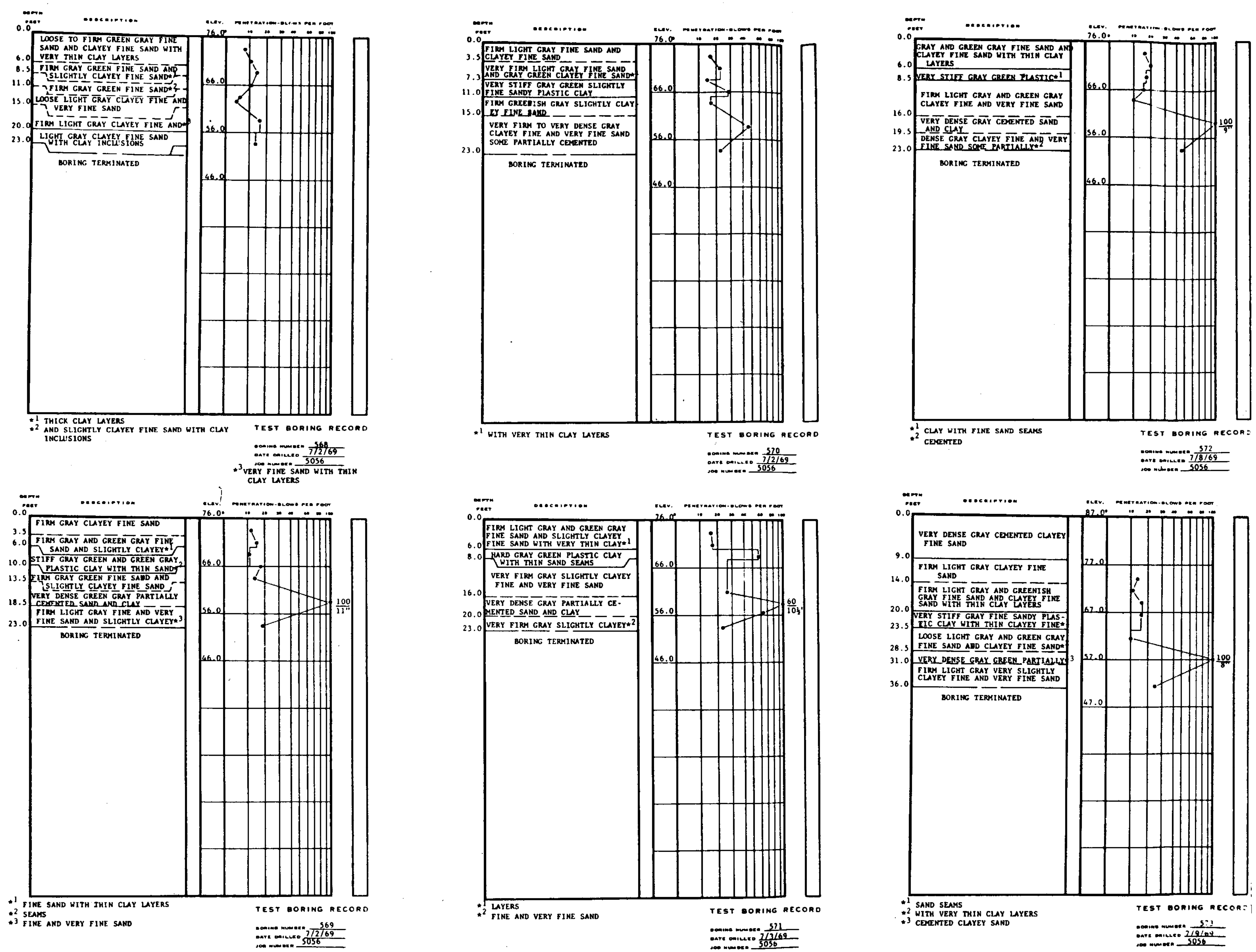
		FIGURE 2B-89



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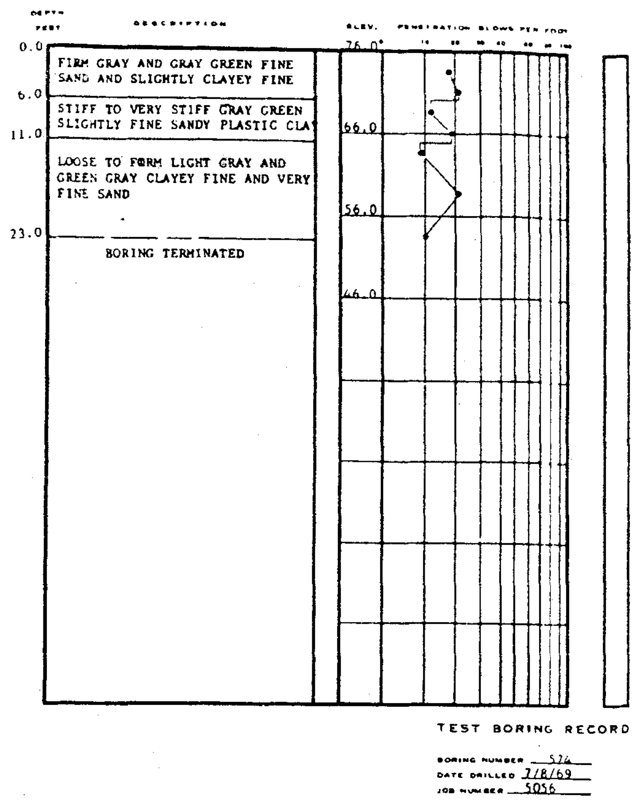
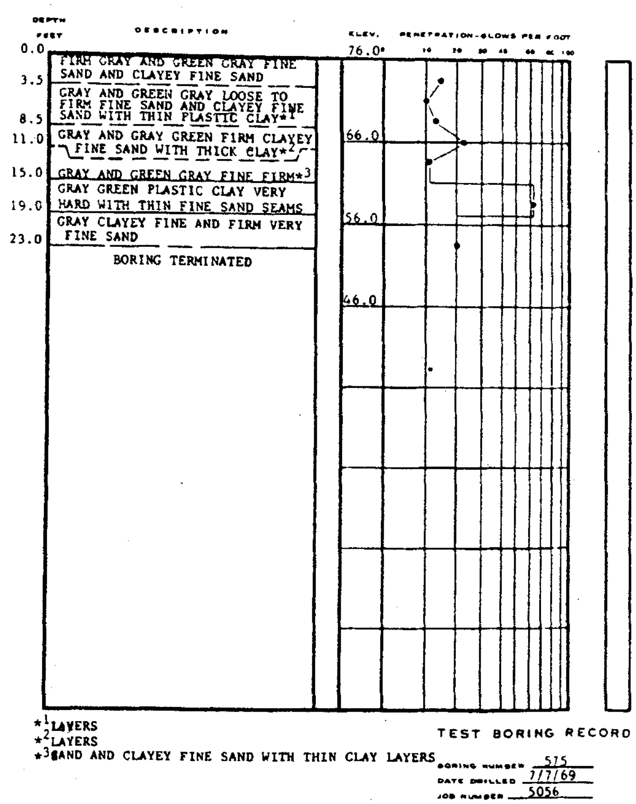
TEST BORING RECORD
BORING NOS. 568 THRU 573

FIGURE 2B-90





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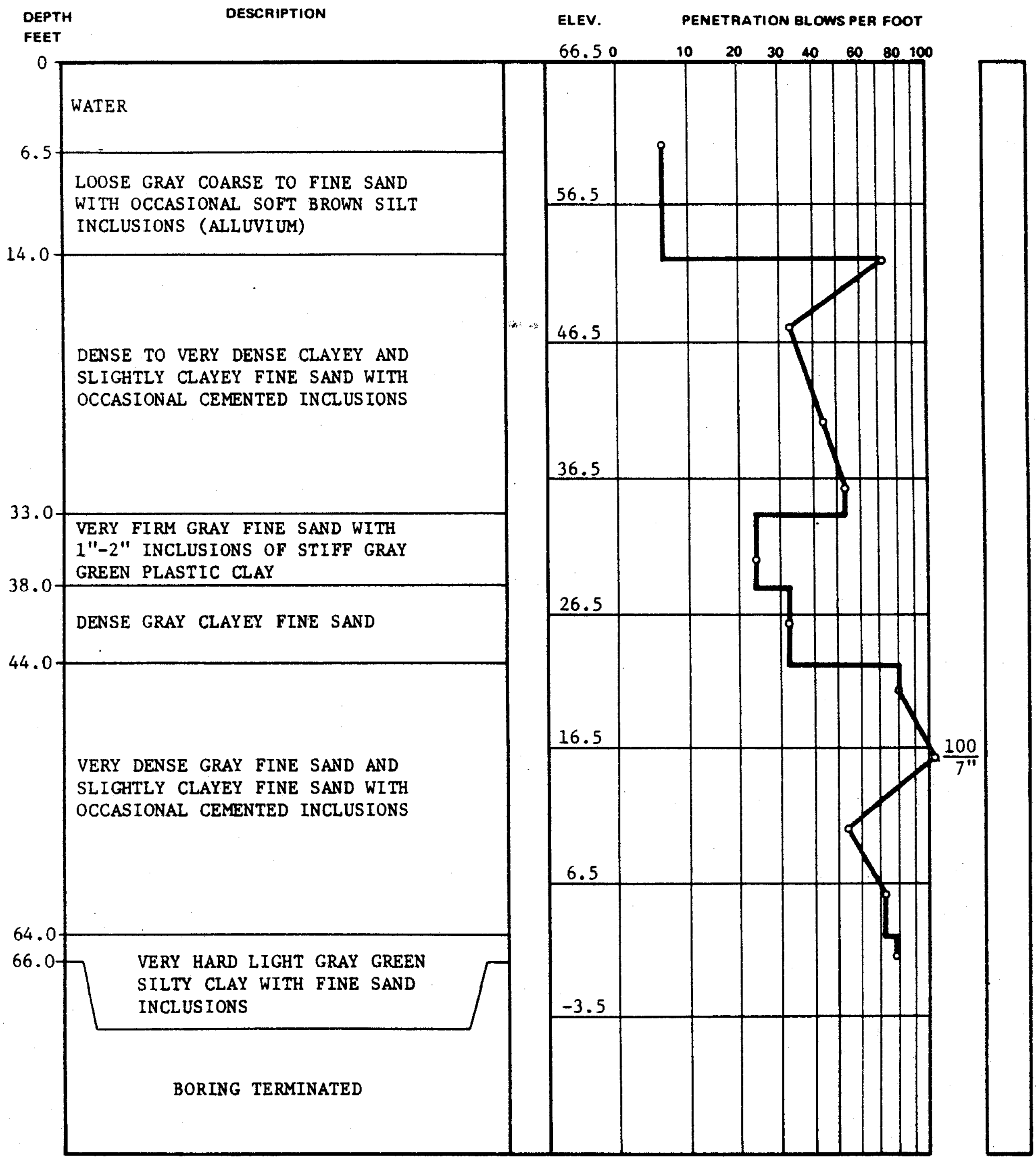
UNIT 2

TEST BORING RECORD
BORING NOS. 574 AND 575

FIGURE 2B-91



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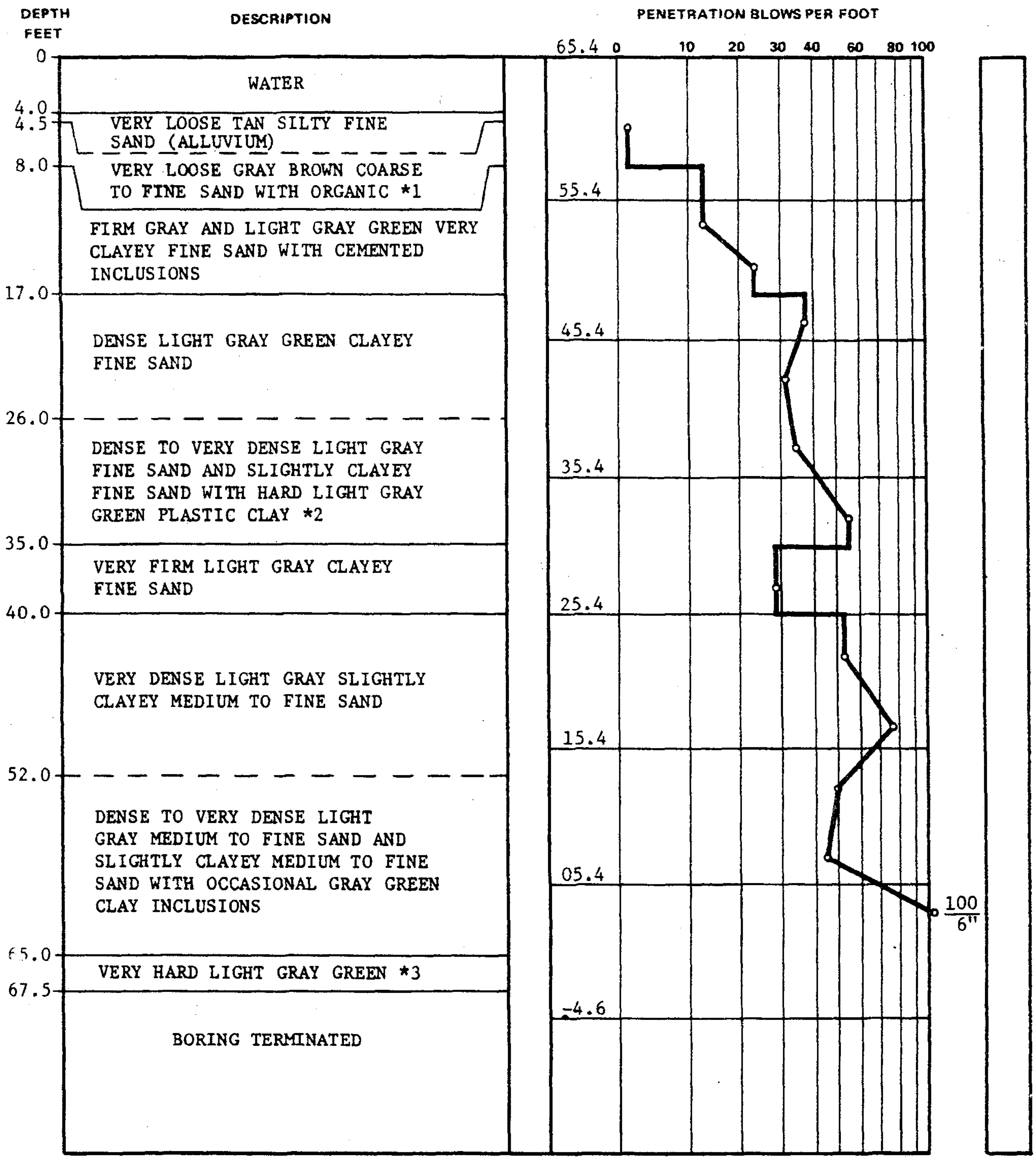
TEST BORING RECORD
BORING NO. 576

FIGURE 2B-92



*1 MATTER (ALLUVIUM)
*2 INCLUSIONS
*3 SILTY CLAY WITH OCCASIONAL FINE SAND INCLUSIONS

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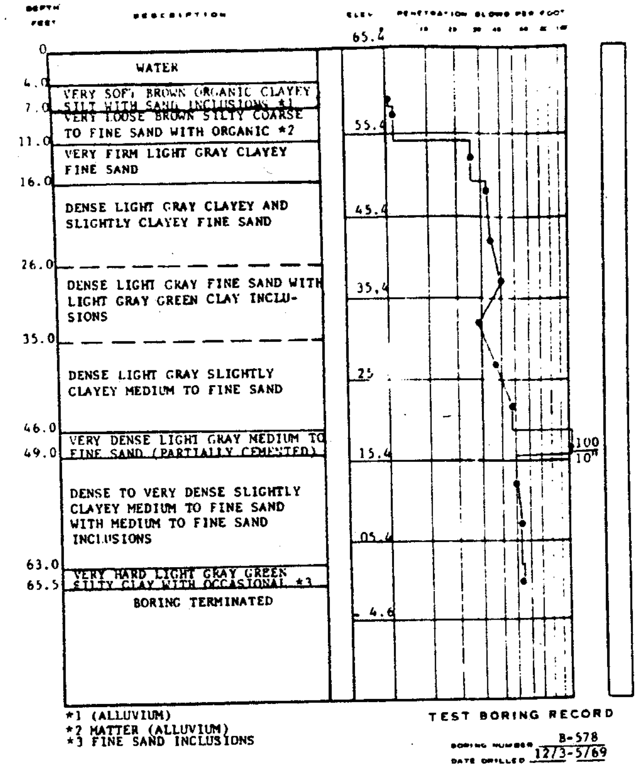
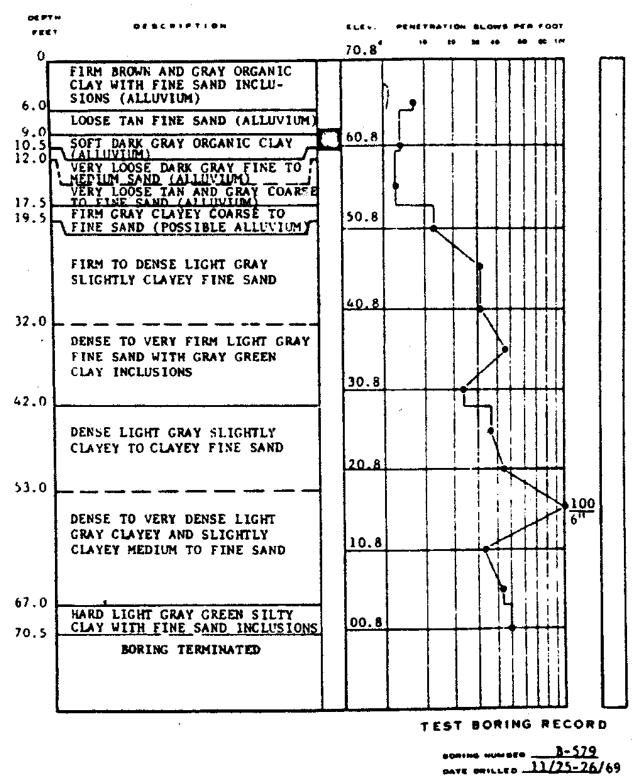
TEST BORING RECORD
BORING NO. 577

FIGURE 2B-93





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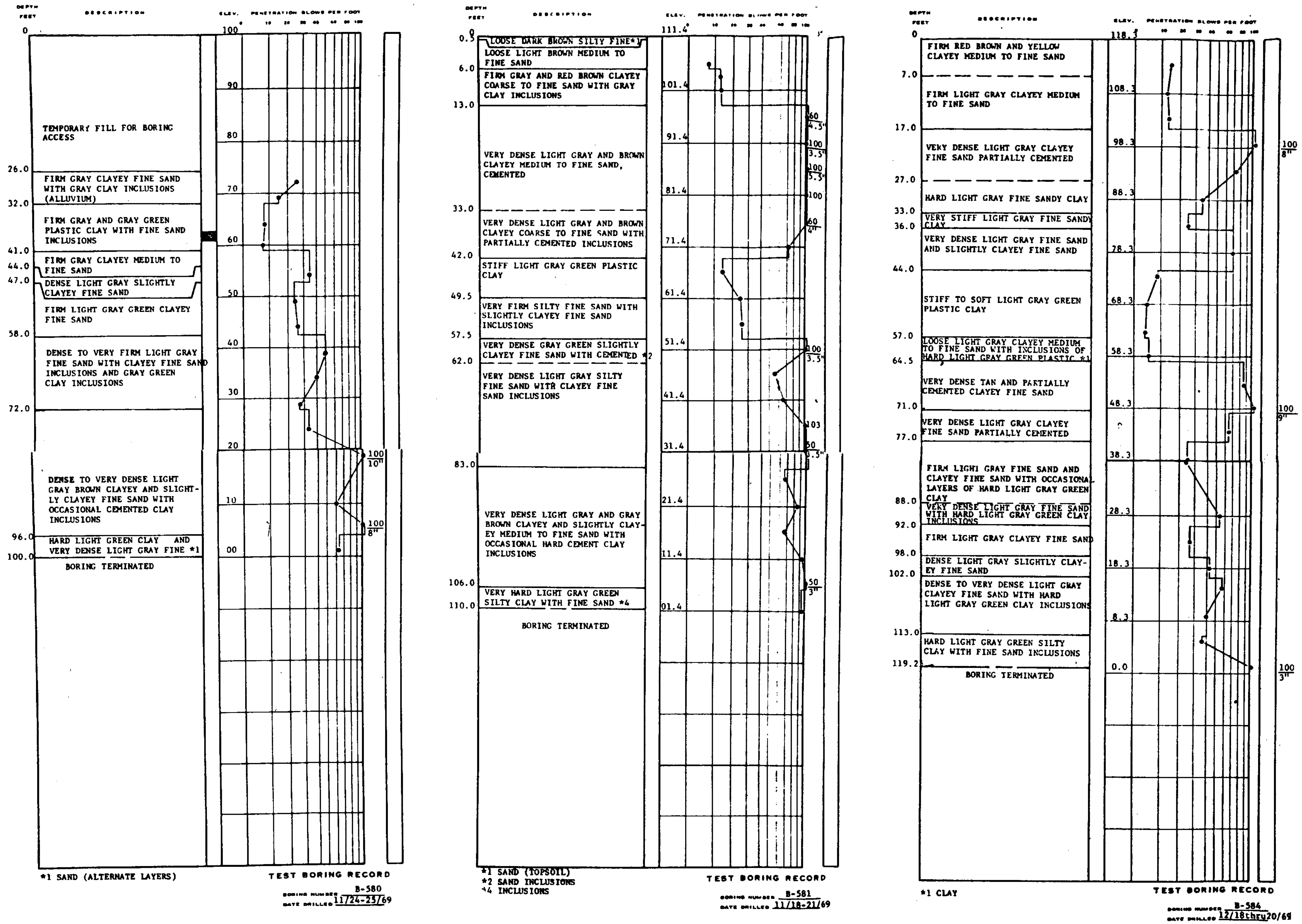
TEST BORING RECORD
BORING NOS. 578 AND 579

FIGURE 2B-94



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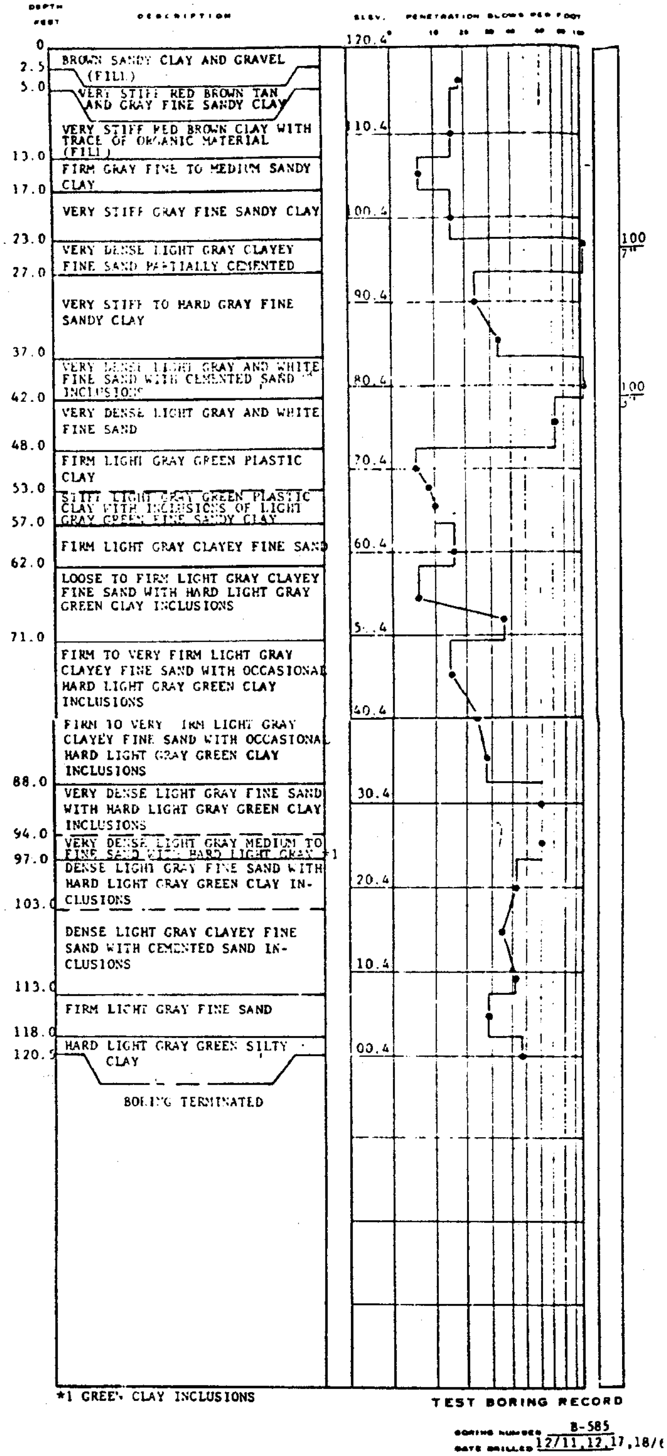
TEST BORING RECORD
BORING NOS. 580, 581, AND 584

FIGURE 2B-95



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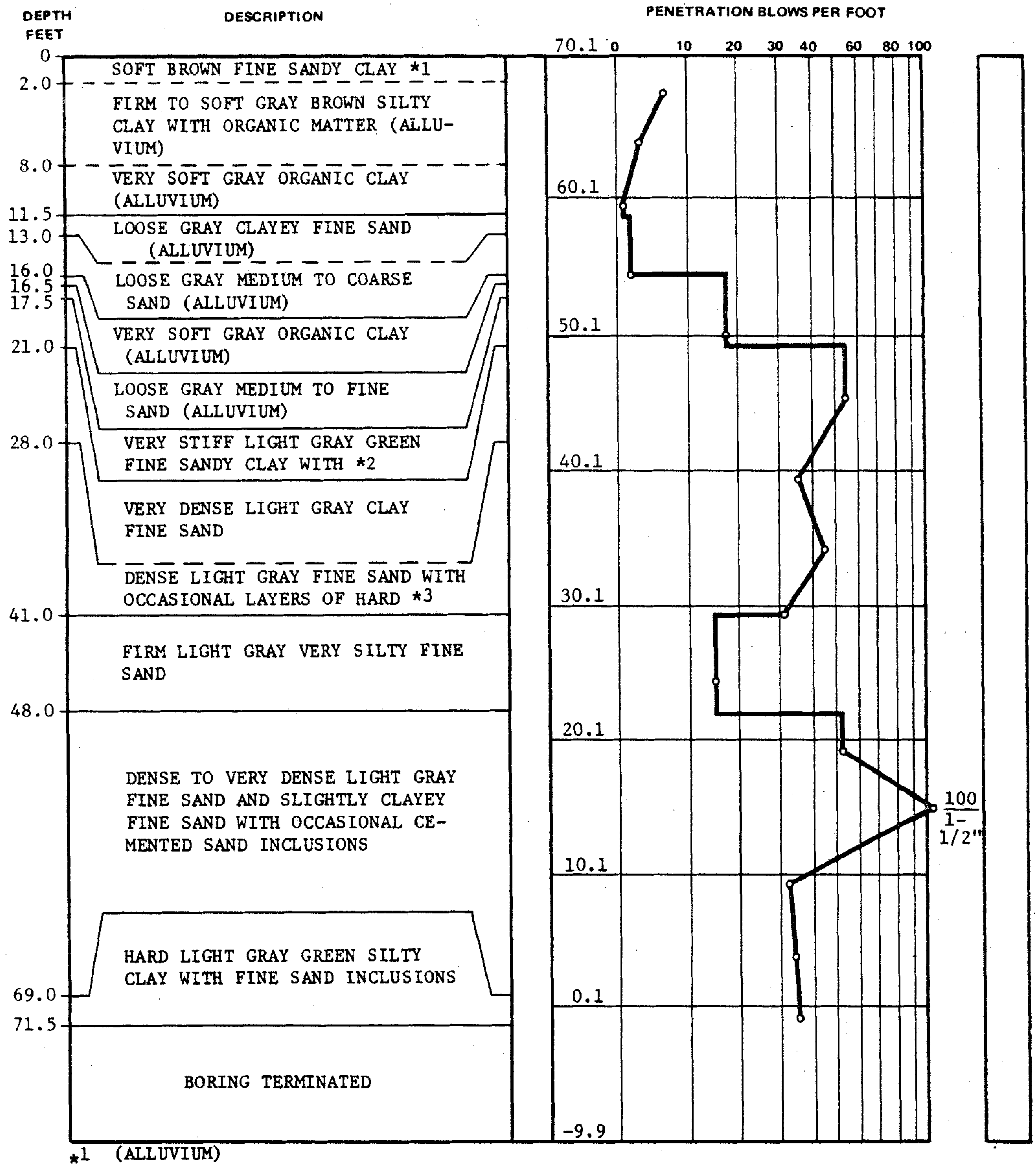
TEST BORING RECORD
BORING NO. 585

FIGURE 2B-96



*1 (ALLUVIUM)
*2 CEMENTED INCLUSIONS
*3 GRAY GREEN PLASTIC CLAY

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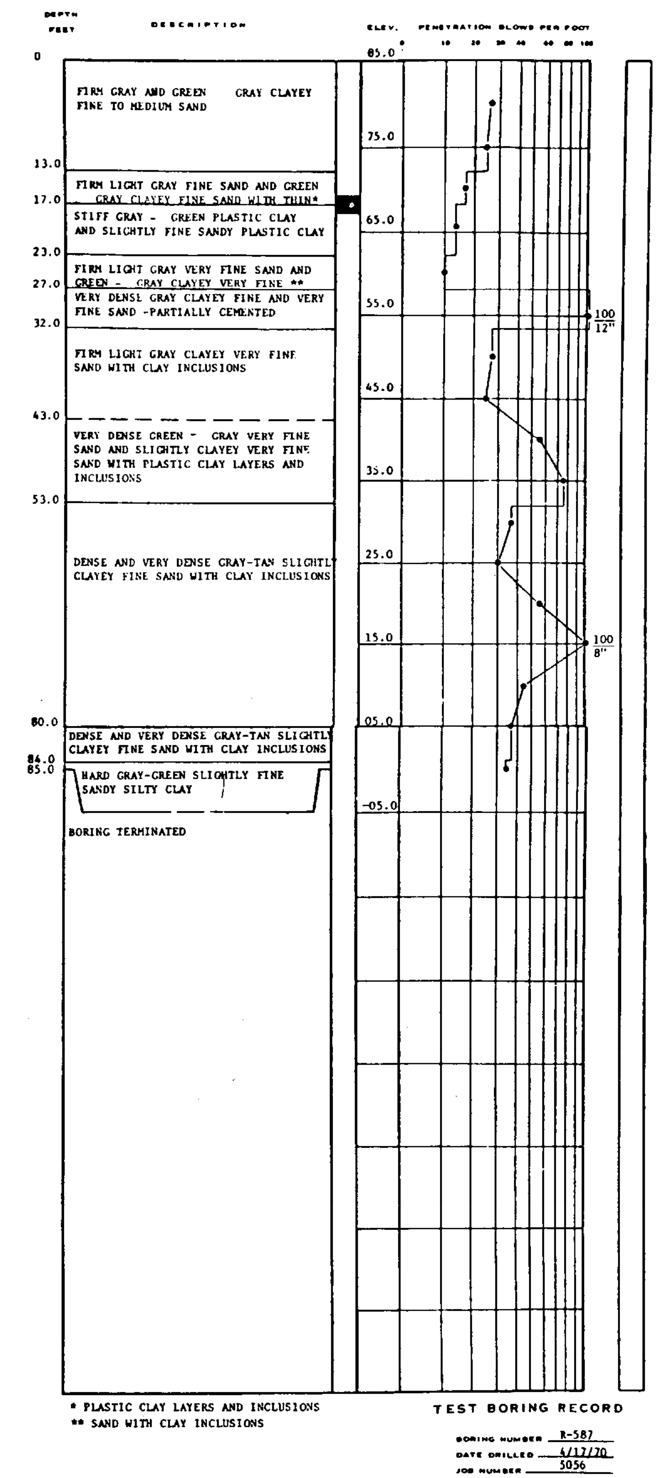
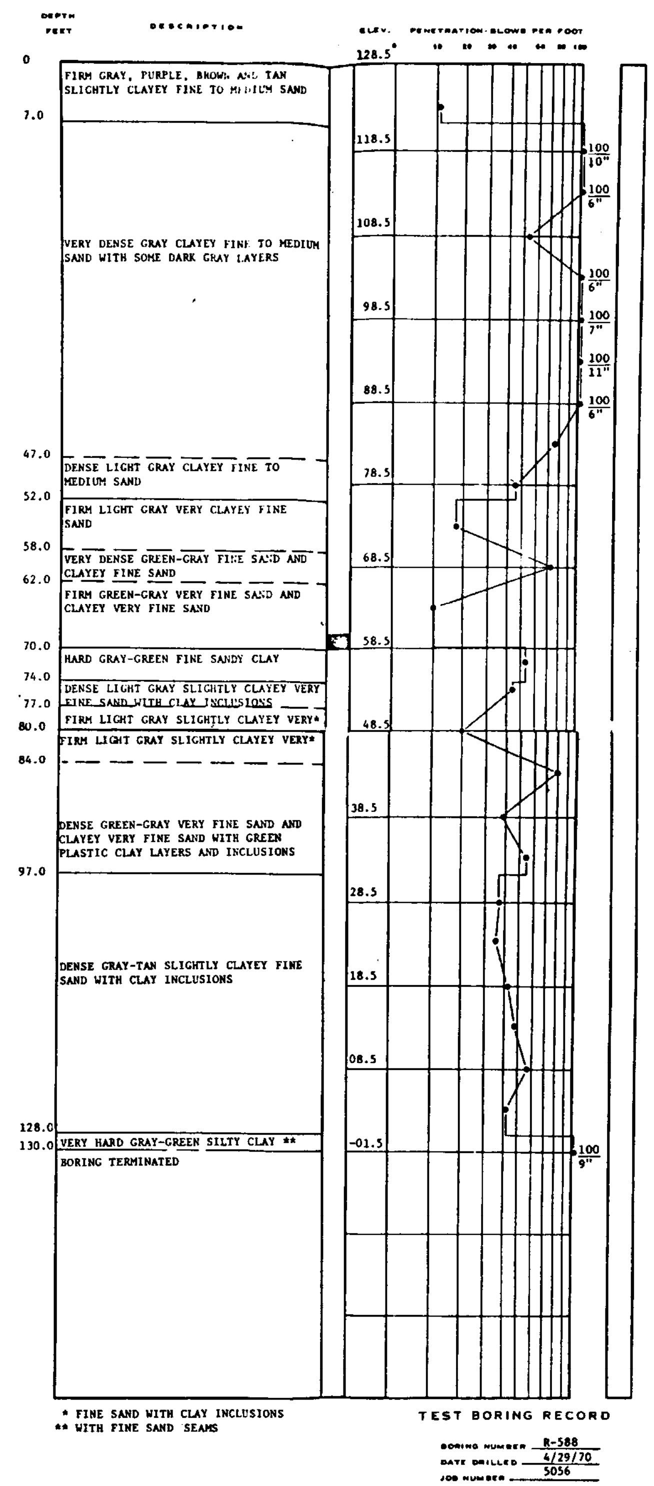
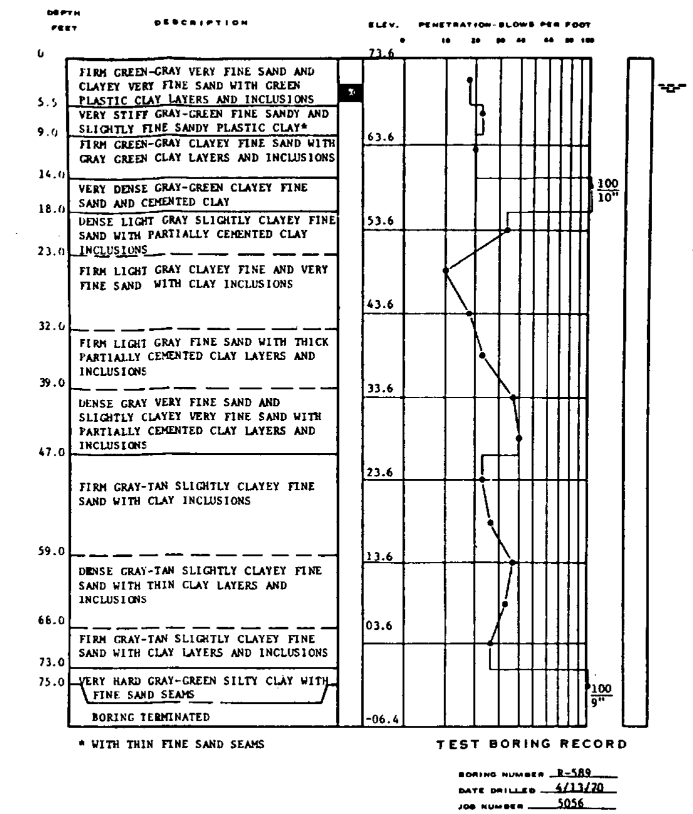
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 586

FIGURE 2B-97







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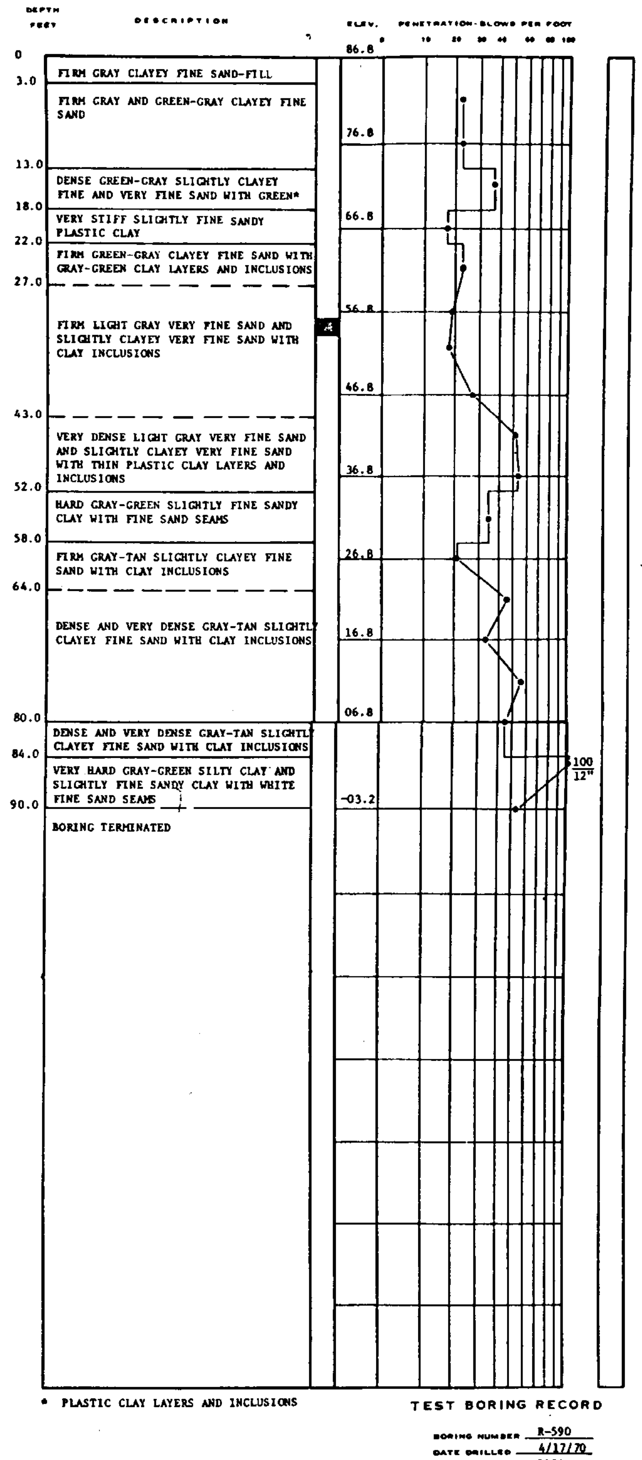
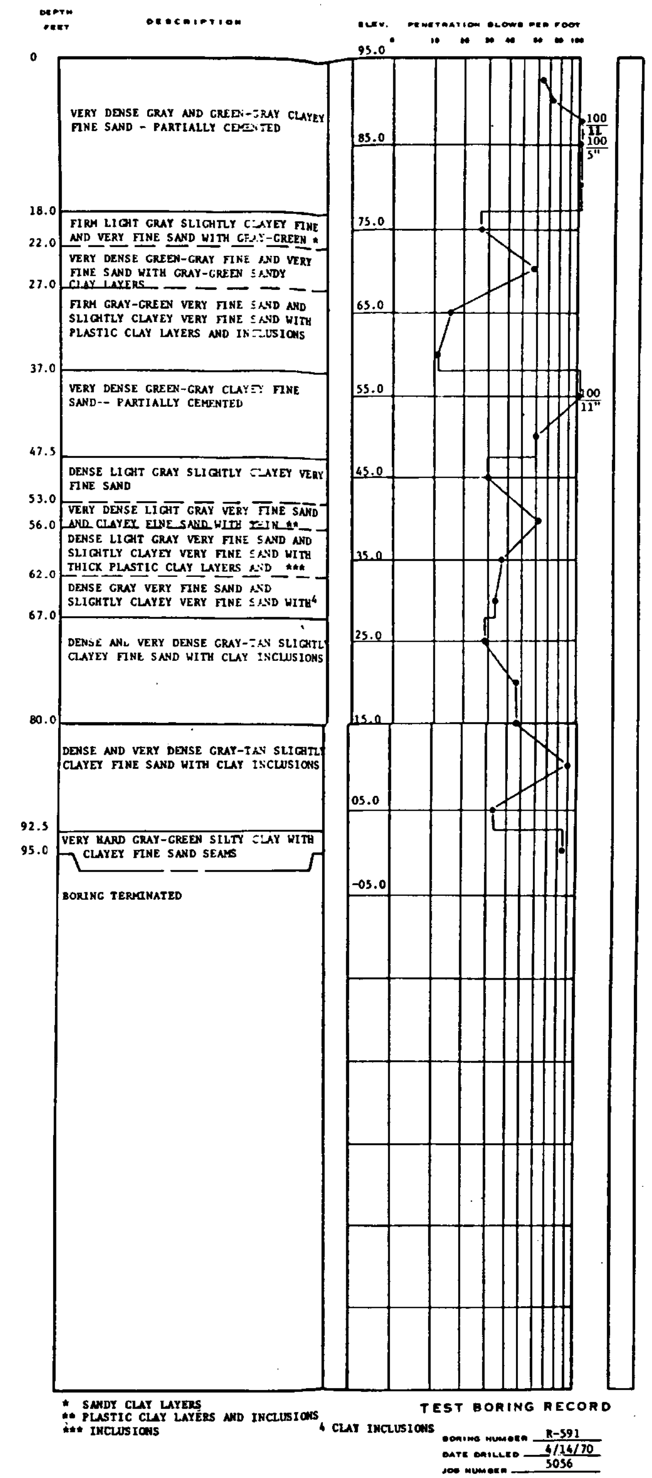
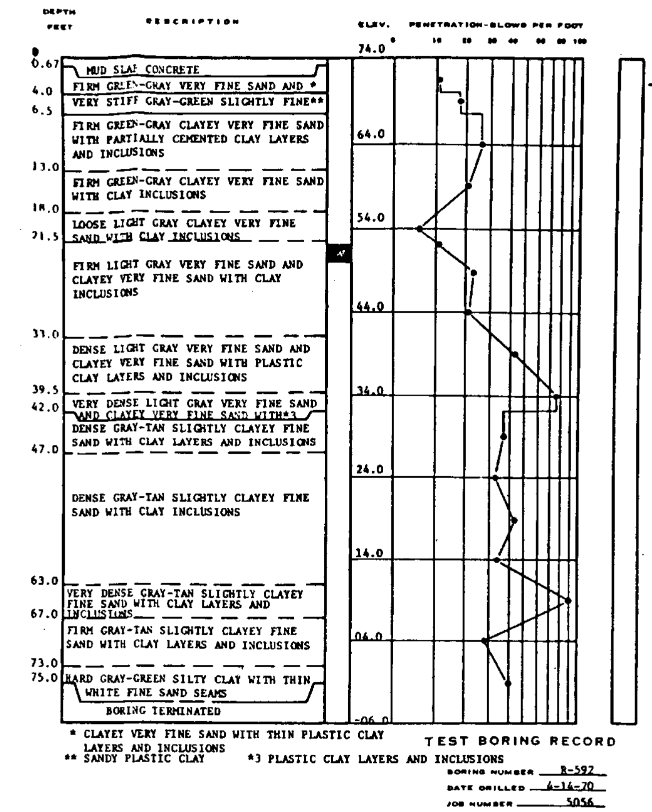
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 587, 588, AND 589

FIGURE 2B-98







HISTORICAL

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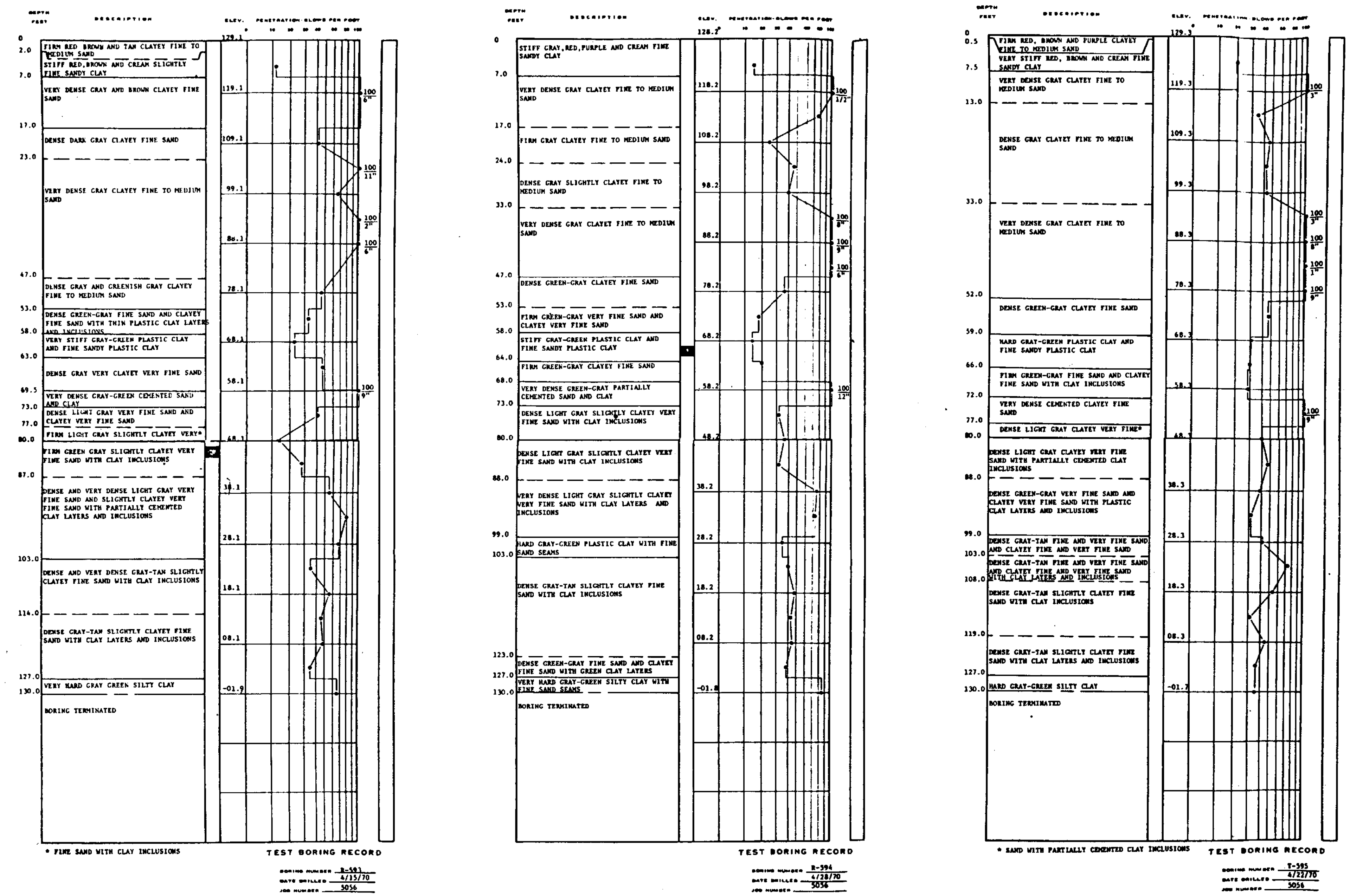
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 590, 591, AND 592

FIGURE 2B-99



HISTORICAL

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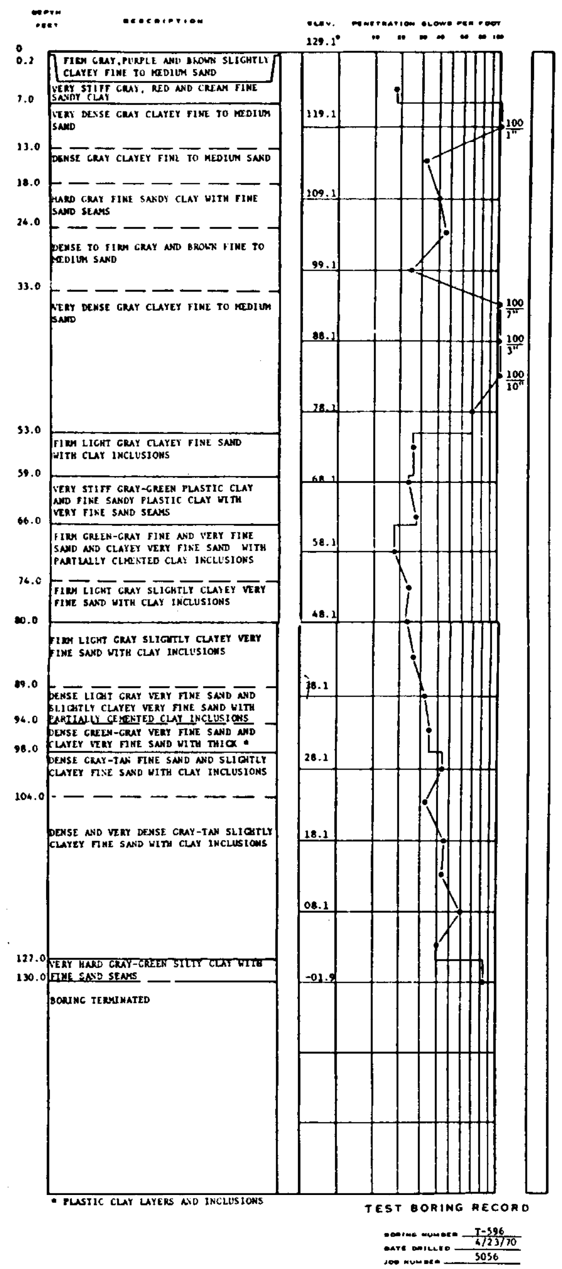
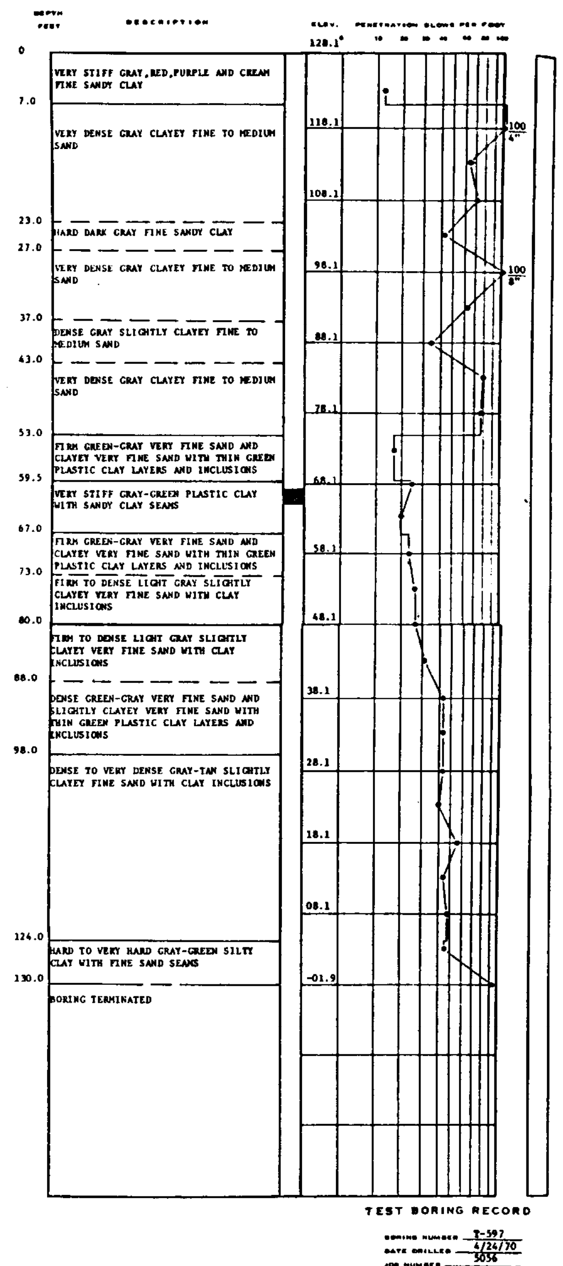
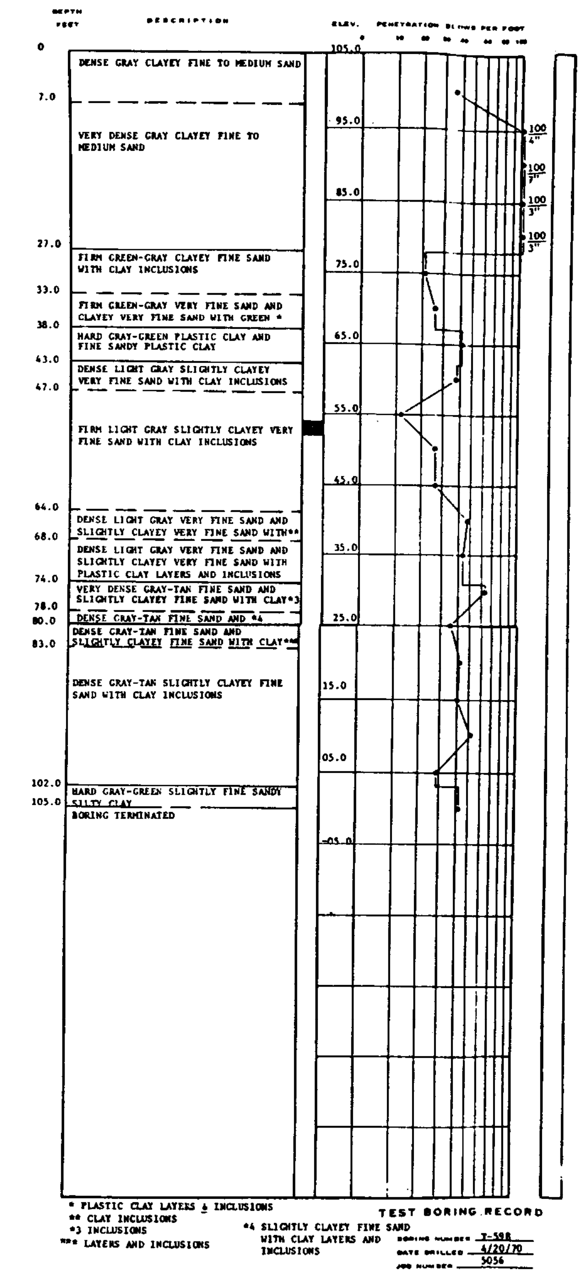
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 593, 594, AND 595

FIGURE 2B-100







HISTORICAL

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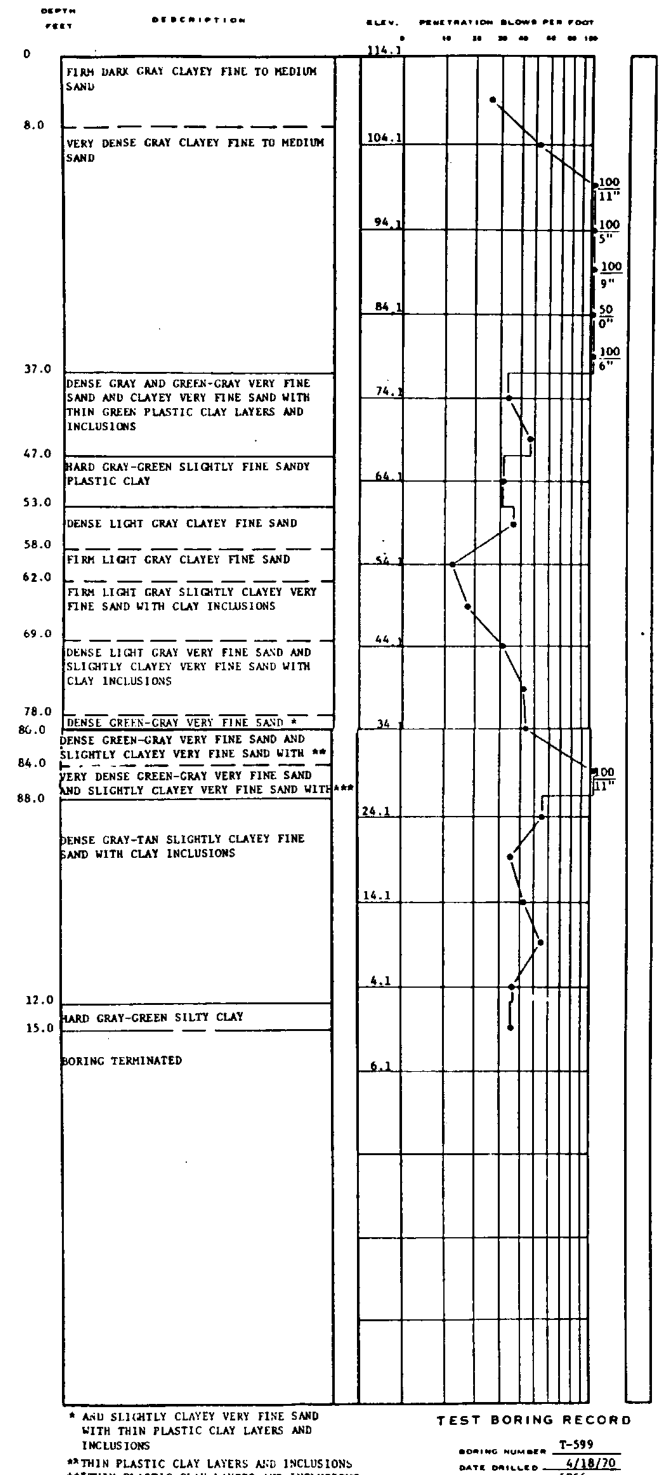
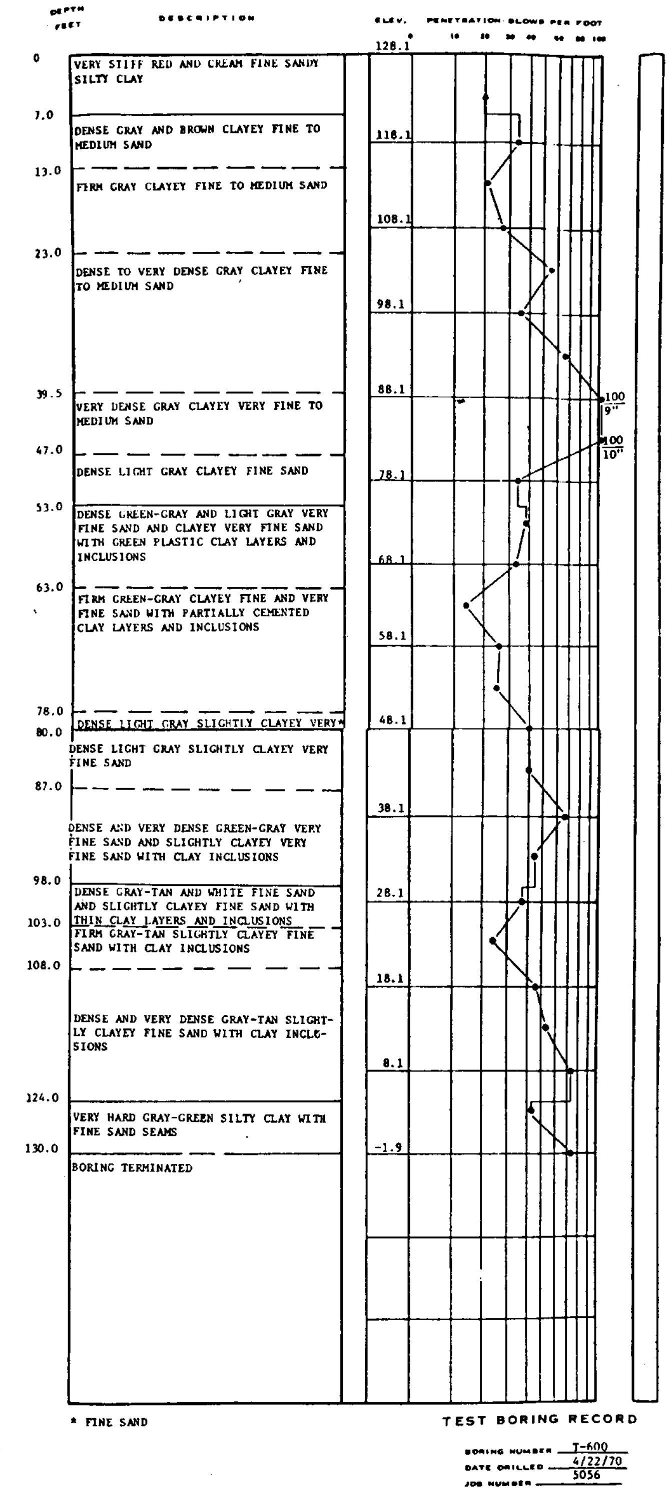
SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
CORNIG NOS. 596, 597, AND 598

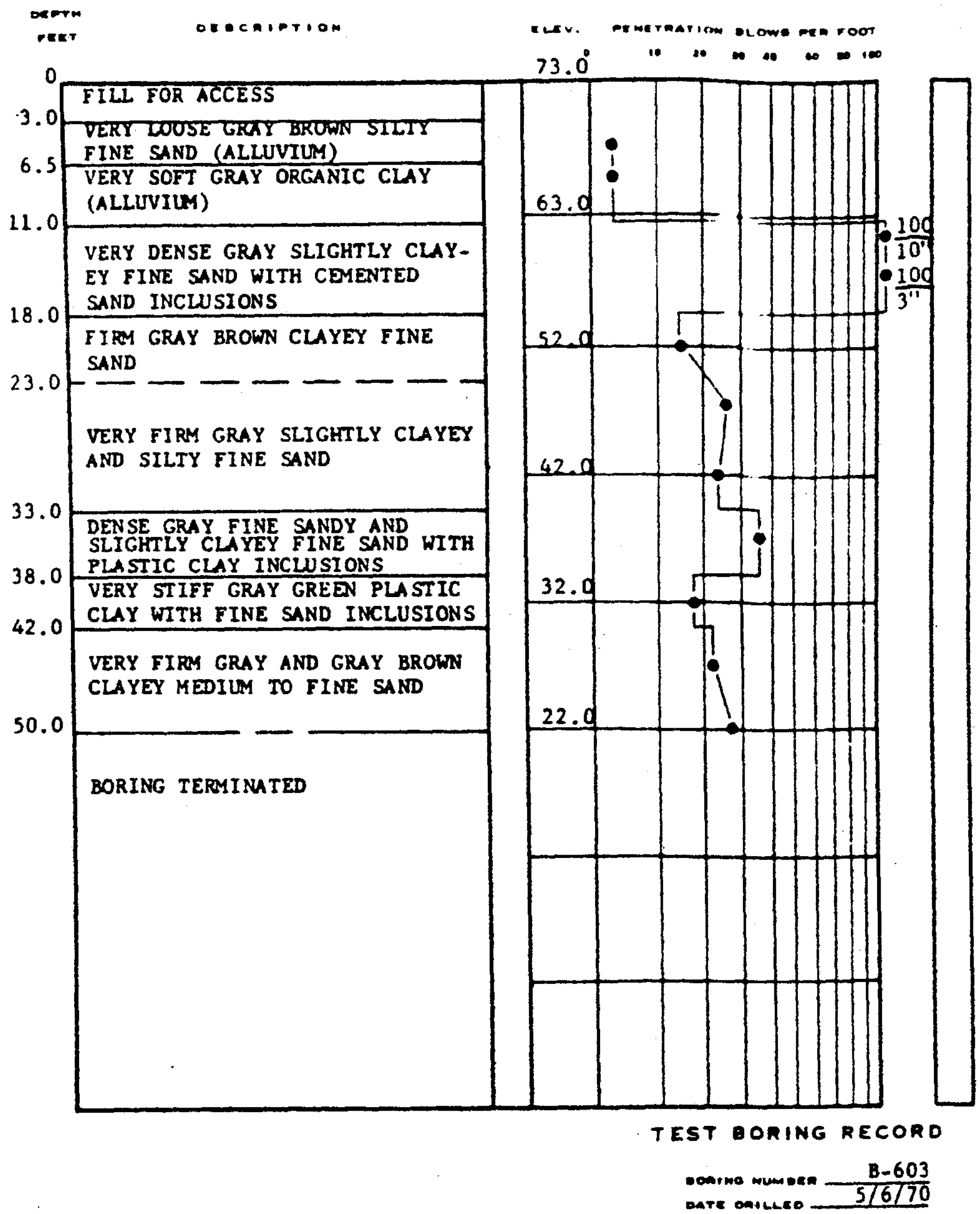
FIGURE 2B-101





HISTORICAL
REV 19 7/01

SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TEST BORING RECORD CORING NOS. 599 AND 600
		FIGURE 2B-102



ACAD

HISTORICAL

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SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. 603

FIGURE 2B-103

GEOLOGIC LOG — BORING NO. 1001 — LOCATION N. 64 + 18 E. 30 + 47

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
125.2	Gray, brown, and red silty clayey fine to coarse SAND and sandy CLAY, with organic material in upper 6 feet.	COLLUVIUM	
113.2	Green-gray clayey fine to medium SAND.	HAWTHORN FORMATION	UNIT 6
92.7	Gray and green fine to coarse sandy partially cemented CLAY.	HAWTHORN FORMATION	UNIT 5
77.2	Green, gray, brown, and tan slightly clayey very fine to coarse SAND with clay inclusion.	HAWTHORN FORMATION	UNIT 4
67.2	Green-gray silty very fine to fine SAND with clayey silt layers and cemented clayey sand seams.	HAWTHORN FORMATION	UNIT 4
42.2	Gray-tan silty slightly clayey fine to medium SAND with partially cemented clay layers and inclusions.	HAWTHORN FORMATION	UNIT 4
28.2	Gray-tan cemented fine to medium sandy CLAY and cemented SAND with clay inclusions.	HAWTHORN FORMATION	UNIT 3
9.7	Light green cemented fine sandy silty CLAY with fine sand seams	HAWTHORN FORMATION	UNIT 2
4.7	Bottom at 4.7 ft MSL		

DEPTH scale: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150

BORING NUMBER 1001

GEOLOGIC LOG — BORING NO. 1002 — LOCATION N. 61 + 95 E. 34 + 42

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
114.0	Tan to green-gray clayey fine to medium SAND.	ALLUVIUM	
92.0	Green-gray very clayey fine to coarse SAND with cemented clay inclusions.	HAWTHORN FORMATION	UNIT 5
79.5	Gray-green fine sandy CLAY with fine sand seams.	HAWTHORN FORMATION	UNIT 4
67.5	Green-gray silty clayey fine SAND with partially cemented sandy clay layers.	HAWTHORN FORMATION	UNIT 4
45.5	Green-gray clayey silty fine SAND with clay inclusions.	HAWTHORN FORMATION	UNIT 4
36.0	Gray-tan clayey silty fine to medium SAND with partially cemented clay and fine sand layers.	HAWTHORN FORMATION	UNIT 3
6.0	Gray-green fine sandy silty CLAY with white fine sand seams.	HAWTHORN FORMATION	UNIT 2
(-1.5)	Bottom at (-1.5) ft MSL.		

DEPTH scale: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150

BORING NUMBER 1002

GEOLOGIC LOG — BORING NO. 1003 — LOCATION N. 57 + 00 E. 34 + 42

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
141.0	Gray and brown clayey fine to coarse SAND and sandy CLAY.	HAWTHORN FORMATION	UNIT 6
124.0	Gray clayey partially cemented fine to coarse SAND and sandy CLAY.	HAWTHORN FORMATION	UNIT 5
78.0	Gray and green silty clayey very fine to fine SAND with clay layers and inclusions.	HAWTHORN FORMATION	UNIT 4
34.0	Gray and tan clayey silty fine to medium SAND with clay inclusions and some clay layers.	HAWTHORN FORMATION	UNIT 3
4.0	Gray fine sandy silty CLAY with fine sand seams.	HAWTHORN FORMATION	UNIT 2
0.5	Bottom at 0.5 ft MSL		

DEPTH scale: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150

BORING NUMBER 1003

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UNIT 2

TEST BORING RECORD
BORING NOS. 1001, 1002, AND 1003

FIGURE 2B-104

GEOLOGIC LOG — BORING NO. 1006 — LOCATION N. 50 + 62 E. 38 + 63

DEPTH	ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
	128.0	Red, brown, and gray clayey fine to medium SAND.	HAWTHORN FORMATION	UNIT 6
	111.5	Gray and gray-green cemented fine sandy CLAY and clayey SAND.	HAWTHORN FORMATION	UNIT 5
	76.5	Gray and gray-green fine sandy CLAY and clayey fine SAND with clay layers, sand seams, and partially cemented clay inclusions.	HAWTHORN FORMATION	UNIT 4
	36.0	Gray-tan silty clayey fine to medium SAND with partially cemented clay and sandy clay inclusions.	HAWTHORN FORMATION	UNIT 3
	5.0	Green fine sandy silty CLAY with fine sand seams.	HAWTHORN FORMATION	UNIT 2
	(-2.5)	Bottom at (-2.5) ft MSL		

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150

BORING NUMBER 1006

GEOLOGIC LOG — BORING NO. 1005 — LOCATION N. 48 + 05 E. 34 + 42

DEPTH	ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
	148.0	FILL: clayey fine to medium sand.	FILL	
	124.0	Gray-brown fine sandy CLAY.	HAWTHORN FORMATION	UNIT 6
	121.0	Gray clayey fine to coarse SAND with cemented sand and clay layers.	HAWTHORN FORMATION	UNIT 5
	76.0	Green-gray clayey silty fine SAND with clay layers and inclusions.	HAWTHORN FORMATION	UNIT 4
	30.5	Gray-tan slightly clayey silty fine SAND with some clay inclusions.	HAWTHORN FORMATION	UNIT 3
	9.0	Light green fine sandy silty CLAY with fine sand seams.	HAWTHORN FORMATION	UNIT 2
	2.5	Bottom at 2.5 ft MSL		

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150

BORING NUMBER 1005

GEOLOGIC LOG — BORING NO. 1004 — LOCATION N. 52 + 29 E. 34 + 42

DEPTH	ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
	144.5	Gray, red, and purple clayey fine to medium SAND.	HAWTHORN FORMATION	UNIT 6
	123.0	Gray to gray-green clayey partially cemented fine to medium SAND and sandy CLAY.	HAWTHORN FORMATION	UNIT 5
	77.0	Gray to gray-green clayey silty fine to medium SAND with clay layers and inclusions.	HAWTHORN FORMATION	UNIT 4
	27.5	Gray-tan silty clayey very fine to fine SAND with clay layers and partially cemented clay inclusions.	HAWTHORN FORMATION	UNIT 3
	7.0	Green slightly fine sandy silty CLAY with fine sand seams.	HAWTHORN FORMATION	UNIT 2
	(-1.0)	Bottom at (-1.0) ft MSL		

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150

BORING NUMBER 1004

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EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 1004, 1005, AND 1006

FIGURE 2B-105

GEOLOGIC LOG — BORING NO. 1007 — LOCATION N. 47 + 20 E. 44 + 80

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT
131.0	Red and gray fine to medium sandy CLAY.	HAWTHORN FORMATION — UNIT 6
124.0	Green-gray clayey fine to coarse SAND.	UNIT 6
116.5	Green-gray cemented clayey fine to coarse SAND with clay inclusions.	UNIT 5
76.0	Green-gray slightly clayey silty fine SAND with clay layers and inclusions. fine sandy silty clay layer, 82.0' - 87.0'	UNIT 4
39.0	Gray-tan silty clayey very fine to medium SAND with occasional cemented clay inclusions.	UNIT 3
8.0	Gray-green slightly fine sandy silty CLAY with fine sand seams.	UNIT 2
1.2	Bottom at 1.2 ft MSL	

BORING NUMBER 1007

GEOLOGIC LOG — BORING NO. 1008 — LOCATION N. 43 + 48 E. 41 + 45

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT
134.0	Gray clayey fine SAND with cemented clay inclusions.	HAWTHORN FORMATION — UNIT 6
127.0	Gray clayey fine to coarse partially cemented SAND with clay layers.	UNIT 5
116.0	Gray and green clayey fine to medium partially cemented SAND with fine sandy clay layers.	UNIT 5
80.0	Gray and tan slightly clayey fine to medium SAND.	UNIT 4
67.0	Gray-green clayey silty fine SAND with some partially cemented clay inclusions.	UNIT 4
46.0	Gray-green silty clayey very fine to fine SAND with clay layers and inclusions.	UNIT 4
21.0	Gray-tan clayey silty fine to medium SAND with clay inclusions.	UNIT 3
2.0	Blue-green silty clayey fine to medium SAND with clay layers.	UNIT 2
(-1.5)	Bottom at (-1.5) ft MSL	

BORING NUMBER 1008

GEOLOGIC LOG — BORING NO. 1009 — LOCATION N. 43 + 35 E. 44 + 60

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT
135.0	Mottled clayey fine to very coarse SAND.	HAWTHORN FORMATION — UNIT 6
124.0	Gray-green fine to medium sandy CLAY with clayey sand seams.	UNIT 6
118.0	Green-gray clayey medium to very coarse SAND.	UNIT 6
105.5	Gray and gray-green partly silty clayey fine SAND.	UNIT 5
92.0	Green-gray cemented silty fine SAND.	UNIT 5
70.0	Green-gray silty clayey fine to medium SAND with cemented clay inclusions.	UNIT 4
48.0	Gray-tan silty clayey very fine to medium SAND with cemented clay and sandy clay layers and inclusions.	UNIT 3
12.0	Blue-green fine sandy silty CLAY with fine to medium sand seams.	UNIT 2
9.5	Bottom at 9.5 ft MSL	

BORING NUMBER 1009

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EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 1007, 1008, AND 1009

FIGURE 2B-106

GEOLOGIC LOG

BORING NO. 1012 | LOCATION N. 64 + 19 E. 25 + 47

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
113.0	Red, gray, and tan clayey fine to coarse SAND.	ALLUVIUM	
94.5	Gray-green very fine sandy partially cemented CLAY.	HAWTHORN FORMATION	UNIT 5
75.0	Green-gray slightly clayey very fine to medium SAND with some partially cemented clay inclusions. very fine to fine sand seams, 69.0' - 76.5'.		UNIT 4
36.5	Gray-tan silty clayey very fine to fine SAND with some clay inclusions. with cemented clay layers, 103.5' - 112.0'		UNIT 3
1.0	Light green fine sandy silty CLAY with fine sand seams.		UNIT 2
(-1.3)	Bottom at (-1.3) ft MSL.		

DEPTH: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150

BORING NUMBER 1012

GEOLOGIC LOG

BORING NO. 1010 | LOCATION N. 45 + 00 E. 47 + 51

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
134.0	Mottled clayey fine to very coarse SAND.		UNIT 6
111.5	Green-gray fine sandy CLAY with cemented clay layers and inclusions and sand seams.	HAWTHORN FORMATION	UNIT 5
96.5	Green-gray slightly clayey fine to very coarse SAND.		
71.5	Green-gray silty clayey very fine to fine SAND with silty clay layers and inclusions.		UNIT 4
37.0	Green-gray cemented silty clayey fine to medium SAND with partially cemented clay inclusions.		UNIT 3
11.0	Blue-green fine sandy silty CLAY with fine to medium sand seams.		UNIT 2
8.5	Bottom at 8.5 ft MSL		

DEPTH: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150

BORING NUMBER 1010

GEOLOGIC LOG

BORING NO. 1011 | LOCATION N. 41 + 90 E. 47 + 40

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
133.0	Mottled clayey fine to coarse SAND with some sandy clay inclusions and layers.		UNIT 6
100.0	Gray-green cemented very fine to fine sandy CLAY and clayey SAND with cemented clay inclusions and sand seams.	HAWTHORN FORMATION	UNIT 5
80.5	Green-gray silty clayey fine SAND with sandy clay layers and inclusions.		UNIT 4
70.0	Gray-green sandy silty CLAY with partially cemented fine sandy clay layers.		
61.0	Green-gray silty clayey fine SAND with partially cemented clay inclusions.		
31.0	Gray and tan silty clayey partly cemented fine to medium SAND.		UNIT 3
20.5	Gray, tan and green fine to medium SAND with some clay inclusions.		
1.0	Blue-green fine to medium sandy silty CLAY with sand seams.		UNIT 2
(-1.8)	Bottom at (-1.8) ft MSL		

DEPTH: 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140, 150

BORING NUMBER 1011

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UNIT 2

TEST BORING RECORD
CORING NOS. 1010, 1011, AND 1012

FIGURE 2B-107

GEOLOGIC LOG — BORING NO. 1013

LOCATION: N. 39 + 95 E. 44 + 40

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
131.0	Mottled clayey fine to coarse SAND.	HAWTHORN FORMATION	UNIT 6
112.0	Gray and brown fine to medium sandy CLAY.	HAWTHORN FORMATION	UNIT 6
100.0	Gray silty partially cemented fine to medium SAND with cemented clay layers.	HAWTHORN FORMATION	UNIT 5
79.0	Green-gray slightly clayey silty fine SAND with fine sandy clay layers and clay inclusions.	HAWTHORN FORMATION	UNIT 4
29.0	Gray-tan fine SAND with partially cemented clay layers.	HAWTHORN FORMATION	UNIT 3
18.0	Gray-tan slightly clayey fine to medium SAND with clay inclusions.	HAWTHORN FORMATION	UNIT 3
3.0	Blue-green fine sandy CLAY with fine sand seams.	HAWTHORN FORMATION	UNIT 2
(-3.5)	Bottom at (-3.5) ft MSL		

BORING NUMBER 1013

GEOLOGIC LOG — BORING NO. 1014

LOCATION: N.38 + 75 E. 48 + 35

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
139.0	Red clayey fine to coarse SAND.	HAWTHORN FORMATION	UNIT 6
132.5	Purple and red fine to coarse sandy CLAY with fine to coarse sand seams.	HAWTHORN FORMATION	UNIT 6
122.0	Tan and brown fine to medium sandy cemented CLAY.	HAWTHORN FORMATION	UNIT 5
112.5	Green-gray clayey fine to coarse partially cemented SAND with clay inclusions.	HAWTHORN FORMATION	UNIT 5
101.5	Gray-green fine to medium sandy cemented CLAY.	HAWTHORN FORMATION	UNIT 5
82.0	Green and gray silty slightly clayey very fine to fine SAND with clay layers and inclusions.	HAWTHORN FORMATION	UNIT 4
27.0	Gray-tan silty slightly clayey very fine to medium SAND with clay inclusions.	HAWTHORN FORMATION	UNIT 3
3.0	Blue-green silty CLAY with fine sand seams.	HAWTHORN FORMATION	UNIT 2
(-0.5)	Bottom at (-0.5) ft MSL		

BORING NUMBER 1014

GEOLOGIC LOG — BORING NO. 1015

LOCATION: N. 39 + 60 E. 51 + 30

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT	
130.0	Red, gray, and purple clayey fine to medium SAND.	HAWTHORN FORMATION	UNIT 6
113.0	Green-gray fine sandy silty CLAY and clayey fine to coarse SAND.	HAWTHORN FORMATION	UNIT 6
89.0	Gray clayey cemented fine to medium SAND and cemented sandy CLAY.	HAWTHORN FORMATION	UNIT 5
78.0	Gray and gray-green clayey fine SAND with clay inclusions and sandy clay layers.	HAWTHORN FORMATION	UNIT 4
25.0	Green-gray cemented clayey silty fine SAND.	HAWTHORN FORMATION	UNIT 4
22.0	Gray-tan silty clayey fine SAND with clay layers and inclusions.	HAWTHORN FORMATION	UNIT 3
2.5	Blue-green and gray fine sandy silty CLAY with fine sand seams.	HAWTHORN FORMATION	UNIT 2
(-5.5)	Bottom at (-5.5) ft MSL		

BORING NUMBER 1015

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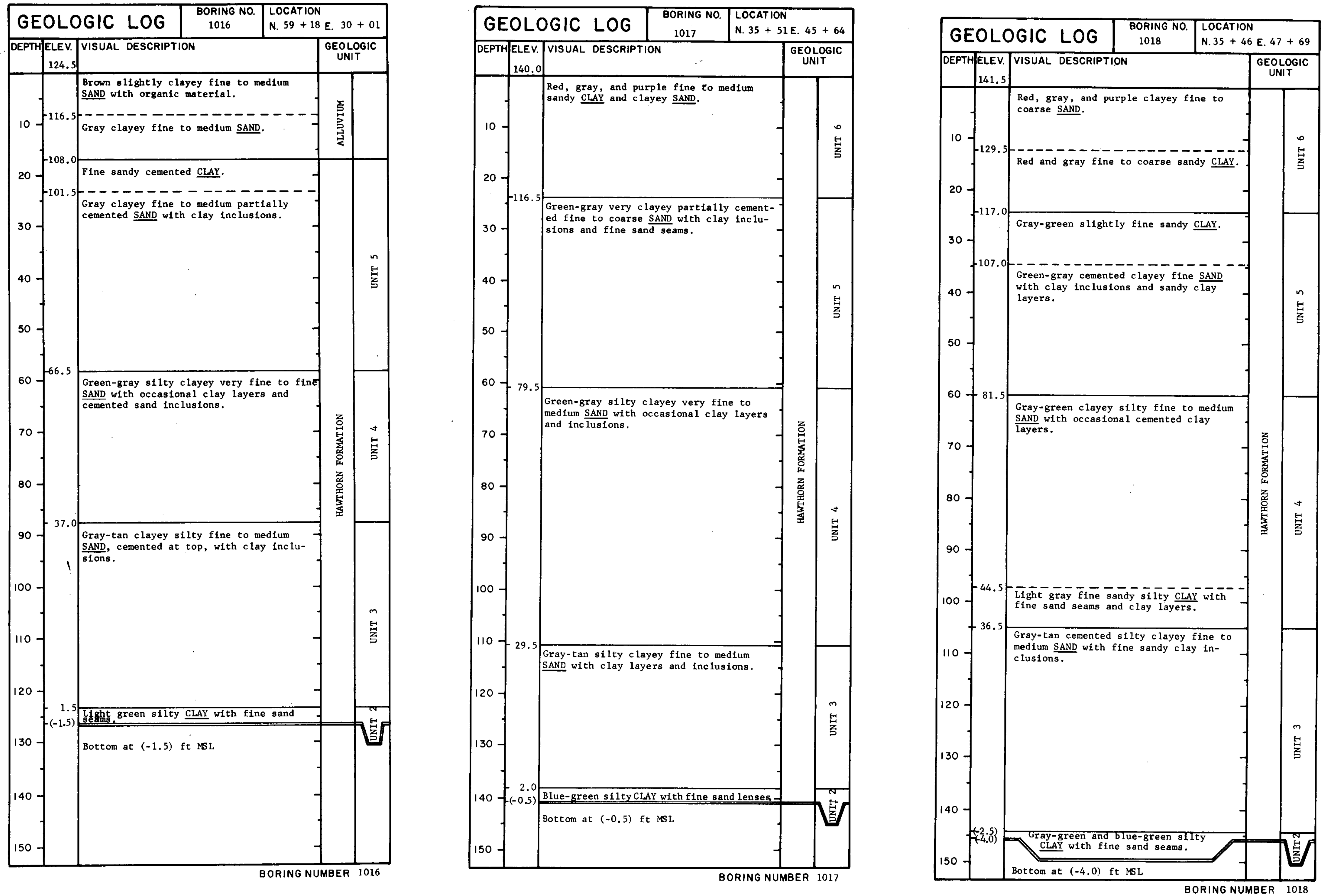
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 1013, 1014, AND 1015

FIGURE 2B-108



HISTORICAL

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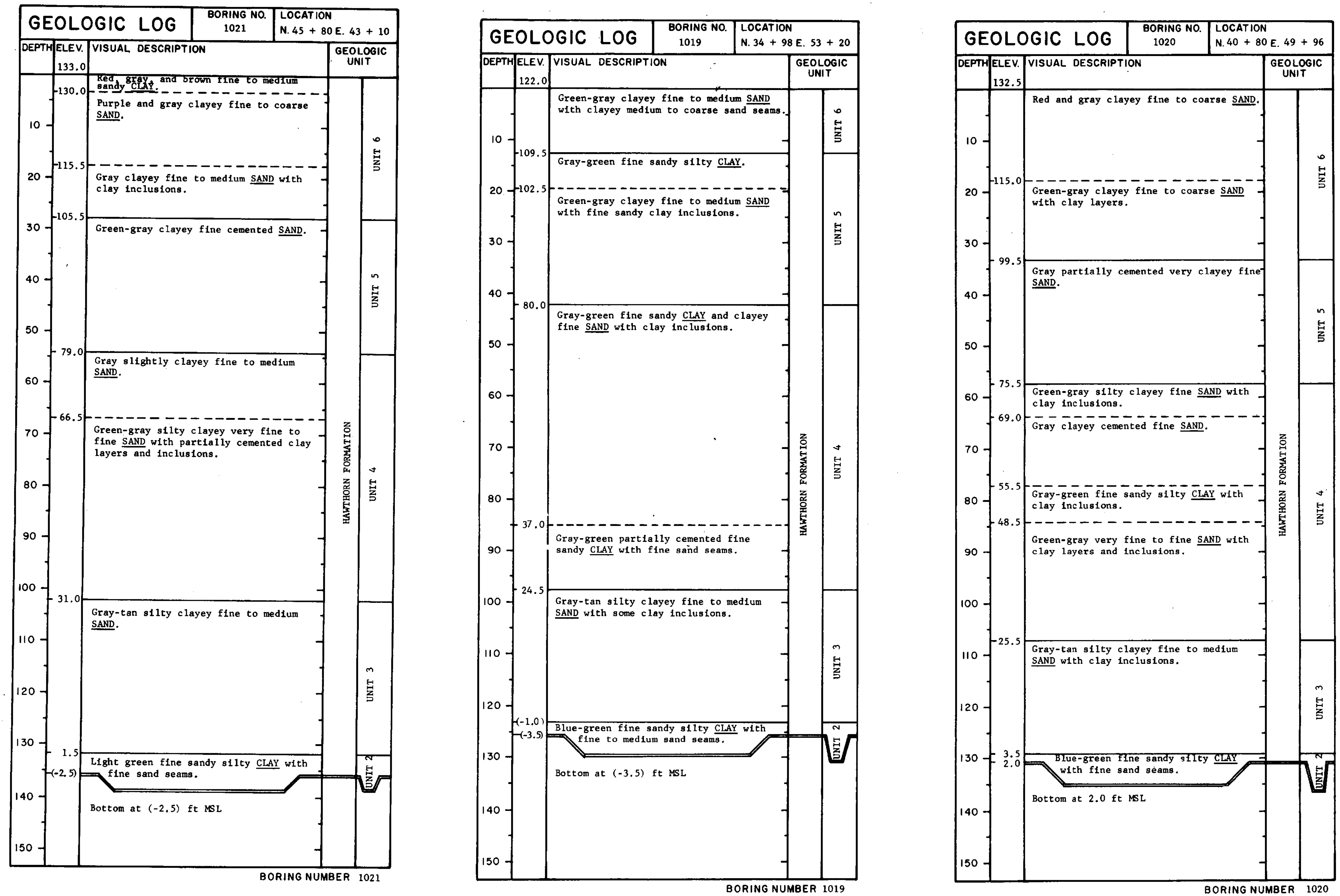
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 1016, 1017, AND 1018

FIGURE 2B-109



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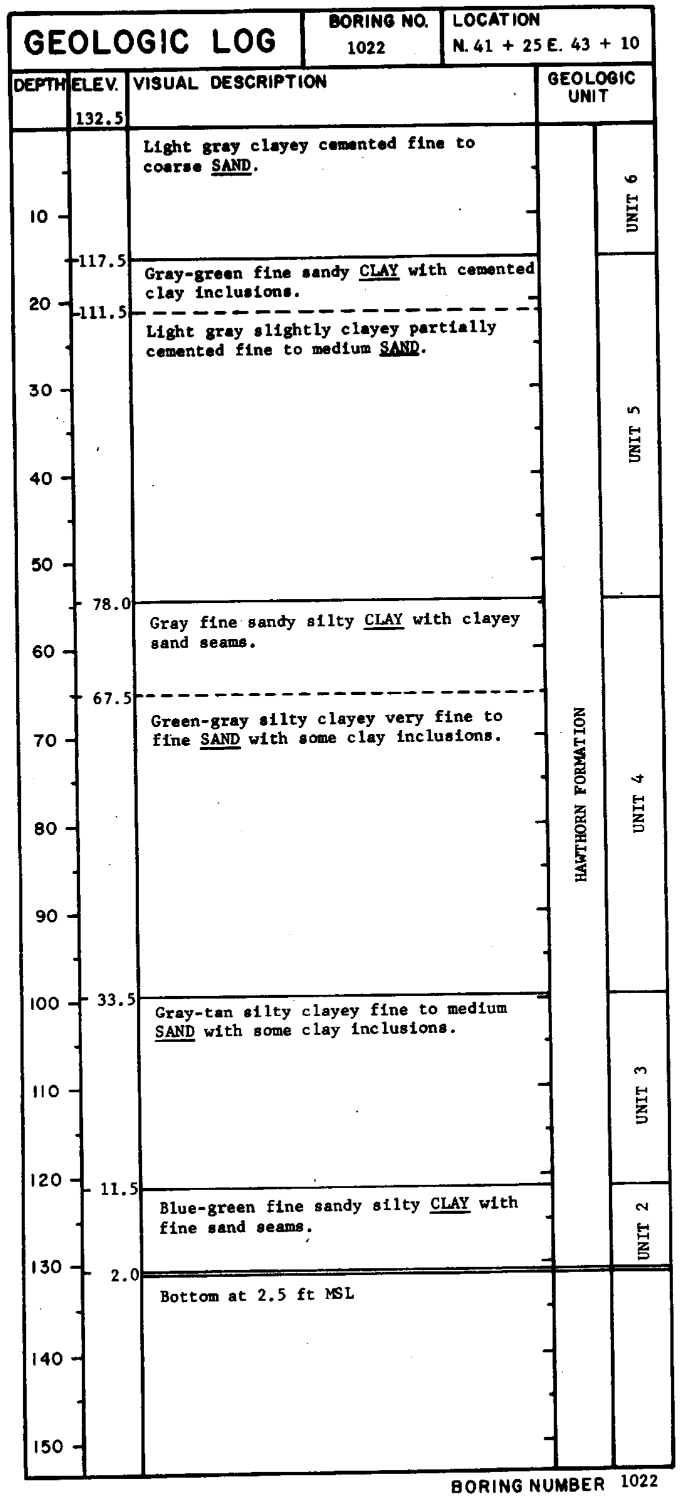
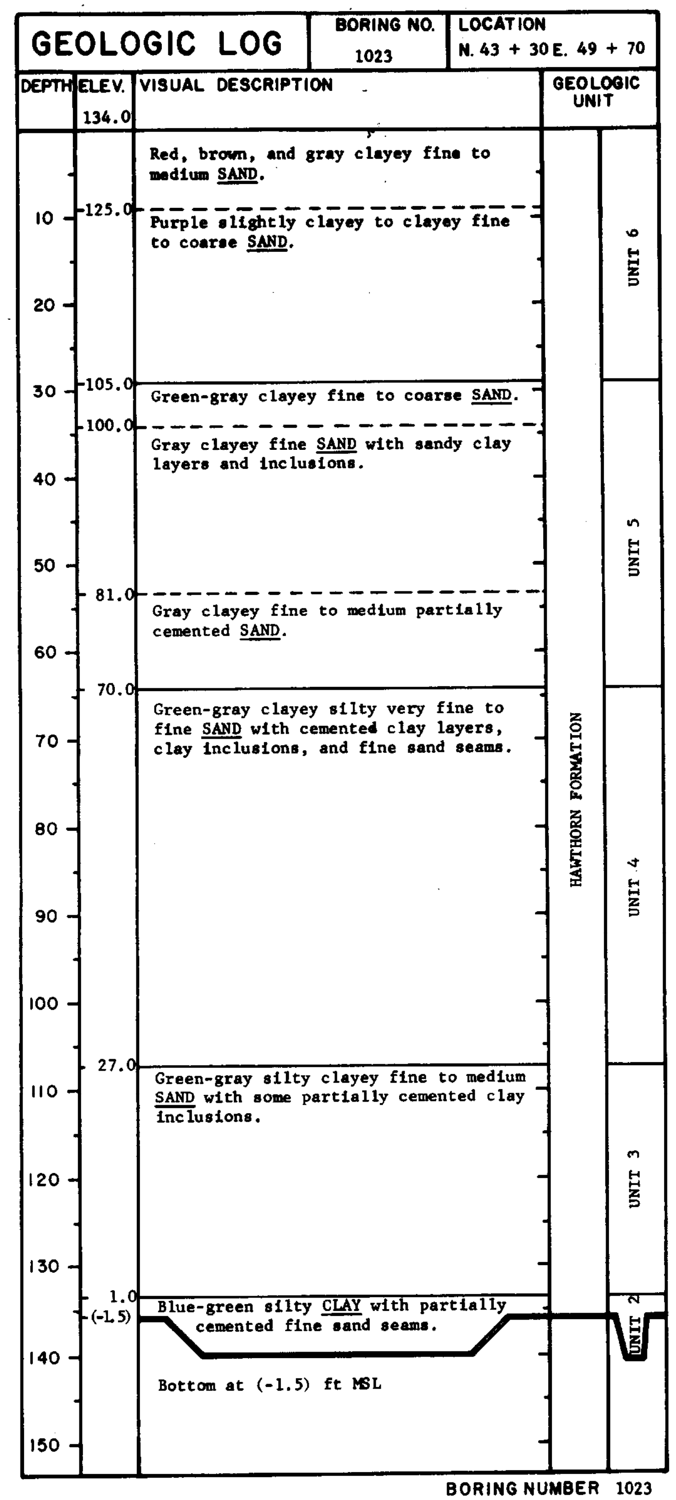
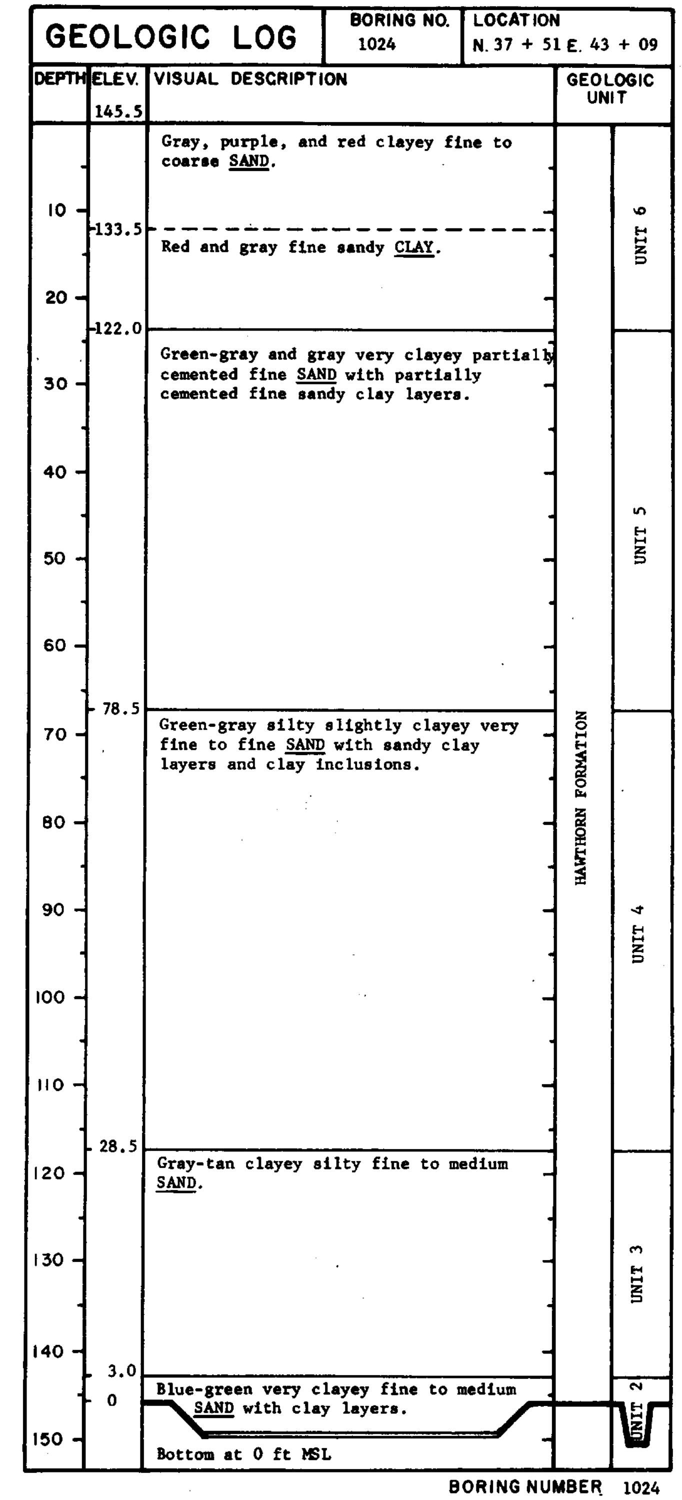
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TEST BORING RECORD BORING NOS. 1019, 1020, AND 1021
		FIGURE 2B-110







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EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NOS. 1022, 1023, AND 1024

FIGURE 2B-111

GEOLOGIC LOG — BORING NO. 1025 — LOCATION N. 37 + 12 E. 47 + 37

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT
143.5	Red, gray, and purple fine sandy CLAY.	HAWTHORN FORMATION — UNIT 6
137.0	Red, gray, and purple very clayey fine to medium SAND.	HAWTHORN FORMATION — UNIT 6
128.5	Red and purple fine sandy CLAY.	HAWTHORN FORMATION — UNIT 6
122.5	Gray clayey fine to coarse partially cemented SAND with some clay layers.	HAWTHORN FORMATION — UNIT 5
96.0	Gray-green slightly fine sandy cemented CLAY.	HAWTHORN FORMATION — UNIT 5
90.5	Green-gray clayey fine to medium SAND with clay inclusions.	HAWTHORN FORMATION — UNIT 5
81.5	Green-gray silty clayey very fine to fine SAND with clay layers and inclusions.	HAWTHORN FORMATION — UNIT 4
42.0	Green-gray silty partially cemented CLAY with fine sand seams.	HAWTHORN FORMATION — UNIT 4
32.5	Green-gray clayey fine SAND with clay inclusions.	HAWTHORN FORMATION — UNIT 4
26.5	Gray-tan silty clayey fine to medium SAND with clay layers and inclusions.	HAWTHORN FORMATION — UNIT 3
5.5	Blue-green fine sandy silty CLAY.	HAWTHORN FORMATION — UNIT 2
3.0	Bottom at 3.0 ft MSL	

BORING NUMBER 1025

GEOLOGIC LOG — BORING NO. 1026 — LOCATION N.39 + 82 E. 41 + 33

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT
132.0	Red, brown, purple, and gray clayey fine to medium SAND and sandy CLAY.	HAWTHORN FORMATION — UNIT 6
113.5	Gray partially cemented clayey fine SAND.	HAWTHORN FORMATION — UNIT 5
79.5	Green-gray slightly clayey very fine to fine SAND and silty CLAY with clay inclusions and fine sand seams. partially cemented, 74.0' - 78.0'	HAWTHORN FORMATION — UNIT 4
30.5	Gray-tan very fine to fine SAND with clay layers and partially cemented sand layers.	HAWTHORN FORMATION — UNIT 3
19.5	Gray-tan fine to medium SAND with clay inclusions and some cemented sand seams.	HAWTHORN FORMATION — UNIT 3
5.0	Blue-green fine sandy silty CLAY with fine sand seams.	HAWTHORN FORMATION — UNIT 2
1.5	Bottom at 1.5 ft MSL	

BORING NUMBER 1026

GEOLOGIC LOG — BORING NO. 1028 — LOCATION N. 62 + 05 E. 31 + 66

ELEV.	VISUAL DESCRIPTION	GEOLOGIC UNIT
134.0	Gray, brown, and red clayey fine to medium SAND with clay layers.	HAWTHORN FORMATION — UNIT 6
122.0	Gray fine sandy clayey partially cemented SILT.	HAWTHORN FORMATION — UNIT 5
111.5	Gray clayey fine SAND and fine sandy silty partially cemented CLAY.	HAWTHORN FORMATION — UNIT 5
97.0	Gray slightly clayey cemented fine SAND.	HAWTHORN FORMATION — UNIT 5
81.0	Gray slightly clayey fine to medium SAND with clay inclusions.	HAWTHORN FORMATION — UNIT 4
62.0	Green-gray slightly clayey silty very fine to fine SAND with clay layers and inclusions.	HAWTHORN FORMATION — UNIT 4
36.0	Gray-tan slightly clayey fine SAND with clay inclusions.	HAWTHORN FORMATION — UNIT 3
16.0	Light green slightly fine sandy very silty CLAY with very fine to fine sand seams.	HAWTHORN FORMATION — UNIT 2
3.5	Bottom at 3.5 ft MSL	

BORING NUMBER 1028

HISTORICAL
REV 19 7/01

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UNIT 2

TEST BORING RECORD
BORING NOS. 1025, 1026, AND 1028

FIGURE 2B-112

DEPTH FEET

DESCRIPTION

1.2 CONCRETE

FIRM GREEN GRAY FINE SAND AND CLAYEY FINE SAND WITH GREEN *

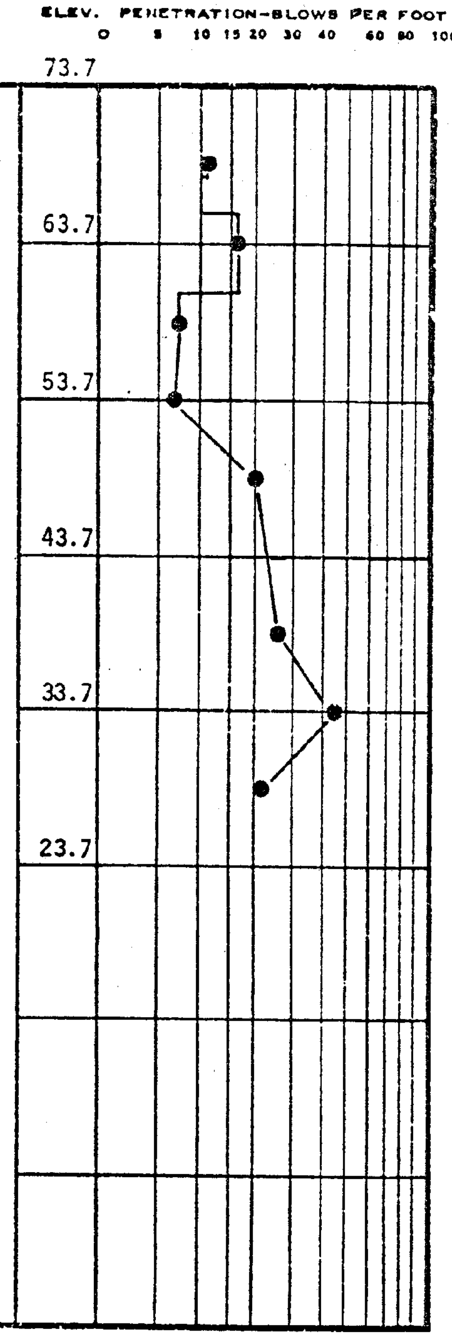
5.5 STIFF GRAY GREEN FINE SANDY PLASTIC CLAY WITH FINE SAND SEAMS

8.0 LOOSE TO FIRM GREEN GRAY VERY FINE SAND AND CLAYEY VERY FINE SAND WITH PLASTIC CLAY LAYERS AND INCLUSIONS

13.0 LOOSE TO FIRM LIGHT GRAY CLAYEY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY INCLUSIONS

27.5 FIRM TO DENSE LIGHT GRAY FINE SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS

45.0 BORING TERMINATED



REMARKS: *PLASTIC CLAY LAYERS AND INCLUSIONS

N- 51+58
E- 49+99

BORING NUMBER RFI-1
DATE DRILLED 10-5-71
JOB NUMBER 5056

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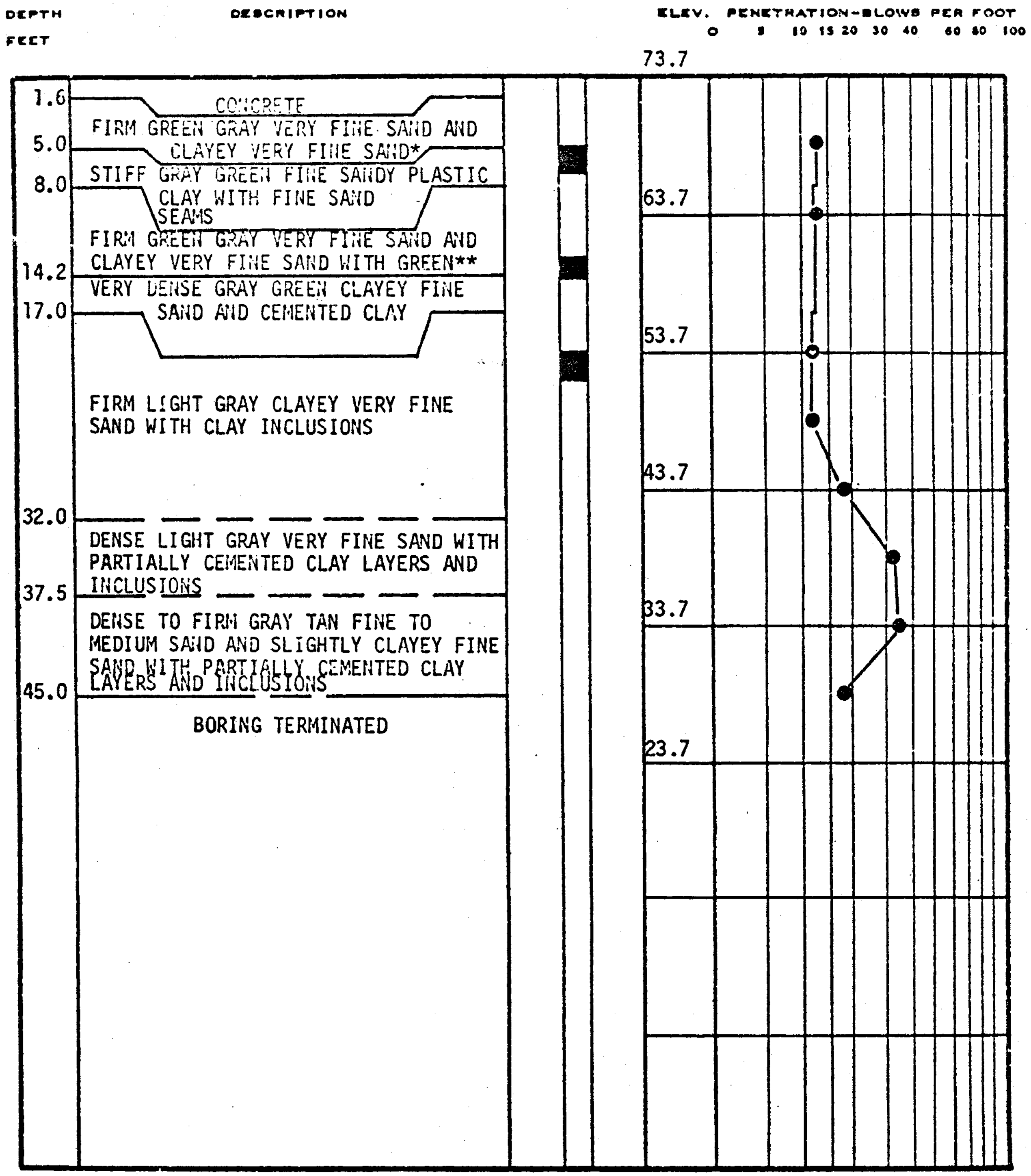
TEST BORING RECORD
BORING NO. RFI-1

FIGURE 2B-113



REMARKS: *WITH GREEN PLASTIC CLAY LAYERS AND INCLUSIONS
**PLASTIC CLAY LAYERS AND INCLUSIONS

N- 51+58
E- 50+57

BORING NUMBER RFI-2
DATE DRILLED 10-4-71
JOB NUMBER 5056

HISTORICAL

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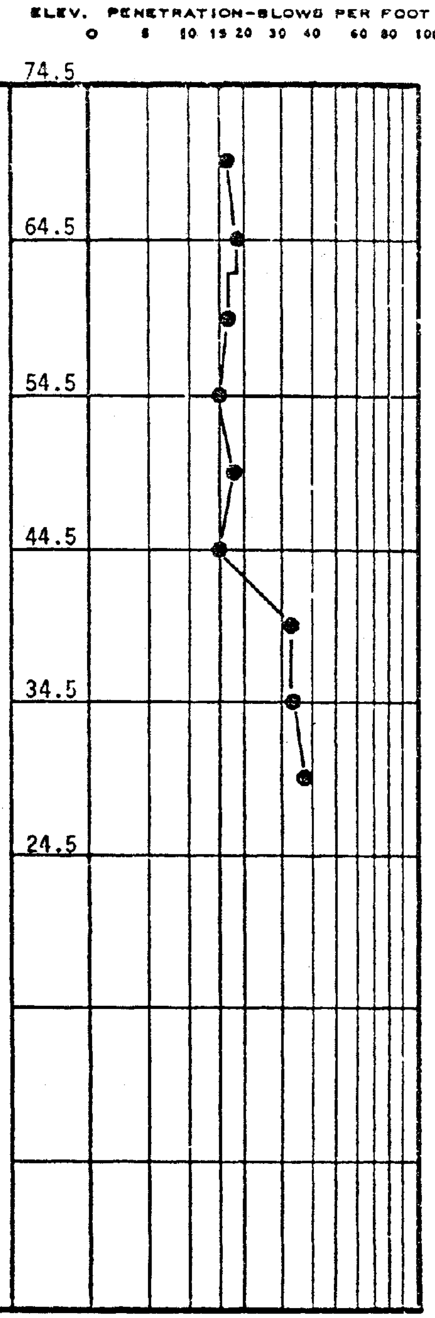
UNIT 2

TEST BORING RECORD
BORING NO. RFI-2

FIGURE 2B-114

DEPTH FEET	DESCRIPTION
2.0	FIRM GREEN GRAY VERY FINE SAND AND CLAYEY VERY FINE SAND WITH THIN PLASTIC CLAY*
12.0	VERY STIFF GRAY GREEN FINE SANDY PLASTIC CLAY WITH FINE SAND SEAMS
18.0	FIRM GREEN GRAY VERY FINE SAND AND CLAYEY VERY FINE SAND WITH THIN PLASTIC CLAY LAYERS AND INCLUSIONS
32.0	FIRM LIGHT GRAY CLAYEY VERY FINE SAND WITH CLAY INCLUSIONS
45.0	DENSE LIGHT GRAY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS
	BORING TERMINATED



REMARKS: *LAYERS AND INCLUSIONS

N-51+22
E-49+70

BORING NUMBER RFI-3
DATE DRILLED 10-6-71
JOB NUMBER 5056

HISTORICAL

REV 19 7/01

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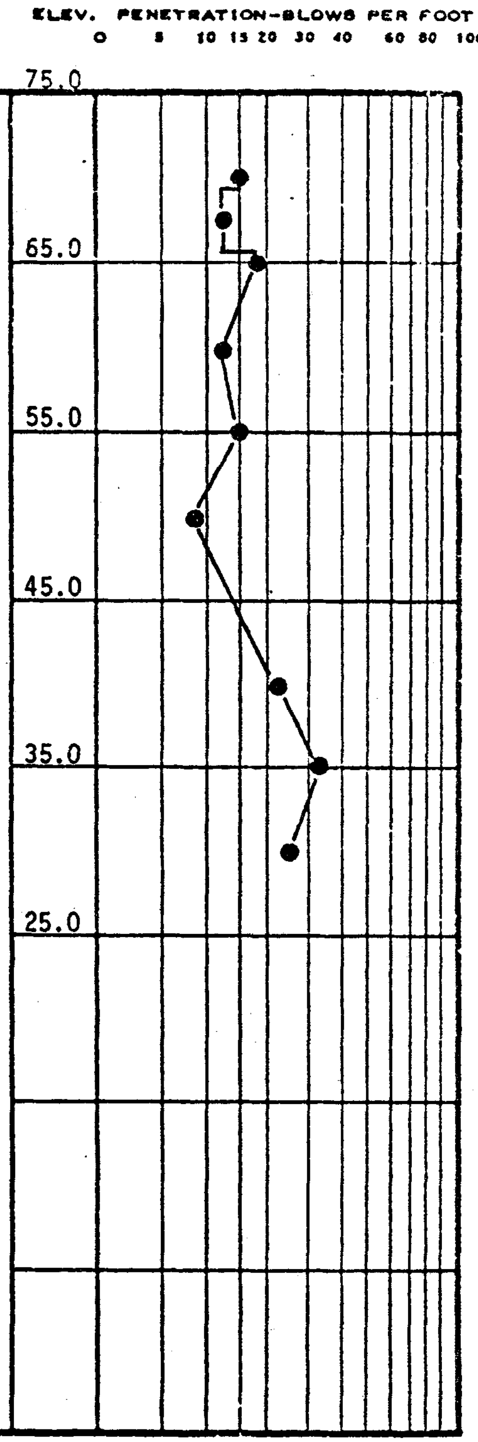
UNIT 2

TEST BORIN RECORD
BORING NO. RFI-3

FIGURE 2B-115

DEPTH FEET	DESCRIPTION
	FIRM GREEN GRAY VERY FINE SAND AND CLAYEY VERY FINE SAND WITH GREEN PLASTIC CLAY LAYERS AND INCLUSIONS
5.5	VERY STIFF GRAY GREEN FINE SANDY PLASTIC CLAY WITH FINE SAND SEAMS
9.5	FIRM GREEN GRAY VERY FINE SAND AND SLIGHTLY CLAYEY VERY FINE SAND WITH GREEN PLASTIC CLAY LAYERS AND *
15.5	LOOSE TO FIRM LIGHT GRAY CLAYEY VERY FINE SAND
32.0	FIRM TO DENSE LIGHT GRAY FINE SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS
41.5	FIRM GRAY TAN FINE SAND WITH PARTIALLY CEMENTED CLAY LAYERS
45.0	



REMARKS: *INCLUSIONS

N-51+22
E-50+28

BORING NUMBER RFI-4
DATE DRILLED 10/11/71
JOB NUMBER 5056

HISTORICAL
REV 19 7/01

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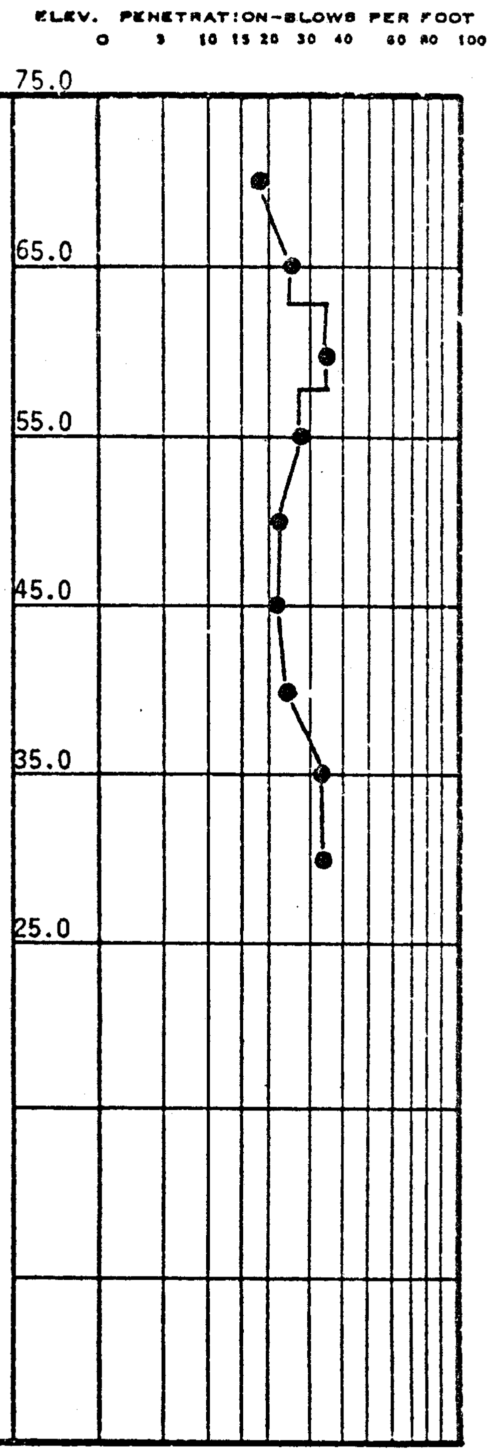
UNIT 2

TEST BORING RECORD
BORING NO. RFI-4

FIGURE 2B-116

DEPTH FEET	DESCRIPTION
5.5	FIRM LIGHT GRAY AND GREEN GRAY FINE SAND WITH GREEN PLASTIC CLAY LAYERS AND INCLUSIONS
12.0	VERY STIFF GRAY GREEN FINE SANDY PLASTIC CLAY WITH FINE SAND SEAMS
17.0	DENSE GREEN GRAY VERY FINE SAND AND CLAYEY VERY FINE SAND WITH CLAY INCLUSIONS
41.5	FIRM TO DENSE LIGHT GRAY VERY FINE SAND AND CLAYEY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS
45.0	DENSE GRAY TAN SLIGHTLY CLAYEY FINE TO MEDIUM SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS
	BORING TERMINATED



REMARKS:

N-51+22
E-50+85

BORING NUMBER RFI-5
DATE DRILLED 10/11/71
JOB NUMBER 5056

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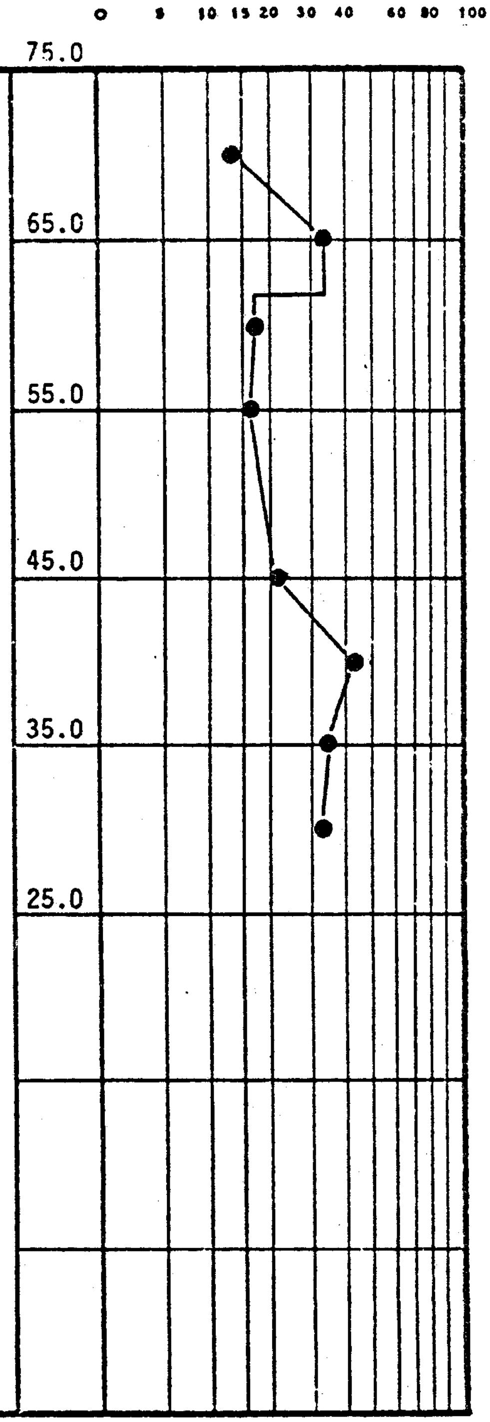
TEST BORING RECORD
BORING NO. RFI-5

FIGURE 2B-117

DEPTH FEET	DESCRIPTION
	VERY STIFF GRAY GREEN FINE SANDY PLASTIC CLAY WITH FINE SAND SEAMS
5.5	HARD GRAY GREEN FINE SANDY CLAY WITH FINE SAND SEAMS
13.0	FIRM GREEN GRAY CLAYEY VERY FINE SAND WITH CLAY INCLUSIONS
32.0	DENSE LIGHT GRAY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS
45.0	BORING TERMINATED

ELEV. PENETRATION—BLOWS PER FOOT



REMARKS:

N-50+86
C-49+99

BORING NUMBER RFI-6
DATE DRILLED 10/7/71
JOB NUMBER 5056

HISTORICAL

REV 19 7/01

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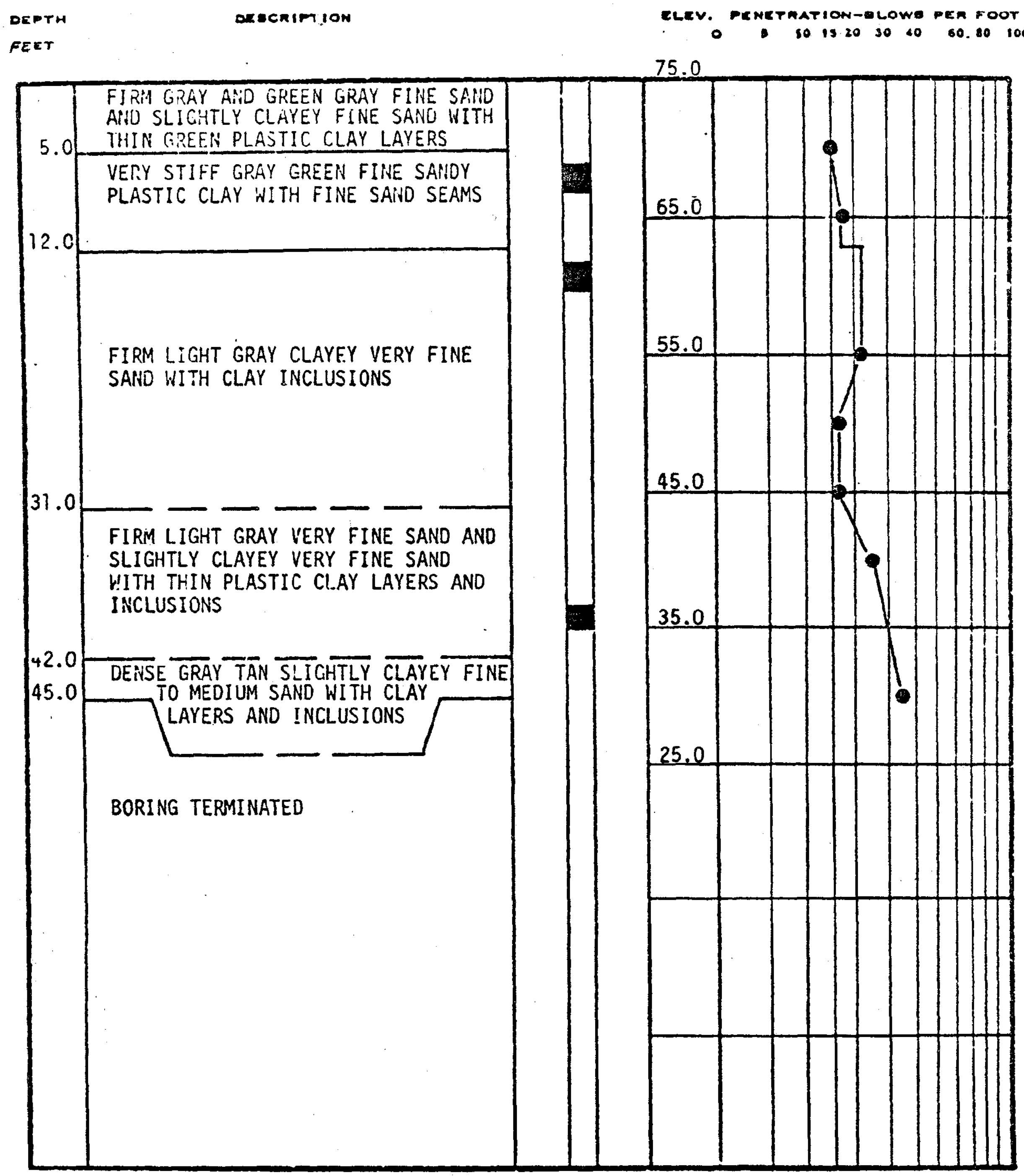
UNIT 2

TEST BORING RECORD
BORING NO. RFI-6

FIGURE 2B-118



REMARKS:

N-50+86
E-50+57

BORING NUMBER RFI-7
DATE DRILLED 10/8/71
JOB NUMBER 5056

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REV 19 7/01

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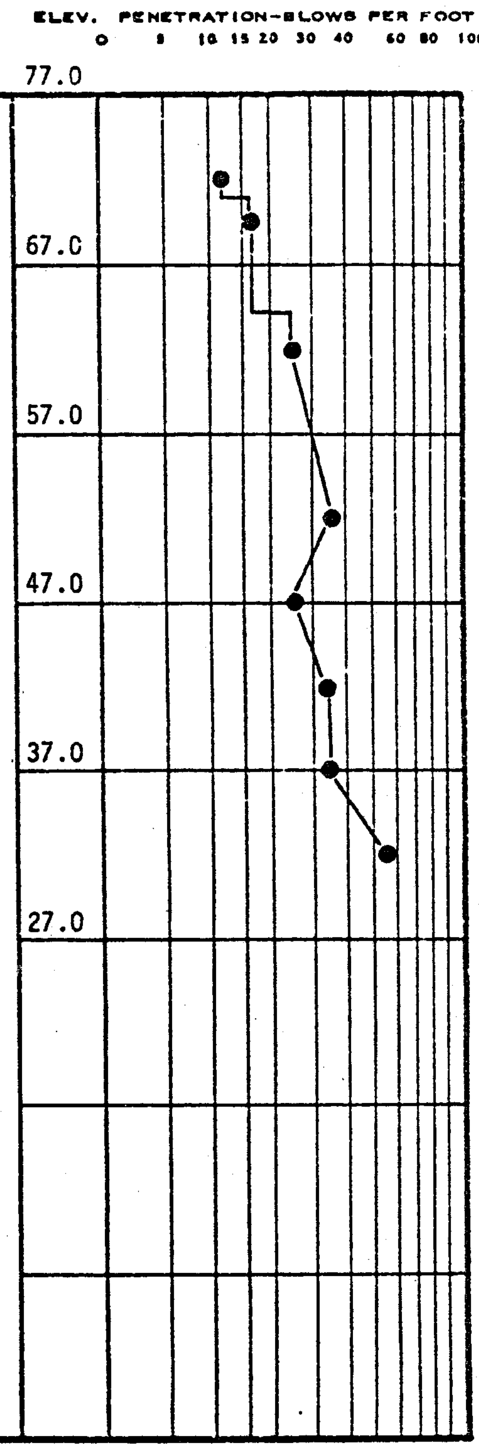
TEST BORING RECORD
BORING NO. RFI-7

FIGURE 2B-119

DEPTH FEET	DESCRIPTION
	FIRM GRAY AND GREEN GRAY CLAYEY FINE SAND WITH GREEN CLAY INCLUSIONS
6.0	VERY STIFF GRAY GREEN FINE SANDY PLASTIC CLAY
12.5	FIRM TO DENSE LIGHT GRAY FINE AND VERY FINE SAND AND SLIGHTLY CLAYEY VERY FINE SAND WITH CEMENTED CLAY LAYERS AND INCLUSIONS
28.5	FIRM TO DENSE LIGHT GRAY SLIGHTLY CLAYEY VERY FINE SAND WITH CLAY INCLUSIONS
45.0	BORING TERMINATED



REMARKS:

N-50+50
E-49+70

BORING NUMBER RFI-8
DATE DRILLED 10/4/71
JOB NUMBER 5056

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REV 19 7/01

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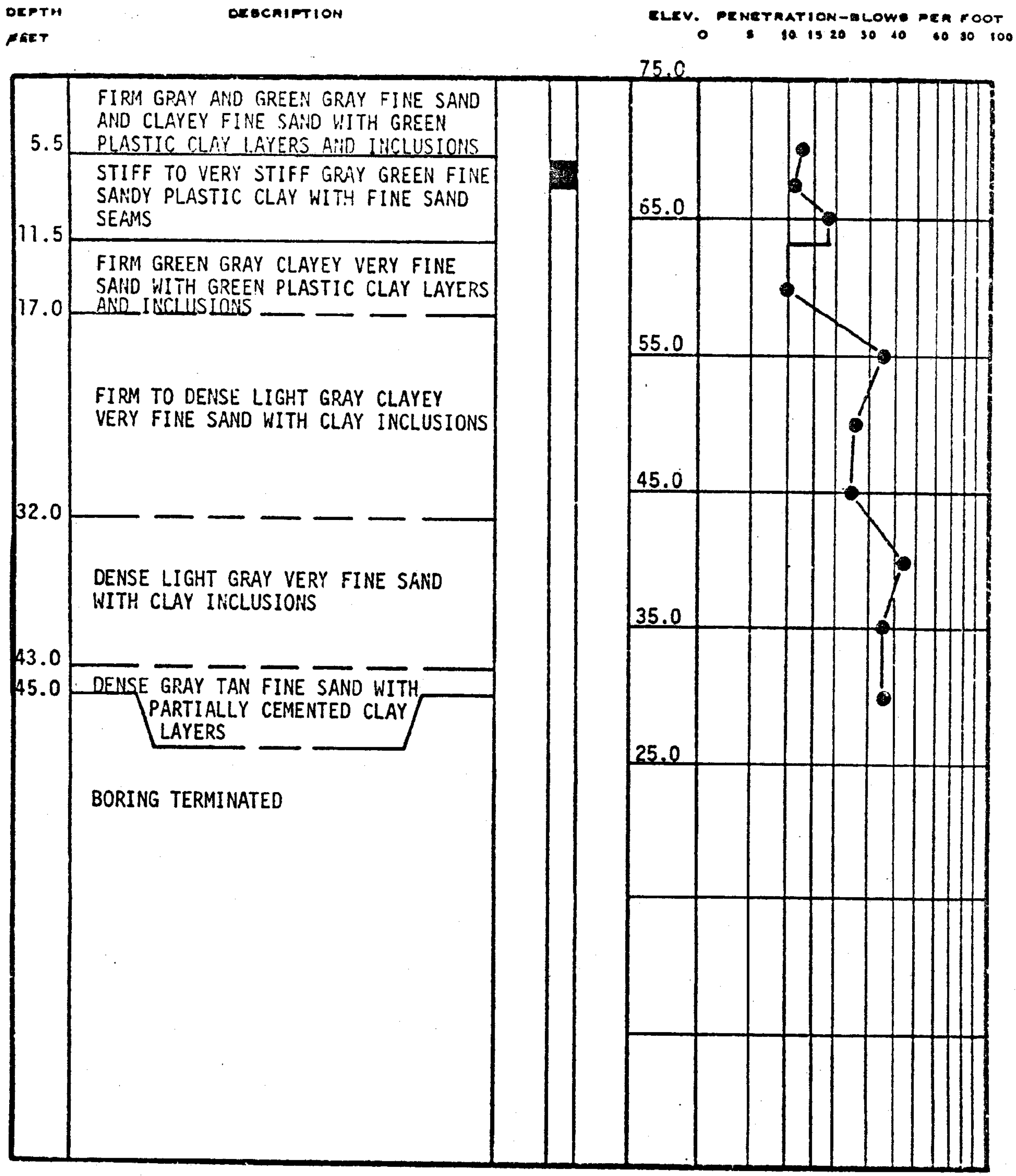
UNIT 2

TEST BORING RECORD
BORING NO. RFI-8

FIGURE 2B-120



REMARKS:

N-50+50
E-50+28

Boring Number RFI-9
Date Drilled 10/7/71
Job Number 5056

HISTORICAL

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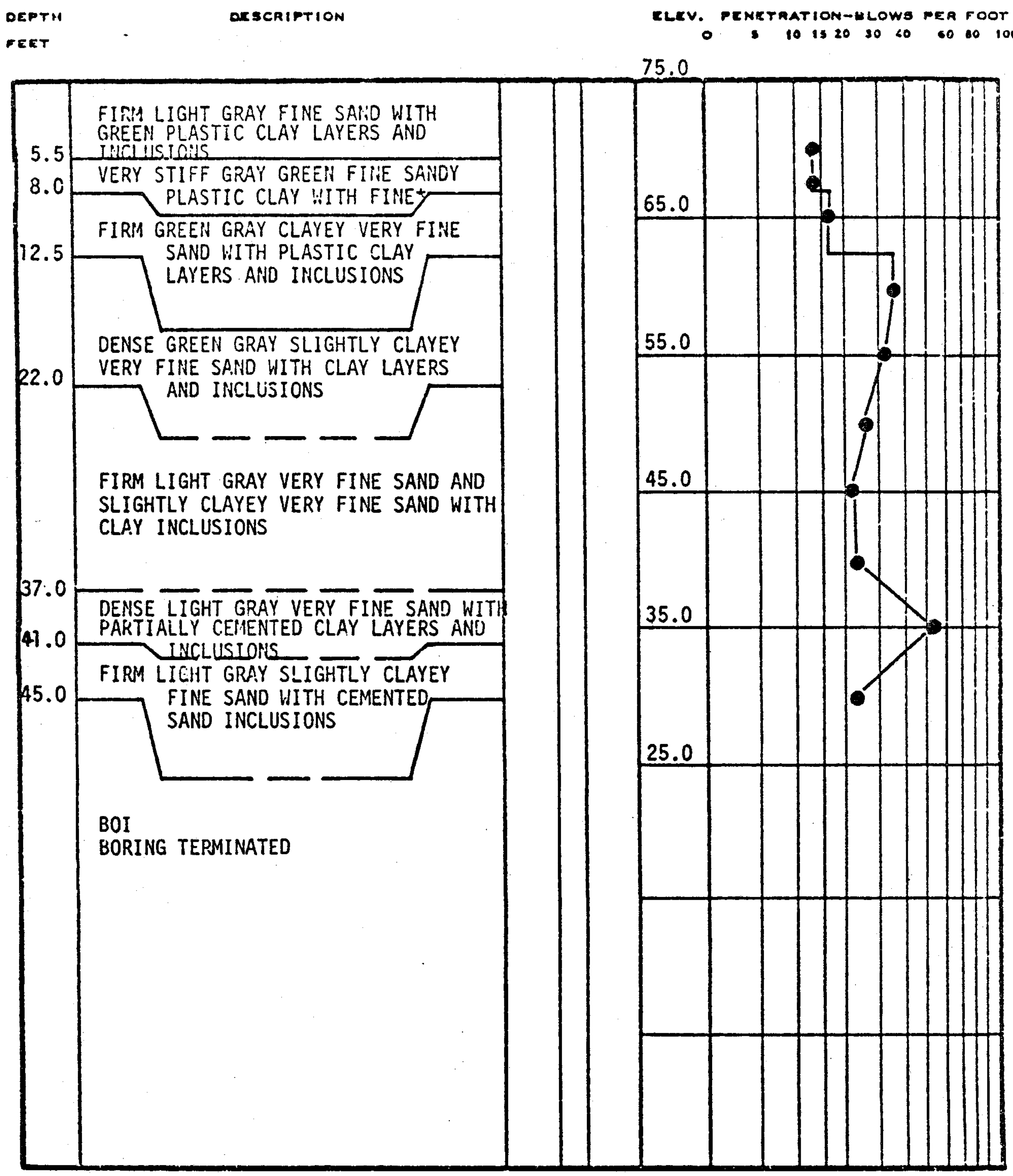
TEST BORING RECORD
BORING NO. RFI-9

FIGURE 2B-121



REMARKS: *SAND SEAMS

N-50+50
E-50+85

BORING NUMBER RFI-10
DATE DRILLED 10/12/71
JOB NUMBER 5056

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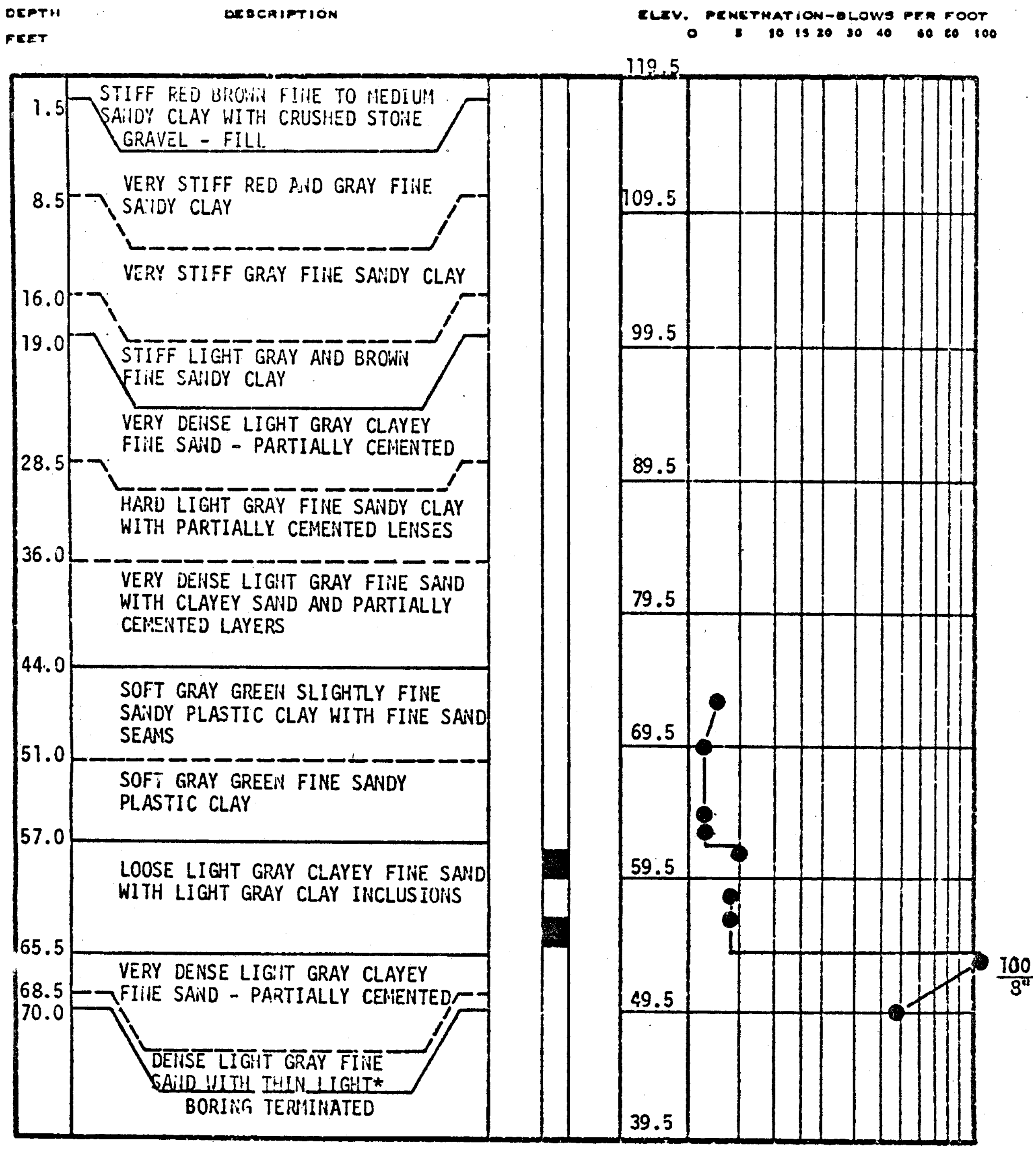
TEST BORING RECORD
BORING NO. RFI-10

FIGURE 2B-122



REMARKS: *GRAY CLAY LAYERS

N- 52+53
E- 56+86

BORING NUMBER B-646
DATE DRILLED 10-15-71
JOB NUMBER 5056

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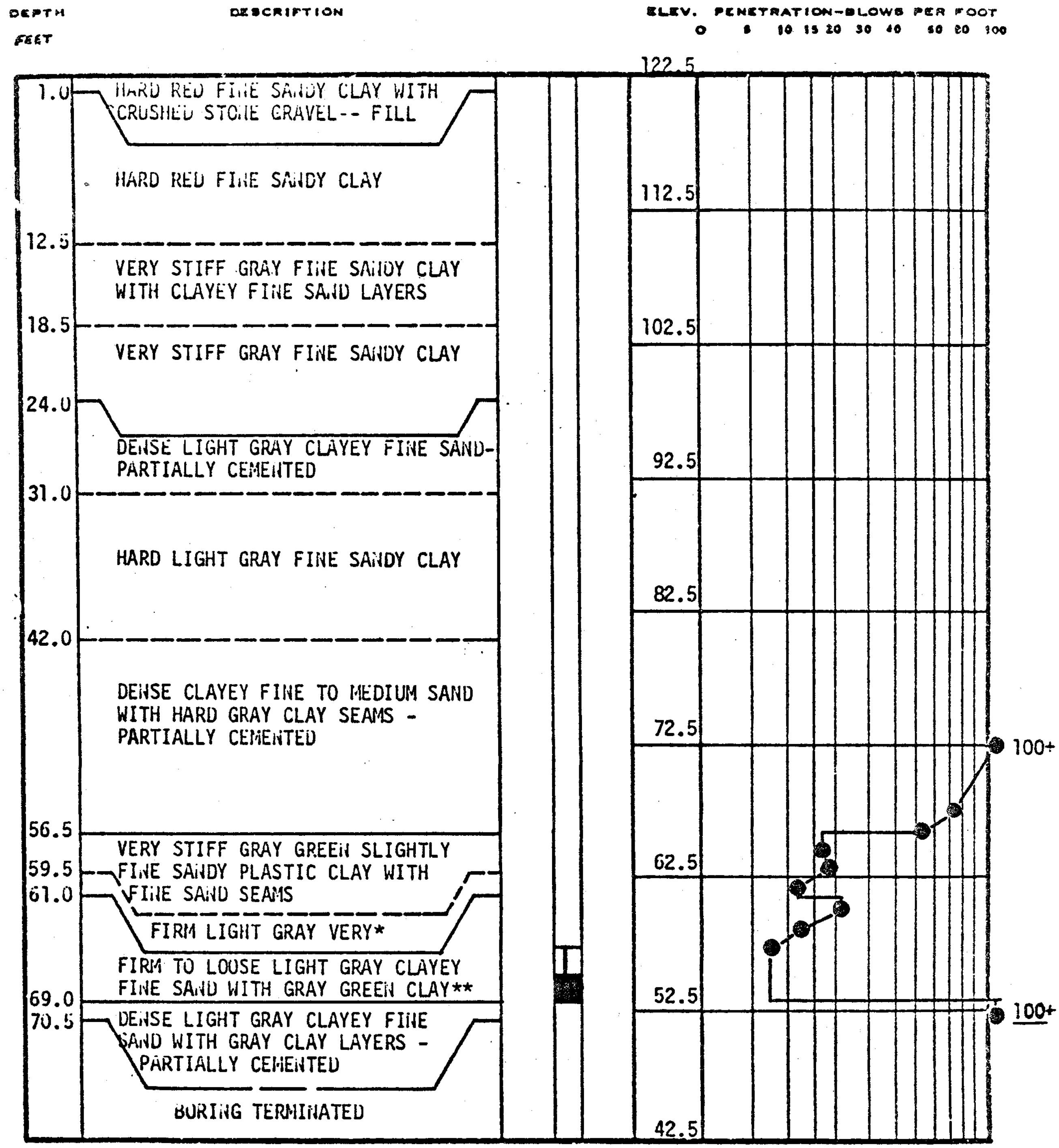
UNIT 2

TEST BORING RECORD
BORING NO. B-646

FIGURE 2B-123



REMARKS: *CLAYEY FINE SAND WITH GRAY GREEN CLAY LAYERS
**LAYERS AND INCLUSIONS

N- 52+69
E- 56 67

BORING NUMBER B-647
DATE DRILLED 10-21/22-71
JOB NUMBER 5056

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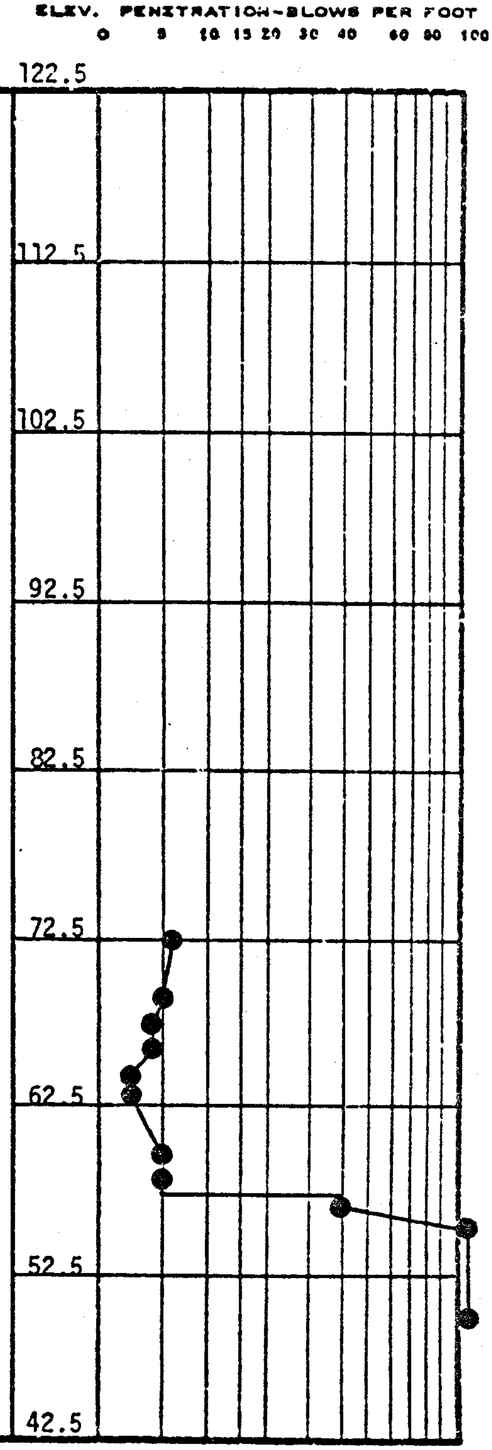
UNIT 2

TEST BORING RECORD
BORING NO. B-647

FIGURE 2B-124

DEPTH FEET	DESCRIPTION
	DENSE CRUSHED STONE GRAVEL AND CLAY - FILL
4.5	HARD RED FINE SANDY CLAY
10.0	VERY STIFF TO HARD LIGHT GRAY FINE SANDY CLAY
18.0	VERY DENSE LIGHT GRAY CLAYEY FINE SAND- PARTIALLY CEMENTED
23.0	HARD GRAY FINE SANDY CLAY
41.0	VERY DENSE GRAY CLAYEY FINE SAND - PARTIALLY CEMENTED
46.5	SOFT GRAY GREEN FINE SANDY PLASTIC CLAY
58.0	LOOSE LIGHT GRAY CLAYEY FINE SAND WITH CLAY INCLUSIONS
65.0	DENSE TO VERY DENSE LIGHT GRAY CLAYEY FINE SAND WITH THIN PARTIALLY CEMENTED CLAY LAYERS
72.5	BORING TERMINATED

ELEV. PENETRATION-BLOWS PER FOOT



N- 51+66
E- 55+70

BORING NUMBER B-648
DATE DRILLED 10-25/26-71
JOB NUMBER 5056

HISTORICAL

REV 19 7/01

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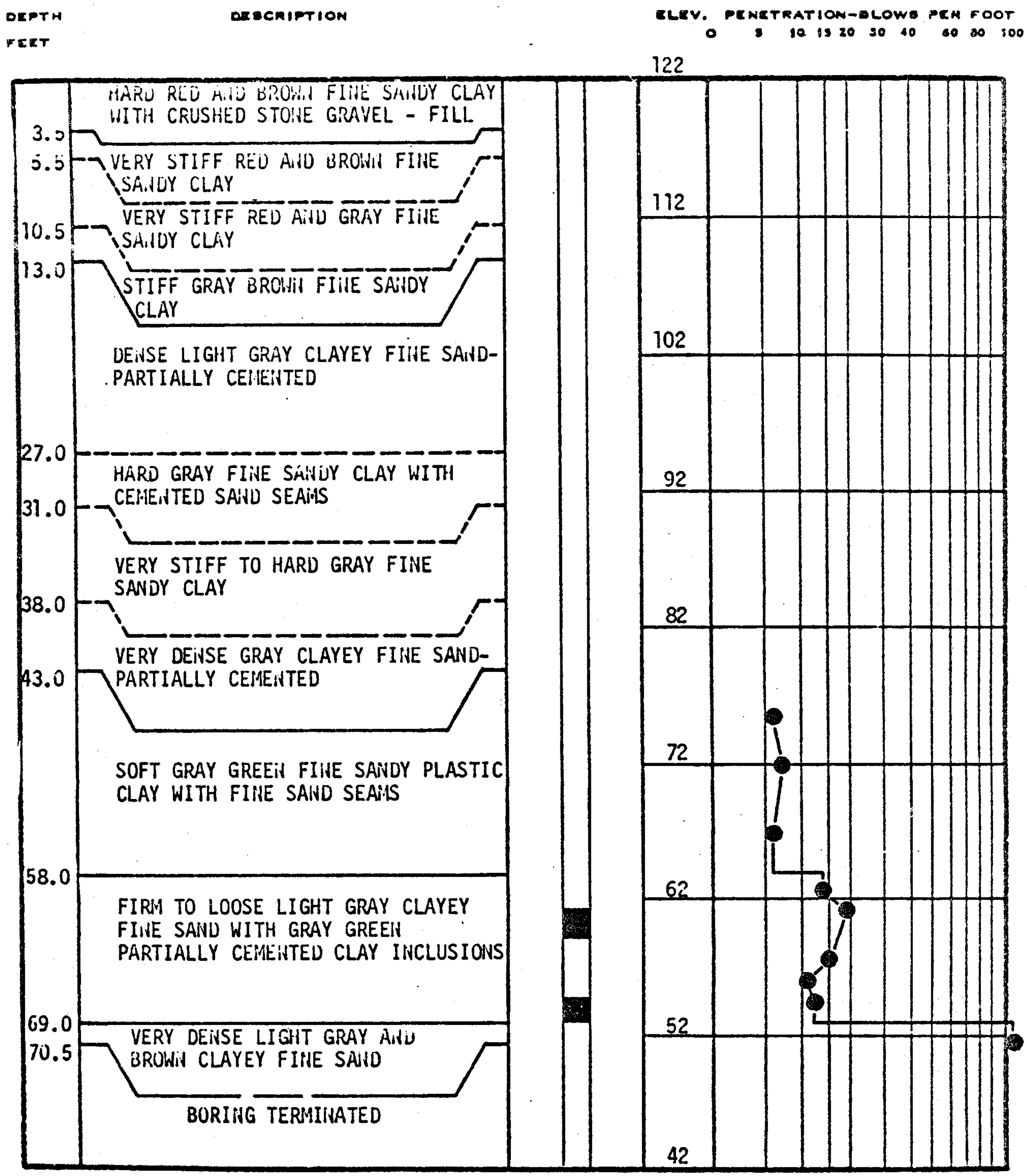
UNIT 2

TEST BORING RECORD
BORING NO. B-648

FIGURE 2B-125



REMARKS:

N- 51+30
E- 56+37

BORING NUMBER B-649
DATE DRILLED 10-71
JOB NUMBER 5056

HISTORICAL
REV 19 7/01

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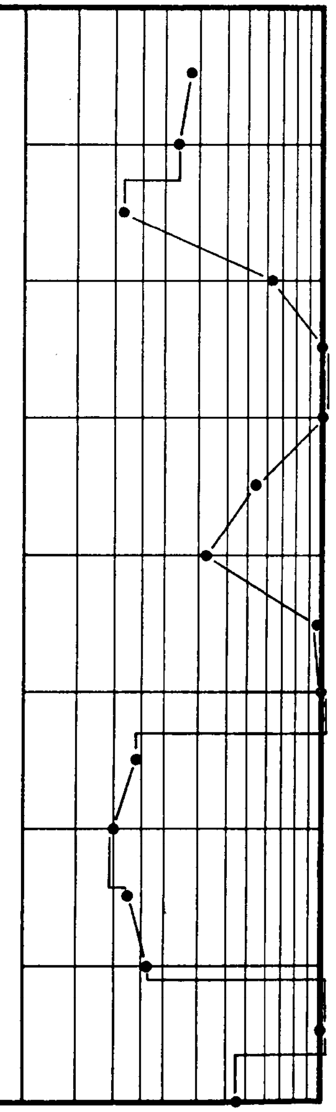
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. B-649

FIGURE 2B-126

ELEV.	DEPTH FEET	DESCRIPTION
122.0		CRUSHED STONE AND SAND FILL
	2.5	FIRM MULTI-COLORED CLAYEY FINE TO MEDIUM SAND-FILL
112.0		
	12.5	FIRM GRAY & BROWN CLAYEY FINE TO MEDIUM SAND WITH THIN CLAY SEAMS
	17.0	
102.0		DENSE TO VERY DARK GRAY CEMENTED CLAYEY FINE TO MEDIUM SAND WITH HARD GRAY SANDY CLAY LAYERS
92.0		
82.0		
	42.0	VERY DENSE GRAY CLAYEY FINE TO MEDIUM SAND
72.0		
	53.0	STIFF LT GRAY PLASTIC CLAY WITH FINE TO MEDIUM SAND SEAMS
	57.0	STIFF GREEN GRAY SLIGHTLY FINE SANDY PLASTIC CLAY WITH FINE SAND SEAMS
62.0		
	64.5	FIRM LIGHT GRAY CLAYEY FINE SAND WITH GRAY GREEN CLAY INCLUSIONS
52.0		
	71.0	VERY DENSE LIGHT GRAY CLAYEY FINE SAND - PARTIALLY CEMENTED
	76.5	DENSE LIGHT GRAY SLIGHTLY CLAYEY FINE SAND WITH THIN CEMENTED CLAY*
42.0		

PENETRATION-BLOWS PER FOOT



REMARKS: * LAYERS

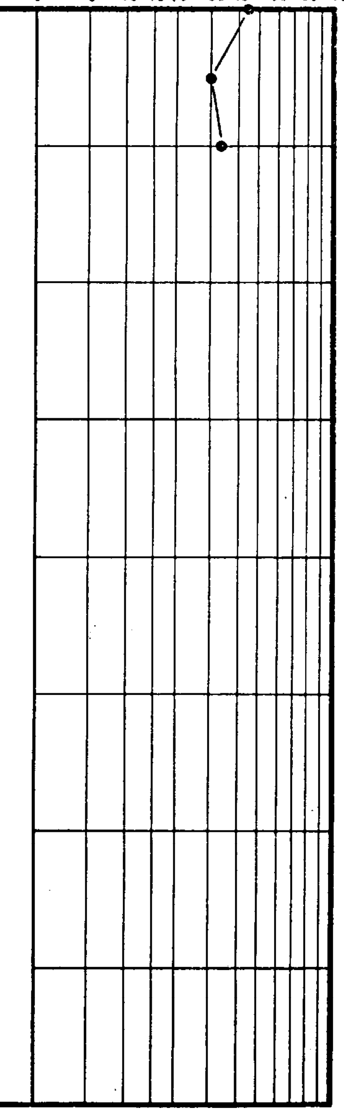
DRILLED BY ________
LOGGED BY ________
CHECKED BY ________

COORDINATES: N-52+30
E-55+20

BORING NUMBER B-650
DATE STARTED ________
DATE COMPLETED 4/8&9/72
JOB NUMBER 5056

ELEV.	DEPTH FEET	DESCRIPTION
42.0	81.0	DENSE LT.GRAY SLIGHTLY CLAYEY FINE SAND W/THIN CEMENTED CLAY LAYERS
		DENSE LIGHT GRAY SLIGHTLY CLAYEY VERY FINE SAND WITH PLASTIC CLAY INCLUSIONS
32.0	90.0	BORING TERMINATED

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED BY ________
LOGGED BY ________
CHECKED BY ________

COORDINATES: N-52+30
E-55+20

BORING NUMBER B-650
DATE STARTED ________
DATE COMPLETED 4/8&9/72
JOB NUMBER 5056

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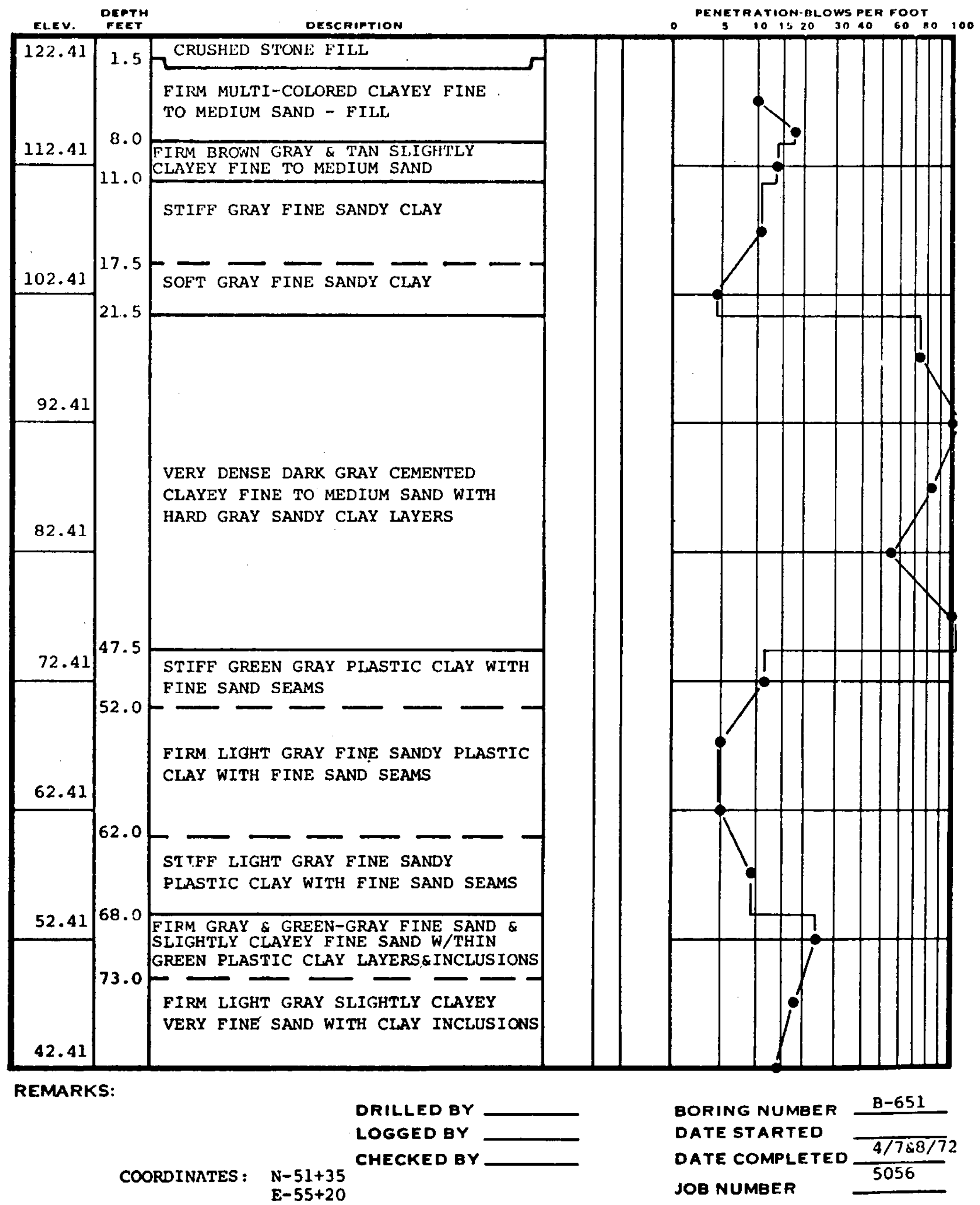
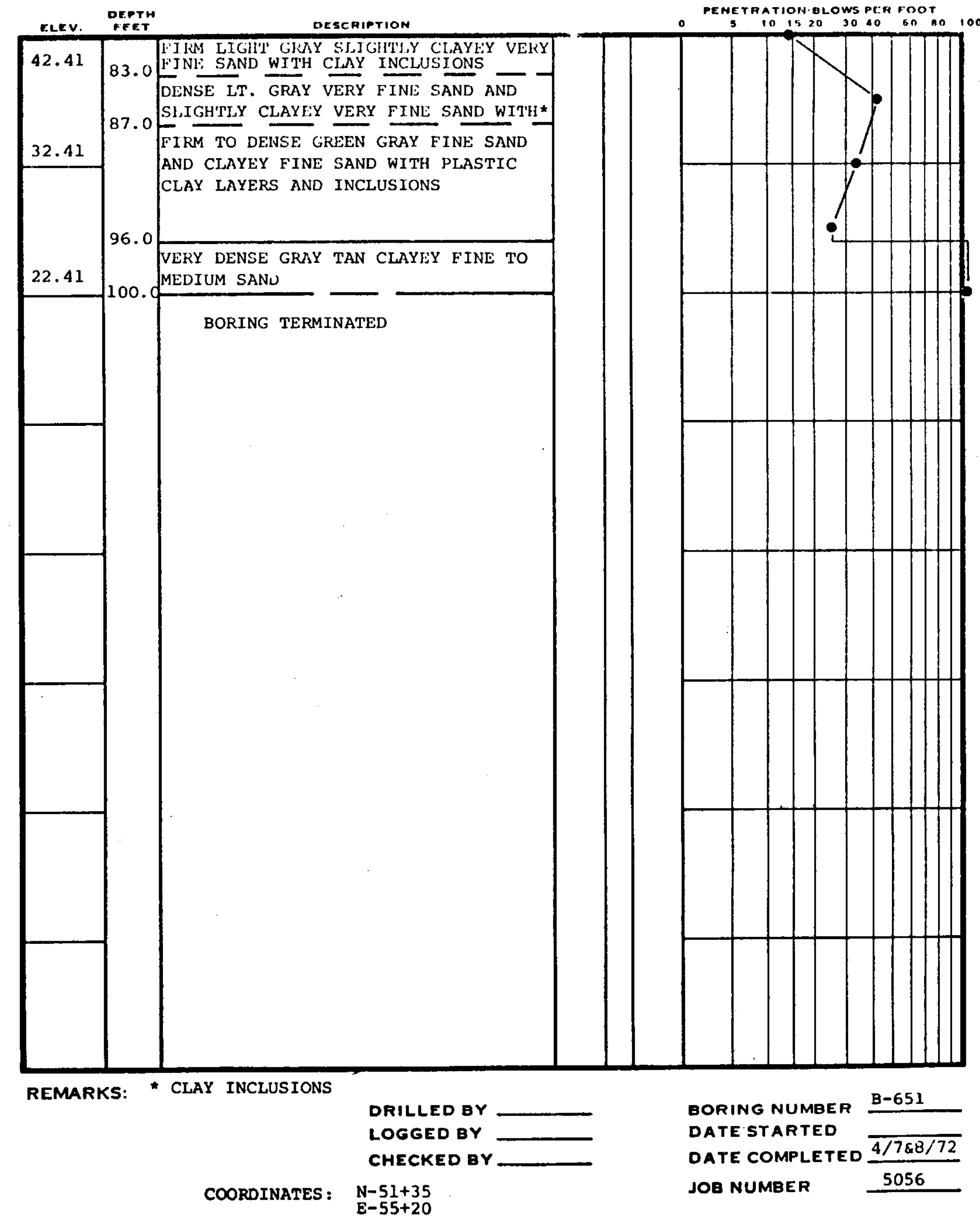
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TEST BORING RECORD BORING NO. B-650
		FIGURE 2B-127





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EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TEST BORING RECORD
BORING NO. B-651

FIGURE 2B-128

BORING LOG

PROJECT	JOB NO.	SHEET NO.	BORING NO.
HATCH UNITS 1 & 2	6511	1 OF 2	B-2001

LOCATION	COORDINATES	ANGLE FROM HORIZ.	BEARING
BLUFFS AT RIVER	N 64 + 31 E 40 + 27	90°	

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	OVERBURDEN (FT)	ROCK (FT.)	TOTAL DEPTH
6-9-76	6-9-76	LETCo. - C. Ivey	CME - 55	5"	100.5		100.5

CORE RECOVERY (FT/%)	CORE BOXES	SAMPLES	EL. TOP CASING	GROUND EL.	DEPTH / EL. GROUND WATER	DEPTH / EL. TOP OF ROCK
		20		121.7	31.3/90.6 @ 24 hr.	

SAMPLE HAMMER WT./FALL	CASING LEFT IN HOLE: DIA. / LENGTH	LOGGED BY:
2 in. O.D. 140#/30"		M. BLENDY

ELEV.	DEPTH	DESCRIPTION – VISUAL	GRAPHIC LOG	PENETRATION RESISTANCE BLOWS/FOOT 0 10 20 30 50 70 100 +100	N
121.7		Red-Tan-Gray Sandy Silty CLAY			
	5				
111.7	10				
	15				
101.7	20	Tan Gray Clayey coarse to fine SAND			
	25	Tan Gray Sandy Silty CLAY			
91.7	30	Tan Gray Clayey Silty coarse to fine SAND			
	35	(cemented)			
81.7	40	(cemented)			
	45				
71.7	50	Tan Gray Clayey fine SAND			45
	55	Light Gray Green Clayey Silty fine SAND			10
61.7	60				8
	65	(clay inclusions)			4

REMARKS	LOCATION	BORING NO.
	BLUFFS AT RIVER	B-2001

BORING LOG

PROJECT	JOB NO.	SHEET NO.	BORING NO.
HATCH UNITS 1 & 2	6511	2 OF 2	B-2001

ELEV.	DEPTH	DESCRIPTION – VISUAL	GRAPHIC LOG	PENETRATION RESISTANCE BLOWS/FOOT 0 10 20 30 50 70 100 +100	N
	65	Light Gray Green Clayey Silty fine SAND			4
51.7	70				2
	75				3
41.7	80				4
		Light Gray Green fine SAND			
	85	(clay inclusions)			25
31.7	90	(clay inclusions)			12
		Light Tan Gray Clayey Silty fine SAND (clay inclusions)			
	95				34
		Light Tan Gray Silty fine SAND			
21.7	100	Boring terminated 6-9-76			43

REMARKS	LOCATION	BORING NO.
	BLUFFS AT RIVER	2001

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UNIT 2

TEST BORING RECORD
BORING NO. B-2001

FIGURE 2B-129

BORING LOG

PROJECT	JOB NO.	SHEET NO.	BORING NO.
HATCH UNITS 1 & 2	6511	1 OF 2	2001A

LOCATION	COORDINATES	ANGLE FROM HORIZ.	BEARING
BLUFFS AT RIVER	N 64 + 31 E 40 + 23	90°	

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	OVERBURDEN (FT)	ROCK (FT.)	TOTAL DEPTH
6-18-76	6-20-76	LETCo - C. Ivey	CME - 55	6"			102.5

CORE RECOVERY (FT/%)	CORE BOXES	SAMPLES	EL. TOP CASING	GROUND EL.	DEPTH / EL. GROUND WATER	DEPTH / EL. TOP OF ROCK
				121.7	N.A.	

SAMPLE HAMMER WT./FALL	CASING LEFT IN HOLE: DIA. / LENGTH	LOGGED BY:
U.D. SAMPLES		M. BLENDY

ELEV.	DEPTH	DESCRIPTION – VISUAL	GRAPHIC LOG	PENETRATION RESISTANCE BLOWS/FOOT 0 10 20 30 50 70 100 +100	N	
121.7						
	5					
111.7	10	Red - Brown - Gray Sandy Silty CLAY				UD - 1 (Shelby Tube)
	15					
101.7	20	Red - Gray Clayey coarse to fine SAND				UD - 2 (Shelby Tube)
	25					
91.7	30	Tan - Gray Clayey Silty coarse to fine SAND (cemented)				UD - 3 (Pitcher Sampler)
	35					
81.7	40	Tan - Gray Clayey Silty coarse to fine SAND (cemented)				UD - 4 (Pitcher Sampler)
	45					
71.7	50	Light Tan Gray Green medium to fine SAND				UD - 5 (Pitcher Sampler)
	55	Light Gray Green Slightly Clayey Silty fine SAND				UD - 6 (Piston Sampler)
61.7	60	Light Tan Gray Green Clayey Silty fine SAND				UD - 7 (Shelby Tube)

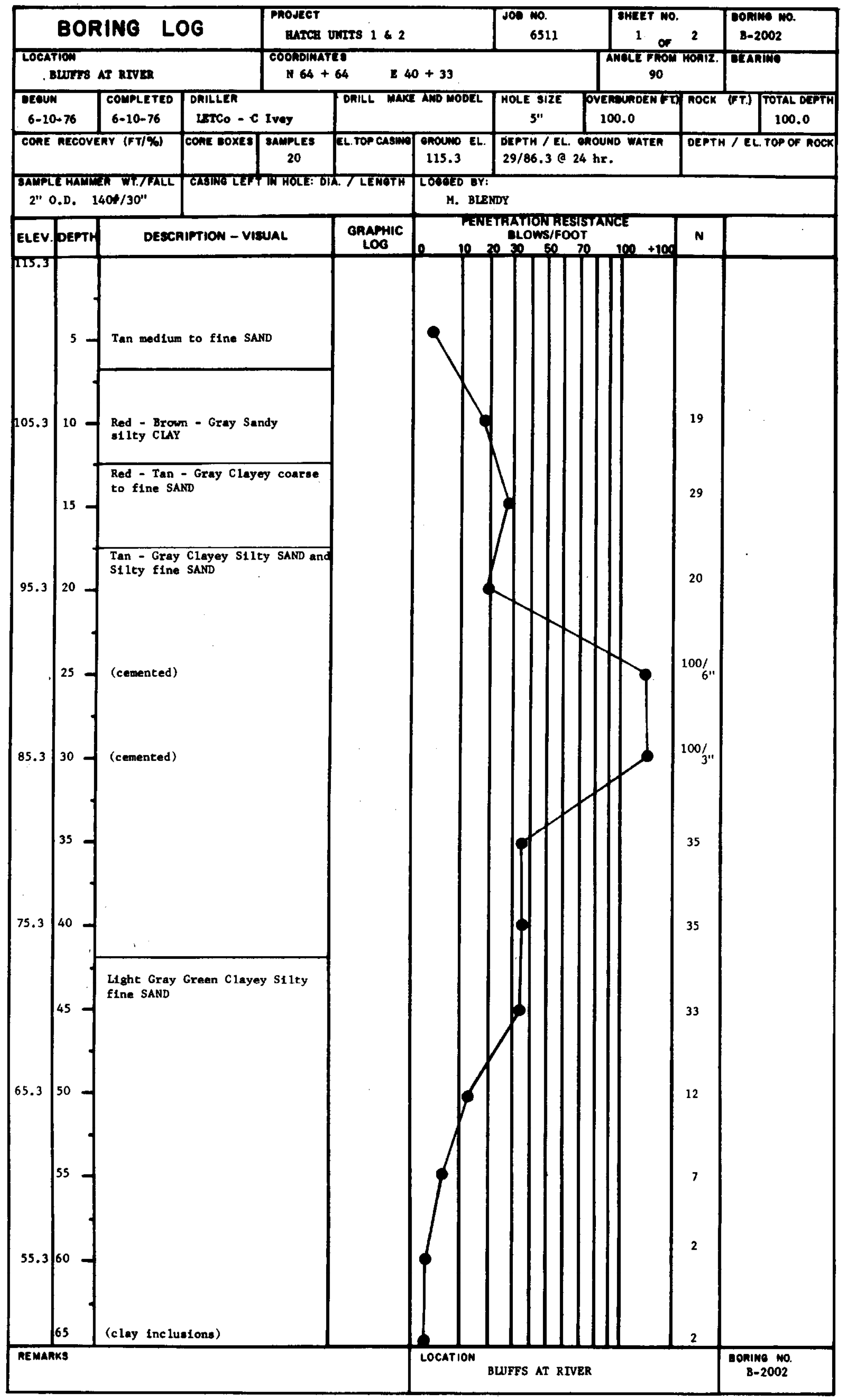
REMARKS	LOCATION	BORING NO.
	BLUFFS AT RIVER	B-2001A

BORING LOG

PROJECT	JOB NO.	SHEET NO.	BORING NO.
HATCH UNITS 1 & 2	6511	2 OF 2	2001A

ELEV.	DEPTH	DESCRIPTION – VISUAL	GRAPHIC LOG	PENETRATION RESISTANCE BLOWS/FOOT 0 10 20 30 50 70 100 +100	N	
	65					
51.7	70	Light Tan Green Clayey Silty fine SAND				UD - 8 (Pitcher Sampler)
	75	Light Gray Green Slightly Silty fine SAND				UD - 9 (Pitcher Sampler)
41.7	80	Light Gray Green Slightly Silty fine SAND				UD - 10 (Pitcher Sampler)
	85	Light Tan Gray Silty Clayey fine SAND				UD - 11 (Pitcher Sampler)
31.7	90	Light Tan Gray Silty fine SAND				UD - 12 (Pitcher Sampler)
	95					
21.7	100	Tan Gray Slightly Clayey Silty fine SAND				UD - 13 (Pitcher Sampler)
		Boring terminated 6-20-76				

REMARKS	LOCATION	BORING NO.
	BLUFFS AT RIVER	2001 A



BORING LOG

PROJECT	JOB NO.	SHEET NO.	BORING NO.
HATCH UNITS 1 & 2	6511	1 OF 2	B-2002

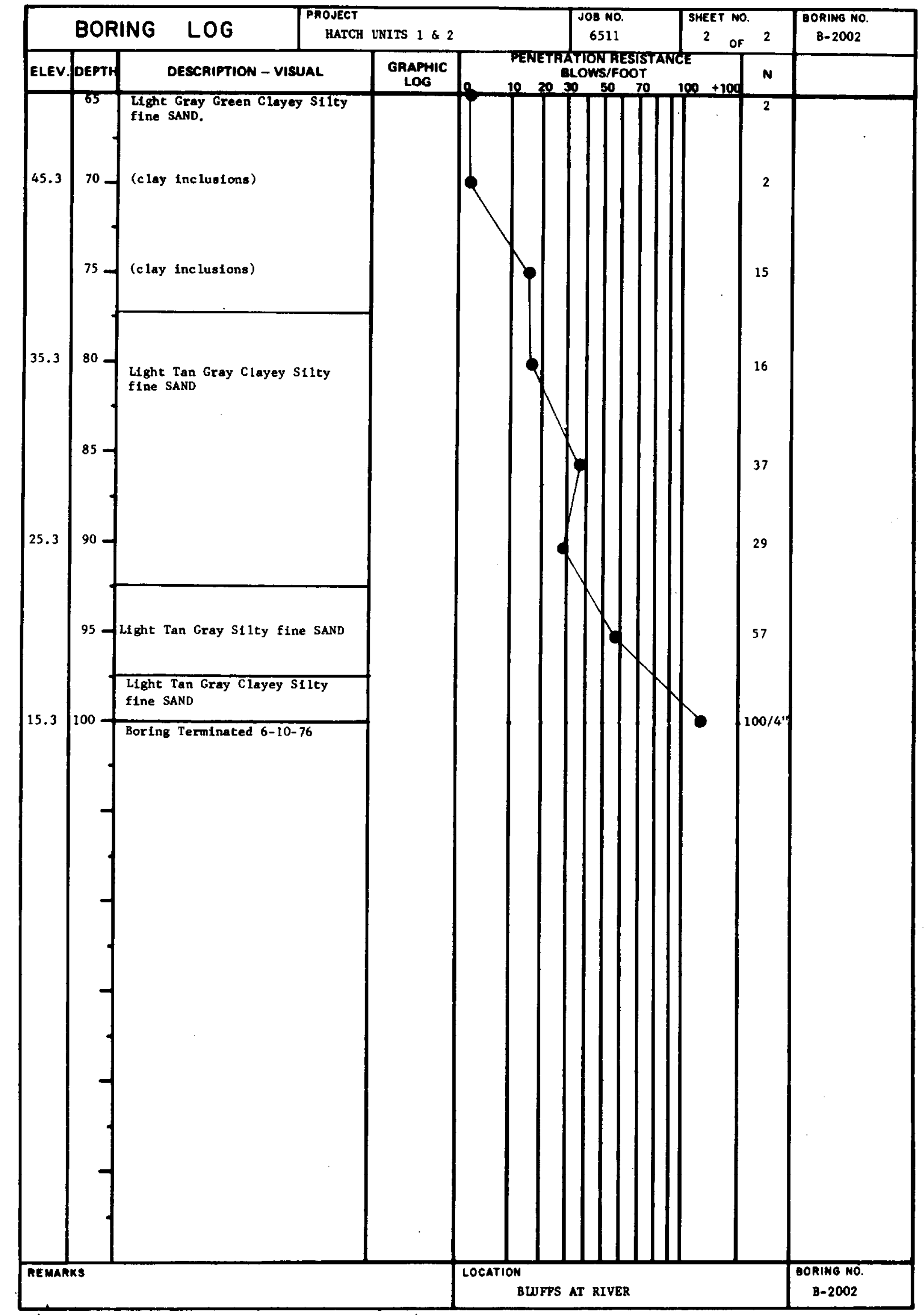
LOCATION	COORDINATES	ANGLE FROM HORIZ.	BEARING
BLUFFS AT RIVER	N 64 + 64 E 40 + 33	90	

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	OVERBURDEN (FT)	ROCK (FT.)	TOTAL DEPTH
6-10-76	6-10-76	LETCo - C Ivey		5"	100.0		100.0

CORE RECOVERY (FT/%)	CORE BOXES	SAMPLES	EL. TOP CASING	GROUND EL.	DEPTH / EL. GROUND WATER	DEPTH / EL. TOP OF ROCK
		20		115.3	29/86.3 @ 24 hr.	

SAMPLE HAMMER WT./FALL	CASING LEFT IN HOLE: DIA. / LENGTH	LOGGED BY:
2" O.D. 140#/30"		M. BLENDY

ELEV.	DEPTH	DESCRIPTION – VISUAL	N
115.3			
	5	Tan medium to fine SAND	
105.3	10	Red - Brown - Gray Sandy silty CLAY	19
		Red - Tan - Gray Clayey coarse to fine SAND	29
	15		
		Tan - Gray Clayey Silty SAND and Silty fine SAND	
95.3	20		20
	25	(cemented)	100/6"
85.3	30	(cemented)	100/3"
	35		35
75.3	40		35
		Light Gray Green Clayey Silty fine SAND	
	45		33
65.3	50		12
	55		7
			2
55.3	60		
	65	(clay inclusions)	2



BORING LOG

PROJECT	JOB NO.	SHEET NO.	BORING NO.
HATCH UNITS 1 & 2	6511	2 OF 2	B-2002

ELEV.	DEPTH	DESCRIPTION – VISUAL	N
	65	Light Gray Green Clayey Silty fine SAND.	2
45.3	70	(clay inclusions)	2
	75	(clay inclusions)	15
35.3	80	Light Tan Gray Clayey Silty fine SAND	16
	85		37
25.3	90		29
	95	Light Tan Gray Silty fine SAND	57
		Light Tan Gray Clayey Silty fine SAND	
15.3	100	Boring Terminated 6-10-76	100/4"

HISTORICAL

REV 19 7/01

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TEST BORING RECORD
BORING NO. B-2002

FIGURE 2B-131

BORING LOG

BORING LOG	PROJECT: HATCH UNITS 1 & 2	JOB NO.: 6511	SHEET NO.: 1 OF 2	BORING NO.: 2003
LOCATION: BLUFFS AT RIVER	COORDINATES: N 62 + 95 E 40 + 38		ANGLE FROM HORIZ.: 90°	BEARING:

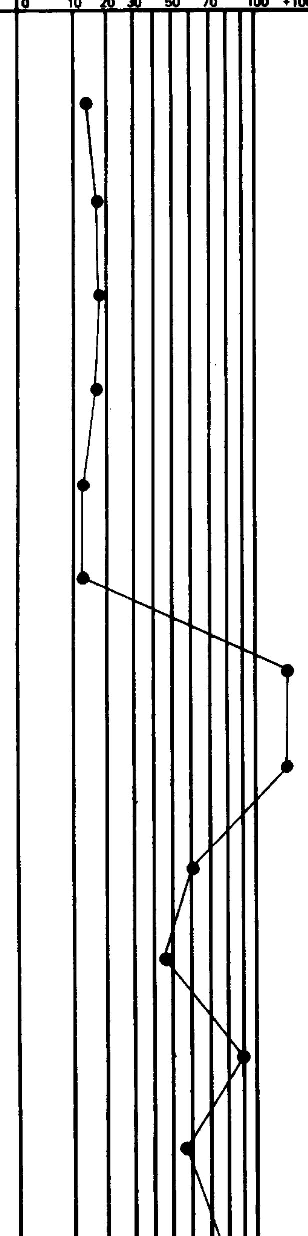
BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	OVERBURDEN (FT.)	ROCK (FT.)	TOTAL DEPTH
6-11-76	6-14-76	LETCo - C. Ivey	CME - 55	5"	120.5		120.5

CORE RECOVERY (FT/%)	CORE BOXES	SAMPLES	EL. TOP CASING	GROUND EL.	DEPTH / EL. GROUND WATER	DEPTH / EL. TOP OF ROCK
		24		127.8	34'/93.8 @ 24 hr.	

SAMPLE HAMMER WT./FALL	CASING LEFT IN HOLE: DIA. / LENGTH	LOGGED BY:
2 in. O.D. 140#/30"		M. BLENDY

ELEV.	DEPTH	DESCRIPTION – VISUAL	GRAPHIC LOG	N
127.8				
	5	Red Brown Clayey coarse to fine SAND (Trace gravel)		13
117.8	10			17
	15	Purple, Brown, and Gray Sandy Silty CLAY		17
107.8	20	Tan Gray Sandy Silty CLAY		16
	25			11
97.8	30			11
	35	Gray Clayey Silty medium to fine SAND (cemented)		35/1"
87.8	40	(cemented)		63/6"
	45	Light Tan Gray Silty medium to fine SAND and Tan Gray coarse to fine SAND		60
77.8	50			45
	55			89
67.8	60			54
	65			73

PENETRATION RESISTANCE BLOWS/FOOT



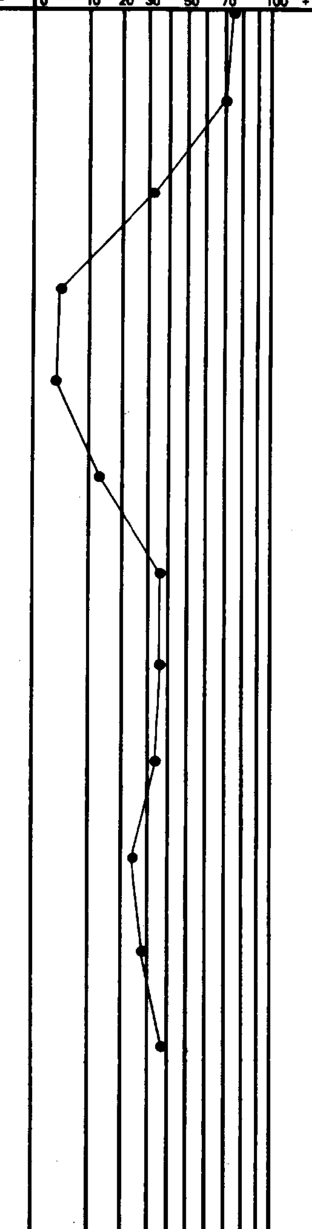
REMARKS	LOCATION	BORING NO.
	BLUFFS AT RIVER	2003

BORING LOG

BORING LOG	PROJECT: HATCH UNITS 1 & 2	JOB NO.: 6511	SHEET NO.: 2 OF 2	BORING NO.: 2003

ELEV.	DEPTH	DESCRIPTION – VISUAL	GRAPHIC LOG	N
	65	Light Tan Gray Silty medium to fine SAND and Tan Gray coarse to fine SAND		73
57.8	70			70
	75			31
47.8	80	Light Gray Green Clayey Silty fine SAND (clay inclusions)		4
	85	(clay inclusions)		
37.8	90	(clay inclusions)		12
	95			36
27.8	100	Tan Gray Silty medium to fine SAND		35
	105	(clay inclusions)		32
17.8	110	(clay inclusions)		24
	119			29
7.8	120			39
		Boring terminated 6-14-76		

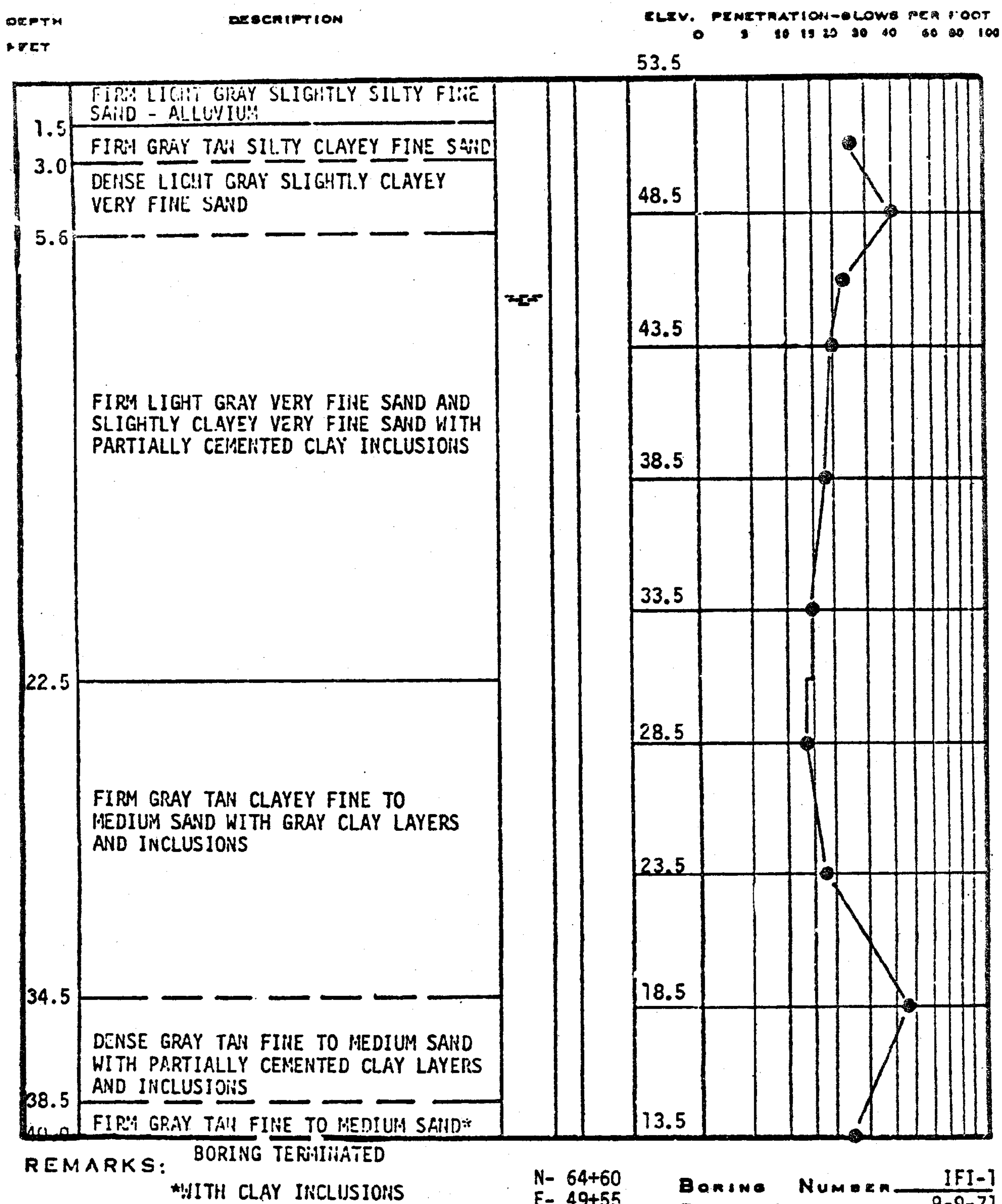
PENETRATION RESISTANCE BLOWS/FOOT



REMARKS	LOCATION	BORING NO.
	BLUFFS AT RIVER	2003

HISTORICAL
REV 19 7/01

SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TEST BORING RECORD BORING NO. B-2003 FIGURE 2B-132



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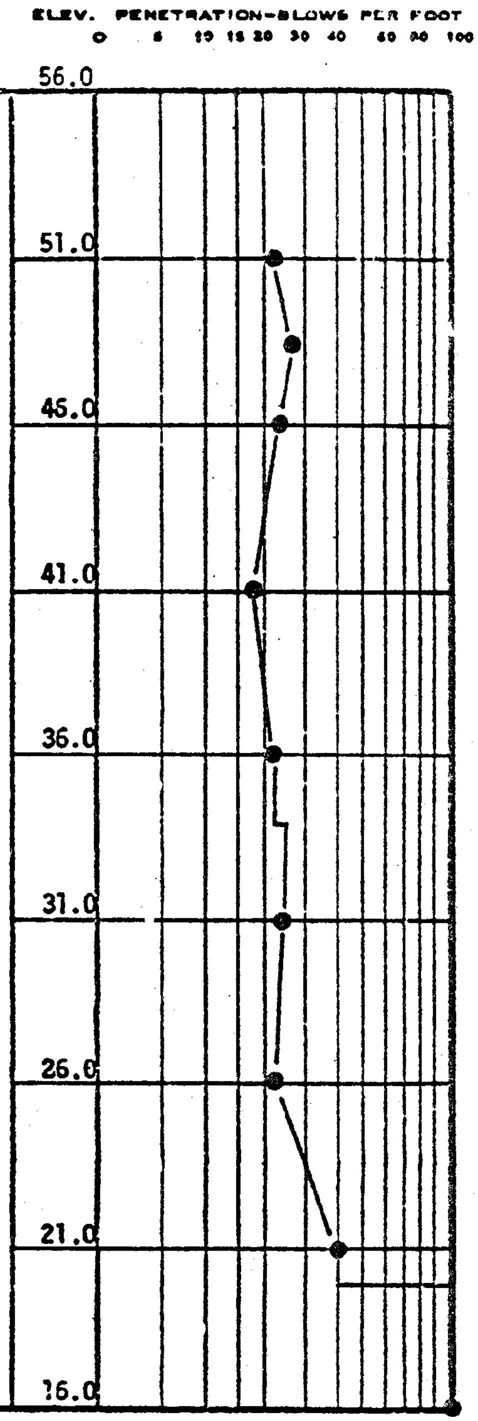
UNIT 2

TEST BORING RECORD
BORING NO. IFI-1

FIGURE 2B-133

DEPTH FEET	DESCRIPTION
0–2.5	FIRM BROWN AND TAN CLAYEY FINE SAND WITH CRUSHED STONE - FILL
2.5–4.0	HARD GREEN GRAY CLAYEY CEMENTED FINE SAND
4.0–22.0	FIRM LIGHT GRAY VERY FINE SAND AND CLAYEY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY INCLUSIONS
22.0–27.0	FIRM GRAY TAN FINE TO MEDIUM SAND WITH LIGHT GRAY LAYERS AND INCLUSIONS
27.0–33.5	FIRM GRAY TAN CLAYEY FINE TO MEDIUM SAND WITH PARTIALLY CEMENTED CLAY INCLUSIONS
33.5–36.0	DENSE GRAY TAN FINE TO MEDIUM SAND WITH PARTIALLY CEMENTED CLAY *
36.0–37.0	VERY HARD GRAY GREEN CEMENTED CLAY
37.0–40.0	VERY DENSE GRAY TAN SLIGHTLY CLAYEY FINE TO MEDIUM SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS



REMARKS: BORING TERMINATED

*INCLUSIONS

N- 63+85
E- 49+63

BORING NUMBER IFI-2
DATE DRILLED 9-9-71
JOB NUMBER 5056

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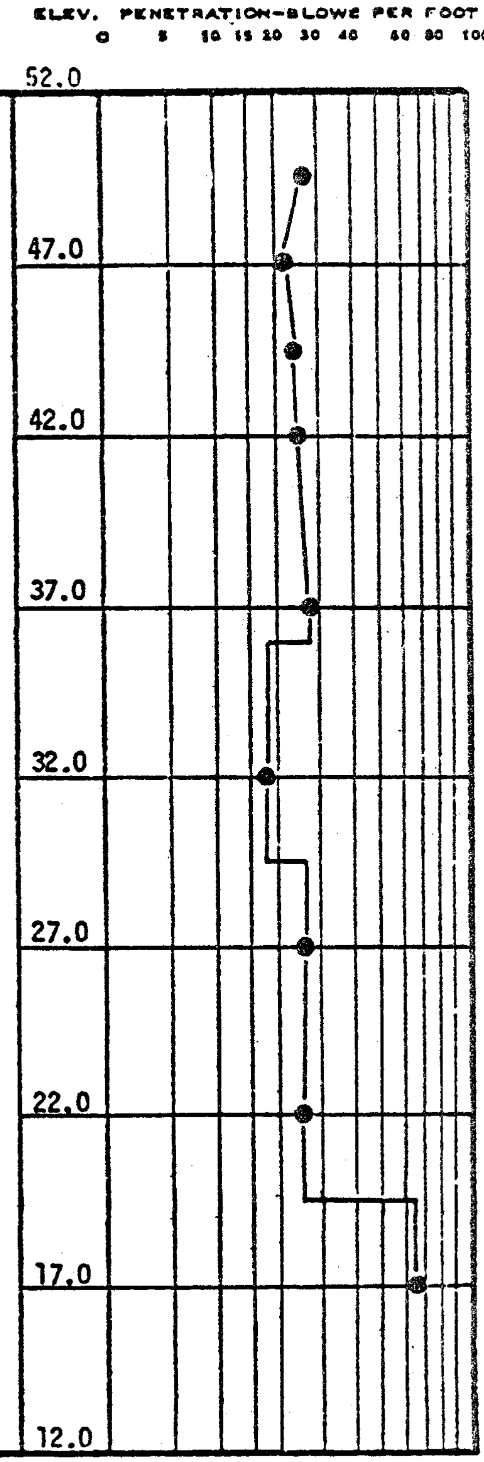
TEST BORING RECORD
BORING NO. IFI-2

FIGURE 2B-134

DEPTH FEET	DESCRIPTION
	FIRM LIGHT GRAY VERY FINE SAND AND CLAYEY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS
16.0	VERY STIFF GRAY GREEN PLASTIC CLAY WITH LIGHT GRAY VERY FINE SAND SEAMS
22.5	FIRM GRAY TAN SLIGHTLY CLAYEY FINE TO MEDIUM SAND
32.5	VERY HARD GRAY GREEN CEMENTED CLAY
33.5	DENSE GRAY TAN FINE TO MEDIUM SAND*
35.0	BORING TERMINATED



REMARKS: *WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS

N- 64+10
E- 49+83

BORING NUMBER IFI-3
DATE DRILLED 9-11-12-71
JOB NUMBER 5056

HISTORICAL
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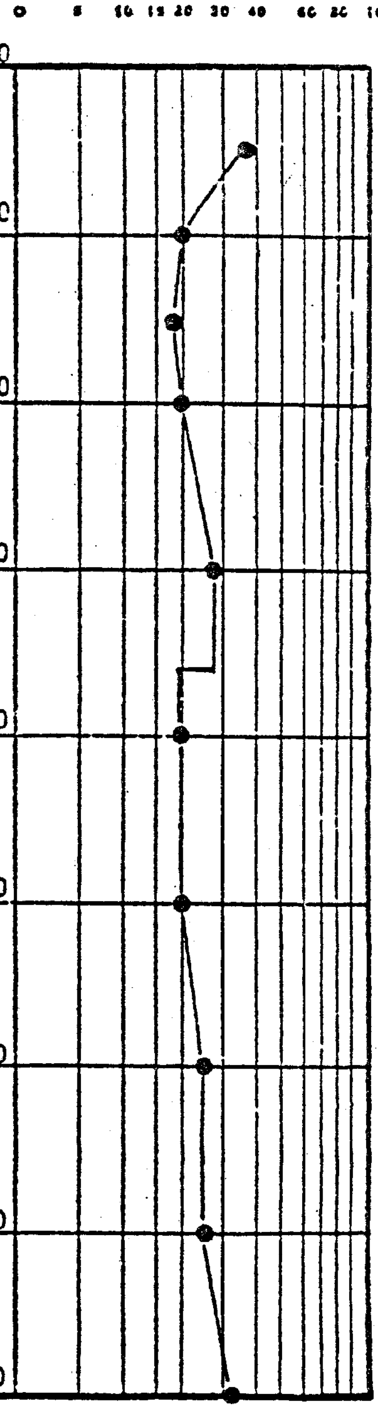
TEST BORING RECORD
BORING NO. IFI-3

FIGURE 2B-135

DEPTH FEET	DESCRIPTION	ELEV.
		52.0
	DENSE LIGHT GRAY CLAYEY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY INCLUSIONS	
3.5	FIRM LIGHT GRAY VERY FINE SAND AND SLIGHTLY CLAYEY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS	47.0
		42.0
		37.0
18.0	VERY STIFF GRAY GREEN PLASTIC CLAY WITH FINE SAND SEAMS	32.0
22.5	FIRM GRAY TAN CLAYEY FINE SAND	27.0
28.5	FIRM GRAY TAN FINE TO MEDIUM SAND WITH PARTIALLY CEMENTED CLAY LAYERS	22.0
32.5	VERY HARD GRAY GREEN CEMENTED CLAY	
33.0	FIRM GRAY TAN FINE TO MEDIUM SAND	17.0
36.5	DENSE GRAY TAN CLAYEY FINE TO MEDIUM SAND WITH PARTIALLY CEMENTED CLAY LAYERS	
40.0		12.0

PENETRATION-BLOWS PER FOOT



REMARKS: BORING TERMINATED

N- 63+55
E- 49+83

BORING NUMBER IFI-4
DATE DRILLED 9-11-71
JOB NUMBER 5056

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HISTORICAL
REV 19 7/01

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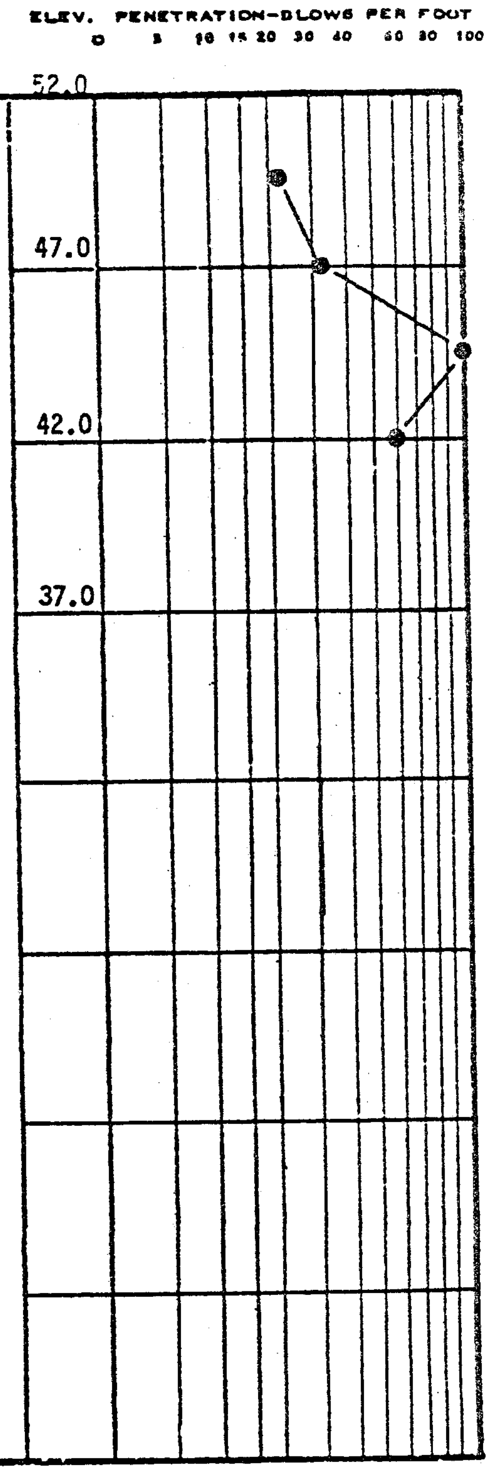
TEST BORING RECORD
BORING NO. IFI-4

FIGURE 2B-136

DEPTH FEET	DESCRIPTION
	FIRM TO DENSE LIGHT GRAY VERY FINE SAND AND CLAYEY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY INCLUSIONS
6.5	VERY DENSE LIGHT GRAY VERY FINE SAND AND SLIGHTLY CLAYEY VERY FINE SAND WITH PARTIALLY CEMENTED CLAY LAYERS AND INCLUSIONS
10.0	BORING TERMINATED



REMARKS: NO GROUND WATER ENCOUNTERED

N- 64+10
E- 49+43

BORING NUMBER IFI-5
DATE DRILLED 9-71
JOB NUMBER 5056

HISTORICAL

REV 19 7/01

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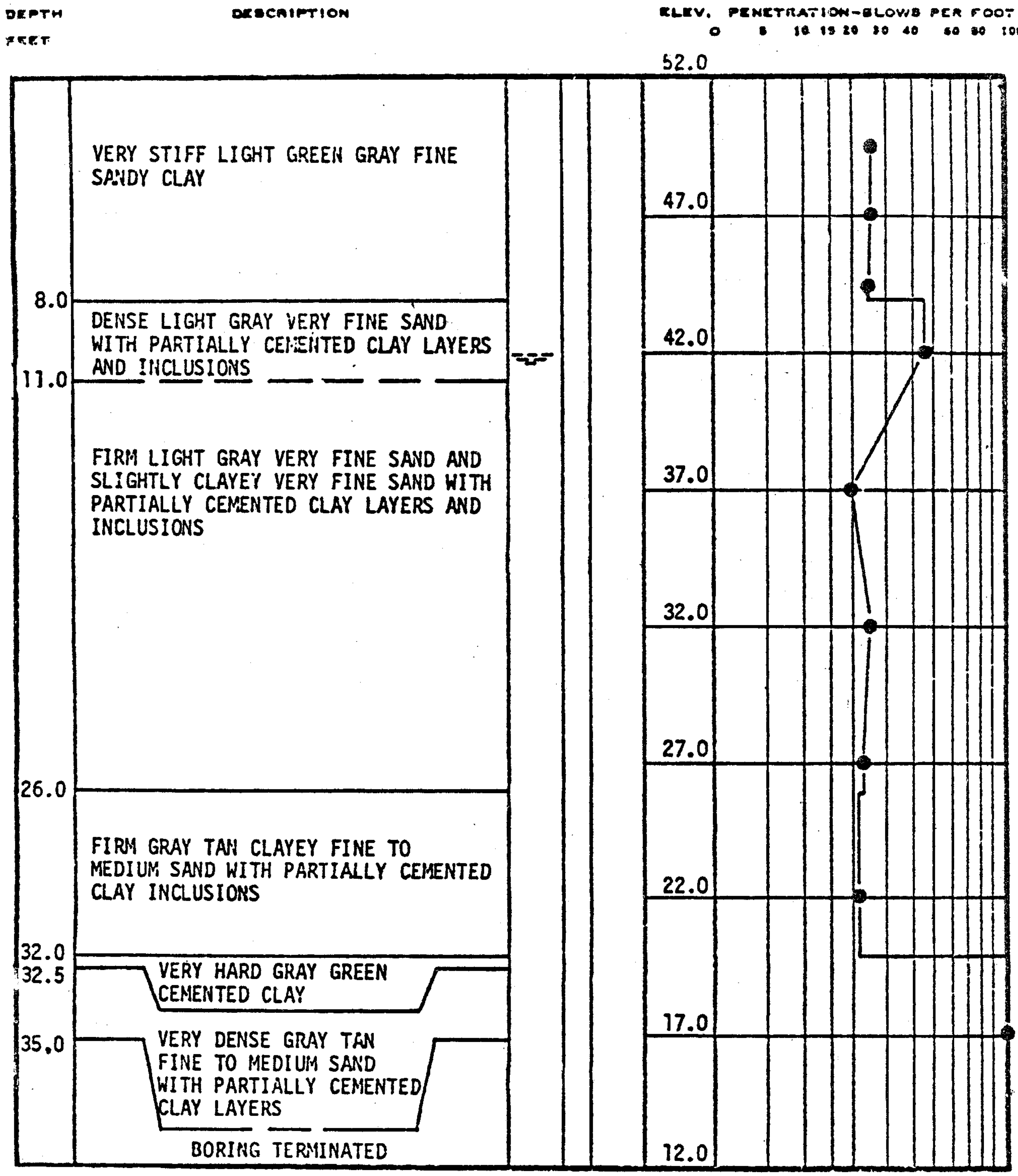
UNIT 2

TEST BORING RECORD
BORING NO. IFI-5

FIGURE 2B-137



REMARKS:

N- 63+55
E- 49+43

BORING NUMBER IFI-6
DATE DRILLED 9-11-71
JOB NUMBER 5056

HISTORICAL
REV 19 7/01

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UNIT 2

TEST BORING RECORD
BORING NO. IFI-6

FIGURE 2B-138

3.0 DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

3.1 CONFORMANCE WITH NUCLEAR REGULATORY COMMISSION (NRC) GENERAL DESIGN CRITERIA

This section discusses the extent to which the design criteria for the Hatch Nuclear Plant-Unit 2 (HNP-2) plant structures, system, and components important to safety meet the General Design Criteria for Nuclear Power Plants, specified in Appendix A to 10 CFR 50. For each criterion, a summary is provided to show how the principal design features meet the criterion. Any exceptions to criteria are identified, with the justification for each exception, in the summary. In the discussion of each criterion, the section of the Final Safety Analysis Report (FSAR) where more detailed information is presented to demonstrate compliance with or exceptions to the criterion is also provided.

Criterion 1 - Quality Standards and Records

"Structures, systems, and components important to safety shall be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. Where generally recognized codes and standards are used, they shall be identified and evaluated to determine their applicability, adequacy, and sufficiency and shall be supplemented or modified as necessary to assure a quality product in keeping with the required safety function. A quality assurance program shall be established and implemented in order to provide adequate assurance that these structures, systems, and components will satisfactorily perform their safety functions. Appropriate records of the design, fabrication, erection, and testing of structures, systems, and components important to safety shall be maintained by or under the control of the nuclear power unit licensee throughout the life of the unit."

Design Evaluation

Structures, systems, and components important to safety are designed, fabricated, erected, and tested under a quality assurance (QA) program which satisfies the intent of Appendix B of 10 CFR 50. The QA program was designed and organized to ensure the HNP is designed, fabricated, and constructed in conformance with the regulatory requirements and design bases outlined in the license application.

Design requirements and other information regarding implementation of the QA program are described in various sections of the FSAR. Codes and standards which apply to safety-related, pressure-retaining piping and equipment are included in subsection 3.2.2. Building codes and standards are discussed in paragraphs 3.8.3.2, 3.8.4.2, and 3.8.5.2. Detailed seismic requirements are outlined in supplements 3.7A and 3.7B.

Structures, systems, and components are first classified with regard to location, service, and relationship to the safety function to be performed. Recognized codes and standards are applied to the equipment in keeping with the appropriate classification. Where codes are not available or where the existing code must be modified, a rigorous justification is provided in the FSAR.

Documents and records are required providing objective evidence that the requirements of the QA program have been satisfied. The documentation shows that the required codes, standards, and specifications were observed, that specified materials were used, that correct procedures were utilized,

that qualified personnel performed the work, and that inspections and tests verify that finished parts and components meet the applicable specifications. All applicable records are maintained during the operational life of the plant and are readily available for reference. The QA program developed by the applicant and his contractors satisfies the requirements of General Design Criterion (GDC) 1.

Criterion 2 - Design Bases for Protection Against Natural Phenomena

"Structures, systems, and components important to safety shall be designed to withstand the effect of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. The design bases for these structures, systems, and components shall reflect: (1) appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated, (2) appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena and (3) the importance of the safety functions to be performed."

Design Evaluation

The design basis for protection against natural phenomena is in accordance with GDC 2. Structures, systems, and components important to safety are designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, and floods without loss of the capability to perform those safety functions necessary to cope with appropriate margin to account for uncertainties in the historical data. The natural phenomena postulated in the design are presented in sections 2.3, 2.4, and 2.5. The design criteria for the structures, systems, and components affected by each natural phenomenon are presented in sections 3.2, 3.3, 3.4, 3.5, and 3.8; and supplements 3.7A and 3.7B. Those combinations of natural phenomena and plant-originated accidents that are considered in the design are identified in sections 3.8, 3.9, 3.10, and 3.11.

Criterion 3 - Fire Protection

"Structures, systems, and components important to safety shall be designed and located to minimize, consistent with other safety requirements, the probability and effect of fires and explosions. Noncombustible and heat resistant materials shall be used wherever practical throughout the unit, particularly in locations such as the containment and control room. Fire detection and fighting systems of appropriate capacity and capability shall be provided and designed to minimize the adverse effects of fires on structures, systems, and components important to safety. Fire-fighting systems shall be designed to assure that their rupture or inadvertent operation does not significantly impair the safety capability of these structures, systems, and components."

Design Evaluation

Structures, systems, and components important to safety are designed to minimize the probability and effect of fires and explosions. Noncombustible and heat-resistant materials are used whenever practical throughout the plant, particularly in the containment, control room, and in areas containing engineered safeguards.

Appropriate equipment and facilities for fire protection, including detection, alarm, and extinguishment of fires, are provided to protect plant equipment and personnel from fire, explosions, and the resultant release of toxic vapors. Automatic and manual types of fire protection equipment are provided.

The fire protection system provides an adequate supply of water to the deluge systems, sprinkler systems, and hose stations located throughout the plant. Carbon dioxide systems are used to protect the cable spreading room, the computer room, and the emergency diesel generators and associated switchgear areas. Portable and mobile chemical fire extinguishers are provided throughout the plant. A complete description of the fire protection design bases is provided in the ***Edwin I. Hatch Nuclear Plant Units 1 and 2 Fire Hazards Analysis and Fire Protection Program (incorporated by reference into the FSAR)*** *submitted to the Nuclear Regulatory Commission on July 22, 1986. Fire-fighting systems are designed to ensure that their rupture or inadvertent operation does not significantly impair safety-related systems.*

The fire protection system consists of a reliable, partially automatic unit designed and installed in accordance with the requirements of the National Fire Protection Association, Nuclear Mutual Limited (NML), and the Occupational Safety and Health Act, in addition to the applicable local codes and regulations.

A fire and smoke detection system is provided throughout the plant for immediate detection and identification of fire and smoke. Ionization-type detectors are provided in the control room.

The equipment and systems are inspected and tested in accordance with the requirements of local and state authorities and have the approval of NML. The fire protection system is provided with test valves and facilities for periodic testing. All equipment is accessible for periodic inspection.

Criterion 4 - Environmental and Dynamic Effects Design Bases

"Structures, systems, and components important to safety shall be designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, including loss-of-coolant accidents. These structures, systems, and components shall be appropriately protected against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids that may result from equipment failures and from events and conditions outside the nuclear power unit."

Design Evaluation

Structures, systems, and components important to safety are designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, including the design basis LOCA. These structures, systems, and components are appropriately protected against dynamic effects and discharging fluids that may result from equipment failures. Normal and postulated accident effects and load combinations are given in sections 3.6, 3.8, 3.9, and 3.10.

Special attention was directed to the effects of pipe movement, jet forces, and missiles within the primary containment. Pipe whip restraints have been provided to the extent practical. The structures, systems, and components important to safety are protected from dynamic effects by separating redundant counterparts so that no single event can prevent a required safety action and by routing and locating, to the extent practical, these components to avoid potentially hazardous areas.

Dynamic effects external to the plant, induced by natural phenomena, i.e., tornado-produced missiles, are appropriately considered in section 3.5.

Section 3.11 contains a discussion of design environmental conditions.

<u>Criterion 5 - Sharing of Structures, Systems, and Components</u>

"Structures, systems, and components important to safety shall not be shared among nuclear power units unless it can be shown that such sharing will not significantly impair their ability to perform their safety functions, including in the event of an accident in one unit an orderly shutdown and cooldown of the remaining units."

Design Evaluation

The two units of the HNP share the facilities and equipment below. Reactor safety is not impaired by sharing these facilities and equipment.

A. *Shared Facilities*

- *Main stack.*
- *Intake structure.*
- *Diesel generator building.*
- *Control building. (Main control room panels are separate; the units are controlled separately.)*
- *Refueling floor of reactor buildings.*
- *Service buildings.*
- *Water treatment building.*
- *Fire protection pump house.*
- *Waste gas treatment building.*
- *Discharge pipe to the river.*
- *High-voltage switchyard.*
- *Decontamination facility.*
- *Chlorine building.*
- *Auxiliary boiler.*

- *Turbine building (above el 164 ft).*
- *Hydrogen storage facility.*
- *Calibration facility.*
- *Hot machine shop.*
- *Central alarm station building.*
- *Hot tool room.*

B. Shared Equipment

- *One standby ac-power supply (diesel generator).*
- *Fuel pool cooling and cleanup (FPCC) systems.*
- *Fire protection system.*
- *Makeup water treatment system.*
- *Plant service water/circulating water chemical addition system.*
- *Potable and sanitary water system.*
- *Plant communication system.*
- *Main control room environmental control system.*
- *Main stack radiation monitoring system.*
- *Turbine building cranes.*
- *Reactor building crane.*
- *Fuel transfer canal.*
- *Seismic instrumentation.*
    - *Free-field strong-motion triaxial, time-history accelerograph.*
    - *Peak accelerographs - intake structure, diesel generator building, and control building.*
    - *Response spectrum recorder.*

- *Plant security system.*
- *Control building chilled water system.*
- *Diesel generator fuel storage and transfer system.*

Criterion 10 - Reactor Design

"The reactor core and associated coolant, control, and protection systems shall be designed with appropriate margin to assure that specified acceptable fuel design limits are not exceeded during any condition of normal operation, including the effects of anticipated operational occurrences."

Design Evaluation

The reactor core components consist of fuel assemblies, control rods, incore ion chambers, neutron sources, and related items. The mechanical design is based on conservative application of stress limits, operating experience and experimental test results. The fuel is designed to provide high integrity over a complete range of power levels, including transient conditions.

The core is sized with sufficient heat transfer area and coolant flow to ensure that there is no fuel damage under normal conditions or anticipated operational occurrences (AOOs).

The reactor protection system (RPS) is designed to monitor certain reactor parameters, sense abnormalities, and scram the reactor, thereby preventing fuel damage when trip points are exceeded. Scram-trip setpoints are selected on operating experience and by the safety design basis. There is no case in which the scram-trip setpoints allow the core to exceed the thermal-hydraulic safety limits. Power for the protection system is supplied by its own high inertia ac motor-generator sets. Alternate electrical power is available to the RPS buses.

An analysis and evaluation of the effects upon core fuel following adverse plant operating conditions were made. The results of AOOs are presented in section 15.2 and show that the specified acceptable fuel design limits are not exceeded, thereby assuring adequate fuel protection.

The reactor core and associated coolant, control, and protection systems are designed to ensure that the specified fuel-design limits are not exceeded during conditions of normal or abnormal operation and, therefore, meet the requirements of GDC 10.

For further discussion, see sections 4.2, 4.3, 4.4, 5.5, and 7.2, and chapter 15.

Criterion 11 - Reactor Inherent Protection

"The reactor core and associated coolant systems shall be designed so that in the power operating range the net effect of the prompt inherent nuclear feedback characteristics tends to compensate for a rapid increase in reactivity."

Design Evaluation

The reactor core is designed to have a reactivity response that regulates or damps changes in power level and spatial distributions of power production to a level consistent with safe and efficient operation. The inherent dynamic behavior of the core is characterized in terms of:

- *Fuel temperature or Doppler coefficient.*
- *Moderator void coefficient.*
- *Moderator temperature coefficient.*

The combined effect of these coefficients in the power range is termed the power coefficient.

Doppler reactivity feedback occurs simultaneously with a change in fuel temperature and opposes the power change that caused it; thus, it contributes to system stability. Since the Doppler reactivity opposes load changes, it is desirable to maintain a large ratio of moderator void coefficient to Doppler coefficient for optimum load-following capability. The boiling water reactor (BWR) has an inherently large moderator-to-Doppler coefficient ratio which permits use of coolant flowrate for load following. Load following is not used at Plant Hatch.

In a BWR, the moderator void coefficient is of primary importance during operation at power. Nuclear design is based on the void coefficient inside the fuel channel being negative. The negative void reactivity coefficient provides an inherent negative feedback during power transients. Because of the large negative moderator coefficients of reactivity, the BWR has a number of inherent advantages, such as:

- *Use of coolant flow as opposed to control rods for load following.*
- *Inherent self-flattening of the radial power distribution.*
- *Ease of control.*
- *Spatial xenon stability.*

The reactor is designed so that the moderator temperature coefficient is small and positive in the cold condition; however, the overall power reactivity coefficient is negative.

The reactor core and associated coolant system are designed so that in the power operating range prompt inherent dynamic behavior tends to compensate for any rapid increase in reactivity in accordance with GDC 11.

For further discussion, see sections 4.3 and 4.4.

<u>Criterion 12 - Suppression of Reactor Power Oscillations</u>

"The reactor core and associated coolant, control, and protection systems are designed to ensure that power oscillations which can result in conditions exceeding specified acceptable fuel design limits are not possible or can be reliably and readily detected and suppressed."

Design Evaluation

The reactor core is designed to ensure that no power oscillation will cause fuel-design limits to be exceeded. The power reactivity coefficient is the composite simultaneous effect of the fuel temperature or Doppler coefficient, moderator void coefficient, and moderator temperature coefficient to the change in power level. It is negative and well within the range required for adequate damping of power and spatial xenon disturbances. Operating experience has shown large BWRs to be inherently stable against xenon-induced power instability. The large negative operating coefficients provide:

- *Good load following with well damped behavior and little undershoot or overshoot in the heat transfer response.*

- *Load following with recirculation flow control.*

- *Strong damping of spatial power disturbances.*

The RPS design provides protection from excessive fuel-cladding temperatures and protects the nuclear system process barrier from excessive pressures which threaten the integrity of the system. Local abnormalities are sensed, and, if protection system limits are reached, corrective action is initiated through an automatic scram. High integrity of the protection system is achieved through the combination of logic arrangement, trip channel redundancy, power supply redundance, and physical separation.

The reactor core and associated coolant, control, and protection systems are designed to suppress any power oscillations which could result in exceeding of fuel-design limits. These systems assure that GDC 12 is met.

For further discussion, see sections 4.2, 4.3, 4.4, 5.2, 7.2, 7.7, and chapter 15.

<u>Criterion 13 - Instrumentation and Control</u>

"Instrumentation shall be provided to monitor variables and systems over their anticipated ranges for normal operations, for anticipated operational occurrences, and for accident conditions as appropriate to assure adequate safety, including those variables and systems that can affect the fission process, the integrity of the reactor core, the reactor coolant pressure boundary, and the containment and its associated systems. Appropriate controls shall be provided to maintain these variables and systems within prescribed operating ranges."

Design Evaluation

The fission process is monitored and controlled for all conditions from source range through power operating range. The neutron monitoring system (NMS) detects core conditions that threaten the overall

integrity of the fuel barrier due to excess power generation and provides a signal to the RPS. Fission counters, located in the core, are used for the source range through power operating range. The detectors are located to provide maximum sensitivity to control rod movement during startup and to provide optimum monitoring in the intermediate and power ranges.

The source range monitor (SRM) subsystem provides neutron flux information during reactor startup and low flux level operations. Detectors are inserted into the core for a reactor startup and may be withdrawn after neutron flux is indicated on the intermediate range monitor (IRM) subsystem. The SRMs can provide detection of less than a 20-s period under the worst possible startup conditions and provides SRM period annunciation.

The IRMs monitor neutron flux from the upper portion of the SRMS to the lower portion of the average power range monitor (APRM) subsystem. The IRMs are capable of generating a trip signal to block rod withdrawal or to scram the reactor.

The local power range monitor (LPRM) subsystem consists of fission chambers located through the core, the signal conditioning equipment, and trip functions. LPRM signals are also used in the average power range monitor (APRM) subsystem, rod block monitor (RBM) subsystem, and process computer. The RBMs are designed to prevent local fuel damage as a result of a single rod withdrawal error under a condition of allowed IRM bypass.

The traversing incore probe (TIP) subsystem provides a signal proportional to the axial neutron flux distribution of the core. This system is used in the calibration of the LPRM signal by correlation with the TIP signal.

The RPS protects the fuel barriers and the nuclear process barrier by monitoring plant parameters and causing a reactor scram when predetermined setpoints are exceeded.

The reactor manual control system (RMCS) consists of the electrical circuitry, switches, indicators, and alarm devices required to provide for the manipulation of the control rods and surveillance equipment. Separation of the scram and normal rod control function prevents failures in the reactor manual control circuitry from affecting the scram circuitry.

Reactor vessel instrumentation monitors the transient reactor vessel process temperatures, water levels, water flow, internal pressure, and water leakage detection from the top head flange. This information is used to assess conditions existing inside the vessel and the physical condition of the reactor vessel. Reactor vessel temperatures are recorded on a multipoint recorder in the control room. Controlled heating and cooling rates allow thermal stress to be appropriately limited. Reactor vessel water level is also indicated in the control room. Recirculation loop flow, core flow, and differential pressure between the reactor vessel annulus outside of the core and the core inlet plenum are indicated in the control room.

To provide protection against the consequences of accidents involving the release of radioactive material from the fuel and nuclear system process barrier, the primary containment and reactor vessel isolation control system initiates automatic isolation of appropriate pipelines which penetrate the primary containment whenever monitored variables exceed preselected operational limits.

Nuclear system leakage limits are established so that appropriate action can be taken to ensure the integrity of the nuclear system process barrier. Nuclear system leakage rates are classified as identified

and unidentified, which correspond respectively to the flow to the equipment drain and drywell floor drain sumps. The permissible total leakage rate limit to these sumps is based upon the makeup capabilities of various reactor component systems. Flow integrator and recorders are used to determine the leakage flow pumped from the drain sumps. The unidentified leakage rate, as established in chapter 5, is limited to a value that is less than the value that has been conservatively calculated to be a minimum leakage from a crack large enough to propagate rapidly but which still allows time for identification and corrective action before integrity of the process barrier is threatened.

A process computer system receives input from plant variables, including all variables of the RPS. The inputs are scanned and monitored for change of state and provide a quick and accurate determination of the core thermal performance. Certain inputs are annunciated to aid in general plant operation. The process computer system provides inputs to the rod block circuitry. The data reduction, accounting, and logging functions supplement procedural requirements for control rod manipulation during reactor startup and shutdown. Although the process computer is a valuable aid to the operator, it is not required for the safe operation of the plant.

For further discussion, see sections 4.2, 6.2, 7.2, 7.3, 7.6, and 7.7.

<u>Criterion 14 - Reactor Coolant Pressure Boundary</u>

"The reactor coolant pressure boundary shall be designed, fabricated, erected, and tested so as to have an extremely low probability of abnormal leakage, of rapidly propagating failure, and of gross rupture."

Design Evaluation

The piping and equipment pressure parts within the reactor coolant pressure boundary (RCPB) through the outer isolation valve are designed, fabricated, erected, and tested to provide a high degree of integrity throughout the plant lifetime. Subsection 3.2.2 classifies the systems and components within the RCPB as Quality Group A. The design requirements and codes and standards applied to the quality group ensure a quality product in keeping with the safety functions to be performed.

In order to minimize the possibility of brittle fracture within the RCPB, the fracture or notch properties and the operating temperature of ferritic materials are controlled to ensure adequate toughness when the system is pressurized to more than 20% of the design pressure. Section 5.2 describes the methods utilized to control toughness properties. Materials to be impact tested are tested by the Charpy V-notch method in accordance with American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section III. Service temperature of these materials is maintained above the nil ductility transition temperature (NDTT). The fracture toughness temperature requirements of the RCPB materials also apply for the RCPB piping which penetrates the containment.

Piping and equipment pressure parts of the RCPB are assembled and erected by welding unless applicable codes permit flanged or screwed joints. Welding procedures are employed which produce welds of complete penetration, of complete fusion, and free of unacceptable defects. All welding procedures, welders, and welding machine operators are qualified in accordance with the requirements of Section IX of the ASME Boiler and Pressure Vessel Code for the materials to be welded. Qualification records, including the results of procedure and performance qualification tests and identification symbols assigned to each welder, are maintained.

Section 5.2 contains the detailed material and examination requirements for the piping and equipment of the RCPB prior to and after its assembly and erection. Leakage testing and surveillance is accomplished as described in the evaluation against GDC 30.

The design, fabrication, erection, and testing of the RCPB assure an extremely low probability of failure or abnormal leakage, thus satisfying the requirements of GDC 14.

For further discussion, see sections 3.2, 5.2, 5.4, 5.5, and 7.6, and chapters 15 and 17.

Criterion 15 - Reactor Coolant System Design

"The reactor coolant system and associated auxiliary, control, and protection systems shall be designed with sufficient margin to assure that the design conditions of the reactor coolant pressure boundary are not exceeded during any condition of normal operation, including anticipated operational occurrences."

Design Evaluation

The RCS consists of the reactor vessel and appurtenances, the reactor recirculation system (RRS), the pressure relief system, the main steam lines, the reactor core isolation cooling (RCIC) system, and the residual heat removal (RHR) system. These systems are designed, fabricated, erected, and tested to stringent quality requirements and appropriate codes and standards which assure high integrity of the RCPB throughout the plant lifetime. The RCS is designed and fabricated to meet the requirements of the ASME Boiler and Pressure Vessel Code, Section III.

The auxiliary, control, and protection systems associated with the RCS act to provide sufficient margin to assure that the design conditions of the RCPB are not exceeded during any condition of normal operation, including AOOs. As described in the evaluation of GDC 13, instrumentation is provided to monitor essential variables to ensure that they are within prescribed operating limits. If the monitored variables exceed their predetermined settings, the auxiliary, control, and protection systems automatically respond to maintain the variables and systems within allowable design limits.

An example of the integrated protective action scheme which provides sufficient margin to ensure that the design conditions of the RCPB are not exceeded is the automatic initiation of the pressure relief system upon receipt of an overpressure signal. To accomplish overpressure protection, a number of pressure-operated relief valves are provided that can discharge steam from the nuclear system to the pressure suppression pool. The pressure relief system also provides for automatic depressurization of the nuclear system in the event of a LOCA in which the vessel is not depressurized by the accident. The depressurization of the nuclear system in this situation allows operation of the low-pressure emergency core cooling system (ECCS) subsystems to supply enough cooling water to adequately cool the core. In a similar manner, other auxiliary, control, and protection systems provide assurance that the design conditions of the RCPB are not exceeded during any conditions of normal operation, including AOOs.

The application of appropriate codes, standards, and high quality requirements to the RCS and the design features of its associated auxiliary, control, and protection systems ensure that the requirements of GDC 15 are satisfied.

For further discussion, see sections 5.2, 5.4, 5.5, 7.6 and chapter 15.

Criterion 16 - Containment Design

"Reactor containment and associated systems shall be provided to establish an essentially leaktight barrier against the uncontrolled release of radioactivity to the environment and to assure that the containment design conditions important to safety are not exceeded for as long as postulated accident conditions require."

Design Evaluation

The reactor is housed within a drywell containment vessel made of steel plates of 13/16-in. to 4-in. thickness. Reinforced concrete ranging in thickness from 5 ft 7 in. to 10 ft is placed around the drywell vessel. The ability of the containment vessel to provide a leaktight barrier against uncontrolled release of radioactivity is verified by a preoperational leakage test and during the life of the plant. Additional description of the primary containment is found in subsection 3.8.2.

In order to prevent the containment design conditions important to safety from being exceeded, the containment is provided with the following:

- *A pressure suppression chamber and vent system by which steam escaping into the drywell is condensed through contact with a supply of stored water (subsection 3.8.2).*

- *Cooling systems to remove heat from the water in the suppression pool (subsection 6.2.2).*

- *Drywell and suppression chamber water spraying systems to condense steam in the drywell and to cool noncondensible gases in the suppression chamber (subsection 6.2.2).*

A description of the primary containment response to the postulated design basis LOCA is provided in chapter 6.

Criterion 17 - Electric Power Systems

"An onsite electric power system and an offsite electric power system shall be provided to permit functioning of structures, systems, and components important to safety. The safety function for each system (assuming the other system is not functioning) shall be to provide sufficient capacity and capability to assure that (1) specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded as a result of anticipated operational occurrences and (2) the core is cooled and containment integrity and other vital functions are maintained in the event of postulated accidents.

The onsite electric power supplies, including the batteries, and the onsite electric distribution system, shall have sufficient independence, redundancy, and testability to perform their safety functions assuming a single failure.

Electric power from the transmission network to the onsite electric distribution system shall be supplied by two physically independent circuits (not necessarily on separate rights of way) designed and located so as to minimize to the extent practical the likelihood of their simultaneous failure under operating and postulated accident and environmental conditions. A switchyard common to both circuits is acceptable.

Each of these circuits shall be designed to be available in sufficient time following a loss of all onsite ac power supplies and the other offsite electric power circuit, to assure that specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded. One of these circuits shall be designed to be available within a few seconds following a loss-of-coolant accident to assure that core cooling, containment integrity, and other vital safety functions are maintained.

Provisions shall be included to minimize the probability of losing electric power from any of the remaining supplies as a result of, or coincident with, the loss of power generated by the nuclear power unit, the loss of power from the transmission network, or the loss of power from the onsite electric power supplies."

Design Evaluation

Both onsite and offsite electric power systems are provided to permit functioning of structures, systems, and components important to safety. With total loss-of-offsite power (LOSP), the onsite power system provides sufficient capacity and capability to assure that:

- *Specified acceptable fuel-design limits and design conditions of the RCPB are not exceeded as a result of AOOs.*

- *The core is cooled, and containment integrity and other vital functions are maintained in the event of postulated accidents.*

The description and design bases of the onsite power system (ac and dc) are discussed in subsections 8.3.1 and 8.3.2. The onsite electric power system has sufficient independence, redundancy, and testability to perform its safety function assuming a single failure.

All the Class 1E ac loads are divided into two load groups and are connected to the three 4160-V essential buses 2E, 2F, and 2G. These two load groups are independent of each other to ensure the measures specified by the bulleted items above are met considering a postulated single failure.

All the Class 1E dc loads are divided into two load groups and are connected to the two 250/125 V-dc essential buses 2A and 2B. These two load groups are independent of each other to ensure that the measures specified by the bulleted items above are met considering a postulated single failure.

Physically independent circuits as described in section 8.2 are provided from the switchyard to the startup auxiliary transformers. These circuits are fed by at least three independent transmission lines, physically separated as they approach the switchyard so that the failure of one line does not cause failure of another line. From the switchyard to the onsite electrical distribution system, separation is also provided so that failure of one circuit does not cause the failure of the other circuit.

Each of the incoming transmission lines is normally connected to the switchyard, except for short maintenance periods. One of these lines is continually connected to startup transformer 2D to supply power immediately to the essential 4160-V buses in the event of a LOCA. In the event of failure of startup transformer 2D, the essential 4160-V buses are automatically transferred to startup transformer 2C. In the event that all offsite circuits are lost, the emergency buses are isolated from the remaining portion of the onsite power system and connected to the onsite emergency diesel generators.

The turbine-generator is automatically isolated from the switchyard following a turbine or reactor trip. Therefore, its loss does not affect the ability of either the transmission network or the onsite power supplies to provide power to the Class 1E system. Transmission system stability studies indicate that the trip of the most critical fully loaded generating unit does not impair the ability of the system to supply plant station service.

Criterion 18 - Inspection and Testing of Electric Power Systems

"Electric power systems important to safety shall be designed to permit appropriate periodic inspection and testing of important areas and features, such as wiring, insulation, connections, and switchboards, to assess the continuity of the systems and the condition of their components. The systems shall be designed with a capability to test periodically (1) the operability and functional performance of the components of the systems, such as onsite power sources, relays, switches, and buses, and (2) the operability of the systems as a whole and, under conditions as close to design as practical, the full operational sequence that brings the systems into operation, including operation of applicable portions of the protection system, and the transfer of power among the nuclear power unit, the offsite power system, and the onsite power system."

Design Evaluation

The primary circuit breakers are inspected, maintained, and tested on a routine basis. This can be accomplished without removing the generators, transformers, and transmission lines from service.

Transmission line protective relaying is tested on a routine basis. This can be accomplished without removing the transmission lines from service. Generator, unit auxiliary transformer, and startup auxiliary transformer relaying is tested during refueling. Automatic transfers of 4160-V buses 2E, 2F, and 2G from startup transformers to emergency standby diesel generators are tested during the refueling of the unit to prove the operability of the system.

The 4160-V and 600-V circuit breakers and associated equipment may be tested while individual equipment is shutdown. The circuit breakers may be placed in the "test" position and tested functionally. The breaker opening and closing may also be exercised. Circuit breakers and contactors for redundant or duplicated circuits may be tested in service without interfering with the operation of the plant.

The dc system has detectors to indicate when there is a ground existing on any portion of the system. A ground on one portion of the dc systems does not cause any equipment to malfunction. Spurious activation of a system in the "safe" direction is not considered a malfunction. The batteries are under continuous automatic charging and are inspected and checked on a routine basis while the unit is in service.

To verify that the emergency power system responds within the required time limit, and properly when required, the following typical tests are performed periodically:

A. *Manually initiated demonstration of the ability of the diesel generators to start and deliver power up to nameplate rating when operating in parallel with normal power sources. Normal plant operation is not affected. The duration of the test is long enough for the diesels to reach equilibrium operating temperatures.*

B. Manual initiation of permanently installed testing devices demonstrate the ability of the control system to automatically start the diesel generator and restore power to vital equipment by simulating an LOSP and/or a LOCA.

These tests include:

- *Test for automatic transfer of emergency buses being supplied by the normal offsite power source to the alternate offsite power source.*

- *Test for automatically starting, connecting the diesel generators to the emergency bus, and loading the diesel generators upon LOSP sources.*

- *Test for automatically starting diesel generators upon a LOCA signal.*

- *Test for automatically starting, connecting diesel generators to the emergency buses, and sequentially loading the diesel generators upon a LOCA signal accompanied by an LOSP signal.*

The capability to perform the above tests complies with the intent of GDC 18.

<u>Criterion 19 - Control Room</u>

"A control room shall be provided from which actions can be taken to operate the nuclear power unit safely under normal conditions and to maintain it in a safe condition under accident conditions, including loss-of-coolant accidents. Adequate radiation protection shall be provided to permit access and occupancy of the control room under accident conditions without personnel receiving radiation exposures in excess of 5 rem whole body, or its equivalent to any part of the body, for the duration of the accident.

Equipment at appropriate locations outside the control room shall be provided (1) with a design capability for prompt hot shutdown of the reactor, including necessary instrumentation and controls to maintain the unit in a safe condition during hot shutdown and (2) with a potential capability for subsequent cold shutdown of the reactor through the use of suitable procedures."

Design Evaluation

A control room has been provided in which appropriate controls and instrumentation are located to permit personnel to safely operate the unit under normal conditions or maintain it in a safe condition under accident conditions. The MCR and associated post-accident ventilation systems are designed in accordance with Category I requirements.

The design of the control room permits access and occupancy during a LOCA. Previous analyses demonstrate that the LOCA is the limiting event for radiological exposures to operators in the MCR. Therefore, for extended power uprate conditions (2763 MWt), only the LOCA was analyzed for MCR radiological exposures. The results of the analysis bound the subsequent power uprate conditions (2804 MWt) including the conditions for the reactor operating pressure increase to 1060 psia. Sufficient shielding and ventilation are provided to permit occupancy of the control room for a period of 30 days

following the LOCA, without receiving more than 5-rem integrated whole-body dose or its equivalent to any part of the body. An analysis of exposures within the control room is presented in section 15.3.

The ability for prompt hot shutdown of the reactor and the potential capability for subsequent cold shutdown through the use of suitable procedures from locations outside the control room is provided by the remote shutdown system, should the control room become inaccessible. The remote shutdown system has the capability for prompt hot shutdown of the reactor, including necessary instrumentation and control to maintain the unit in a safe condition during hot shutdown, and subsequent cold shutdown of the reactor through use of administrative procedures. The remote shutdown system panel contains controls for the following equipment:

A. *RHR system - The controls for one loop of the RHR system are provided on the remote shutdown panel. All modes of the RHR system operation, low-pressure coolant injection (LPCI), suppression pool cooling, containment spray cooling, and shutdown cooling can be operated from the remote shutdown panel.*

B. *RCIC system - All basic RCIC equipment can be controlled from the remote shutdown panel.*

C. *Reactor recirculation system - The suction valve of one recirculation pump can be controlled from the remote shutdown panel.*

D. *Automatic depressurization system (ADS) - Two manual blowdown valves can be operated from the remote shutdown panel.*

In addition, the diesel generator can be operated from the local panel in the diesel generator building.

<u>Criterion 20 - Protection System Functions</u>

"The protection system shall be designed (1) to initiate automatically the operation of appropriate systems including the reactivity control system, to assure that specified acceptable fuel design limits are not exceeded as a result of anticipated operational occurrences and (2) to sense accident conditions and to initiate the operation of systems and components important to safety."

Design Evaluation

The RPS is designed to provide timely protection against the onset and consequences of conditions that threaten the integrity of the fuel barrier and nuclear system process barrier. Fuel damage is prevented by initiation of an automatic reactor shutdown if monitored nuclear system variables exceed preestablished limits of AOOs. Scram trip settings are selected and verified to be far enough above or below operating levels to provide proper protection but not be subject to spurious scrams. The RPS includes the motor-generator power system, sensors, relays, bypass circuitry, and switches that signal the control rod system to scram and shutdown the reactor. The scrams initiated by NMS variables, nuclear system high-pressure, turbine stop valve closure, turbine control valve fast closure, and reactor vessel low water level prevent fuel damage following AOOs. Specifically, these process parameters initiate a scram in time to prevent the core from exceeding thermal-hydraulic safety limits during AOOs. Response by the RPS is prompt, and the total scram time is short.

A fully withdrawn control rod (withdrawn to 144 in.) traverses 90% of its full stroke in less than 5 s, which is sufficient to assure that acceptable fuel-design limits are not exceeded.

In addition to the RPS which provides for automatic shutdown of the reactor to prevent fuel damage, protection systems are provided to sense accident conditions and initiate automatically the operation of other systems and components important to safety. Subsystems, such as the ECCS, are initiated automatically to limit the extent of fuel damage following a LOCA. Other systems automatically isolate the reactor vessel or the primary containment to limit the extent of fuel damage following a postulated LOCA and prevent the release of significant amounts of radioactive material from the fuel and the nuclear system process barrier. The control and instrumentation for the ECCS and the isolation systems are initiated automatically when monitored variables exceed preselected operational limits.
The design of the protection system satisfies the functional requirements as specified in GDC 20.

For further discussion, see sections 4.2, 6.3, 7.2, 7.3, and 7.6 and chapter 15.

Criterion 21 - Protection System Reliability and Testability

"The protection system shall be designed for high functional reliability and inservice testability commensurate with the safety functions to be performed. Redundancy and independence designed into the protection system shall be sufficient to assure that (1) no single failure results in loss of the protection function and (2) removal from service of any component or channel does not result in loss of the required minimum redundancy unless the acceptable reliability of operation of the protection system can be otherwise demonstrated. The protection system shall be designed to permit periodic testing of its functioning when the reactor is in operation, including capability to test channels independently to determine failures and losses of redundancy that may have occurred."

Design Evaluation

The RPS design fulfills single-failure criteria by providing redundant channels. No single component failure, intentional bypass maintenance operation, calibration operation, or test to verify operational availability impairs the ability of the system to perform its intended safety function. Additionally, the system design assures that when a scram trip point is exceeded there is a high scram probability. However, should a scram not occur, other monitored components scram the reactor if their trip points are exceeded. There is sufficient electrical and physical separation between channels and between trip logics monitoring the same variable to prevent environmental factors, electrical transients, and physical events from impairing the ability of the system to respond correctly.

The RPSs include design features that permit inservice testing. This ensures the functional reliability of the system should the reactor variable exceed the corrective action setpoint.

The RPS initiates an automatic reactor shutdown if the monitored plant variables exceed preestablished limits. The protection system consists of two independently powered trip systems. Each trip system has three trip logics, two of which produce an automatic trip signal. The logic scheme is a one-out-of-two-taken-twice arrangement.

The RPS can be tested during reactor operation. Manual scram testing is performed by operating the two manual scram controls. This tests one trip system. The total test verifies the ability to deenergize the

scram pilot valve solenoids. Indicating lights verify that the actuator contacts have opened. This capability for a thorough testing program significantly increases reliability.

Control rod drive (CRD) operability can be tested during normal reactor operation. Drive position indicators and the incore neutron detectors are used to verify control rod movement. Each control rod can be withdrawn one notch and then reinserted to the original position without significantly perturbing the reactor system. One control rod is tested at a time. Control rod mechanism overdrive demonstrates rod-to-drive coupling integrity. Hydraulic supply subsystem pressures can be observed on control room instrumentation. More importantly, the hydraulic control unit scram accumulator and the scram discharge volume level are continuously monitored.

The main steam isolation valves (MSIVs) may be tested during full reactor operation. They can be closed to 90% of full-open position without affecting the reactor operation. If reactor power is reduced to 75% of full power, an isolation valve may be fully closed. Provisions are provided to evaluate valve stem leakage during reactor shutdown. During refueling operation, valve leakage rates can be determined.

The RHR system testing can be performed during normal operation. Main system pumps can be evaluated by taking suction from the suppression pool. System design and operating procedures also permit testing the discharge valves to the reactor recirculation loops and discharge valves to the containment spray headers. The LPCI mode can be tested after reactor shutdown. Each ECCS active component which operates in a DBA is designed to be testable.

The high functional reliability, redundancy, and inservice testability of the protection system satisfy the requirements specified in GDC 21.

For further discussion, see sections 9.2, 5.5, 6.2, 6.3, 7.2, 7.3, and chapter 15.

Criterion 22 - Protection System Independence

"The protection system shall be designed to assure that the effects of natural phenomena and of normal operating, maintenance, testing, and postulated accident conditions on redundant channels do not result in loss of the protection function, or shall be demonstrated to be acceptable on some other defined basis. Design techniques, such as functional diversity or diversity in component design and principles of operation, shall be used to the extent practical to prevent loss of the protection function."

Design Evaluation

The components of protection systems are designed so that the mechanical and thermal environment resulting from any emergency situation in which the components are required to function do not interfere with that function. Wiring for the RPS outside of the control room enclosures is run in rigid metallic conduits or raceways segregated from all other wiring. The wires from duplicate sensors on a common process tap are run in separate conduits. The system sensors are electrically and physically separated. Only one trip actuator logic circuit from each trip system may be run in the same wireway.

The RPS is designed to permit maintenance and diagnostic work while the reactor is operating without restricting the plant operation or hindering the output of any safety functions. The flexibility in design embodied in the protection system allows operational system testing by the use of an independent trip channel for each trip logic input. When an essential monitored variable exceeds its scram trip point, it is

sensed by at least two independent sensors in each trip system. An intentional bypass, maintenance operation, calibration operation, or test results in a single channel trip. This leaves at least two trip channels per monitored variable capable of initiating a scram. Only one trip channel in each trip system must trip to initiate a scram. Thus, the arrangement of two trip channels per trip system assures that a scram occurs as a monitored variable exceeds its scram setting.

The protection system meets the design requirements for functional and physical independence as specified in GDC 22.

For further discussion, see sections 4.2, 5.5, 6.3, 7.2, 7.3, and 7.6.

<u>Criterion 23 - Protection System Failure Modes</u>

"The protection system shall be designed to fail into a safe state or into a state demonstrated to be acceptable on some other defined basis if conditions such as disconnection of the system, loss of energy (e.g., electric power, instrument air), or postulated adverse environments (e.g., extreme heat or cold, fire, pressure, steam, water, and radiation) are experienced."

Design Evaluation

The RPS is designed to fail into a safe state. Use of an independent trip channel for each trip logic allows the system to sustain any trip channel failure without preventing other sensors monitoring the same variable from initiating a scram. A single sensor or trip channel failure causes a channel trip. Only one trip channel must trip in each trip system to initiate a scram. Intentional bypass, maintenance operation, calibration operation, or test results in a single channel trip. A failure of any one RPS input or subsystem component produces a trip in one of two channels. This condition is insufficient to produce a reactor scram, but the system is ready to perform its protection function upon another trip.

The environmental conditions in which the instrumentation and equipment of the RPS must operate were considered in establishing the component specifications. Instrumentation specifications for the reactor and turbine buildings are based on the worst expected ambient conditions in which the instruments must operate.

The failure modes of the protection system are such that it fails into a safe state as required by GDC 23.

For further discussion, see sections 3.11, 6.3, 7.2, 7.3, 7.6, and chapter 8.

<u>Criterion 24 - Separation of Protection and Control Systems</u>

"The protection system shall be separated from control systems to the extent that failure of any single control system component or channel, or failure or removal from service of any single protection system component or channel which is common to the control and protection systems leaves intact a system satisfying all reliability, redundancy, and independence requirements of the protection system. Interconnection of the protection and control systems shall be limited to assure that safety is not significantly impaired."

Design Evaluation

There is separation between the RPS and the process systems. Sensors, trip channels, and trip logics of the RPS are not used directly for automatic control of process systems. Therefore, failure in the controls and instrumentation of process systems cannot induce failure of any portion of the protection system. High scram reliability is designed into the RPS and hydraulic control unit for the CRD. The scram signal and mode of operation overrides all other signals.

The containment and reactor vessel isolation control systems are designed so that any one failure, maintenance operation, calibration operation, or test to verify operational availability not impair the functional ability of the isolation control system to respond to essential variables.

The RPS is separated from control systems as required in GDC 24.

For further discussion, see sections 6.3, 7.2, 7.3, and 7.6.

<u>Criterion 25 - Protection System Requirements for Reactivity Control Malfunctions</u>

"The protection system shall be designed to assure that specified acceptable fuel design limits are not exceeded for any single malfunction of the reactivity control systems, such as accidental withdrawal (not ejection or dropout) of control rods."

Design Evaluation

The RPS provides protection against the onset and consequences of conditions that threaten the integrity of the fuel barrier and the nuclear system process barrier. Any monitored variable which exceeds the scram setpoint initiates an automatic scram and does not impair the remaining variables from being monitored and, if one channel fails, the remaining portions of RPS shall function.

The RMCS is designed so that no single failure can negate the effectiveness of a reactor scram. The circuitry for the RMCS is completely independent of the circuitry controlling the scram valves. This separation of the scram and normal rod control functions prevents failures in the reactor manual control circuitry from affecting the scram circuitry. Because each control rod is controlled as an individual unit, a failure that results in energizing any of the insert or withdraw solenoid valves can affect only one control rod. The effectiveness of a reactor scram is not impaired by the malfunctioning of any one control rod.

The most serious rod withdrawal errors occur when an out-of-sequence rod is continuously withdrawn while the reactor is just subcritical. The rod worth minimizer (RWM) would normally prevent the withdrawal of out-of-sequence control rods.

If such a continuous rod withdrawal were to occur, the increase in fuel temperature subsequent to scram would not be sufficient to exceed acceptable fuel-design limits.

The design of the protection system assures that specified acceptable fuel-design limits are not exceeded for any single malfunction of the reactivity control systems as specified in GDC 25.

For further discussion, see sections 4.2, 4.3, 4.4, 7.2, 7.7, and chapter 15.

<u>Criterion 26 - Reactivity Control System Redundancy and Capability</u>

"Two independent reactivity control systems of different design principles shall be provided. One of the systems shall use control rods, preferably including a positive means for inserting the rods, and shall be capable of reliably controlling reactivity changes to assure that under conditions of normal operation, including anticipated operational occurrences, and with appropriate margin for malfunctions such as stuck rods, specified acceptable fuel design limits are not exceeded. The second reactivity control system shall be capable of reliably controlling the rate of reactivity changes resulting from planned, normal power changes (including xenon burnout) to assure that acceptable fuel design limits are not exceeded. One of the systems shall be capable of holding the reactor core subcritical under cold conditions."

Design Evaluation

Two independent reactivity control systems utilizing different design principles are provided. The normal method of reactivity control employs control rod assemblies which contain boron-carbide (B_4C) powder. Control of reactivity is operationally provided by a combination of these movable control rods, burnable poisons, and reactor coolant recirculation system flow. These systems accommodate fuel burnup, load changes, and long-term reactivity changes.

Reactor shutdown by the CRD system is sufficiently rapid to prevent exceeding of acceptable fuel-design limits for normal operation and all AOOs. The circuitry for manual insertion or withdrawal of control rods is completely independent of the circuitry for reactor scram. This separation of the scram and normal rod control functions prevents failures in the reactor manual control circuitry from affecting the scram circuitry. Because each control rod is controlled as an individual unit, a failure that results in energizing any of the insert or withdraw solenoid valves can affect only one control rod. Two sources of scram energy (accumulator pressure and reactor vessel pressure) provide needed scram performance over the entire range of reactor pressure, i.e., from operating conditions to cold shutdown.

The design of the CRD system includes appropriate margin for malfunctions, such as stuck rods, in the highly unlikely event that they do occur. Control rod withdrawal sequences and patterns are selected prior to operation to achieve optimum core performance, and, simultaneously, low individual rod worths. The operating procedures to accomplish such patterns are supplemented by the RWM program of the process computer, which prevents rod withdrawals yielding a rod worth greater than permitted by the preselected rod withdrawal pattern. An additional safety design basis of the CRD system requires that the core in its maximum reactivity condition be subcritical with the control rod of the highest worth fully withdrawn and all other rods fully inserted. Because of the carefully planned and regulated rod withdrawal sequence, prompt shutdown of the reactor can be achieved with the insertion of a small number of the many independent control rods. In the event that a reactor scram is necessary, the unlikely occurrence of a limited number of stuck rods does not hinder the capability of the CRD system to render the core subcritical.

A standby liquid control system containing neutron absorbing sodium pentaborate solution is the independent backup system. This system has the capability to shut the reactor down from full power and maintain it in a subcritical condition at any time during the core life. The reactivity control provided to reduce reactor power from rated to a shutdown condition with the control rods withdrawn in the power pattern accounts for the reactivity effects of xenon decay, eliminating steam voids, change in water density due to the reduction in water temperature, Doppler effect in uranium, changing neutron leakage from boiling to cold, and changing rod worth as boron affects neutron migration length.

The redundancy and capabilities of the reactivity control systems for the BWR satisfy the requirements of GDC 26.
For further discussion, see sections 4.2, 7.4, 7.6, and 7.7.

Criterion 27 - Combined Reactivity Control Systems Capability

"The reactivity control systems shall be designed to have a combined capability, in conjunction with poison addition by the emergency core cooling system, of reliably controlling reactivity changes to assure that, under postulated accident conditions and with appropriate margin for stuck rods, the capability to cool the core is maintained."

Design Evaluation

There is no credible event applicable to the BWR which requires combined capability of the CRD system and poison additions by the emergency core cooling network. The primary reactivity control system for the BWR during postulated accident conditions is the CRD system. Abnormalities are sensed, and, if protection system limits are reached, corrective action is initiated through an automatic scram. High integrity of the protection system is achieved through the combination of logic arrangement, trip channel redundancy, and physical separation. High reliability of reactor scram is further achieved by separation of scram and manual control circuitry, individual control units for each control rod, and fail-safe design features built into the CRD system. Response by the RPS is prompt, and the total scram time is short.

In operating the reactor, there is a spectrum of possible control rod worths, depending on the reactor state and the control rod pattern chosen for operation. Control rod withdrawal sequences and patterns are selected to achieve optimum core performance and low individual rod worths. The RWM prevents rod withdrawal other than by the preselected rod withdrawal pattern. These functions assist the operator with an effective backup control rod monitoring routine that enforces adherence to established startup, shutdown, and low-power-level operations. As a result of this carefully planned procedure, prompt shutdown of the reactor can be achieved with scram insertion of less than half of the many independent control rods. If accident conditions require a reactor scram, this can be accomplished rapidly with appropriate margin for the unlikely occurrence of malfunctions such as stuck rods.

The reactor core design assists in maintaining the stability of the core under accident conditions as well as during power operation. Reactivity coefficients in the power range that contribute to system stability are:

- *Fuel temperature or Doppler coefficient.*
- *Moderator void coefficient.*
- *Moderator temperature coefficient.*

The overall power reactivity coefficient is negative and provides a strong negative reactivity feedback under severe power transient conditions.

The design of the reactivity control systems assures reliable control of reactivity under postulated accident conditions with appropriate margin for stuck rods. The capability to cool the core is maintained under all postulated accident conditions; thus, GDC 27 is satisfied.

For further discussion, see sections 4.2, 4.3, 4.4, 7.2, 7.6, 7.7, 7.10, and chapter 15.

Criterion 28 - Reactivity Limits

"The reactivity control systems shall be designed with appropriate limits on the potential amount and rate of reactivity increase to assure that the effects of postulated reactivity accidents can neither (1) result in damage to the reactor coolant pressure boundary greater than limited local yielding nor (2) sufficiently disturb the core, its support structures or other reactor pressure vessel internals to impair significantly the capability to cool the core. These postulated reactivity accidents shall include consideration of rod ejection (unless prevented by positive means), rod dropout, steam line rupture, changes in reactor coolant temperature and pressure, and cold water addition."

Design Evaluation

The CRD system design incorporates appropriate limits on the potential amount and rate of reactivity increase. Control rod withdrawal sequences and patterns are selected to achieve optimum core performance and low individual rod worths. The RWM prevents withdrawal other than by the preselected rod withdrawal pattern. These functions assist the operator with an effective backup control rod monitoring routine that enforces adherence to established startup, shutdown, and low-power-level control rod procedures.

The control rod mechanical design incorporates a hydraulic velocity limiter in the control rod which prevents rapid rod ejection. This engineered safeguard protects against a high reactivity insertion rate by limiting the control rod velocity to < 5 ft/s.

The safety analysis (chapter 15) evaluates the postulated reactivity accidents as well as the AOOs in detail. Analyses are included for rod dropout, steam line rupture, changes in reactor coolant temperature and pressure, and cold water addition. The initial conditions, assumptions, calculational models, sequence of events, and anticipated results of each postulated occurrence are covered in detail. The results of these analyses indicate that none of the postulated AOOs or accidents result in damage to the RCPB. In addition, the integrity of the core, its support structures, or other reactor pressure vessel internals are maintained so that the capability to cool the core is not impaired for any of the postulated reactivity accidents described in the safety analysis.

The design features of the reactivity control system which limit the potential amount and rate of reactivity increase ensure that GDC 28 is satisfied for all postulated reactivity accidents.

For further discussion, see sections 4.2, 4.3, 4.5, 5.2, 5.4, 5.5, and chapters 3 and 15.

Criterion 29 - Protection Against Anticipated Operational Occurrences

"The protection and reactivity control systems shall be designed to assure an extremely high probability of accomplishing their safety functions in the event of anticipated operational occurrences."

Design Evaluation

The high functional reliability of the protection and reactivity control systems is achieved through the combination of logic arrangement, redundancy, physical and electrical independence, functional separation, fail-safe design, and inservice testability. These design features are discussed in GDC 21, 22, 23, 24, and 26.

An extremely high probability of correct protection and reactivity control systems response to AOOs is maintained by a thorough program of inservice testing and surveillance. Active components can be tested or removed from service for maintenance during reactor operation without compromising the protection or reactivity control functions even in the event of a subsequent single failure. Components important to safety such as CRDs, MSIVs, pumps, etc., are tested during normal reactor operation. Functional testing and calibration schedules are developed using available failure rate data, reliability analyses, and operating experience. These schedules represent an optimization of protection and reactivity control system reliability by considering, on one hand, the reliability effects during individual component testing on the portion of the system not undergoing test. The capability for inservice testing ensures the high functional reliability of protection and reactivity control systems should a reactor variable exceed the corrective action setpoint.

The capabilities of the protection and reactivity control systems to perform their safety functions in the event of AOOs are satisfied in agreement with the requirements of GDC 29.

For further discussion, see sections 4.2, 5.5, 6.2, 6.3, 7.2, 7.3, 7.6, and chapter 15.

Criterion 30 - Quality of Reactor Coolant Pressure Boundary

"Components which are part of the reactor coolant pressure boundary shall be designed, fabricated, erected, and tested to the highest quality standards practical. Means shall be provided for detecting and, to the extent practical, identifying the location of the source of reactor coolant leakage."

Design Evaluation

By utilizing conservative design practices and detailed quality control procedures, the pressure retaining components of the RCPB are designed and fabricated to retain their integrity during normal and postulated accident conditions. Accordingly, components which comprise the RCPB are designed, fabricated, erected, and tested in accordance with recognized industry codes and standards listed in chapter 5. Further product and process quality planning is provided to assure conformance with the applicable codes and standards and to retain appropriate documented evidence verifying compliance. Because the subject matter of this criterion deals with the aspects of the RCPB, further discussion on this subject is treated in the response to GDC 14.

Means are provided for detecting reactor coolant leakage. The leak detection system consists of sensors and instruments to detect, annunciate, and, in some cases, isolate the RCPB from potential hazardous leaks before predetermined limits are exceeded. Small leaks are detected by temperature and pressure changes, increased frequency of sump pump operation, and by measuring fission product concentration in the primary containment atmosphere. In addition to these means of detection, large leaks are detected by flowrates in process lines and changes in reactor water level. The allowable leakage rates have been based on the predicted and experimentally determined behavior of cracks in pipes, the ability to makeup

coolant system leakage, the normally expected background leakage due to equipment design, and the detection capability of the various sensors and instruments. The total leakage rate limit is established so that, in the absence of normal ac power concomitant with a loss of feedwater supply, makeup capabilities are provided by the CRD and RCIC systems. While the leak detection system provides protection from small leaks, the ECCS network provides protection for the complete range of discharges from ruptured pipes. Thus, protection is provided for the full spectrum of possible discharges.

The RCPB and the leak detection system are designed to meet the requirements of GDC 30.

For further discussion, see sections 5.2, 5.4, 5.5, 7.6, and chapters 3 and 15.

Criterion 31 - Fracture Prevention of Reactor Coolant Pressure Boundary

"The reactor coolant pressure boundary shall be designed with sufficient margin to assure that, when stressed under operating, maintenance, testing, and postulated accident conditions (1) the boundary behaves in a nonbrittle manner and (2) the probability of rapidly propagating fracture is minimized. The design shall reflect consideration of service temperatures and other conditions of the boundary material under operating, maintenance, testing, and postulated accident conditions and the uncertainties in (1) determining material properties, (2) the effects of irradiation on material properties, (3) residual, steady state, and transient stresses, and (4) size of flaws."

Design Evaluation

Brittle fracture control of pressure-retaining ferritic materials is provided to ensure protection against nonductile fracture. To minimize the possibility of brittle fracture failure of the reactor pressure vessel, it is designed to meet the requirements of the ASME Boiler and Pressure Vessel Code, Section III.

The NDTT is defined as the temperature below which ferritic steel breaks in a brittle rather than ductile manner. The NDTT increases as a function of neutron exposure at integrated neutron exposures greater than about 3.8×10^{17} nvt with neutrons of energies in excess of 1 MeV. Since the material NDTT dictates the minimum operating temperature at which the reactor vessel can be pressurized, it is desirable to keep the NDTT as low as possible.

The reactor assembly design provides an annular space from the outermost fuel assemblies to the inner surface of the reactor vessel that serves to attenuate the fast neutron flux incident upon the reactor vessel wall. This annular volume contains the core shroud, jet pump assemblies, and reactor coolant. Assuming plant operation at rated power availability of 100% and a plant life of 40 years, the fast neutron fluence at the inner surface of the vessel is calculated to be 3.8×10^{17} nvt. (A fast neutron fluence consists of neutrons having energies > 1 MeV.)

The end-of-life NDTT provides a substantial margin for the prevention of brittle fracture because the vessel cannot be pressurized until coolant temperatures exceed 212°F. For hydrostatic test, the vessel is not pressurized until the vessel temperature exceeds the NDTT by at least 60°F. Therefore, during operation when pressure depends on temperature, brittle failure of the vessel is not possible until the neutron fluence of the reactor vessel reaches a value of the order of 10^{20} nvt. This value is more than 250 times the maximum fast neutron fluence calculated during the lifetime of this plant.

The RCPB is designed, maintained, and tested such that adequate assurance is provided so that the boundary behaves in a nonbrittle manner throughout the life of the plant.

For further discussion, see sections 5.2, 5.4, and chapter 15.

Criterion 32 - Inspection of Reactor Coolant Pressure Boundary

"Criteria which are part of the reactor coolant pressure boundary shall be designed to permit (1) periodic inspection and testing of important areas and features to assess their structural and leaktight integrity, and (2) an appropriate material surveillance program for the reactor pressure vessel."

Design Evaluation

The HNP-2 design conforms with the intent of GDC 32. The unit's RCPB design meets the requirements of the ASME Boiler and Pressure Vessel Code, Section XI, including Summer 1971 Addenda, except replacement of recirculation piping, stainless steel portions of RHR, and portions of RWC which meet the inspection requirements of the Winter 1980 Addenda which requires access for all required inspections. The design also permits the conduct of a material surveillance program for the reactor pressure vessel. Additional details of these features can be found in subsections 5.2.8 and 5.2.4.

The reactor recirculation piping and main steam piping were hydrostatically tested with the reactor pressure vessel at a test pressure that is in accordance with Section III of the ASME Code. Current hydrostatic testing is in accord with Section XI requirements (subsection 5.2.8).

Vessel material surveillance samples are located within the reactor pressure vessel to enable periodic monitoring of material properties with exposure. The program includes specimens of the base metal, heat-affected zone within the base metal, and weld metal.

For further discussion, see sections 5.2, 5.4, 5.5, and chapter 3.

Criterion 33 - Reactor Coolant Makeup

"A system to supply reactor coolant makeup for protection against small breaks in the reactor coolant pressure boundary shall be provided. The system safety function shall be to assure that specified acceptable fuel design limits are not exceeded as a result of reactor coolant loss due to leakage from the reactor coolant pressure boundary and rupture of small piping or other small components which are part of the boundary. The system shall be designed to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished using the piping, pumps, and valves used to maintain coolant inventory during normal reactor operation."

Design Evaluation

The total leakage rate limit is established so that, in the absence of normal ac power concomitant with a loss of feedwater supply, makeup capabilities are provided by the CRD and RCIC systems. While the leak detection system provides protection from small leaks, the ECCS provides protection for the complete range of discharges from ruptured pipes. Thus, protection is provided for the full spectrum of possible discharges to the extent that fuel-cladding temperature limits are not exceeded.

The plant is designed to provide ample reactor coolant makeup for protection against small leaks in the RCPB for AOOs and postulated accident conditions. The design of these systems meets the requirements of GDC 33.

For further discussion, see sections 5.2, 5.6, 6.3, and 7.6.

Criterion 34 - Residual Heat Removal

"A system to remove residual heat shall be provided. The system safety function shall be to transfer fission-product decay heat and other residual heat from the reactor core at a rate such that specified acceptable fuel design limits and the design conditions of the reactor coolant pressure boundary are not exceeded.

Suitable redundancy in components and features and suitable interconnections, leak detection, and isolation capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure."

Design Evaluation

The RHR system provides the means to:

- *Remove decay heat and residual heat from the nuclear system so that refueling and nuclear system servicing can be performed.*

- *Supplement the FPCC system capacity during shutdown to provide additional cooling capacity.*

The major equipment of the RHR system consists of two heat exchangers, four main system pumps, and four service water pumps. The equipment is connected by associated valves and piping, and the controls and instrumentation are provided for proper system operation. The main system pumps are sized on the basis of the flow required during the LPCI mode of operation, which is the mode requiring the maximum flowrate. The heat exchangers are sized on the basis of the required duty for the shutdown cooling function, which is the mode requiring the maximum heat exchanger capacity.

One loop, consisting of a heat exchanger, two main system pumps in parallel, and associated piping, is located in one area of the reactor building. The other heat exchanger, pumps, and piping forming a second loop, are located in another area of the reactor building to minimize the possibility of a single physical event causing the loss of the entire system. The two loops of the RHR system are

cross-connected by a single header, making it possible to supply either loop from the pumps in the other loop. Either of these redundant loops can meet fully the most limiting of the three modes of operation.

The RHR system is designed for three modes of operation:

- *Shutdown cooling.*
- *Containment cooling.*
- *LPCI.*

Both normal ac power and auxiliary onsite power systems provide adequate power to operate all the auxiliary loads necessary for plant operation. The power sources for the plant auxiliary power system are sufficient in number and of such electrical and physical independence that no single probable event could interrupt all auxiliary power at one time.

The plant auxiliary buses supplying power to engineered safety features (ESFs) and RPSs and those auxiliaries required for safe shutdown are connected by appropriate switching to the standby diesel-driven generators located in the plant. Each power source, up to the point of its connection to the auxiliary power buses, is capable of complete and rapid isolation from any other source.

Loads important to plant operation and safety are split and diversified between switch gear sections, and means are provided for detection and isolation of system faults.

The plant layout is designed to effect physical separation of essential bus sections, standby generators, switchgear, interconnections, feeders, power centers, motor control centers, and other system components.

Three standby diesel generators (one shared with HNP-1) are provided to supply a source of electrical power which is self-contained within the plant and is not dependent on external sources of supply. The standby generators produce ac power at a voltage and frequency compatible with the normal bus requirements for essential equipment within the plant. Each of the diesel generators has sufficient capacity to start and carry the essential loads it is expected to drive. All of the auxiliary loads required for safe and orderly shutdown, including components of the RHR system, are duplicated and connected to separate buses.

The RHR systems are adequate to remove residual heat from the reactor core to assure fuel and RCPB design limits are not exceeded. Redundant offsite and onsite electric power systems are provided. The design of the RHR system, including their power supplies, meets the requirements of Criterion 34.

For further discussions, see sections 5.5, 6.3, 7.3, 8.3, 9.2, and chapter 15.

Criterion 35 - Emergency Core Cooling

"A system to provide abundant emergency core cooling shall be provided. The system safety function shall be to transfer heat from the reactor core following any loss of reactor coolant at a rate such that

(1) fuel and clad damage that could interfere with continued effective core cooling is prevented and (2) clad metal-water reaction is limited to negligible amounts.

Suitable redundancy in components and features and suitable interconnections, leak detection, isolation, and containment capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure."

Design Evaluation

The ECCS consists of the following subsystems:

- *High-pressure coolant injection (HPCI) system.*
- *ADS.*
- *Core spray (CS) system.*
- *LPCI (an operating mode of the RHR system).*

The ECCS is designed to limit fuel-cladding temperature over the complete spectrum of possible break sizes in the nuclear system process barrier, including a complete and sudden circumferential rupture of the largest pipe connected to the reactor vessel.

The HPCI system consists of a steam turbine, a constant-flow pump, system piping, valves, controls, and instrumentation. The HPCI system is provided to assure that the reactor core is adequately cooled to prevent excessive fuel-cladding temperatures for breaks in the nuclear system which do not result in rapid depressurization of the reactor vessel. The HPCI system continues to operate until reactor vessel pressure is below the pressure at which LPCI operation or CS system operation maintains core cooling. Two sources of water are available, namely the condensate storage tank or the suppression pool.

In case the capability of the feedwater pumps, CRD water pumps, and RCIC and HPCI systems is not sufficient to maintain the reactor water level, the ADS functions to reduce the reactor pressure so that flow from LPCI and the CS system enters the reactor vessel in time to cool the core and prevent excessive fuel-cladding temperature. The ADS uses several of the nuclear system pressure relief valves to relieve the high pressure steam to the suppression pool.

Two independent loops are provided as a part of the CS system. Each loop consists of a centrifugal water pump driven by an electric motor, a spray sparger in the reactor vessel above the core, piping, and valves to convey water from the suppression pool to the sparger, and the associated controls and instrumentation. In case of low water level in the reactor vessel or high pressure in the drywell, the CS system automatically sprays water onto the top of the fuel assemblies in time and at a sufficient flowrate to cool the core and prevent excessive fuel temperature. LPCI starts from the same signals which initiate the CS and operates independently to achieve the same objective by flooding the reactor vessel.

In case of low water level in the reactor or high pressure in the containment drywell, the LPCI mode of operation of the RHR system pumps water into the reactor vessel in time to flood the core and prevent

excessive fuel temperature. LPCI operation provides protection to the core for the case of a large break in the nuclear system when the feedwater pumps and the HPCI system are unable to maintain reactor vessel water level. Protection provided by LPCI also extends to a small break where the ADS has operated to lower the reactor vessel pressure so LPCI and the CS system start to provide core cooling.

Results of the performance of the ECCS for the entire spectrum of liquid line breaks are discussed in section 6.3.

The ECCS provided is adequate to prevent fuel and cladding damage which could interfere with effective core cooling and do limit clad metal-water reaction to a negligible amount. Redundant offsite and onsite electric power systems are provided. The design of each ECCS subsystem, including power supplies, meets the requirements of GDC 35.

For further discussion, see sections 5.5, 6.3, 7.3, 8.3, 9.2, and chapter 15.

Criterion 36 - Inspection of Emergency Core Cooling System

"The emergency core cooling system shall be designed to permit appropriate periodic inspection of important components, such as spray rings in the reactor pressure vessel, water injection nozzles, and piping, to assure the integrity and capability of the system."

Design Evaluation

The CS spargers within the vessel are accessible for remote visual inspection during refueling outages. Removable plugs in the sacrificial shield and/or panels in the insulation provide access for examination of nozzles from the vessel outside diameter. Removable insulation is provided on the ECCS piping out to and including the first isolation valve outside containment. Inspection of the ECCS is in accordance with the intent of Section XI of the ASME Code insofar as is practicable.

During plant operations, the pumps, valves, piping, instrumentation, wiring, and other components outside the primary containment can be visually inspected at any time. Components inside the primary containment can be inspected when the drywell is open for access. When the reactor vessel is open for refueling or other purposes, the spargers and other internals can be inspected. Portions of the ECCS which are part of the RCPB are designed to specifications for inservice inspection to detect defects which might affect the cooling performance. Particular attention is given to the reactor nozzles, CS, and feedwater spargers. The design of the reactor vessel and internals for inservice inspection and the plant testing and inspection program ensures that the requirements of GDC 36 are met. Refer to subsection 5.2.8 for a further discussion on ECCS inservice inspection.

<u>Criterion 37 - Testing of Emergency Core Cooling System</u>

"The emergency core cooling system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leaktight integrity of its components, (2) the operability and performance of the active components of the system, and (3) the operability of the system as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the system into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of the associated cooling water system.

Design Evaluation

The ECCS consists of the HPCI system, ADS, LPCI mode of the RHR system, and the CS system. Each of these systems is provided with sufficient test connections and isolation valves to permit appropriate periodic pressure testing to assure the structural and leaktight integrity of its components.

The HPCI system, LPCI, and CS system are designed to permit periodic testing to assure the operability and performance of the active components of each system.

The complete ECCS is subjected to tests to verify the performance of the full operational sequence that brings each system into operation.

The operation of the associated cooling-water systems is discussed in the response to GDC 46."

<u>Criterion 38 - Containment Heat Removal</u>

"A system to remove heat from the reactor containment shall be provided. The system safety function shall be to reduce rapidly, consistent with the functioning of other associated systems, the containment pressure and temperature following any loss-of-coolant accident and to maintain them at acceptably low levels.

Suitable redundancy in components and features and suitable interconnections, leak detection, isolation, and containment capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure."

Design Evaluation

In the event of a LOCA within the reactor containment, the pressure suppression system rapidly condenses the steam to prevent containment overpressure. The containment feature of pressure suppression employs two separate compartmented sections of the primary containment: the drywell that houses the nuclear system and the suppression chamber containing a large volume of water. Any increase in pressure in the drywell from a leak in the nuclear system is relieved below the surface of the suppression chamber water pool by connecting vent lines, thereby condensing steam being released to the drywell. The pressure buildup in the suppression chamber is equalized with the drywell by a vent line and vacuum breaker arrangement. Cooling systems remove heat from the reactor core, the drywell, and from the water in the suppression chamber during accident condition and, thus, continuous cooling of the primary containment is provided.

The ECCS is actuated to provide core cooling in the event of a LOCA. Low water level in the reactor vessel or high pressure in the drywell initiates the ECCS to prevent excessive fuel temperature. Sufficient water is provided in the suppression pool to accommodate the initial energy which can transiently be released into the drywell from the postulated pipe failure.

The suppression chamber is sized to contain this water plus the water displaced from the reactor primary system, together with the free air initially contained in the drywell.

Either or both RHR heat exchangers can be manually activated to remove energy from the containment. The redundancy and capability of the offsite and onsite electrical power systems for the RHR system is presented in the evaluation against GDC 34.

The pressure suppression system is capable of rapid containment pressure and temperature reduction following a LOCA to assure that the design limits are not exceeded. Redundant offsite and onsite electrical power systems are provided. The design of the containment RHR meets the requirements of GDC 38.

For further discussion, see sections 5.5, 6.2, 6.3, 7.3, and chapters 8, 9, and 15.

<u>Criterion 39 - Inspection of Containment Heat Removal System</u>

"The containment heat removal system shall be designed to permit appropriate periodic inspection of important components, such as the torus, sumps, spray nozzles, and piping, to assure the integrity and capability of the system."

Design Evaluation

Provisions are made to facilitate periodic inspections of active components and other important equipment of the containment pressure-reducing systems. During plant operations, the pumps, valves, piping, instrumentation, wiring, and other components outside the primary containment can be visually inspected periodically. Components inside the primary containment can he inspected when the drywell is open for access. The testing frequencies of most components are correlated with the component inspection.

The suppression chamber is designed to permit appropriate periodic inspection. Space is provided outside the chamber for inspection and maintenance. There are two hatches that permit access to the suppression chamber for inspection.

The containment heat removal system is designed to permit periodic inspection of major components both outside and within the primary containment. This design meets the requirements of Criterion 39.

For further discussion, see sections 5.5, 6.2, 6.3, 7.3, and 9.2.

<u>Criterion 40 - Testing of Containment Heat Removal System</u>

"The containment heat removal system shall be designed to permit appropriate periodic pressure and functional testing to (1) assure the structural and leaktight integrity of its components, (2) the operability and performance of the active components of the system, and (3) the operability of the system as a whole

and, under conditions as close to the design as practical, the performance of the full operational sequence that brings the system into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of the associated cooling water system."

Design Evaluation

The containment heat removal function is accomplished by the containment cooling mode of the RHR system. This mode consists of the suppression pool cooling subsystem and containment spray subsystem.

The RHR system is provided with sufficient test connections and isolation valves to permit periodic pressure testing. The pumps and valves of the RHR system are operated periodically to verify operability.

Criterion 41 - Containment Atmosphere Cleanup

"Systems to control fission products, hydrogen, oxygen, and other substances which may be released into the reactor containment shall be provided as necessary to reduce, consistent with the functioning of other associated systems, the concentration and quantity of fission products released to the environment following postulated accidents and to control the concentration of hydrogen or oxygen and other substances in the containment atmosphere following postulated accidents to assure that containment integrity is maintained.

Each system shall have suitable redundancy in components and features and suitable interconnections, leak detection, isolation, and containment capabilities to assure that, for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available), its safety function can be accomplished, assuming a single failure."

Design Evaluation

Fission products released into the reactor building following postulated accidents are automatically processed by the SGTS. The SGTS initiation follows high radiation signals from monitors in the refueling floor exhaust duct, monitors in the reactor building exhaust duct or from the primary containment isolation system. The ability of this system to remove radioactivity from the process stream is discussed in subsection 6.2.4. The SGTS is composed of two trains which are separated physically and electrically so that a single failure does not prevent its function. The redundancy of this system is also discussed in subsection 6.2.4.

A combustible gas control system (CGCS), which is now removed and piping retired in place, consisted of redundant hydrogen recombiners to maintain hydrogen and oxygen concentrations below flammable limits following a postulated LOCA. The system processed the primary containment atmosphere continuously following manual initiation. The system was designed in accordance with the Branch Technical Position CSB 6-2, attached to USNRC Regulatory Standard Review Plan, Subsection 6.2.5, March 1975, and met the requirements of an ESF system. The title 10 CFR 50.44 no longer defines a design-basis LOCA hydrogen release and no longer requires hydrogen control systems to mitigate such a release. The hydrogen and oxygen concentrations are maintained below flammable limits following a postulated LOCA by the containment atmospheric dilution (CAD) system, a subsystem to the primary containment purge and inerting system. A detailed description is provided in paragraph 6.2.5.6.

Criterion 42 - Inspection of Containment Atmosphere Cleanup Systems

"The containment atmosphere cleanup systems shall be designed to permit appropriate periodic inspection of important components, such as filter frames, ducts, and piping, to assure the integrity and capability of the system."

Design Evaluation

Inspection of the internal structure of the SGTS filter banks is facilitated by access doors installed in each unit to allow entry to the unit for visual inspection of structural members and filter faces.

Each charcoal bed is provided with facilities for taking a sample of charcoal.

For a further discussion of the SGTS inspection features, refer to paragraph 6.2.4.5.

Criterion 43 - Testing of Containment Atmosphere Cleanup Systems

"The containment atmosphere cleanup systems shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leaktight integrity of its components, (2) the operability and performance of the active components of the systems such as fans, filters, dampers, pumps, and valves, and (3) the operability of the systems as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the system into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of associated systems."

Design Evaluation

Each unit of the SGTS is operated periodically to ascertain the operability and performance of the major active components such as fans, filters, motors, and valves, and structural integrity of the unit. This test also verifies the operability of the system as a whole and operability of all associated subsystems. See paragraph 8.3.1.1 for a discussion of the testing of the auxiliary power system.

The leaktightness of the high-efficiency particulate air (HEPA) filters is measured by the dioctyl phthalate (DOP) test. The charcoal beds are checked for bypass with halogenated hydrocarbon. The efficiency of the charcoal adsorbers is checked by lab testing. For further discussion on testing, refer to paragraph 6.2.4.5.

Criterion 44 - Cooling Water

"A system to transfer heat from structures, systems, and components important to safety to an ultimate heat sink shall be provided. The system safety function shall be to transfer the combined heat load of these structures, systems, and components under normal operating and accident conditions.

Suitable redundancy in components and features and suitable interconnections, leak detection, and isolation capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure."

Design Evaluation

The RHR service water (RHRSW) system (subsection 9.2.7) and the plant service water (PSW) system (subsection 9.2.1) transfer the heat loads from structures, systems, and components important to safety during normal operating, shutdown, and accident conditions to the ultimate heat sinks (subsection 9.2.5).

The RHRSW and PSW systems are designed with sufficient redundancy of components and piping so that no single failure can prevent the achieving of the safety cooling objective. Assuming a single failure, the electrical power supplies to valving are such that at least one train of cooling water is provided. Sufficient redundancy exists in the electrical power supply to ensure minimum safety pumping requirements are met. (See paragraph 8.3.1.4.)

Criterion 45 - Inspection of Cooling-Water System

"The cooling water system shall be designed to permit appropriate periodic inspection of important components, such as heat exchangers and piping, to assure the integrity and capability of the system."

Design Evaluation

To the extent practical and consistent with other design considerations, the components of the RHRSW and PSW systems are located to facilitate visual inspection. (See subsections 9.2.7 and 9.2.1.)

Criterion 46 - Testing of Cooling-Water System

"The cooling water system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leaktight integrity of its components, (2) the operability and the performance of the active components of the system, and (3) the operability of the system as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the system into operation for reactor shutdown and for loss-of-coolant accidents, including operation of applicable portions of the protection system and the transfer between normal and emergency power sources."

Design Evaluation

The pumps and automatic valves are tested periodically to verify operation. Since the PSW system is normally in operation, no special tests are required to ensure that the system can operate in an emergency. Periodic tests are conducted to verify the operation of the RHRSW system. The specific tests which are to be conducted are discussed more fully in the Technical Specifications, the associated Bases, and plant procedures. Chapter 8 discusses the tests which are conducted to ensure the availability of electrical power. The pumps and valves of these systems which must operate in an emergency are powered from a standby ac distribution system.

Criterion 50 - Containment Design Basis

"The reactor containment structure, including access openings, penetrations, and the containment heat removal system, shall be designed so that the containment structure and its internal compartments can accommodate, without exceeding the design leakage rate and, with sufficient margin, the calculated pressure and temperature conditions resulting from any loss-of-coolant accident. This margin shall

reflect consideration of (1) the effects of potential energy sources which have not been included in the determination of the peak conditions, such as energy in steam generators and energy from metal-water and other chemical reactions that may result from degraded emergency core cooling functioning, (2) the limited experience and experimental data available for defining accident phenomena and containment responses, and (3) the conservatism of the calculational model and input parameters."

Design Evaluation

The containment structure, access openings, penetrations, heat removal system, and internal compartments are designed with sufficient margin to meet the intent of GDC 50.

Criterion 51 - Fracture Prevention of Containment Pressure Boundary

"The reactor containment boundary shall be designed with sufficient margin to assure that, under operating, maintenance, testing, and postulated accident conditions, (1) its ferritic materials behave in a nonbrittle manner and (2) the probability of rapidly propagation fracture is minimized. The design shall reflect consideration of service temperatures and other conditions of the containment boundary material during operation maintenance, testing, and postulated accident conditions and the uncertainties in determining (1) material properties, (2) residual, steady-state, and transient stresses, and (3) size of flaws."

Design Evaluation

The reactor containment vessel is fabricated to the requirements of the ASME Boiler and Pressure Vessel Code, Section III, Subsection NE for Class ML components. This code in article NE-2000 gives due recognition to the requirement that containment materials behave in a ductile manner for all conditions of service, thus assuring that its ferritic materials behave in a nonbrittle manner and that the probability of rapidly propagating fracture is minimized. The lowest design service temperature is conservatively taken as 30°F. The actual service temperature is calculated to be ~ 135°F. Thus, sufficient margin is inherent in the design to account for the various uncertainties involved in design and fabrication.

Criterion 52 - Capability for Containment Leakage Rate Testing

"The reactor containment and other equipment which may be subjected to containment test conditions shall be designed so that periodic integrated leakage rate testing can be conducted at containment design pressure."

Design Evaluation

The primary reactor containment and other equipment including the personnel airlock and isolation valves are designed to permit type A, B, and C leakage tests to be conducted in accordance with Appendix J of 10 CFR 50. A more complete discussion can be found in subsection 3.8.2 and in the Technical Specifications.

Criterion 53 - Provisions for Containment Testing and Inspection

"The reactor containment shall be designed to permit (1) appropriate periodic inspection of all important areas, such as penetrations, (2) an appropriate surveillance program, and (3) periodic testing at

containment design pressure of the leaktightness of penetrations which have resilient seals and expansion bellows."

Design Evaluation

The reactor containment is designed to optimize the accessibility of important areas to permit required inspection and surveillance.

All penetrations with resilient seals or expansion bellows are the double seal type. The space between the seals may be periodically pressurized to containment design pressure and their leaktightness verified. This is discussed in subsection 3.8.2.

Criterion 54 - Piping Systems Penetrations Containment

"Piping systems penetrating primary reactor containment shall be provided with leak detection, isolation, and containment capabilities having redundancy, reliability, and performance capabilities which reflect the importance to safety of isolating these piping systems. Such piping systems shall be designed with a capability to test periodically the operability of the isolation valves and associated apparatus and to determine if valve leakage is within acceptable limits."

Design Evaluation

Piping systems which penetrate the drywell have been accorded special design considerations to reflect their importance in accomplishing safety-related functions and in achieving isolation, if required. The penetrations are discussed in subsections 3.8.2 and 6.2.1. Both the isolation valving and system which initiates isolation use components whose quality maximizes reliability and are provided with sufficient independence and redundancy, to optimize the isolation function should it be required. Containment isolation is discussed in subsections 3.8.2 and 6.2.5, and the system which initiates isolation is discussed in subsection 7.3.2.

The operation of remote manual isolation valves is periodically verified according to the Technical Specifications. Sufficient test connections are provided to each of these piping systems to ensure that minimal valve leakage is achieved and maintained (paragraph 6.2.1.4).

Criterion 55 - Reactor Coolant Pressure Boundary Penetrating Containment

"Each line that is part of the reactor coolant pressure boundary and that penetrates the primary reactor containment shall be provided with containment isolation valves as follows unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

(1) One locked-closed isolation valve inside and one locked-closed isolation valve outside containment.

(2) One automatic isolation valve inside and one locked-closed isolation valve outside containment.

(3) One locked-closed isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment.

(4) One automatic isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment."

Isolation valves outside containment shall be located as close to the containment as practical and upon loss of actuating power automatic isolation valves shall be designed to take the position that provides greater safety.

Other appropriate requirements to minimize the probability or consequences of an accidental rupture of these lines or of lines connected to them shall be provided as necessary to assure adequate safety. Determination of the appropriateness of these requirements, such as higher quality in design, fabrication, testing, additional provisions for inservice inspection, protection against more severe natural phenomena, and additional isolation valves and containment, shall include consideration of the population density, use characteristics, and physical characteristics of the site environs."

Design Evaluation

The RCPB consists of the RPV, the pressure-retaining appurtenance attached to the vessel, and valves and pipes which extend from the RPV up to and including the outermost isolation valve. The lines of the RCPB which penetrate the primary containment are capable of isolating the containment, thereby precluding any significant release of radioactivity. Similarly, for lines which do not penetrate the primary containment but which form a portion of the RCPB the design ensures that isolation from the RCPB can be achieved.

Influent Lines

Influent lines which penetrate the primary containment and connect directly to the RPV are equipped with two isolation valves; one inside the containment and the other outside located as close to the containment as possible or a single isolation valve outside but as close to containment as possible with a closed system as the second isolation barrier.

Table 3.1-1 lists those influent pipes that comprise the RCPB. The purpose of this table is to review the design of each line with respect to the requirements imposed by GDC 55. The following discussion provides additional detail on the specific conformance of each line with GDC 55. (The comment numbers link applicable information to specific influent lines in table 3.1-1.)

Comment 55.1

The portion of the feedwater line which forms part of the RCPB and penetrated the primary containment has three isolation valves. The valve inside the containment is a simple check valve while the two valves outside the containment are air assisted check valves.

The air-assisted check valve is an internal balance type in that the disc arm is cut out and mated with a shaft dog. With instrument air established on the spring loaded air cylinder, the valve disc is free to swing without movement of the shaft.

During normal feedwater flow, the valve disc is fully open by action of the force on the disc, due to flow alone. Upon reversal of flow the valve closes as a free swinging check valve. In addition, the control room operator may assist in starting valve closure by removing the open signal form the solenoid valve. The solenoid valve isolates the air supply to the cylinder and exhausts the cylinder air to the atmosphere. The air cylinder closing spring forces the piston downward, which in turn, acting through the piston rod, pulls down the closing lever on the shaft, and by means of closing dogs on the shaft and disc arm, closes and holds the disc to its seat.

The valve remains in this position until air pressure is again established in the cylinder and the piston is moved upward. During accidental loss of instrument air, the valve remains open because the force of the flow overcomes the spring force. The amount of valve opening is proportionate to the amount of flow. Should a break occur in the feedwater line, the check valves prevent significant loss of inventory and offer immediate isolation. During the postulated LOCA, it is desirable to maintain reactor coolant makeup from all sources of supply. For this reason, the outer feedwater isolation gate valve does not automatically isolate upon a signal from the primary containment isolation protection system.

Comment 55.2

Influent lines which connect to process piping but do not penetrate the primary containment must adequately reflect the importance to safety of isolating these piping systems. Pipes of this type include those portions of the RCIC, the reactor water cleanup (RWC), and the HPCI lines that tie into the feedwater line. The RCIC and HPCI lines have motor-operated, automatic, and remote-manually actuated gate valves that are closed during normal operation, whereas the RWC line is open during operation and has a simple check valve to provide positive assurance of isolation in the event of a break upstream of this valve. In addition to the check valve, the RWC line has a normally open, remote, manually actuated, motor-operated globe valve capable of providing leakage control.

Comment 55.3

The RHR suction and return lines to and from the recirculation system and the CS lines have a single, normally closed, automatic, and remote-manually actuated isolation valve as the inboard isolation barrier in each line. The isolation valve is installed outside primary containment and as close to the containment as possible. Since the CS and RHR systems meet the criteria for closed systems outside primary containment and are operating post-LOCA, adequate containment isolation provisions are provided and no additional primary containment isolation valves are required.

However, additional valves are provided inside primary containment on each of the subject lines to afford dual barriers between high- and low-pressure systems. The CS and RHR return lines utilize a check valve for this purpose. The RHR suction from the recirculation line uses a motor-operated gate valve.

The motor-operated gate valves in the suction line automatically close when primary coolant pressure is greater than the RHR system design pressure. Both suction gate valves also receive automatic primary containment isolation signals. Thus, the gate valve inside primary containment also serves as a backup to the isolation valve.

For the postulated LOCA, the protection system initiates automatic opening of the CS and LPCI valves at the appropriate time.

Comment 55.6

The standby liquid control line utilizes a simple check valve as the isolation valve inside as well as outside the primary containment. GDC 55 states that a simple check valve may not be used as the automatic isolation valve outside the containment; however, should insertion of the liquid poison become necessary, it is imperative that the injection line be open. In the design of this system, it has been the accepted practice to omit an automatic valve that opens on signal as this introduces a possible failure mechanism. As a means of providing assurance for reliable timely actuation, an explosive valve is used. In this manner, the availability of the line is assured. Because the standby liquid control line is a closed, nonflowing line, rupture of this line is very remote. However, should a break occur, the check valves provide positive actuation for immediate isolation.

Comment 55.7

<u>Other</u>

1. *CRD Insert and Withdraw Lines*

   GDC 55 applies to lines of the RCPB which penetrate the primary reactor containment. The CRD insert and withdraw lines are not part of the RCPB.

   The basis to which the CRD lines are designed is commensurate with the safety importance of isolating these lines. Since these lines are vital to the scram function, their operability is of utmost concern.

   In the design of this system, it has been accepted practice to omit automatic valves for isolation purposes as this introduces a possible failure mechanism. As a means of providing positive actuation, manual shutoff valves are used. In the event of a break of these lines, the manual valves may be closed to ensure isolation. In addition, a ball valve located in the insert line is designed to automatically seal this line in the event of a break.

   Finally, several breaks and combinations of breaks in the CRD lines have been postulated and analyzed (chapter 4). The results of these analyses indicate that the worst situation causes a leak rate which is negligible compared to the makeup capability.

2. *TIP System*

   Since the TIP system lines do not communicate freely with the containment atmosphere and since they do not comprise a portion of the RCPB, GDC 56 and 55 are not directly applicable to this specific class of lines. The basis to which these lines are designed is more closely described by GDC 54, which states, in effect, that isolation capability of a system be commensurate with the safety importance of the isolation. Furthermore, even though the failure of the TIP system lines presents no safety hazard, the TIP system has redundant isolation capabilities. These and other safety features are described in the following paragraphs.

When the TIP system cable is inserted, the ball valve of the select tube opens automatically so that the probe and cable may advance. A maximum of four valves may be opened at any one time to conduct the calibration, and any one guide tube is used, at most, a few hours per year.

If closure of the line is required during calibration, a signal causes the cable to be retracted and the ball valve to close automatically after completion of cable withdrawal. To ensure isolation capability if a TIP cable fails to withdraw or a ball valve fails to close, an explosive shear valve is installed in each line. Upon receipt of a signal, this explosive valve shears the TIP cable and seal the guide tube.

Comment 55.8

The recirculation pump seal water line extends from the recirculation pump through the drywell and connects to the CRD supply line outside the containment. Isolation is provided by a check valve installed inside primary containment and a check valve outside primary containment, installed as close to containment, as possible. Since the seal water line does form a part of the RCPB the consequence of breaking this line has been evaluated. This evaluation shows that the consequence of breaking this line is less severe than that of failing an instrument line. The recirculation pump seal water line is 3/4-in. Quality Group B from the recirculation pump through the second check valve. From this valve to the CRD connection the line is Quality Group D. Should this line be postulated to fail and either one of the check valves is assumed not to close, the flowrate through the broken line has been calculated to be substantially less than that permitted for a broken instrument line.

Therefore, the two check valves in series provide sufficient isolation capability for postulated failure of this line. Installation of an automatic power actuated valve outside primary containment would increase the probability of inadvertent isolation of the seal water supply, which could result in failure of the recirculation pump seal and a possibly avoidable breach of the primary coolant boundary during normal reactor operation.

Effluent Lines

Effluent lines, except instrument sensing lines, and the RHR recirculation system suction which form part of the RCPB and penetrate the primary containment are equipped with two isolation valves; one inside the containment and the other outside located as close to the containment as possible.

Table 3.1-2 lists those effluent pipes that comprise the reactor coolant pressure boundary and which penetrate the primary containment.

Aside from the MSIVs, each valve is a motor-operated, automatic, or remote-manually actuated gate valve capable of providing adequate isolation protection in the event of a break in these lines. The MSIVs are air-operated, automatic, and remote-manually actuated globe valves which provide two distinct barriers against containment leakage. Upon loss of actuating power, automatic isolation valves assume the position that provides greater safety. The protection system initiates automatic isolation under accident conditions for effluent lines which are normally open during operation and which are not part of the overall safety system network.

Instrument lines are designed in accordance with Regulatory Guide 1.11, as discussed in subsection 6.2.5.

Summary

In order to assure protection against the consequences of accidents involving the release of radioactive material, pipes which form the RCPB have been shown to provide adequate isolation capabilities on a case-by-case basis. In all cases, a minimum of two barriers is shown to protect against the release of radioactive materials. Adequate isolation capabilities were also demonstrated for pipes that connect to the feedwater line outside the primary containment.

In addition to meeting the isolation requirements stated in GDC 55, the pressure retaining components which comprise the RCPB are designed to meet other appropriate requirements which minimize the probability or consequences of an accidental rupture. The quality requirements for these components ensure that they are designed, fabricated, and tested to the highest quality standards of all reactor plant components.

It can, therefore, be concluded that the design of piping systems which comprise the RCPB satisfies GDC 55.

For further discussion, see subsections 6.2.5 and 7.3.2.

<u>Criterion 56 - Primary Containment Isolation</u>

"Each line that connects directly to the containment atmosphere and penetrates the primary reactor containment shall be provided with containment isolation valves, as follows, unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

(1) One locked-closed isolation valve inside and one locked-closed isolation valve outside containment; or

(2) One automatic isolation valve inside and one locked-closed isolation valve outside containment; or

(3) One locked-closed isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment; or

(4) One automatic isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment.

Isolation valves outside containment shall be located as close to the containment as practical and, upon loss of actuating power, automatic isolation valves shall be designed to take the position that provides greater safety."

Design Evaluation

Lines which penetrate the primary containment and communicate with the containment interior may be grouped into four categories:

- *Pipes which communicate with the drywell or suppression chamber atmosphere.*
- *Influent lines to the suppression pool.*
- *Effluent lines from the suppression pool.*
- *Instrumentation sensing lines.*

Lines Which Communicate with the Drywell or Suppression Chamber Atmosphere

Several lines which penetrate the primary containment and communicate with the drywell or suppression pool atmosphere are provided with isolation valves outside primary containment, rather than one isolation valve inside and one isolation valve outside primary containment. These lines are identified in ***Technical Requirements Manual (TRM) table T7.0-1 (incorporated by reference into the FSAR)****.*

This deviation from the GDC is considered safe and adequate for the following reasons:

A. *Lines which penetrate the containment for atmosphere sampling or processing presently terminate at the inboard end of the welded-in drywell penetration sleeve. The sleeve and the piping connected to it on the outboard side of the primary containment are Seismic Category I, Quality Group B up to and including at least the second primary containment isolation valve.*

   Installation of the inboard isolation valve inside primary containment would require supporting the valves from the drywell shell, resulting in additional welds, and/or extending the piping, and adding supports from other structural members within the drywell. Since there is limited space within the drywell, placing these valves inside would severely impede accessibility for inspection and maintenance of the valves and other equipment.

B. *Placing the valves inside the containment would subject them to an inimical environment and, thus, increase the probability of failure.*

   The environment within the drywell and torus post-LOCA could be especially detrimental to the operation of the drywell and torus spray valves, since these modes of RHR would be used after the postulated event and the spray lines valves would be required to function during the postulated containment pressure transient.

   The design spray coverage further necessitates the location of the spray header as close as practical to the interior of the drywell and torus shells.

   Therefore, the isolation valve for each torus spray and drywell spray line is installed outside primary containment. The outboard isolation barrier is the closed RHR system. This system is Quality Group B, Seismic Category I, and has been evaluated for missile hazards.

In addition to the two barriers required by GDC 56, the torus and drywell spray lines each have a motor-operated valve installed inboard of the containment isolation valve. This design reflects the importance of avoiding an inadvertent initiation of containment sprays during plant operation. These valves also contribute additional conservatism to the containment isolation provisions since they will shut, if open, upon receipt of the same isolation signals as the isolation valves.

C. *Several of the lines which fall into this category are not in use during normal operation and are, therefore, isolated. Valves which are normally closed during plant operation are identified in* ***TRM table T7.0-1****.*

D. *Valves are accessible in systems which must be available for long-term operation following an accident. Examples are the containment atmosphere monitoring lines and the* H_2 *recombiner system.*

E. *Isolation valves installed outside primary containment are compatible with minimizing personnel exposure during maintenance and inspections.*

*Isolation valves for this category of line are either locked closed, administratively closed, or are automatically closed upon receipt of an isolation signal (****TRM table T7.0-1****).*

The isolation valves in each line are installed as close together and as close to the primary containment as practical.

Influent Lines of Suppression Pool

The reasons for not placing valves inside the suppression chamber are similar to those mentioned in the preceding section. The following discussion provides unique considerations as to the types of valves and isolation capabilities for the RCIC and HPCI turbine exhaust lines, HPCI turbine condensate line, and RCIC vacuum pump discharge line.

These lines penetrate the torus and discharge below the minimum suppression pool water level. Two primary containment isolation valves are provided outside the torus on each line. The inboard isolation valve for each line is a locked open globe stop check valve. When in its normal position, open, the valve allows flow into the suppression pool. The valve may be manually closed by a local handwheel to provide long-term leakage control. When closed, the valve exhibits characteristics identical to a standard globe valve and tightly seals against flow in either direction. The outboard isolation valve is a simple swing check valve and functions as a redundant isolation valve to ensure backflow from the suppression pool is prohibited.

The HPCI and RCIC turbines are designed to operate with an exhaust pressure < 65 psia. A high-exhaust pressure trip and isolation signal are provided to avoid damage to the turbine and/or steam release to the secondary containment through the turbine seals. The installation of remotely operated or automatically operated isolation valves in these lines would increase the probability of inadvertent isolation of the exhaust lines for the HPCI and RCIC turbines and would, therefore, be detrimental to the operability of the systems.

The HPCI turbine condensate line is the normal drain path from the HPCI turbine exhaust line drain pot. Failure to maintain the drain pot adequately drained could ultimately result in a high-exhaust pressure. Therefore, the same design considerations apply to the isolation valves on this line as on the exhaust lines.

Operation of the turbine gland-seal condenser components is required to prevent outleakage from the turbine shaft seals. Therefore, the RCIC vacuum pump discharge line utilizes two check valves as isolation valves to reduce the probability of inadvertent isolation.

It should be noted that each of the lines in this category are Seismic Category I and Quality Group B, as required for ESF-related systems. Also, leak detection is provided, as described in paragraph 5.2.7.2.3.7.

Suppression Chamber-to-Reactor Building Vacuum Relief Lines.

The suppression chamber-to-reactor building vacuum breakers consist of two lines penetrating into the suppression chamber atmosphere, each with a self actuating check valve and an air operated butterfly valve. This design does not meet the explicit statement of GDC 56 because the GDC prohibits the use of a simple check valve. However, the GDC also states that other criteria are acceptable if "...it can be demonstrated that the containment isolation provisions for a specific class of lines,..., are acceptable on some other defined basis."

In the case of the suppression chamber-to-reactor building vacuum relief lines, the "other defined basis" is the GE design specification for these valves which explicitly states that one of these vacuum breaker valves may be self actuated instead of remotely operated, i.e., a simple check valve. This was reviewed and approved by the NRC and, as a result, the HNP-2 design for the suppression chamber to reactor building vacuum breakers complies with GDC 56 via the "other defined basis" portion of the GDC.

Minimum Flow and Test Lines

These lines have isolation capabilities which are commensurate with the importance to safety of isolating these lines. The HPCI and RCIC minimum flow lines have two valves in series, both of which are located outside the primary containment. The RHR and CS minimum flow lines have single isolation valves, each a normally open, motor-operated gate valve, and the closed RHR or CS system serves as the second containment isolation barrier.

The HPCI and RCIC minimum flow lines utilize a normally shut, motor-operated globe valve as the inboard isolation barrier and an upstream check valve as the outboard isolation barrier. The globe valves also function as flow control devices and automatically shut when no start signal is present for the HPCI and RCIC turbines or when flow is established in the system.

The CS and RHR minimum flow line isolation valves cycle as required by system flow conditions. However, the valves may be remote-manually shut and leakage detection is provided as described in subsection 5.2.7.

In addition to the isolation valve, each CS and RHR minimum flow line has a swing check valve installed for reverse flow prevention. The check valve in each CS minimum flow line is installed inboard of the isolation valve, the RHR minimum flow check valves are installed outboard of the isolation valve. In each

case, the check valve acts against flow from the primary containment direction and, therefore, provides additional conservatism in the containment isolation provisions designed for this class of line.

The jockey pump system maintains the CS and RHR pump discharge lines constantly full of water with two continuously running pumps. Minimum flow lines are provided for the jockey pumps to avoid overheating and damaging cavitation, either of which could occur if the pumps were continually running at shutoff head. The jockey pump minimum flow lines return water to the suppression pool via the CS test line. A single-globe stop check valve is installed in each minimum flow line as an isolation valve. Since the system is continually pressurized and operating, further isolation provisions are not required. Since the operation of the bypass lines discussed in the previous paragraphs is required to ensure the pumps are not damaged, the lines are important to the operations of the safety systems. Since automatic isolation valves could degrade the reliability of these systems, no further valving has been incorporated.

The CS test line has a single automatic isolation valve installed outside primary containment. The CS system is a Quality Group B, Seismic Category I, missile protected systems. Therefore, the closed system is the second isolation barrier and no further isolation is required. Also, CS operating pressure post-LOCA is > P_a.

The RHR test line has a single automatic isolation valve outside primary containment. RHR relief valves 2E11-F025A&B, -F029, and -F097 discharge into the test lines downstream of the containment isolation valves. Each relief valve has a setpoint < 1.5 times the containment design pressure. Since RHR is a closed system and operates post-LOCA at a pressure greater than peak pressure, no further containment isolation provisions are provided in the relief valve lines or the RHR test line.

RHR relief valves 2E11-F055A&B and recombiner valves 2T49-F009A&B discharge into the suppression pool through two other penetrations. Design characteristics and isolation provisions of these lines are the same as for the relief valves discharging into the RHR test line.

All lines in this category terminate below the water level of the suppression pool. Each line is ESF related, is Seismic Category I, Quality Group B, and is part of a Seismic Category I, Quality Group B system. Leakage detection is available for this category of lines, as discussed in subsection 5.2.7.

Effluent Lines from Suppression Pool (RHR, CS, HPCI, RCIC)

It should be noted that GDC 56 does not reflect consideration of the BWR suppression pool design. Certain lines, such as the RHR, CS, HPCI, and RCIC suction lines, penetrate below the waterline in the suppression pool and, therefore, do not communicate with the containment atmosphere. These lines do not have an isolation valve located inside the containment as this would necessitate placement of the valve under water. In effect, this would result in introducing a potentially unreliable valve in a highly reliable system, thereby compromising design. For this reason, these lines incorporate isolation valves outside the containment.

The HPCI and RCIC suction lines have two remote-manually operated isolation valves. The inboard valve is an air-operated butterfly valve located as close to the containment as possible. The second valve is a motor-operated gate valve.

The RHR and CS suction lines have a single, remote-manual, gate valve as a containment isolation valve. The second isolation barrier is provided by the respective closed system. In addition, the RHR and CS

suction lines have an air-operated butterfly valve installed as close to the containment as possible, to facilitate isolating the gate valve for maintenance and testing.

Because of the importance of the above suction lines to core cooling, none of these valves receive an automatic isolation signal. Leakage detection capabilities are for those lines which have remote-manually operated isolation valves. The leakage detection provisions are discussed in paragraph 5.2.7.2.3.1 and subsection 9.3.3.

The torus drainage and purification system vacuum drag line has two air-operated butterfly valves with automatic isolation signals. The torus cleanup transfer system has two locked-closed manual gate valves and the piping downstream is blank flanged during normal operation to assure greater protection from fluid leakage.

Instrumentation Lines which Communicate with the Primary Containment Atmosphere

Instrumentation sensing lines are designed in accordance with Regulatory Guide 1.11. Each line has a manual blocking valve for maintenance and testing and a remotely operated control valve for isolation.

Leakage in one of these sensing lines would be indicated by abnormal instrument outputs.

The instrumentation lines are further discussed in subsection 6.2.5.

Hydrogen Recombiner System (This system equipment is removed and the piping retired in place.)

The hydrogen recombiner supply and return lines utilize a single remote-manual, motor-operated gate valve installed in each line as the inboard primary containment isolation barrier. The second, or outboard isolation barrier is provided by the closed system.

The remote-manually operated isolation valve has been utilized in consideration of the fact that the system is designed to be operated post-LOCA and is ESF related. Installation of the valve outside of primary containment ensures accessibility of the system for maintenance and testing.

In lieu of leakage detection, which is difficult and unreliable for a gaseous system operating under the conditions of the recombiner system, the hydrogen recombiner system is designed to minimize the possibility of leakage.

The blower and motor are contained in a single canned housing to ensure any leakage through the blower seal would be contained.

An additional motor-operated gate valve is installed in each supply and return line inboard of the isolation valve to facilitate testing of the isolation valve and to ensure that any part of the system, including the isolation valves, may be isolated from the primary containment for maintenance, if required.

Drywell Pneumatic System

The design of the drywell pneumatic system satisfies the requirements of 10 CFR 50, Appendix A, GDC 56, concerning containment isolation. Two automatic isolation valves are provided for the compressor inlet and for each discharge header in the drywell pneumatic system.

Isolation of the two discharge headers during a LOCA is provided by the pneumatic system instrumentation which will generate a high-flow signal and automatically close the redundant isolation valves should an air header be reptured inside the drywell. The redundant isolation valves ensure single-failure proof containment isolation in the event that the system is breached during a LOCA. Provisions are included to allow containment leakage testing per 10 CFR 50, Appendix J.

Summary

To assure protection in the event of accidents involving release of significant amounts of radioactive materials into the primary containment, pipes that penetrate the primary containment have been provided with isolation capabilities in accordance with the intent of GDC 56. In all cases, these pipes are provided with a minimum of two protective barriers against containment leakage and in some cases more.

<u>Criterion 57 - Closed-System Isolation Valves</u>

"Each line that penetrates the primary reactor containment and is neither part of the reactor coolant pressure boundary nor connected directly to the containment atmosphere shall have at least one containment isolation valve which shall be either automatic, locked closed, or capable of remote manual operation. This valve shall be outside the containment and located as close to the containment as practical. A simple check valve may not be used as the automatic isolation valve."

Design Evaluation

The drywell chilled water (2P64) and reactor building closed cooling water (2P42) systems are closed systems inside primary containment. Piping for both systems is Seismic Category I, missile protected and Quality Group D. The Quality Group D classification is improved by including both systems, inside primary containment, in the ASME Section XI inservice inspection program for Nuclear Class 3 piping.

A single remote-manually operated isolation valve is provided outside containment and as close to the containment as practical on each supply and return line for the subject systems. Leakage detection is provided by the monitored drywell sump as described in subsection 5.2.7 and by process instrumentation.

<u>Criterion 60 - Control of Releases of Radioactive Materials to the Environment</u>

"The nuclear power unit design shall include means to control suitably the release of radioactive materials in gaseous and liquid effluents and to handle radioactive solid wastes produced during normal reactor operation, including anticipated operational occurrences. Sufficient holdup capacity shall be provided for retention of gaseous and liquid effluents containing radioactive materials, particularly where unfavorable site environmental conditions can be expected to impose unusual operational limitations upon the release of such effluents to the environment."

Design Evaluation

The liquid radwaste system is designed with the intent to process and recycle the liquid waste collected in the waste holdup tanks to the extent practicable. Liquid waste collected in chemical or floor drain tanks is normally discharged to the environment after treatment and dilution with cooling tower blowdown. A floor drain filter was added to the liquid radwaste system to provide further capability for minimizing liquid radioactive releases. During normal plant operation, the annual average whole-body radiation dose to individuals from both reactors on the site, resulting from these routine liquid waste discharges, is expected to be ~ 3% of 10 CFR 20.1 - 20.601 (found in 10 CFR published before January 1994) limits. Short-term release from the plant resulting from equipment malfunctions or operational transients is within the 10 CFR 20.1 - 20.601 (found in 10 CFR published before January 1994) limits. Solid wastes are packaged in suitable containers for offsite shipment and burial.

The air ejector off-gas radioactive wastes are treated by an ambient charcoal bed adsorption system before discharge to the environment. An off-gas recombiner was added downstream of the steam jet air ejectors (SJAE) to recombine hydrogen and oxygen and thereby, increase holdup time. The charcoal adsorption system was also added. This system increases the effective holdup time for the isotopes of krypton and xenon and significantly reduces their release to the environment. The annual average exposure at the site boundary due to noble gases from both units during normal operation is not expected to exceed 30 mrem.

The liquid and gaseous effluents from the treatment systems are continuously monitored, and the discharges are terminated if the effluents exceed preset radioactivity levels.

The radioactive waste treatment system design discussed in this section limits the radioactivity releases to the environment from HNP to levels as low as reasonably achievable.

<u>*Criterion 61 - Fuel Storage and Handling and Radioactivity Control*</u>

"The fuel storage and handling, radioactive waste, and other systems which may contain radioactivity shall be designed to assure adequate safety under normal and postulated accident conditions. These systems shall be designed (1) with a capability to permit appropriate periodic inspection and testing of components important to safety, (2) with suitable shielding for radiation protection, (3) with appropriate containment, confinement, and filtering systems, (4) with a residual heat removal capability having reliability and testability that reflects the importance to safety of decay heat and other residual heat removal, and (5) to prevent significant reduction in fuel storage coolant inventory under accident conditions."

Design Evaluation

A. New-Fuel Storage

New fuel is placed in dry storage in the new-fuel storage vault which is located inside the secondary containment reactor building. The storage vault within the reactor building provides adequate shielding for radiation protection. Storage racks preclude accidental criticality. (See evaluation against GDC 62.) The new-fuel storage racks do not require any special inspection and testing for nuclear safety purposes.

B. Spent-Fuel Handling and Storage

The handling of new- and spent-fuel assemblies for reactor refueling is within the reactor building which serves as the primary containment during refueling operations. The reactor well and spent-fuel pool are filled directly from the condensate storage tank in order to provide shielding above the reactor and spent fuel. Fuel pool water is circulated through the fuel pool cooling and cleanup (FPCC) system to maintain fuel pool water temperature, purity, water clarity, and water level. Storage racks preclude accidental criticality. (See evaluation against GDC 62.)

Reliable decay heat removal is provided by a single train, closed-loop FPCC system. One of two cooling trains of HNP-1 is shared during refueling. The HNP-2 system consists of one circulating pump, one heat exchanger, one filter-demineralizer, two skimmer surge tanks, and the required piping, valves, and instrumentation. The pool water is circulated through the system, and suction is taken from surge tanks by the pump. Flow passes through the heat exchanger and filters and is discharged through diffusers at the bottom of the fuel pool. Pool water temperature is maintained below 125°F when removing the normal heat load from the pool with the reactor building closed cooling water temperature at its maximum. If the pool temperature exceeds 150°F, the FPCC system is connected to the RHR system. This increases the cooling capacity of the FPCC system up to a full core.

The decay heat removal (DHR) system is provided for use during refueling outages to remove decay heat from the spent fuel pool. This additional cooling system allows the RHR system and/or the FPCC system to be taken out of service for inspections, repairs, and/or modifications during outages. There are no connections to the fuel storage pool which could allow the fuel pool to be drained below the pool gate between the reactor well and fuel pool. Check valves are provided on lines terminating at the bottom of the pool to prevent syphoning of the pool water. The high and low level switches indicate pool water level changes in the control room and annunciator indication on 2H21-P157 at the 185-ft level of the reactor building. Fission product concentration in the pool water is minimized by use of the filter-demineralizer. This minimizes the release from the pool to the reactor building environment. No special tests are required because the FPCC system is continuously in operation while fuel is stored in the pool. Routine visual inspection of the system components, instrumentation, and trouble alarms is adequate to verify system operability.

C. Radioactive Waste Systems

The radioactive waste systems provide all the equipment necessary to collect, process, and prepare for disposal all radioactive liquids, gases, and solid waste produced as a result of reactor operation.

Liquid radwastes are classified, contained, and treated as high or low purity, chemical, detergent, or sludges. Processing includes filtration, ion exchange, and dilution. Liquid wastes are also decanted, and sludge is accumulated for disposal as solid radwaste. Wet and dry solid wastes are packaged in appropriate containers and dewatered for shipment. Gaseous radwastes are monitored, processed, recorded, and controlled so that radiation doses to persons outside the controlled area are below those allowed by 10 CFR 20.1 - 20.601 (found in 10 CFR published before January 1994).

Accessible portions of the reactor and radwaste buildings shall have sufficient shielding to maintain dose rates within the limits set forth in 10 CFR 20.1 - 20.601 (found in 10 CFR published before January 1994). The radwaste building is designed to preclude accidental release of radioactive materials to the environs.

The radwaste systems are used on a routine basis and do not require specific testing to assure operability. Performance is monitored by radiation monitors during operation.

The fuel storage and handling and radioactive waste systems are designed to assure adequate safety under normal and postulated accident conditions.

The design of these systems meets the requirement of GDC 61.

For further discussion, see chapter 11; sections 6.2, 9.4, and 12.1; and subsections 5.5.7, 9.1.1, 9.1.2, and 9.1.3.

Criterion 62 - Prevention of Criticality in Fuel Storage and Handling

"Criticality in the fuel storage and handling system shall be prevented by physical systems or processes, preferably by use of geometrically safe configurations."

Design Evaluation

Appropriate plant fuel handling and storage facilities are provided to preclude accidental criticality for new and spent fuel. Criticality in new- and spent-fuel storage is prevented by the geometrically safe configuration of the storage rack. There is sufficient spacing between the assemblies to assure that the array, when fully loaded, is substantially subcritical. Fuel elements are limited by rack design to only top loading and fuel assembly positions. The new- and spent-fuel storage racks are designed to Seismic Category I requirements.

New fuel is placed in dry storage in the top-loaded new-fuel storage vault. The vault contains a drain to prevent the accumulation of water. The new-fuel storage vault racks (located inside the secondary containment reactor building) are designed to prevent an accidental critical array, even in the event the vault becomes flooded or subjected to seismic loadings. The 6.625-in. center-to-center new-fuel assembly spacing limits the effective multiplication factor of the array to not more than 0.90 for new dry fuel. K_{eff} *does not exceed 0.95 if the new fuel is flooded.*

Spent fuel is stored under water in the spent-fuel pool. The racks in which spent-fuel assemblies are placed are designed and arranged to ensure subcriticality in the storage pool. Spent fuel is maintained at a subcritical multiplication factor K_{eff} *of < 0.90 under normal conditions and 0.95 for abnormal conditions. Abnormal conditions may result from an earthquake, accidental dropping of equipment, or damage caused by the horizontal movement of fuel-handling equipment without first disengaging the fuel from the hoisting equipment.*

Refueling interlocks include circuitry which senses conditions of the refueling equipment and the control rods. These interlocks reinforce operational procedures that prohibit making the reactor critical. The fuel-handling system is designed to provide a safe, effective means of transporting and handling fuel and is designed to minimize the possibility of mishandling or maloperation.

The use of geometrically safe configurations for new- and spent-fuel storage and the design of fuel-handling systems precludes accidental criticality in accord with GDC 62.

For further discussion of criticality monitoring, reference section 9.1.

Criterion 63 - Monitoring Fuel and Waste Storage

"Appropriate systems shall be provided in fuel storage and radioactive waste systems and associated handling areas (1) to detect conditions that may result in loss of residual heat removal capability and excessive radiation levels and (2) to initiate appropriate safety actions."

Design Evaluation

Appropriate systems are provided to meet the requirements of this criterion. A malfunction of the FPCC system which could result in loss of RHR capability and excessive radiation levels is alarmed in the control room. Alarmed conditions include low fuel pool cooling water pump discharge pressure, high and low levels in the fuel storage pool, low level in the skimmer surge tank, and flow in the drain lines between fuel pool gates located between the fuel pool and the reactor well. System temperature is also continuously monitored and alarmed in the control room. Spent-fuel storage is discussed in subsection 9.1.2, and the FPCC system is discussed in subsection 9.1.3.

The reactor building and refueling floor ventilation radiation monitoring systems detect abnormal amounts of radioactivity and initiate appropriate action to control the release of radioactive material to the environs. These systems are discussed in sections 9.4 and 11.4.

Area radiation and tank and sump levels are monitored and alarmed to give indication of conditions which may result in excessive radiation levels in radioactive waste system areas. These systems are discussed in chapter 11.

Criterion 64 - Monitoring Radioactivity Releases

"Means are provided for monitoring the reactor containment atmosphere, spaces containing components for recirculation of loss-of-coolant accident fluids, effluent discharge paths, and the plant environs for radioactivity that may be released from normal operations, including anticipated operational occurrences, and from postulated accidents."

Design Evaluation

A fission products monitoring system samples the containment (both drywell and torus) atmosphere for radioactive particulates, noble gases, and iodine during normal operation. A hydrogen-oxygen analyzer system monitors the oxygen-hydrogen concentration in the containment during normal operation and following and accident.

Radioactive effluent discharge paths and the site environs are monitored for radioactivity releases.

<u>DOCUMENTS INCORPORATED BY REFERENCE INTO THE FSAR</u>

Technical Requirements Manual Table T7.0-1, Primary Containment Penetrations.

Edwin I. Hatch Nuclear Plant Units 1 and 2 Fire Hazards Analysis and Fire Protection Program.

TABLE 3.1-1

REACTOR COOLANT PRESSURE BOUNDARY INFLUENT LINES

Penetration	Influent Lines	Inner Barrier	Outer Barrier	Comments
X9A&B	Feedwater	CV	AO-CV	55.1
	• HPCI return		MOV	55.2
	• RCIC return		MOV	55.2
	• Cleanup return		CV	55.2
X13A&B	RHR return to recirculation	MOV[(a)]	Closed system	55.3
X16A&B	CS	MOV[(a)]	Closed system	55.3
X17	Disconnected			
X42	Standby liquid control	CV	CV	55.6
X27C & X57C	Recirculation pump seal water	CV	CV	55.8
	Other			55.7

LEGEND

CV - Check valve
MOV - Motor-operated valve
AO-CV - Air-operated check valve

a. Both barriers are outside the primary containment.

TABLE 3.1-2

REACTOR COOLANT PRESSURE BOUNDARY EFFLUENT LINES

Penetration	Effluent Lines	Inside Drywell	Outside Drywell
X7A-D	Main steam	AOV	AOV, MOV
X14	RWC	MOV	MOV
X12	RHR shutdown cooling		MOV, closed system
X8	Main steam line drain	MOV	MOV
X10	RCIC turbine steam	MOV	MOV
X11	HPCI turbine steam	MOV	MOV

3.2 CLASSIFICATION OF STRUCTURES, COMPONENTS, AND SYSTEMS

3.2.1 SEISMIC CLASSIFICATION

A two-level system is used for the seismic classification of the structures, components, and systems of the facility:

- Seismic Category I structures, components, and systems.
- Category II structures, components, and systems.

3.2.1.1 Definitions

Seismic Category I structures, components, and systems are those that must function for safe shutdown, immediate or long-term core cooling, or for activity confinement following a loss-of-coolant accident to ensure that the public is protected in accordance with 10 CFR 100 guidelines.

Seismic Category I structures, components, and systems are designed to withstand the effects of the design basis earthquake (DBE) and operating basis earthquake (OBE) as discussed in supplements 3.7A and 3.7B.

When a system as a whole is referred to as Seismic Category I, portions not associated with loss of function of the system may be designated as Category II.

Category II structures, components, and systems are those whose failure would not result in the release of significant radioactivity and would not prevent reactor shutdown. All equipment not specifically listed as Seismic Category I is included as Category II. The failure of Category II structures, components, and systems may interrupt power generation.

All Category II structures are designed to conform to paragraph 2.3.1.4 of the 1970 edition of the Uniform Building Code.

None of the structures in the Hatch Nuclear Plant (HNP) have classifications that are partially Seismic Category I and partially Category II; however, portions of nonseismic Category II systems are seismically supported if their failure could cause damage to Seismic Category I components.

Seismic classification of structures, systems, and components is in accordance with Regulatory Guide 1.29, (August 1973).

3.2.1.2 Seismic Category I Structures

Reactor building

- Primary containment structure
- Spent-fuel pool
- New-fuel storage vault

Diesel generator building

Control building

Intake structure

Main stack

Structures supporting or housing Seismic Category I equipment

- Wall around condensate storage tank (CST)
- Liquid nitrogen storage tank and foundation
- Diesel generator fuel oil storage tanks

3.2.1.3 Seismic Category I Mechanical Components and Systems

Seismic Category I mechanical components and systems are listed in table 3.2-1.

3.2.1.4 Seismic Category I (Class 1E) Electrical Equipment

Switchgear & buses

- 4160-V buses 2E, 2F, and 2G
- 600-V load centers 2C and 2D
- 250 V-dc buses 2A and 2B
- 4160-V recirculation pump trip (RPT) switchgear

Transformers

- 1190-1368-kVA, 4160-600-V essential transformers
- 112.5 kVA, 600-208/120-V essential transformers
- 225-kVA, 75-kVA 4160-600-V transformers 2F1 and 2F2

600-V bus duct associated with 600-V load centers 2C and 2D and 4160-600-V transformer 2CD

Motors (4 kV)

- Residual heat removal service water (RHRSW) pump motors (4)
- Plant service water (PSW) pump motors (4)
- Residual heat removal (RHR) pump motors (4)
- Core spray (CS) pump motors (2)

ac & dc lighting and miscellaneous power cabinets - control and diesel generator buildings

ac & dc motor control centers (MCCs)

- 600 V-ac essential MCC - reactor building (7)
- 250 V-dc essential MCC - reactor building (2)
- 600-208 V-ac essential MCC - diesel generator building (3)
- 600 V-ac essential MCC - intake structure (2)

Batteries and chargers

- 125-250-V station batteries 2A and 2B
- 125-V diesel batteries 2A and 2C
- 125-V battery chargers 2A-2F (station batteries)
- 125-V battery chargers 2G and 2J (diesel batteries)

Diesel generators 2A, 2C, and 1B (1B is shared with HNP-1.)

Neutral grounding resistors for diesels

Primary electrical penetrations (drywell)

Power and control cable for essential equipment and instruments

Raceway supports associated with essential systems & equipment

- Pull boxes and junction boxes
- Underground ducts, fittings, and encasement

Reactor protection system (RPS) breaker protection panels (2C71-P003A through F)

3.2.1.5 Seismic Category I Instrumentation and Control Systems Equipment

RPS (except distribution cabinets and motor-generator sets)

Primary containment and reactor vessel isolation control system

Power range monitors in nuclear boiler system

Emergency core cooling system (ECCS) initiating channels and logic and automatic depressurization system initiating channels and logic

Essential instrumentation and controls on the following systems:

- Nuclear boiler
- Control rod drive (CRD)
- RHR and RHRSW
- CS
- High-pressure coolant injection (HPCI) system
- Reactor core isolation cooling (RCIC) system
- Standby gas treatment system (SGTS)
- PSW

Instrumentation and controls for the following:

- Standby liquid control system (SLCS)
- Safeguard equipment emergency room coolers and main control room (MCR) air-handling and condensing units

Neutron monitoring system (NMS)

Portions of radiation monitoring systems

Oxygen and hydrogen analyzer

MCR panels and local instrument racks for the above instrumentation and control systems

Leak detection systems in the HPCI system and pipe chase rooms

Switchboards and panels

- Control boards
  - Reactor and containment cooling and isolation board (2H11-P601)
  - Power range neutron monitoring cabinet (2H11-P608)
  - Channel A primary isolation and RPS vertical board (2H11-P609)
  - Channel B primary isolation and RPS vertical board (2H11-P611)
  - Steam, feedwater condensate, circulating, and service water bench board (2H11-P650)
  - Emergency diesel generator 2A vertical board
  - Emergency diesel generator 1B vertical board
  - Emergency diesel generator 2C vertical board

    (2H11-P652)
    (with physical partitions)
  - Reactor water cleanup (RWC) and recirculation benchboard (2H11-P602)
    - Reactor control benchboard (2H11-P603)

Protective relay board

- Channel A RHR relay vertical board (2H11-P617)
- Channel B RHR relay vertical board (2H11-P618)
- HPCI relay vertical board (2H11-P620)
- RCIC relay vertical board (2H11-P621)
- Inboard isolation valve relay vertical board (2H11-P622)
- Outboard isolation valve relay vertical board (2H11-P623)
- Channel A CS relay vertical board (2H11-P626)
- Channel B CS relay vertical board (2H11-P627)
- Auto blowdown relay vertical board (2H11-P628)
- Heating, venting, and air-conditioning systems control boards (2H11-P654 and 2H11-P657)

Other panels

- Reactor building instrument enclosure (2H21-P151)
- Diesel generator 2A relay panel (2H21-P200)
- Diesel generator 1B relay panel (2H21-P201)
- Diesel generator 2C relay panel (2H21-P202)
- Diesel generator 2A relay panel (2H21-P230)
- Diesel generator 1B relay panel (2H21-P231)
- Diesel generator 2C relay panel (2H21-P232)
- 600-V bus 2C control panel (2H21-P245)
- 600-V bus 2D control panel (2H21-P246)
- 250-V bus, switchgear 2A control panel (2H21-P248)
- 250-V bus, switchgear 2B control panel (2H21-P249)

- Motor-operated valve (MOV) and fuel pumps control panel Division I - diesel building (2H21-P255)
- MOV and fuel pumps control panel Division II - diesel building (2H21-P256)
- Diesel generator 2A heat and ventilation control panel - diesel building (2H21-P257)
- Diesel generator 2C heat and ventilation control panel - diesel building (2H21-P259)
- Switchgear 2E - room heat and ventilation control panel - diesel building (2H21-P260)
- Switchgear 2F - room heat and ventilation control panel - diesel building (2H21-P261)
- Switchgear 2G - room heat and ventilation control panel - diesel building (2H21-P262)
- Station battery 2A shunt panel P - control building (2H21-P285)
- Station battery 2A shunt panel PN - control building (2H21-P286)
- Station battery 2A shunt panel N - control building (2H21-P287)
- Station battery 2B shunt panel P - control building (2H21-P288)
- Station battery 2B shunt panel PN - control building (2H21-P289)
- Station battery 2B shunt panel N - control building (2H21-P290)
- Diesel battery 2A shunt panel - diesel building (2H21-P291)
- Diesel battery 2C shunt panel - diesel building (2H21-P293)
- Radiation monitor battery 2A shunt panel - control building (2H21-P294)
- Radiation monitor battery 2B shunt panel - control building (2H21-P295)
- Radiation monitor battery 2A charger shunt box - control building (2H21-P296)
- Radiation monitor battery 2B charger shunt box - control building (2H21-P297)

- Diesel generator 2A leading timer panel (2H21-P303)
- Diesel generator 1B leading timer panel (2H21-P304)
- Diesel generator 2C leading timer panel (2H21-P305)
- Vital ac battery fuse panel (2H21-P317)
- Diesel generator 2A battery metering panel (2R43-P002A)
- Diesel generator 2C battery metering panel (2R43-P002C)
- Remote shutdown panels (2H21-P173 and 2C82-P001)
- Analyzer ventilation and leak detection panel (2H11-P700)
- Diesel battery 2A shunt box (2H21-P198)
- Diesel battery 2C shunt box (2H21-P199)
- Local instrument racks containing Class 1E components (65)
- Radwaste building ventilation control panel (2H21-P182)
- Analog transmitter trip system panels and associated distribution buses (H11-P921, H11-P922, H11-P923, H11-P924, H11-P925, H11-P926, H11-P927, and H11-P928)

Radiation monitors of MCR intake

3.2.2 SYSTEM QUALITY GROUP CLASSIFICATIONS

System quality group classifications, as defined in Nuclear Regulatory Commission (NRC) Regulatory Guide 1.26 (September 1974), were determined for each water, steam, or radioactive waste containing component of those applicable fluid systems relied upon to:

- Prevent or mitigate the consequences of accidents and malfunctions originating within the reactor coolant pressure boundary (RCPB).
- Provide safe shutdown capability of the reactor and maintain it in a safe shutdown condition.
- Contain radioactive material.

A tabulation showing the quality group classification for each mechanical component so defined is shown in table 3.2-1 under the heading, Quality Group Classification. Drawing no. H-26095 is

a diagram which depicts the relative locations of these components along with their quality group classifications. Table 3.2-4 lists the individual systems and the drawing number in which the quality group classifications for the components of the system are indicated.

System quality group classifications and design and fabrication requirements, as indicated in tables 3.2-1 and 3.2-3, meet the requirements of Regulatory Guide 1.26 (September 1974).

Because of the long lead time between purchase and delivery of valves at HNP-2, it was necessary, to avoid construction delays, to consider the use on HNP-2 of certain valves purchased as spares for HNP-1. The valves utilized (table 3.2-6) are small (2 in. and under), manually operated, quality group B valves purchased in compliance with the 1968 Draft American Society of Mechanical Engineers (ASME) Code for Pumps and Valves.

Regulatory Guide 1.26 refers to 10 CFR 50.55a for guidance when determining the code and addenda to be applied for quality group B valves. This guidance indicates that the applicable code for HNP-2, based on its construction permit date of December 1972, should be the 1971 edition of the ASME Boiler and Pressure Vessel (B&PV) Code. The following is a summary report which demonstrates that these valves meet the intent of the 1971 ASME Code.

A. Function of Valves

The valves described in table 3.2-6 are generally used as follows: (See table 3.2-6 notes for exceptions.)

- Low point drain connections for any process lines.
- High point vent connections for any process lines.
- Root valves for instruments.

These valves are not used in process lines which may be required for emergency shutdown of the plant but may be used in drain and vent lines for such process lines.

B. Degree of Compliance with 1968 Code

The subject valves are in full compliance with the 1968 Draft ASME Code for Pumps and Valves and ASME Section III addenda through March of 1970, with the interpretation that, prior to 1971, N stamping of data reports was not required for nuclear Class 2 or 3 valves.

The valves given in table 3.2-6 are designed to the criteria of American National Standards Institute (ANSI) B-16.5 and, thus, meet the requirements of paragraph NC-3511 of the 1971 edition of Section III of the ASME Code.

C. Comparison With HNP-2 Code With Justification of Differences

For valves ordered more than 12 months prior to the construction permit, the effective ASME Code, according to the guidance of 10 CFR 50.55a, should be the 1971 edition. However, the only discernible difference between the ASME Code, 1971 edition, and the 1968 Draft ASME Pump and Valve Code is the requirement for a code data report and official N stamp. The design, fabrication, inspection, and testing requirements are common to both codes.

The code data report was not required for nuclear Class 2 and 3 valves by the 1968 Code and therefore was not generated. However, all the required documentation for the code data report is available. This consists of chemical and physical mill certificates, nondestructive examination, plus hydro and seal leakage tests. This is the exact information required by the 1971 NPV-1 form except that an authorized inspector must witness the hydro tests and certify the documentation package. The valves are hydrotested again with the piping system, although not at the higher pressures of SP-61.

The official N stamp was not applied to the valves, although the manufacturer did have the authorization for nuclear Class I valve stamping. A nuclear symbol is applied to certain valves, and all valves are tagged with the manufacturer's name and the design pressure at coincident temperature.

Therefore, because these valves are designed, fabricated, inspected, and tested in accordance with the 1968 Draft ASME Pump and Valve Code, it is considered that these valves meet the intent of the 1971 ASME B&PV Code.

TABLE 3.2-1 (SHEET 1 OF 11)

SEISMIC CATEGORY I SYSTEMS AND MECHANICAL COMPONENTS

Principal Components	Scope of Supply[b]	Seismic Category[c]	Quality Group Classification[d]	Principal Construction/ Design Code[f]	Notes
Reactor System					
Reactor pressure vessel (RPV)	GE	I	A	III-A	
RPV support	GE	I	NA	None	
RPV appurtenances pressure-retaining portions	GE	I	NA	III-A	
CRD housing	GE	I	A	III-A	
Reactor internal structures, engineered safety features, shroud	GE	I	NA	None	
Reactor internal structures, CS lines	GE	I	NA	None	10
Control rods	GE	I	NA	None	
CRDs	GE	I	NA	III-A	
Core support structures	GE	I	NA	None	
Power range detector hardware in core guide tube seal	GE	I	B	III-B	
Fuel assemblies - RPV stabilizer	GE	I	NA	None	
Nuclear Boiler System					
Vessels, level instrumentation chambers	GE	I	A	III-A	
Vessels, air accumulators	B	I	B	III-2	
Piping, relief valve discharge	B	I	C	III-3	
Piping, main steam, within outermost isolation valve	GE	I	A	III-1	1
Piping, feedwater from vessel through third shutoff valve	B	I	A	III-1	7
Piping, feedwater, from third shutoff valve through fourth shutoff valve	B	I	B	III-2	7
Pipe supports, main steam	GE	I	NA	None	
Pipe restraints, main steam	B/GE	I	NA	None	
Piping, other within outermost isolation valves	B	I	A	III-1	1
Piping instrumentation beyond outermost isolation valves	B	NA	D	B31.1	1
Safety & relief valves	GE	I	A	III-1	
Valves, MSIVs	GE	I	A	NPVC-1	
Valves, within outermost isolation valves	B	I	A	NPVC-1	1
Valves, instrumentation beyond outermost isolation valves	B	I	D	B31.1	1

TABLE 3.2-1 (SHEET 2 OF 11)

Principal Components	Scope of Supply[b]	Seismic Category[c]	Quality Group Classification[d]	Principal Construction/ Design Code[f]	Notes
Reactor Recirculation System					
Piping (replaced in 1984)	GE	I	A	III-1	1, 12
Pipe suspension, recirculation line	GE	I	NA	None	1, 19
Pipe restraints, recirculation line	GE	I	NA	None	
Pumps	GE	I	A	NPVC-1	
Valves	GE	I	A	NPVC-1	1
Motor/pump	GE	S	NA	None	
CRD Hydraulic (CRDH) System					
Valves, isolation, water return line	B	I	B	III-1	7
Valves, scram discharge volume lines	B	I	B	III-2	1
Valves, insert & withdraw lines	GE	I	A	NPVC-II	
Valves, other	B	NA	D	B31.1	1
Piping, water return line within isolation valves	B	I	B	III-1	
Piping, scram discharge volume lines	B	I	B	III-2	
Piping, insert & withdraw lines	B	I	A	III-1	
Piping, other	B	NA	D	B31.1	1
Hydraulic control unit	GE	I	D	None	2
SLCS					
Standby liquid control tank	GE	I	B	API 650/VIII	
Pump	GE	I	B	NPVC-2	
Pump motor	GE	S	NA	None	
Valves, explosive	GE	I	B	NPVC-2	
Valves, isolation & within	B	I	A	III-1	1
Valves beyond isolation valves	B	I	B	III-2	1
Piping within isolation valves	B	I	A	III-1	1
Piping beyond isolation valves	B	I	B	III-2	1
Accumulator	GE	I	B	III-C	
NMS					
Piping, traversing incore probe (TIP)	B	I	B	III-2	23
Valves, isolation, TIP subsystem	GE	I	B	III-2	23

TABLE 3.2-1 (SHEET 3 OF 11)

Principal Components	Scope of Supply[b]	Seismic Category[c]	Quality Group Classification[d]	Principal Construction/ Design Code[f]	Notes
RHR System					
Heat exchangers, primary side	GE	I	B	III-C & TEMA-C	11
Heat exchangers, secondary side	GE	I	C	VIII & TEMA-C	
Piping (only SS) within outermost isolation valves	B/GE,	I	A	III-1	1, 13
Piping beyond outermost isolation valves	B	I	B	III-2	1
Pumps	GE	I	B	NPVC-2	20
Pump motors	GE	S	NA	None	
Valves, isolation, LPCI line	B	I	A	III-1	
Valves, isolation, other	B	I	A/B	III-1/2	1
Valves beyond isolation valves	B	I	B	III-2	1
CS					
Piping within outermost isolation valves	B	I	A	III-1	1
Piping beyond outermost isolation valves	B	I	B	III-2	1
Pumps	GE	I	B	NPVC-2	1
Pump motors	GE	S	NA	None	
Valves, isolation & within	B	I	A	III-1	1
Valves beyond outermost isolation	B	I	B	III-2	1
HPCI System					
Piping, suction line to CST	B	I	B	III-2	
Piping, turbine steam supply & discharge	B	I	B	III-2	
Piping, return test line to CST beyond second isolation valve	B	I	D	B31.1	
Piping within outermost isolation valve	B	I	A	III-1	
Piping, suppression pool suction & pump discharge	B	I	B	III-2	1
Pump	GE	I	B	NPVC-2	
Turbine	GE	I	NA	None	3
Valves beyond outermost isolation valves	GE	I	B	NPVC-2	
Valves, outer isolation & within	B	I	A	III-1	1
Valves beyond isolation valves, motor operated	GE	I	B	HPVC-2	
Valves, other	B	I	B	III-2	

TABLE 3.2-1 (SHEET 4 OF 11)

Principal Components	Scope of Supply[b]	Seismic Category[c]	Quality Group Classification[d]	Principal Construction/ Design Code[f]	Notes
RCIC System					
Piping within outermost isolation valves	B	I	A	III-1	1
Piping beyond outermost isolation valves	B	I	B	III-2	1
Piping, return test line to CST beyond second isolation valve	B	I	D	B31.1	
Vacuum pump discharge line to containment isolation valves	B	I	B	III-2	1
Pumps	GE	I	B	NPVC-2	
Valves, isolation & within	B	I	A	III-1	1
Valves, return test line to condensate storage beyond second isolation valve, & vacuum pump discharge line to containment isolation valves	B	I	B	III-2	
Valves, other	B	I	B	III-2	1
Turbine	GE	I	NA	None	3
Fuel Service Equipment					
Fuel preparation machine	GE	I	NA	None	
General purpose grapple	GE	I	NA	None	
RPV Service Equipment					
Steam line plugs	GE	I	NA	None	
Dryer, separator sling, & head strongback	GE	I	NA	None	
In-Vessel Service Equipment					
Control rod grapple	GE	I	NA	None	
Refueling Equipment					
Refueling equipment platform assembly	GE	II/I	NA	None	18
Refueling bellows	GE	I	B	III-2	
Storage Equipment					
Fuel storage racks	GE	I	NA	None	
Defective fuel storage container	GE	I	NA	None	

TABLE 3.2-1 (SHEET 5 OF 11)

Principal Components	Scope of Supply[b]	Seismic Category[c]	Quality Group Classification[d]	Principal Construction/ Design Code[f]	Notes
Radwaste System					
Tanks, atmospheric vessels	GE/B	NA	D/C	API-650, III-3, & B96.1	22
Heat exchangers & evaporators	GE/B	NA	D	VIII & TEMA-Cq	4, 22
Piping	B	NA	C/D	III-3 and B31.1	4, 22
Pumps	GE/B	NA	C/D	III-3 and B31.1	1, 4, 22
Valves, containment isolation	B	I	B	III-2	4, 22
Valves, flow control & filter system	B	NA	C	III-3	4, 22
Valves, other	B	NA	C	III-3	22
RWC System					
Vessels, filter-demineralizer	GE	NA	C	III-3	
Heat exchangers regenerator & nonregenerating	GE	NA	C	VIII & TEMA-C	
Piping within outermost isolation valves	GE	I	A	III-1	14
Piping beyond outermost isolation valves	B	NA	C	III-3	
Pumps	GE	NA	C	NPVC-3	
Valves, isolation valves & within	B	I	A	III-1	1, 5
Valves beyond outermost isolation valves	B	NA	C	III-3	1, 5
Phase separator	GE	NA	C	III-3	1, 5
Spent-Fuel Pool Cooling & Cleanup System					
Vessels (filter-demineralizer & resin trap)	B	NA	C	III-3	
Vessels (precoat tank)	B	NA	D	Atmospheric	
Heat exchanger	B	NA	D	VIII & TEMA-B	24
Pumps (main & holding)	B	NA	C	III-3	
Precoat pump	B	NA	D	Hydraulic Institute	
Precoat piping & valves	B	NA	D	B31.1	
All other piping & valves	B	I/NA	C	III-3	
Off-Gas System					
Tanks	GE	NA	D	API 650	
Heat exchangers	GE	NA	D	VIII & TEMA	
Piping	B	NA	D	B31.1	
Valves, flow control	B	NA	D	B31.1	
Valves, other	B	NA	D	B16.5	
Pressure vessels	GE	NA	D	VIII	

TABLE 3.2-1 (SHEET 6 OF 11)

Principal Components	Scope of Supply[b]	Seismic Category[c]	Quality Group Classification[d]	Principal Construction/ Design Code[f]	Notes
RHRSW System					
Piping	B	I	C	III-3	
Pumps	B	I	C	III-3	
Pump motors	B	I	NA	None	
Valves	B	I	C	III-3	
PSW System					
Piping (intake structure, reactor building, diesel generator building)	B	I	C	III-3	
Piping (underground)	SCS	I	C	USAS B31.7	
Pumps	B	I	C	III-3	
Pump motors	B	I	NA	None	
Valves	B	I	C	III-3	
All other piping & valves	B	NA	D	B31.1	
Reactor Building Closed Cooling Water (RBCCW) System					
Piping & valves forming part of primary containment boundary	B	I	B	III-2	21
Instrument & Service Air System					
Accumulators & piping between accumulators & valves for outboard MSIVs	B	I	B	III-2	
Accumulators & piping in noninterruptible air system	B	I	D	B31.1 VIII	
Piping & valves forming part of containment boundary	B	I	B	III-2	16
Compressors	B	NA	D	B31.1	
All other piping, valves, & receivers	B	NA	D	B31.1	
Drywell Pneumatic System					
Piping forming a part of containment	B	I	B	III-2	
Piping, valves, & accumulators inside drywell	B	I	B	III-2	
Compressors, piping, & valves to receivers	B	NA	D	B31.1	
Receivers, piping, & valves to containment isolation	B	I	B	III-2	

TABLE 3.2-1 (SHEET 7 OF 11)

Principal Components	Scope of Supply[(b)]	Seismic Category[(c)]	Quality Group Classification[(d)]	Principal Construction/ Design Code[(f)]	Notes
Diesel Generator System					
Day tanks	SCS	I	C	VIII	8
Piping & valves fuel oil system	SCS	I	C	B31.1	9
Piping & valves expansion tank makeup inside diesel generator building	B	I	D	B31.1	
Piping & valves diesel service water system	B	I	C	III-3	
Pumps fuel oil system	B	I	C	III-3	
Pumps diesel service water system	B	I	C	III-3	
Pump motor fuel oil system	B	I	NA	None	
Pump motors diesel service water system	B	I	NA	None	
Diesel generators	SCS	I	NA	None	
CAD					
Primary containment purge (includes CAD)	B	I	B	III-2	
SGTS					
Filter train housing	B	I	B	III-2	17
Valves	B	I	B	III-2	
Piping	B	I	B	III-2	
Damper in reactor building to maintain integrity of secondary containment	B	I	C	NA	
ECCS Equipment Area Cooling System					
All components with safety function	B	I	C	III-3	
Power Conversion Systems					
Main steam piping to turbine stop valves, bypass valves, steam seal isolation valve, steam jet air ejector isolation valve, reactor feed pump turbine isolation valve, moisture-separator reheater isolation valves	B	I	B	III-2	
All above valves except turbine stop & bypass valves	B	I	B	III-2	6
Piping & valves, others	B	NA	D	B31.1	

TABLE 3.2-1 (SHEET 8 OF 11)

Principal Components	Scope of Supply[b]	Seismic Category[c]	Quality Group Classification[d]	Principal Construction/ Design Code[f]	Notes
Condensate Storage &Transfer System					
CST	B	NA	C	III-3	
Piping & valves	B	NA	C/D	III-3 & B31.1	
Condensate transfer pump	B	NA	D	Hydraulic Institute	

a. (deleted)
b. GE - General Electric.
B - Bechtel Power Corporation.
SCS - Southern Company Services, Inc.
c. I - The equipment is constructed in accordance with the seismic requirements for the DBE described in section 3.7.
NA - The seismic requirements for the DBE are not applicable to the equipment.
S - The equipment meets the seismic requirements described in the purchase specification.
d. The equipment is constructed in accordance with the codes listed in table 3.2-2.
e. (deleted)
f. Notations for principal construction/design codes are:
- III-A, B, C, 1, 2, 3, NF, NE - ASME Boiler and Pressure Vessel Code, Section III, Class A, B, C, 1, 2, 3, NF or NE. (Earlier versions of the code used the Class A, B, C designation, while later versions used the Class 1, 2, 3, NE designation. Equipment was ordered throughout a period requiring use of both designations.)
- VIII - ASME Boiler and Pressure Vessel Code, Section VIII, Pressure Vessels, Division I.
- B31.7 - USAS Code for Pressure Piping.
- B31.1 - ANSI B31.1 Standard Code for Pressure Piping, Power Piping.
- NPVC-1,2,3 - Draft ASME Code for Pumps and Valves for Nuclear Power, Class I, II, III.
- TEMA-B, C - Tubular Exchanger Manufacturers Association (TEMA), Standards Class B, C.
- API 650 - Welded Steel Tanks for Oil Storage; Atmospheric Tanks.
- API 620 - Standards for Large Welded Low-Pressure Storage Tanks.
- B96.1 - USAS B96.1 - Welded Aluminum Alloy Field-Erected Storage Tanks.
- B16.5 - ANSI B16.5 - Steel Pipe Flanges and Flanged Fittings.

NOTES:

1. All instrument lines connected to the RCPB and are not used to actuate safety systems are Quality Group D from the outer isolation valve or the process shutoff valve (root valve) to the sensing instrumentation.

TABLE 3.2-1 (SHEET 9 OF 11)

All other instrument lines:

- Through the root valve, shall be of the same classification as the system to which they are attached.
- Beyond the root valve, if used to actuate a safety system, shall be of the same classification as the system to which they are attached.
- Beyond the root valve, if not used to actuate a safety system, are Quality Group D.

All sample lines from the outer isolation valve or the process root valve through the remainder of the sampling system are Quality Group D.

2. The hydraulic control unit (HCU) is a GE factory assembled engineered module of valves, tubing, piping, and stored water which controls a single CRD by the application of precisely timed sequences of pressures and flows. This is accomplished by slow insertion or withdrawal of the control rods for power control and rapid insertion for reactor scram.

   Although the HCU, as a unit, is field installed and connected by process piping, many of its internal parts differ markedly from process piping components because of the more complex functions they must provide. Although the codes and standards invoked by the Quality Group A, B, C, and D pressure integrity quality levels clearly apply at all levels to the interfaces between the HCU and the connecting conventional piping components; e.g., pipe nipples, fittings, simple hand valves, etc., it is considered that they do not apply to the specialty parts; e.g., solenoid valves, pneumatic components, and instruments.

   The design and construction specifications for the HCU do invoke such codes and standards as can be reasonably applied to individual parts in developing required quality levels, but these codes and standards are supplemented with additional requirements for these parts and for the remaining parts and details. For example, all welds are low-pressure inspected, all socket welds are inspected for gap between pipe and socket bottom, all welding is performed by qualified welders, and all work is done according to written procedures.

   Quality Group D is generally applicable because the codes and standards invoked by that group contain clauses which permit the use of manufacturer's standards and proven design techniques which are not explicitly defined within the codes of Quality Groups A, B, or C. This is supplemented by the quality control techniques described above.

3. The RCIC and HPCI turbines do not fall within the applicable design codes. To ensure that the turbines are fabricated to the standards commensurate with their safety and performance requirements, GE established specific design requirements for this component. These requirements are given in the appropriate GE internal documents.

4. Quality Group D, Section VIII of the ASME Boiler and Pressure Vessel (B&PV) Code, Division I, and ANSI B31.1 apply downstream of the outermost isolation valves.

TABLE 3.2-1 (SHEET 10 OF 11)

5. RWC influent to the system has one isolation valve inside and one outside the containment. Downstream of the outside isolation valve, the system is Quality Group C.

   RWC return to the feedwater is Quality Group C upstream of the check valve and Quality Group B downstream of the spring loaded piston actuated check valve.

6. The turbine stop and control valves, bypass valves, and main steam leads meet all the requirements of quality group certification. Certification is defined in the April 19, 1974, letter from Mr. J. M. Hendrie, Deputy Director for Technical Review, Directorate of Licensing to Mr. J. A. Hinds, Manager, Safety and Licensing, General Electric Company.

7. The outermost valve of the three isolatable valves in the feedwater lines and the CRD hydraulic system water return line is similar to a boiler feed pump check valve.

   The spring loaded piston operator of the valve is held open by air pressure during normal operation. Fail-open solenoid valves are used to release air pressure and to permit the check valve piston operators to close. The valves are remote manually operated from the main control room, using signals which indicate loss of feedwater flow or loss of CRD hydraulic system water return line flow, respectively.

8. The day tanks were purchased prior to July 1, 1971. Nondestructive examination (NDE) requirements were in accordance with ASME Section VIII, Division I.

9. Piping and valves for the fuel oil system that were installed with HNP-2 meet the requirements for ASME Section III, Class 3, except for materials traceability and N-stamp requirements. NDE and testing are performed according to the requirements for Quality Group C (ASME, Section III, Class 3).

10. This portion of the CS system is designed in accordance with the methods of the 1971 edition of the ASME B&PV Code, Section III for Class 1 components (documentation was not required). Due to the design, hydrostatic testing was not possible.

11. The containment spray system is a mode of the RHR system and is Seismic Category I. No credit is taken for the containment spray system operation in the mitigation of accidents analyzed in chapter 15.

12. In the replacement of the recirculation system, the piping and fittings are in accordance with ASME B&PV Code, Section III, Subsection NB, 1980 Edition, up to and including Winter 1980 Addenda for materials, testing, and manufacturing. Design for the replacement is in accordance with ASME B&PV Code, Section III, Subsection NB, 1980 Edition, up to and including Winter 1981 Addenda.

13. The stainless steel portion of the RHR piping (suction and return) between the tee connection to the recirculation piping and the first RHR isolation valve was replaced. This replaced portion of piping was supplied by GE to the same code of construction as the recirculation piping. (See note 12.)

14. Portions of the RWC piping, from its connection to the contour nozzle on the 20-in. RHR suction up to the first valve (MO-F004) beyond the penetration (with the exception of the penetration) and a small section of pipe between valve MO-F001 and the penetration, were replaced. These replaced portions were supplied by GE to the same code of construction as the recirculation piping. (See note 12.)

TABLE 3.2-1 (SHEET 11 OF 11)

15. (deleted)

16. The torus-to-drywell vacuum breaker air test lines were installed as ANSI B31.1 (the stainless steel lines between the test solenoid valves and the air operators for vacuum breakers). These lines were subsequently modified to be Seismic Category I. The NRC granted an exemption for the lines to be treated as ANSI B31.1 upgraded to Class 2 for ASME Section III, with Section XI inspection and testing requirements (NRC letter to GPC, dated March 17, 1988).

17. The SGTS filter train housing is constructed to meet the intent of ASME Section III, Nuclear Class 2 requirements. The rules of subsection NC were followed even though the filter train manufacturer did not have an N stamp.

18. All load-supporting members of the bridge, all load-supporting members of the trolley, and the support frame for the frame-mounted and monorail auxiliary hoists are considered nonsafety related but shall be designed for DBE loads. Also, the pertinent quality assurance requirements of Appendix B to 10 CFR Part 50 shall be applied to the refueling platform supporting structure. The following refueling platform subassemblies and components are considered nonsafety related: the fuel grapple and associated cable; all electrical control components; wiring, relays, limit switches, etc; the drive train for the bridge, trolley, and monorail; the frame-mounted auxiliary hoist; the monorail auxiliary hoist; the pneumatic system; the brake assemblies for all the hoists; the hoisting cables for all hoists; and the main hoist. For the nonsafety-related components as specified in the GE Master Parts List F15-E003, the components and/or subassemblies are purchased commercially; consequently, they do not fall under any 10 CFR 50, Appendix B, quality assurance program, nor are any seismic requirements imposed on the equipment.

19. Modified recirculation pipe hangers HA3, HA4, HB3, and HB4 were installed in accordance with ASME Section III, Subsection NF.

20. The RHR pump seal coolers were replaced with similar coolers with shells made of cast steel instead of cast iron. The replacement coolers were purchased as commercial-grade components dedicated to safety-related service in accordance with the requirements of NRC Generic Letter 89-09.

21. The sections of piping from valves 2P42-F051 and 2P42-F052 through the containment penetration sleeves are Quality Group B. The remainder of the system inside containment is Quality Group D, Seismic Category I, and is covered under the plant's ASME Section XI inservice inspection and surveillance program for Nuclear Class 3 piping.

22. Except for primary containment isolation valves and associated piping from these valves to the containment penetrations, replacement components may be designed to the requirements specified in table 11.2-1.

23. This equipment is designed to GE Code, Class E specifications, which are equivalent to ASME Code, Section III, Class 2 requirements.

24. Heat exchanger 2G41-B001 purchased to ASME Code Section VIII, Division 1 2004 w/ 2006 Addenda, TEMA Class B, 8th Edition. TEMA-B meets or exceeds TEMA-C requirements.

TABLE 3.2-2 (SHEET 1 OF 3)

CODE REQUIREMENTS FOR BWR COMPONENTS AND SYSTEMS

ORDERED PRIOR TO JULY 1, 1971

	CLASSIFICATION GROUP			
	A	B	C	D
Reactor containment pressure vessels (steel)	-	ASME B&PV Code, Section III, Class B	-	-
Pressure vessels	ASME B&PV Code, Section III, Class A	ASME B&PV Code, Section III, Class C	ASME B&PV Code, Section VIII, Division 1	ASME B&PV Code, Section VIII, Division I
Piping	USAS B31.7 Nuclear Power Piping, Class I	USAS B31.7 Nuclear Power Piping, Class II	USAS B31.7 Nuclear Power Piping, Class III	USAS B31.1 Code for Pressure Piping
Pumps and valves	ASME Code for Pumps and Valves for Nuclear Power, Class I	ASME Codes for Pumps and Valves for Nuclear Power, Class II	ASME Code for Pumps and Valves for Nuclear Power, Class III	USAS B31.1 Code for Pressure Piping
Low-pressure storage tanks 0 to 15 psig	-	API-620 with NDT supplementary examination requirements per applicable code or standard	API-620 with NDT requirements in accordance with ASME Section VIII, Division I	API-620 or equivalent
Atmospheric storage tank	-	Applicable storage tank codes such as API-650 AWWAD110 or ANSI B96.1 with the NDT supplementary examination requirements per applicable code or standard	Applicable storage tank codes such as API-650 AWWAD100 or ANSI B96.1 with the NDT examination requirements in accordance with ASME Section VIII, Division 1 or API	API-650, AWWAD100 or ANSI B96.1 or equivalent
Heat exchangers	ASME B&PV Code, Section III, Class A	ASME B&PV Code, Section III, Class C, and TEMA Class C	ASME B&PV Code, Section VIII, Division 1, and TEMA Class C	ASME B&PV Code, Section VIII, Division 1, and TEMA Class C

TABLE 3.2-2 (SHEET 2 OF 3)

ORDERED AFTER TO JULY 1, 1971

	CLASSIFICATION GROUP			
	A	B	C	D
Reactor containment pressure vessels (steel)	-	ASME B&PV Code, Section III, Class MC	-	-
Pressure vessels	ASME B&PV Code, Section III, Class 1	ASME B&PV Code, Section III, Class 2	ASME B&PV Code, Section III, Class 3	ASME B&PV Codes, Section VIII, Division 1
Piping	ASME B&PV Code, Section III, Class 1	ASME B&PV Code, Section III, Class 2	ASME B&PV Code, Section III, Class 3	ANSI B31.1 Code for Pressure Piping
Pumps and valves	ASME B&PV Code, Section III, Class 1	ASME B&PV Code, Section III, Class 2	ASME B&PV Code, Section III, Class 3	ANSI B31.1 Code for Pressure Piping
Low-pressure storage tanks 0 to 15 psig	-	API-620 with the NDT supplementary examination requirements	API-620 with the NDT supplementary examination requirements	API-620 or equivalent
Atmospheric storage tank	-	Applicable storage tank codes such as API-650 AWWAD100 or ANSI B96.1 with the NDT supplementary examination requirements	Applicable storage tank codes such as API-650 AWWAD100 or ANSI B96.1 with the NDT examination requirements	API-650, AWWAD100 or ANSI B96.1 or equivalent
Heat exchangers	ASME B&PV Code, Section III, Class 1, and TEMA Class C	ASME B&PV Code, Section III, Class 2, and TEMA Class C	ASME B&PV Code, Section III, Class 3, and TEMA Class C	ASME B&PV Code, Section VIII, Division 1, and TEMA Class C

TABLE 3.2-2 (SHEET 3 OF 3)

NOTES:

1. Pumps operating above 150 psi and 212°F, ASME Section VIII, Division 1 of the B&PV Code, are used as a guide for calculating the thickness of pressure retaining parts. In sizing cover bolting below 150 psi and 212°F, manufacturer standards for service intended are used.

2. Nuclear piping, pumps, and valves meet the provisions of ASME B&PV Code, Section III, paragraph N-153, including Summer 1969 Addenda, with the exception of the recirculation piping and stainless steel portions of the RHR and portions of the RWC piping which were replaced in 1984 in accordance with the ASME B&PV Code, Section III, Subsection NB, 1980 Edition including Winter of 1980 Addenda.

3. Cast parts > 2 in. and < 4 in. in Quality Groups A and B are nondestructively tested where possible in accordance with the 1971 Winter Addenda of ASME Section III.

4. Supplementary examination requirements for tanks ordered after 7/1/71:

    a. 100% volumetric examination of the sidewall and roof weld joints for plates over 3/16-in. thick and 100% surface examination of weld joints for plates 3/16-in. thick or less and the sidewall-to-bottom and sidewall-to-roof joints. These examination requirements are to be performed in accordance with the rules of applicable codes.

    b. 100% volumetric examination of the sidewall weld joints for plates over 3/16-in. thick and 100% surface examination of weld joints for plates 3/16-in. thick or less and the sidewall-to-bottom joint. These examination requirements are to be performed in accordance with the rules of applicable codes.

5. This table documents an agreement reached with the NRC that, for HNP Units 1 and 2, all new components ordered after July 1, 1971, would meet the ASME Code. This agreement applied to new components ordered as part of the construction process and did not address repair and replacement components or parts. For Unit 1, this agreement is documented in the FSAR Questions and Answers (see Q&A 1.1).

LEGEND:

NDT = Nondestructive testing
API = American Petroleum Institute

TABLE 3.2-3

CODE STATUS OF CLASS I PRIMARY PRESSURE BOUNDARY COMPONENTS

Component Description	Plant Identification System No.	Code Specified	Code Required in Accordance with 10 CFR 50.55a
RPV	B11-A001	ASME III, 70S	ASME III, 70S
RPV head nozzle	B11-D072	ASME III, 70S	ASME III, 73
CRD housings	B11-D141,142,143,144	ASME III, 70S	ASME III, 70S
CRD	B11-D146	ASME III, 69W	ASME III, 70S
Incore housing	B11-D190,198	ASME III, 70S	ASME III, 70S
Jet pump instrument penetration	B11-D235	ASME III, 70S	ASME III, 70S
Safety relief valve	B21-F013	ASME III, 1968-1970 Addenda	ASME III, 1968-1970 Addenda
MSIV inboard	B21-F022	ASME III, 71W	ASME III, 71
MSIV outboard	B21-F028	ASME III, 71W	ASME III, 71
Primary steam piping	B21-G001	ASME III, 71W	ASME III, 71W
Main steam flow element	B21-N005	B31.7, 69 & ASME III, 71	ASME III, 71
Recirculation pump	B31-C001	NPVC, 70	NPVC, 70
Recirculation gate valve	B31-F023	NPVC, 70	NPVC, 70
Recirculation gate valve	B31-F031	NPVC, 70	NPVC, 70
Recirculation piping	B31-G001	ASME III, 1980 Edition, 80W	ASME III, 71W
Recirculation flow element	B31-N013	ASME III, 1980 Edition, 80W	ASME III, 71S

TABLE 3.2-4 (SHEET 1 OF 4)

SYSTEM P&IDs SHOWING QUALITY GROUP CLASSIFICATIONS

System	Drawing No.
CRD	H-26007
CRDH	H-26006
SLC	H-26009
Nuclear boiler	H-26000, H-26001, H-26189
RRS	H-26003
RCIC	H-26023, H-26024
RHR	H-26014, H-26015
RWC	H-26036, H-26037
Primary containment purge	H-26084
SGT	H-26078
Primary containment atmosphere H_2-0_2	H-26048, H-26049
HPCI	H-26020, H-26021
CS	H-26018
Jockey pump	H-26019
Post-accident reactor coolant and containment atmosphere sampling	H-26384
Spent-fuel pool cooling	H-26039
Spent-fuel pool cleanup	H-26040
PSW	H-21033, H26050, H-26051

TABLE 3.2-4 (SHEET 2 OF 4)

System	Drawing No.
RBCCW	H-26054, H-26055
Reactor and radwaste buildings condensate storage and transfer	H-26046
RHRSW	H-21039
Plant service noninterruptible instrument air	H-21028, H-21077, H-26064, H-26070, H-26260, H-26261
Leak detection	H-26076
Control building drainage	H-21063
Drywell pneumatic	H-26066, H-28023
Torus drainage and purification	H-26042
Auxiliary steam	H-26063
Control room ventilation	H-16042, H-26094
Reactor zone HVAC	H-26067
Reactor building safeguard equipment emergency cooling	H-26071
Reactor building refueling floor HVAC	H-26072
Radwaste building HVAC	H-26090
Turbine building HVAC	H-26086
Diesel generator building ventilation	H-12619
Primary containment (drywell) HVAC	H-26074
Control building ventilation	H-16041, H-26093
Control building - computer, waste analysis, and cold lab rooms air-conditioning	H-16035, H-40056

TABLE 3.2-4 (SHEET 3 OF 4)

System	Drawing No.
Waste gas treatment building HVAC	H-16549
Technical support center HVAC	H-26002
Reactor and radwaste buildings chilled water	H-26025, H-26008, H-50563
Primary containment (drywell) chilled water	H-26080, H-26081
Control building chilled water	H-51178
Control building chilled water cooling units	H-51179
Diesel engine and fuel oil	H-21074
Fuel oil diesel oil	H-11037
Main steam	H-21012, H-21056
Main condenser gas removal	H-21030, H-21056
Circulating water	H-21026
Condensate polishing demineralizer	H-21018, S-60192
Condensate and feedwater	H-21037, Sheets 1-5 H-21038, Sheets 1-3
Radwaste	H-26026 through H-26032
Radwaste support	H-26035
Offgas	H-26045
Process radiation monitoring	H-26011, H-26012 H-26013, H-16564
Fission products monitoring	H-16173, H-16274

TABLE 3.2-4 (SHEET 4 OF 4)

GENERAL NOTES:

Piping classes on the referenced piping and instrumentation diagrams are designated by a three-letter code. The first letter indicates the primary valve and flange rating, the second letter the type of material, and the third letter the code to which the piping is designed.

The designations are as follows:

First Letter

A	Specified pressure at specific temperature
B	2500# ANSI
C	1500# ANSI
D	900# ANSI
E	600# ANSI
F	400# ANSI
G	300# ANSI
H	150# ANSI
J, K, L, M	For general use as designated on pipe class sheets

Second Letter

A	Stainless steel
B	Carbon steel
C, D, E, F, G, H, K, L, M	For general use

Third Letter		AEC Quality GP Class
A	Nuclear Power Piping, ASME Section III, Class 1	A
B	Nuclear Power Piping, ASME Section III, Class 2	B
C	Nuclear Power Piping, ASME Section III, Class 3	C
E	Code for Pressure Piping, ANSI B31.1	D
F	No code requirements	

TABLE 3.2-6 (SHEET 1 OF 2)

PLACEMENT OF HNP-1 VALVES IN NUCLEAR SYSTEMS IN HNP-2

Nominal Pipe Size (in.)	Valve Type	Service System MPL No.	Service[1]	Total Valves Used	System Design Pressure	System Design Temperature	Valve Pressure Rating	Explanatory Notes
1	Globe	2T48	R	4	62	353	600	-
1	Globe	2G31	D	5	150	150	600	-
1	Globe	2P41	D	11	150	150	600	-
1	Globe	2P11	D	1	150	150	600	-
1	Globe	2G11	O	1	150	150	600	5
1	Globe	2G11	D	5	150	150	600	-
1	Globe	2G11	R	2	150	150	600	-
1	Globe	2C41	O	2	150	150	1500	2
1	Check	2G11	O	5	150	150	600	3
1	Check	2P41	O	1	150	150	600	4
3/4	Globe	2G11	R	26	150	150	600	-
3/4	Globe	2G11	V	8	150	150	600	-
3/4	Globe	2P41	R	24	150	150	600	-
3/4	Globe	2P41	V	8	150	150	600	-
3/4	Globe	2P70	R	5	150	150	600	-
3/4	Globe	2G41	R	1	150	150	600	-
3/4	Globe	2G41	O	2	150	150	600	6
3/4	Globe	2G41	V	1	150	150	600	-
3/4	Globe	2P11	V	1	150	150	600	-
3/4	Globe	2E21	V	8	377	400	600	-
3/4	Globe	2E21	D	8	377	400	600	-
1/2	Check	2G11	O	1	150	150	600	7

TABLE 3.2-6 (SHEET 2 OF 2)

NOTES:

1. R = Instrument root valve

   V = System high point vent valve

   D = System low point drain valve

   O = Other: see explanatory notes column

2. These valves function as service air and demineralized water system boundary valves. In service, the valves are normally closed, locked closed.

3. These valves function in the radioactive waste treatment system as reverse flow preventers. Failure of the valves to perform their design function would not result in adverse plant conditions, nor would their failure to function result in any increase in the release of radioactive effluents.

4. This valve serves as a reverse flow preventer in the swing-diesel generator service water fill piping. Failure of this valve to perform its intended function will not adversely affect diesel generator operation.

5. This valve serves as an isolation valve for an air-hose connection to the floor drain filter backwash piping. In service, the valve is normally closed. Failure of this valve to perform its intended function would not result in adverse conditions within the plant because the valve is backed up in its function by a check valve and a pipe cap fitting.

6. These valves function as sample connection isolation valves. In service, these valves are normally open. Failure of these valves to perform their intended function would not result in adverse conditions for the system or the plant.

7. This valve serves as a reverse flow preventer in a filtered, service air connection to the radwaste system waste filter vessel. Failure of the valve to perform its intended function would not result in adverse conditions for the system or the plant.

3.3 WIND AND TORNADO LOADINGS

3.3.1 WIND LOADINGS (HNP-1 AND HNP-2)

Wind loadings for Seismic Category I structures were selected on the basis of American Society of Civil Engineers (ASCE) Paper No. 3269, "Wind Forces on Structures."[1]

3.3.1.1 Design Wind Velocity

Seismic Category I structures are designed to withstand a basic wind velocity of 105 mph. The recurrence interval of this wind velocity is estimated to be at least 100 years.[1] The variation of wind velocity with height is shown in table 3.3-1.

3.3.1.2 Basis for Wind Velocity Selection

The fastest mile of wind at the Hatch plant site is shown, according to Figure 1(b) in the ASCE paper,[1] to be 105 mph.

3.3.1.3 Vertical Velocity Distribution and Gust Factor

The wind pressures resulting from the wind velocities shown in table 3.3-1 incorporate the shape factors in both horizontal and vertical directions.

The gust factor of 1.1 was selected which allows for a gust of ~ 10-s duration which, in a 105-mph basic wind, would have a length downwind of ~ 1540 ft; this factor is described as adequate in ASCE Paper No. 3269 for structures having a horizontal dimension, transverse to the wind, of 125 ft and larger.

3.3.1.4 Determination of Applied Forces

The design wind dynamic pressure is calculated by:

$$q = 0.002558 \left(V^2\right)$$

where:

q = pressure (lb/ft^2).

V = velocity (mph).

A shape coefficient of 1.3 is applied with all wind loads. Of the total of 1.3 q, 0.8 q is applied as positive pressure to the windward walls, and 0.5 q is applied as negative pressure on the leeward walls, where applicable.

Wind loads are applied to the structures as uniform static loads on the vertical areas of the walls.

The applied force of the magnitude and the distribution for Seismic Category I structures is shown in table 3.3-1.

3.3.2 TORNADO LOADINGS

All above-ground Seismic Category I structures are designed to withstand tornado loadings and tornado-generated missiles.

If tornadic winds traverse the site, the reactor is capable of being shut down and secured in a safe shutdown mode. Superstructure damage could be incurred to the reactor building, turbine building, storage tanks, and incoming power lines without affecting the ability to shut down the reactor and maintain integrity of containment and essential heat removal systems during and following a tornado which might traverse the site. Simultaneous damage to all of these items is not expected. However, as a design objective, the reactor is capable of being safely shut down and maintained in a safe-shutdown condition with the loss of all such equipment.

Components which directly affect the ultimate safe shutdown of the plant are located either under the protection of reinforced concrete or underground. These components include the following:

- Reactor coolant system.
- Control rod drive system.
- Standby liquid control system.
- Primary containment and isolation valves.
- Reactor core isolation cooling system.
- Residual heat removal system and associated cooling systems.
- Battery system.
- Standby diesel generator system.
- Electrical controls and instrumentation (for above systems).

- Main control room (MCR).
- Plant instrument air system.
- Intake structure (portions essential to systems listed above).

With the above equipment protected, the plant has the capability to maintain a safe shutdown condition for prolonged periods.

3.3.2.1 Applicable Design Parameters

For Seismic Category I structures designed to withstand tornadoes and tornado-generated missiles, the following parameters are applied in combinations producing the most critical conditions:

A. Dynamic Wind Pressure

The dynamic wind pressure is caused by a tornado funnel having a peripheral tangential velocity of 300 mph and a forward progression of 60 mph. The applicable portions of wind design methods described in ASCE Paper No. 3269 are used to determine the proper drag and shape coefficients. The provisions for gust factors and variation of wind velocity with height are not applied. The average tornado design dynamic wind pressure is q = 230 lb/ft^2 based on an average wind velocity of 300 mph.

B. Pressure Differential

The structure interior bursting pressure is taken as rising 1 psi/s for 3 s, followed by a 3-s calm, then decreasing at 1 psi/s for 3 s. This cycle accounts for reduced pressure in the eye of a passing tornado. All fully enclosed Category I structures are designed to withstand the full 3-psi pressure differential.

C. Missile Impingement

A tornado missile is defined as any object dangerously set in motion and erratically propelled by a tornado. Two types of tornado missiles are considered; each type is assumed to act independently and only one type may be generated at any one time. It is also assumed that the missiles do not tumble while in flight, and are at any time oriented to have the maximum value:

$$\frac{C_d A}{W}$$

where:

C_d = drag coefficient.

A = projected area of missile exposed to wind.

W = weight of missile.

The two types of missiles are as follows:

- A 12-ft-long piece of wood 4 in. x 12 in. in size (108 lb) traveling end-on at a speed of 300 mph and striking the structure at any elevation.
- A 4000-lb automobile, traveling end-on at a speed of 50 mph and striking the structure on an impact area of 20 ft^2, with any portion of the impact area being not more than 25 ft above grade.

D. Torsional Moment

A torsional moment results from applying the wind specified in A on one-half of the structure and a wind velocity equal to one-half that specified in A applied to the other half of the building in the opposite direction.

3.3.2.2 Critical Load Combination for Tornado Load

The loading combination (7) listed below controls the design in determining total tornado load W_t . Other loading combinations were found not to control the design in determining total tornado load.

1. $W_t = W_w$
2. $W_t = W_p$
3. $W_t = W_m$
4. $W_t = W_w + .5W_p$
5. $W_t = W_w + W_m$
6. $W_t = W_w + .5W_p + W_m$
7. $W_t = W_w + W_p + W_m$

where:

W_t = total tornado load.

W_w = tornado wind load.

W_p = tornado differential pressure load.

W_m = tornado missile load.

3.3.2.3 Depressurization and Blowout Panels

A rapid depressurization of the air surrounding the structures occurs if the funnel of a tornado suddenly engulfs the structure. Necessary provisions are made for venting the structures (other than primary containment) which affect equipment necessary for safe shutdown. Venting is accomplished by placing blowout panels, designed to fail at a pressure lower than the safe building capability for internal pressure, that relieve excess pressure in all essential parts of such structures.

For those compartments that are vented, a flow analysis of all air volumes and interconnecting vent areas is performed and the maximum transient pressure differential across every wall, floor, and roof are calculated using the principles of fluid mechanics to determine its maximum transient pressure differential. Finally, each structural component is checked to ensure that it can withstand the maximum calculated transient pressure differential which it experiences.

The blowout panels used for venting the reactor and control building roofs are designed and tested initially to:

- Open against a wind velocity of 300 mph and remain open.
- Open when the internal static pressure in the building is increased to 55 lb/ft^2.
- Release when the internal static pressure in the building is increased to 55 lb/ft^2 if the principal mechanism fails to operate.

Since they are designed to open during a tornado, the impact of tornado missiles was not considered in the designing of the blowout panels.

3.3.2.4 Structural Strength Considerations

The structural steel frame, precast concrete wall panels, and roof deck of the reactor building above the refueling floor are designed to withstand the forces of a tornado.

3.3.2.5 Determination of Forces on Structures

Tornado loads are applied to the Seismic Category I structures in the same manner as the wind loads described in paragraph 3.3.1.4 with the exception that gust factor and variation of wind velocity with height do not apply. The load combinations involving tornadoes are given in paragraphs 3.8.2.3, 3.8.3.3, 3.8.4.3, and 3.8.5.3.

The load factor selected for tornado loadings is 1.0, based on the short duration of the loading condition, the low probability of a tornado striking a specific geographic point, and the degree of conservatism in the selection of design tornado velocity.

The exterior walls of the reactor building are selected as representative of the design procedure. Using a model of the building and normalized Hoecker pressure profile, suctions and airflows within the building were computed using the principles of compressible fluid flow. The maximum transient crushing and bursting pressures were computed. These pressures were applied to the walls as uniform loads to develop moment and shear diagrams. Additionally, the exterior walls were designed for dynamic concentrated loads representing the tornado missile impacts. These loads were obtained from dynamic analysis of the walls subjected to a pulse loading. The pulse was fitted to each case, i.e. span length, thickness, and missile energy, by trial and correction to satisfy energy and momentum principles. The moments and shears due to missiles were combined with those from crushing. The bursting moments and shears, or carryover moments from missile impact if larger, were used to design the opposite face reinforcement.

In most cases practical wall designs required a portion of the missile impact energy to be dissipated in the plastic range in the struck span. The ductility ratio as a general rule was limited to 10. This ratio in no case exceeds 20.

3.3.2.5.1 Safety Consideration for Tornado Relief Vent Openings

The structural steel angle framed safety grills provided under each of the tornado relief vent openings probably deflect the postulated tornado missile upon entering. However, even considering no deflection of the missile, the energy area ratio is not sufficient to cause failure of the spent-fuel racks in the reactor building spent-fuel pool or the heating, ventilation, and air-conditioning (HVAC) ductwork on the control building roof at el 180 ft-0 in. Failure of the MCR HVAC equipment, such as the MCR chiller units, could possibly result from the falling missile; however, failure of this equipment would not be sufficient to render the plant incapable of being shut down safely.

A sketch of the grill system is shown in figure 3.3-1. This grill was installed for safety reasons and was not designed for missile protection. The analysis of the effect of the Hatch 2 missile that could penetrate the tornado relief vent openings; i.e., the 4-in. x 12-in. x 12-ft wooden plank weighing 108 lb on the spent-fuel pool and control building roof, did not assume any deflection (change in direction from the end-on missile) from the grill, although deflection would most likely take place.

The mechanism of failure for the structural grill system upon the impact of the 12-ft-long wooden plank, 4-in. x 12-in., weighing 108 lb, and traveling end-on at a speed of 300 mph, is based on the energy absorption capacity of each angle of the grill. The angles may fail either by bending or by shearing, depending on where the missile strikes.

The angles in the structural grill system would most likely not fail in a manner to generate secondary missiles since the angles are welded at both ends. However, should secondary missiles be generated from the grill, neither the plank missiles nor the angles from the grill have

targets available which are required for safe plant shutdown either on the control room roof at el 180 ft or on the refueling floor at el 228 ft. There is only one relief vent directly over the spent-fuel pool, and if the plank hits the grill, a maximum of three angles could be generated as secondary missiles with a maximum energy of 2000 ft-lb each. The General Electric spent-fuel storage racks, which are protected by 21 ft of water cover, are designed to withstand an impact of ~ 9000 ft-lb[2] over a 3-in.-diameter (or larger) area before the racks would be damaged to the extent that special tooling is required to remove the fuel bundles. Additionally, the Holtec spent-fuel storage racks are able to withstand an impact of ~ 6600 ft-lb.[3]

The Seismic Category I structures, systems, and components which may be impacted by the postulated plank missile falling through the reactor building tornado relief vents or the secondary missiles generated by the failure of the angles in the grill system on the refueling floor include the reactor building overhead crane, the refueling bridge, the spent-fuel storage racks in the spent-fuel pool, and a service water system hose station, as well as the refueling floor itself. Seismic Category I structures which may be impacted by the postulated plank missile or the secondary missiles generated by the failure of the angles in the grill system on the control room roof (el 180 ft) include the MCR environmental control system equipment.

These items were not designed to resist postulated vertical missiles. However, the refueling floor has a slab thickness of a minimum of 18 in. with concrete of f'_c = 5000 psi. The control room roof at el 180 ft has a thickness of 30 in. with concrete of f'_c = 4000 psi.

After passing through the safety grill at the tornado vents and due to the hydrodynamic and buoyancy effects of the water in the fuel pool, the plank missile will have ~ 920 ft-lb of kinetic energy at a depth of 21 ft in the fuel pool. Since the General Electric spent-fuel storage racks are designed to withstand an impact of ~ 9000 ft-lb before the racks are damaged to the extent special tooling is required to remove the fuel bundles, the effect of the plank missile is negligible. The effect of the plank missle on the Holtec racks, which are able to withstand ~ 6600 ft-lb, is also neglible. This energy represents the design capability of the fuel racks even though considerably more energy would be required before fuel damage occurs.

Damage to other components identified above does not prevent safe shutdown of the plant.

3.3.2.6 Ability of Seismic Category I Structures to Perform Despite Failure of Structures not Designed for Tornado Loads

Failure of Category II structures not designed for tornado loads does not affect the ability of Seismic Category I structures to perform their functions for the following reasons:

A. Tornado missiles that may be formed by the failure of Category II structures do not exceed the force of those postulated and described in paragraph 3.3.2.1, against which Seismic Category I structures are designed.

B. The structural frame of the Category II turbine building has been designed against collapse when subjected to tornado loadings.

REFERENCES

1. "Wind Forces on Structures," Transactions of the ASCE, Paper No. 3269, 1961.

2. "Tornado Protection for the Spent-Fuel Storage Pool," APED 5696, General Electric, November 1968.

3. Southern Nuclear Operating Company letter HL-5752, "Spent Fuel Pool Storage Expansion Request for License Amendment," from H. L. Sumner, Jr., to NRC, dated April 6, 1999.

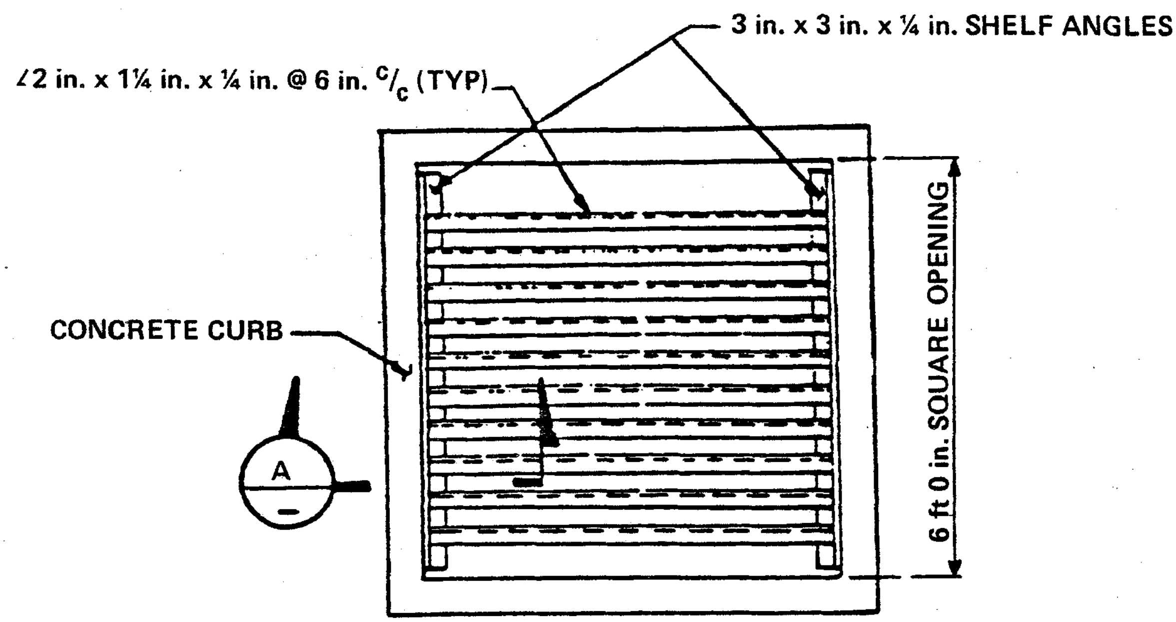
TABLE 3.3-1

**WIND LOADS
(HNP-1 AND HNP-2)**

Height (ft)	Velocity (mph)	Dynamic Pressure q (lb/ft^2)	Wall Load (lb/ft^2) Pressure 0.8q	Wall Load (lb/ft^2) Suction 0.5q	Roof Load (lb/ft^2) Suction 0.75q
0 - 50	105	28	22	14	21
50 - 150	131	44	35	22	33
150 - 400	161	66	53	33	50

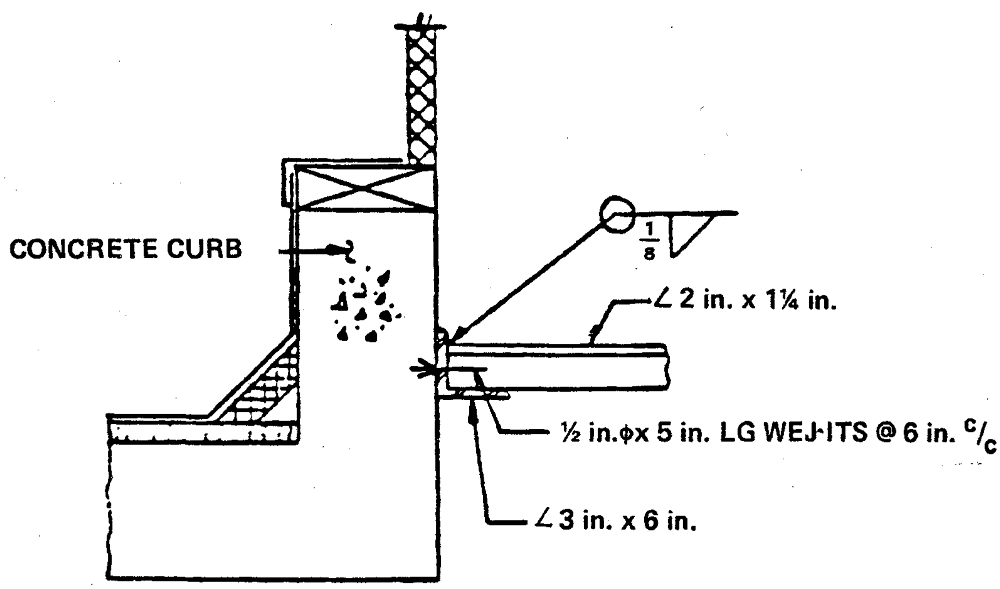
The design wind loads for the main stack are as follows:

ft	Effective Pressure (lb/ft^2)
0-50	19
50-150	28
150-400	43
400-TOP	57



PLAN

STRUCTURAL GRILL SYSTEM



SECTION A

ACAD

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SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

STRUCTURAL GRILL SYSTEM

FIGURE 3.3-1

3.4 WATER LEVEL (FLOOD) DESIGN (HNP-1 and HNP-2)

Seismic Category I structures and components are designed for the protection of safety-related equipment from external flooding. The design maximum flood elevations used in the design of each Seismic Category I structure are described in subsection 2.4.3 and are as follows:

- Primary containment - el 105 ft.
- Reactor building - el 105 ft.
- Diesel generator building - el 105 ft.
- Control building - el 105 ft.
- Intake structure - el 105 ft.
- Main stack - el 105 ft.

Since all Category I structures except the intake structure are not affected by the maximum flood level of the Altamaha River, the design basis for buoyancy and static-water force effects is the ground water level at el 122 ft msl. For the intake structure, the maximum water level considered was 105 ft.

3.4.1 FLOOD PROTECTION

All Seismic Category I systems and components located below flood level are protected by watertight Seismic Category I structures designed to withstand the flood condition.[1]

A. There are no safety-related systems or components located below the design maximum flood elevation of 105 ft that are not protected against flood. When safety-related equipment is located in subgrade levels of Seismic Category I structures, the equipment is protected from the effects of ground water and natural flooding levels by the design of the structures.

   The grade elevations for the Seismic Category I structures are as follows:

   - Reactor building - el 129 ft.
   - Diesel generator building - el 129 ft.
   - Control building - el 129 ft.
   - Intake structure - el 110 ft.
   - Main stack - el 120 ft.

All exterior entrances are at or above the grade level for these structures. Therefore, flood protection for the entrances is not required. A list of exterior entrances is found in table 3.4-1.

Safety-related equipment is protected from the effects of ground water level by sealing each below-grade penetration with an appropriate seal. Seal designs used are shown on drawing no. H-16110.

B. Structures described above housing safety-related equipment do not have exterior or access openings and penetrations below the design flood levels. Access to these structures is possible only from above grade level, which is el 129 ft.

C. Such means as pumping systems, stoplogs, watertight doors, and drainage systems for flood protection are not required because access openings and penetrations are not provided below the design flood levels. Drains and sumps of adequate sizes are provided in all Seismic Category I structures and components to cope with potential inleakage from such phenomena as cracks in structure walls and leaking waterstops.

The wave crest at maximum discharge in the Altamaha River is calculated to reach an elevation of 108.3 ft. The wave runup may splash water onto the roadfill around the intake structure which is at el 110 ft and eventually leak into the building. Floor drains in the intake structure building are designed to handle such an inleakage.

3.4.2 ANALYSIS PROCEDURES

The foundation slabs and exterior walls of safety-related structures are designed to resist upward and lateral pressures caused by the maximum flood level given in the preceding paragraph.

The hydrostatic pressure acting uniformly at the bottom of the structure is the product of the height to the design flood level and the water weight taken as 63 lb/ft^3 (figure 3.4-1). The horizontal pressure acting on the exterior walls varies with height, from maximum value at the bottom of the wall to zero value at the design flood level (figure 3.4-1).

The grade elevations for all safety-related structures, with the exception of the intake structure, are provided in subsection 3.4.1. Based on these high grade elevations, such phenomena as flood current, wind wave, hurricane, or tsunami were not considered to generate dynamic water forces on these structures. The possible maximum wind wave height considered for the design of the intake structure was based on the procedure described in reference 2. Based on a maximum upstream fetch of 18 miles, a maximum sustained wind velocity of 45 mph, and with a duration of more than 1 h, the wave crest at maximum discharge would reach an elevation of 108.3 ft. The reinforced concrete intake pump structure walls which may be affected by the wave runup are designed for an impact load of 4000 lb at 50 mph over an area of 25 ft^2.

REFERENCES

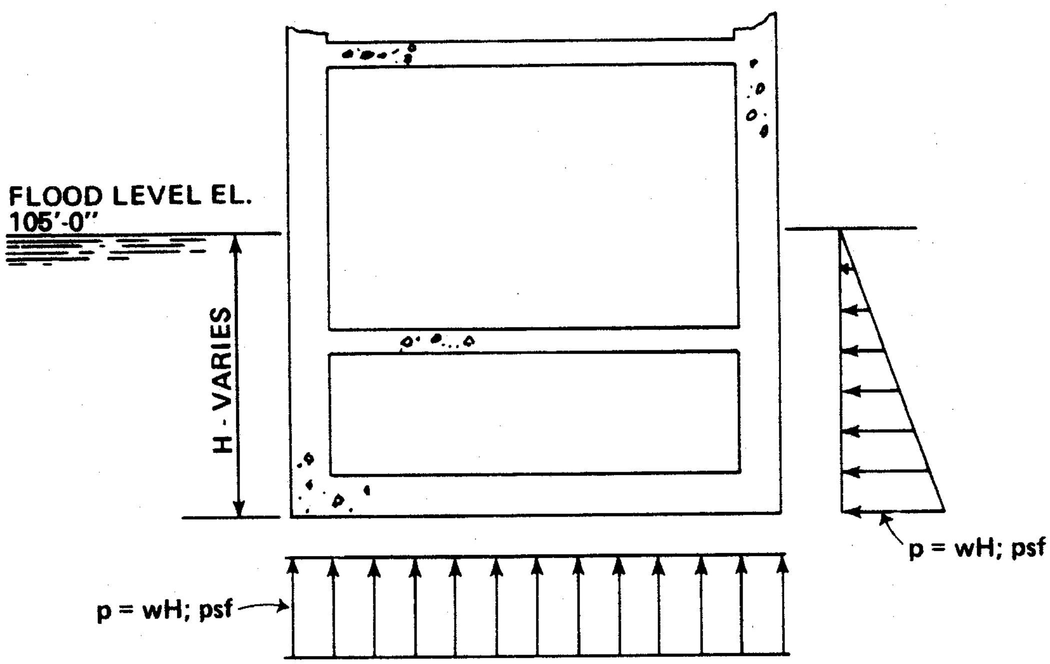
1. "Design Basis for Protection Against Natural Phenomena," 10 CFR 50, Appendix A, General Design Criterion 2.

2. Saville, McClendon, and Cochran, "Freeboard Allowances for Water in Inland Reservoirs," Journal of Water Ways Division, ASCE, May 1962.

TABLE 3.4-1

SEISMIC CATEGORY I STRUCTURES EXTERIOR ENTRANCE (HNP-1 AND HNP-2)

Structure	Entrance Size	Elevation	Type of Door
Reactor building	12 ft x 14 ft	130 ft 0 in.	Airtight door
	3 ft x 7 ft	130 ft 0 in.	Airtight door
	3 ft x 7 ft	130 ft 0 in.	Airtight door
Diesel generator building	6 ft x 7 ft	130 ft 6 in.	Double security doors
	6 ft x 7 ft	130 ft 6 in.	Double security doors
	6 ft x 7 ft	130 ft 6 in.	Double security doors
	6 ft x 7 ft	130 ft 6 in.	Double security doors
	6 ft x 7 ft	130 ft 6 in.	Double security doors
	6 ft x 7 ft	130 ft 6 in.	Double security doors
Control building	9 ft 1/2 in. x 10 ft	130 ft 0 in.	Metal rollup door
	3 ft 6 in. x 7 ft	130 ft 0 in.	Security door
Intake structure	3 ft 8 in. x 7 ft	111 ft 0 in.	Security door
	3 ft 8 in. x 7 ft	111 ft 0 in.	Security door
Main stack	3 ft x 7 ft	120 ft 0 in.	Security door
	3 ft x 7 ft	120 ft 0 in.	Security door
	16 ft x 16 ft	120 ft 0 in.	Rollup door



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EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

WATER PRESSURE ON STRUCTURES

FIGURE 3.4-1

3.5 MISSILE PROTECTION

Seismic Category I structures are designed to protect safety-related equipment and components from damage by internal and exterior missiles.

Systems and components required to ensure integrity of the reactor coolant pressure boundary (RCPB), as defined section 5.1, are contained within two structures -- the primary containment drywell and the reactor building, below el 228 ft 0 in. The primary containment drywell vessel described in paragraphs 3.8.2.1.1 and 6.2.1.2.1.1 is wholly contained within the concrete biological shield having a minimum concrete thickness of 60 in. The biological shield is in turn housed within the reactor building described in paragraphs 3.8.4.1 and 6.2.1.2.2. Structural design features of the reactor building are illustrated on drawing nos. H-26096 and H-26098 through H-26105. As may be determined from the referenced drawings, the reactor building has a minimum wall thickness of 18 in. and a minimum cover; i.e., floor at el 228 ft 0 in., of 18 in. ($f'_c = 5000$ psi).

Systems and components required to ensure the capability of shutting down (and cooling down) the reactor and maintaining the reactor in a safe condition may be considered to include, exclusive of the RCPB components discussed above, the following:

- Reactor core isolation cooling (RCIC).
- Residual heat removal (RHR).
- Residual heat removal service water (RHRSW).
- Plant service water (PSW).
- Standby ac power and diesel generator auxiliary systems.
- dc power systems.
- Emergency core cooling system (ECCS) pump room coolers.

The RCIC and RHR systems, along with the ECCS pump room coolers described in subsections 5.5.6, 5.5.7, and 9.4.2, respectively, are housed in the reactor building. The standby ac power system and associated diesel generator auxiliary systems are housed in the diesel generator building described in paragraph 3.8.4.1. The diesel generator building is constructed with a minimum wall thickness of 30 in. and a minimum roof thickness of 24 in. The dc power system is housed in the control building described in paragraph 3.8.4.1. The control building is provided with 24-in. walls and a 30-in. roof. The standby ac power system and the dc power system are described in chapter 8. A description of the diesel generator auxiliary systems is provided in subsections 9.5.4 through 9.5.7. The PSW and RHRSW systems are housed in the intake structure described in paragraph 3.8.4.1. The intake structure is constructed with minimum wall and roof thicknesses of 30 in. Portions of the RHRSW and the PSW systems are also housed in the diesel generator building, the reactor building, and the

control building; some of the piping is buried a minimum of 3 1/2 ft underground. Descriptions of the PSW and RHRSW systems are provided in subsections 9.2.1 and 9.2.7, respectively.

Systems and components required to ensure the capability of preventing accidents that could result in potential offsite exposures not within the guideline values of Title 10 Code of Federal Regulations (CFR) Part 50.67 are included in the previous discussions.

A discussion of turbine missiles is presented in subsection 10.2.3.

Table 3.5-3 is a tabulation summary of the foregoing discussion.

3.5.1 MISSILE BARRIERS AND LOADINGS

The missile shields and barriers are designed to resist the selected missiles described in subsection 3.5.2.

3.5.1.1 Accident/Incident-Generated Missiles (Inside Primary Containment)

The design philosophy is that no missiles are allowed to penetrate the primary containment. This is accomplished, in practice, through the specific design of the containment and containment systems that takes into account the potential for missile generation and minimizes the possibility of containment violation.

Safety-related equipment within the primary containment is protected from the effects of missiles by redundancy and separation. Therefore, missile barriers are not required to ensure the capability of safely shutting down the plant.

3.5.1.2 Internally Generated Missiles (Outside Primary Containment)

Safety-related equipment is primarily protected from internally generated missiles by separation through the design of the Seismic Category I structures.

3.5.1.3 Environmentally Generated Missiles

Seismic Category I structures, housing equipment and components vital to a safe shutdown, were designed to withstand penetration of the tornado missiles described in paragraph 3.3.2.1C. These structures, having at least 1-ft 6-in.-thick concrete exterior walls, constitute barriers against missile penetration. Calculations show that the deepest missile penetration of the concrete barriers would be 10 in. Therefore, the 1-ft 6-in.-thick slabs provide ample protection. A tabulation of all outdoor safety-related components, including the ventilation intakes and exhausts and the vents for safety-related tanks, is provided in table 3.5-4. This table indicates the tornado missile protection provided for all listed items.

The condensate storage tank provides the source of makeup water to the steam cycle on the boiling water reactor (BWR). This tank is protected from the penetration of tornado missiles by a Seismic Category I structure as described in section 9.2.6.3.

The BWR is not fitted with an auxiliary feedwater system.

The PSW pump and the RHRSW system pump intake structure are described in paragraph 3.8.4.1D and, as designed, afford tornado missile protection for the service water pumps.

The RHRSW and PSW Division I and II piping headers pass through the valve pit en route to the service connections in the plant complex. The automatic backwash strainers for the PSW system are located in the valve pit. Flooding of the valve pit was evaluated, and the result would be the loss of the automatic feature of the backwashable strainers. Loss of this automatic feature would not jeopardize the continued operation of the PSW system. Pipe routing and equipment location in the valve pit are shown on drawing no. H-21102. The Borden metal grating on the valve pit is capable of withstanding the energy imposed by the postulated missile impacts.

3.5.1.4 Site Proximity Missiles

The nearest airport with scheduled passenger service is located in Savannah, Georgia, ~ 67 miles northeast of the site. Small municipal fields not used for scheduled commercial services are located in Baxley ~ 13 miles south; in Hazlehurst, ~ 16 miles southwest; in Vidalia, ~ 20 miles north; in Alma, 28 miles south, and in Waycross, ~ 48 miles south. The nearest defense facility is Fort Stewart, with the nearest boundary 30 miles from the site. The closest firing range is R 3006 located ~ 45 miles from the site. For these reasons, the site is considered sufficiently distant from aircraft and guided missile installations; therefore, the plant is neither designed nor operated with special provisions to protect the facility against the effects of the installations and/or the aircraft.

3.5.2 MISSILE SELECTION

3.5.2.1 Accident/Incident-Generated Missiles (Inside Primary Containment)

The following missiles are postulated to be generated within the primary containment:

- Recirculation pump missiles.
- Relief valve bonnets.
- Thermowells - with and without resistance temperature detectors (RTDs).
- Valve bonnets (small and large).

- Valve stems.

- Control rod drive (CRD) housings.

The valve bonnets and the thermowells are jet-propelled missiles. Valve stems are piston-type missiles.

Recirculation pump missiles are discussed in paragraph 1.5.1.2.5.

The CRD housings are restrained from becoming missiles by supports designed to allow a maximum deflection of 3 in. A detailed discussion of the CRD housings and the support members is included in paragraph 4.2.3.2.3.1.

3.5.2.2 Internally Generated Missile Selection (Outside Primary Containment)

There are three types of postulated internally generated missiles outside the primary containment:

- Rotating-equipment missiles.

- Piston-type missiles.

- Jet-propelled missiles.

Pump impellers are considered to remain within the pump casing; therefore, the only postulated missiles from rotating equipment are couplings and turbine-generator missiles discussed in subsection 10.2.3. Equipment for which coupling missiles were evaluated is as follows:

- Core spray pump.

- High-pressure coolant injection pump.

- Hydraulic fluid power unit.

- RCIC pump.

- RHR pump.

- RHRSW pump.

- PSW pump.

- Reactor water cleanup (RWC) pump.

Other internal missiles selected are piston-type and jet-propelled missiles. These missiles are:

- Valve stems.
- Valve bonnets (large and small).
- Thermowells.

The valve stems are considered as piston-type missiles; valve bonnets and thermowells are considered as jet-propelled missiles.

3.5.2.3 Environmentally Generated Missile Selection

The tornado-generated missiles selected for the design of Seismic Category I structures and the structural frame of the Category II turbine building are described in subsection 3.3.2.

3.5.3 SELECTED MISSILES

3.5.3.1 Selected Accident/Incident-Generated Missiles (Inside Primary Containment)

Velocities for missiles postulated within the primary containment were determined by the same methods used for the missiles postulated outside the primary containment (paragraph 3.5.3.2). Missile velocities and steel penetrations are tabulated in table 3.5-2.

The maximum steel thickness required to prevent perforation by the worst case postulated missile is 0.61 in. Since the drywell shell thickness in the region where postulated missiles would be generated is a minimum of 1.125 in., no missile is capable of penetrating the primary containment.

3.5.3.2 Selected Internal Missiles (Outside Primary Containment)

Missiles from rotating equipment described in paragraph 3.5.2.2 were evaluated for protection requirements.

The hydraulic fluid power unit, the RCIC pump, and the RWC pump use full coupling guards in their constructions. The coupling guards minimize the consequences of a coupling failure, in addition to providing personnel protection.

The remaining pumps have couplings installed within the motor support. The cylindrical motor support has access openings through which parts of a coupling may escape. However, the area of the access openings is small relative to the area of the motor support walls. Therefore, the coupling would probably ricochet and lose energy before leaving the confines of the motor support.

Just as the aforementioned equipment is safety-related and protected by redundancy and separation, the consequences of rotating-equipment missiles do not exceed the safety-related equipment design spectra.

The internal missiles postulated for HNP-2 are the missiles resulting from the turbine-generator failure and the missiles originating in the recirculation system pump following a postulated pipe rupture. The turbine missile analysis and evaluation are discussed in subsection 10.2.3.

For HNP-2, the break locations are defined by the criteria recommended in the Nuclear Regulatory Commission Standard Review Plans 3.6.1 and 3.6.2. The American Society of Mechanical Engineers (ASME) Code, Section III, Class I piping stress report was used to establish break locations.

Scope

Studies conducted on plants closely resembling HNP-2 indicate that HNP-2 is adequately safeguarded against adverse effects that may result from recirculation pump-generated missiles. The following indicates the work performed to evaluate the consequences of recirculation pump overspeed that leads to potential pump impeller missile generation following a postulated design basis loss-of-coolant accident for a typical General Electric BWR 4, Mark I containment nuclear power plant. The power plants selected for this probability study were Browns Ferry Nuclear Plant-Units 1, 2, and 3.

A. Probabilistic Model

The analysis used for evaluation of a typical BWR 4, Mark I containment plant was based on the USNRC pipe break criteria.

B. Break Locations

The break locations were determined using the criteria described in section 3.6.

C. Conclusions

Application of break location criteria delineated in section 3.6 indicated no damage to the primary containment, any major piping system, or to an inboard main steam isolation valve. Absence of damage is because the trajectories of postulated missiles do not intersect these systems.

D. Definition

Safe - A break is considered safe if the postulated missile:

- Is contained within the piping system.

- May leave the piping system at a velocity insufficient to perforate the containment or an essential piping system.

- Will impact a nonessential target that does not escalate the consequences of the accident.

Using conservative assumptions, it was determined that neither piston-type missiles nor jet-propelled missiles from fluid lines have sufficient velocity to penetrate the walls of the area in which they are produced.

The piston-type missile velocity was determined by assuming that all work done during ejection of the missile is converted to the kinetic energy of the missile. No friction loss or air resistance was assumed. The velocity of the missile is expressed as:[1]

$$V = \left(\frac{2PAL}{m}\right)^{1/2}$$

where:

V = velocity at end of piston stroke (ft/s).

P = pressure of fluid (psia).

A = cross-sectional area of piston (in.^{2}).

L = length of stroke (ft) (assumed to be stem length).

m = mass of missile (lb-s^2/ft).

The operating pressure of the HNP-2 safety systems or systems in proximity to safety systems is not sufficient to impart a velocity to stem missiles great enough to penetrate the drywell or the walls of the area in which the missile occurs. Therefore, the safety systems are protected from valve stem missiles by the walls of the Seismic Category I structures.

Jet-propelled missile (valve bonnets and thermowells) velocity was determined by first determining the jet velocity at the throat of the line penetration.

The velocity of the missile is induced by the mass and the velocity of the escaping fluid and is predicted by the following equation:[1]

$$\frac{-V}{V_f} - \ell n\left(1 - \frac{V}{V_f}\right) = \frac{K_2}{r_O} - \frac{K_2}{r_O + X\tan\beta}$$

where:

V = missile velocity (ft/s).

V_f = jet velocity at throat (ft/s).

r_o = radius of throat (ft).

X = distance traveled (ft).

β = half-angle expansion of jet.

and $K_2 = \dfrac{A_O A_m}{\overline{V}_f m \pi \tan\beta}$ = constant for a particular missile.

where:

A_O = break area (ft^2).

A_m = missile area (ft^2).

$\overline{V}_f$ = specific volume of jet fluid at break (ft^3/lbm).

m = mass of missile.

and zero initial velocity is assumed.

Velocity analyses for jet-propelled missiles were conservative in that a half-angle expansion of only 15 degrees was assumed, and the missile was assumed to retain the velocity attained at the point where the fluid jet assumed asymptotic properties.

Velocities were determined for missiles from pressurized systems containing saturated steam, saturated water, and subcooled water. Velocities of the fluid jets were determined by methods outlined in references 2 and 3. Missile velocities, in all cases, were not sufficient to enable significant concrete penetration or any spalling. Therefore, safety-related equipment is considered to be protected from jet-propelled missiles by the walls of the Seismic Category I structures. No additional missile barriers are required. The selected missiles are described in table 3.5-1.

3.5.3.3 <u>Selected Environmentally Generated Missiles</u>

The origin, weight, size, impact velocity, orientation, and material composition for each selected missile are given in paragraph 3.3.2.1.

The two types of tornado-generated missiles considered in the HNP-2 design are as follows:

- A 12-ft-long piece of wood, 4 in. x 12 in. in size (108 lb), traveling end-on at 300 mph and striking the structure at any elevation.

- A 4000-lb automobile traveling end-on at 50 mph and striking the structure on an impact area of 20 ft^2, with any portion of the impact area being no more than 25 ft above grade.

These missiles were tested, using a 16-in.-thick concrete slab, by the Environmental Test Department of Sandia Laboratories; and the results are reported by A. E. Stephenson in "Tornado Vulnerability Nuclear Production Facilities," April 1975, and also in "Addendum to Tornado Vulnerability Nuclear Production Facilities." The tests demonstrate that the two tornado-generated missiles under consideration do not penetrate or cause spalling; therefore, it can be concluded that they will not impair the structural integrity of the HNP-2 facilities.

Because of the test results, the material coefficient of penetration is not applicable.

As discussed in subsection 3.5.4, the overall structural response was limited to checking the flexural adequacy of the barriers, using a ductility ratio of 5.0 for computing the maximum flexural resistance. Thus, the equivalent static load and the deflection were not computed. Based on the evaluation of this structural response, typical percentages of steel reinforcement provided for the exterior walls are:

- el 130 to 158 ft - 0.00441.
- el 158 to 185 ft - 0.00208.
- el 185 to 203 ft - 0.00278.
- el 203 to 228 ft - 0.00204.

To assess the degree of protection comparability against tornado missiles provided by the HNP-2 design accepted at the construction permit stage of review, the following information is submitted:

Category I structures and appurtenances having walls or roofs with thicknesses < 2 ft are as follows:

Category I Structures/Appurtenances	Minimum Wall Thickness	Minimum Roof Thickness
Reactor building	1-ft 6-in. concrete wall; 8-in. concrete panel between el 185 and 228 ft	3-in. concrete over Robertson Q-floor 21-18
Reactor building vestibule area on el 228-ft floor	1 ft 0 in.	10 in.
Reactor building precast concrete panels above el 228 ft 0 in.	0 ft 8 in.	-
Intake structure valve pit	Below grade	Borden 2 1/4-in. by 3/16-in. type W/D grating
Control building precast concrete panels above el 164 ft 0 in.	0 ft 8 in.	3-in. concrete over Robertson Q-floor section 3
Main stack	1 ft 0 in.	0 ft 6 in.

Tornado-generated missiles from a vertical direction were not considered in the design of roofs. The main stack was not designed for tornado winds and associated missiles.

The precast panels and the walls previously identified as having thicknesses < 2 ft are located above 30 ft from the finished grade, and only an 8-lb steel rod that is 1 in. in diameter and 3 ft in length was considered in computing the following velocities:

Description	Velocity Required For Complete Penetration[4]	Velocity Below Which Generation of Secondary Missiles Will Not Occur[4]
1-ft 6-in.-thick concrete walls in reactor building above el 185 ft 0 in. made of concrete with minimum compressive strength $f_c' = 4000$ psi.	438 ft/s	260 ft/s
1-ft 0-in.-thick concrete walls in reactor building vestibule area above el 228 ft 0 in. made of concrete with $f_c' = 4000$ psi. No equipment is placed in this area.	323 ft/s	192 ft/s
8-in.-thick precast concrete panel in control building above el 164 ft 0 in. and in reactor building above el 228 ft 0 in. made of concrete with $f_c' = 5000$ psi.	259 ft/s	154 ft/s

3.5.3.4 Turbine-Generated Missiles

The turbine-generated missile analysis and evaluation are discussed in subsection 10.2.3.

3.5.4 BARRIER DESIGN PROCEDURES

The missile barriers were designed to resist missile penetration. The analysis for the depth of missile penetration was conducted using the following modified Petry formula for concrete:[5]

$$D = KApV'.$$

$$V' = \log_{10}\left[1 + \frac{V^2}{215{,}000}\right].$$

$$D' = D\left[1 + e^{-4(a'-2)}\right].$$

$$a' = \frac{T}{D}.$$

D = depth of penetration of an infinitely thick slab.

K = an experimentally obtained materials coefficient penetration (K = 0.00276 for 4000-psi reinforced concrete).

Ap = sectional pressure, obtained by dividing weight of missile by maximum cross-sectional area (expressed as lb/ft^2).

V′ = velocity factor.

V = terminal or striking velocity in ft/s.

D′ = actual depth of penetration.

T = thickness of resisting slab.

Missile penetration in a steel barrier was determined by using the Ballistics Research Laboratory formula, modified by setting a material constant K = 1.[4] The steel thickness, T, which will just be perforated by a missile, is solved as follows:

$$T = \frac{\left(\frac{MV_S^2}{2}\right)^{2/3}}{672D}$$

T = steel plate thickness to just perforate (in.).

M = mass of missile ($lb\text{-}s^2/ft$).

D = diameter of missile (in.).

V_s = striking velocity of missile, normal to target.

The thickness of steel required to prevent perforation is tp, where:

tp = 1.25 (reference 4)

The overall response evaluation of structural barriers to missile impact was not a design requirement at the time of the HNP-2 application; however, the exterior walls forming the structural barriers were considered to act as simple spans supported by various floors and checked for flexural adequacy of missile impact, using equation 5.16[6] for computing the required maximum resistance, and assuming a value of 5.0 for a ductility ratio and the ultimate strength design method for calculating the actual resistance of the slab barrier.

The structural steel framing was designed to carry all the loads from the floors and the roof. Typical compressive loads on the exterior walls that form a structural barrier are those due to the weight of the barrier itself. A typical structural barrier for the reactor building was evaluated. The computed compressive stresses on the barrier did not exceed a maximum of 110 psi, which is significantly lower than the allowable compressive stress; therefore, limiting values of the ductility ratios subjected to compressive and combined (flexure and compression) loadings were not calculated.

3.5.5 MISSILE BARRIER FEATURES

Drawing nos. H-12192, H-12320, H-12405, H-12406, H-12627, H-12629, H-12631, H-16249, H-22250, H-25000, H-26096, H-26098 through H-26105, H-40429, and H-40430 show the layout and principal design features of the barriers and structures designed to resist missiles. Additional information is provided in subsection 3.5.4 and paragraph 3.5.3.3.

REFERENCES

1. R. C. Gwaltney, "Missile Generation and Protection in Light-Water-Cooled Power Reactor Plants," Oak Ridge National Laboratory, ORNL-NSIC-22, September 1968.

2. F. J. Moody, "Prediction of Blowdown Thrust and Jet Forces," ASME Publication 69-HT-31, August 1969.

3. F. J. Moody, "Maximum Two Phase Vessel Blowdown from Pipes," General Electric, APED-4827, April 20, 1965.

4. "Design of Structures for Missile Impact," Bechtel Topical Report BC-TOP-9A, Rev 2, September 1974.

5. A. Amirikian, "Design of Protective Structures," Bureau of Yards and Docks, Publication No. NAVDOCKS P-51, Department of Navy, Washington D. C., August 1950.

6. Biggs, J. M., Introduction to Structural Dynamics, McGraw Hill Book Company, chapter 5, p. 223, 1964.

TABLE 3.5-1

SELECTED MISSILES OUTSIDE PRIMARY CONTAINMENT

Missile Description	Missile Weight (lb)	Maximum Velocity (ft/s)	Kinetic Energy (ft-lb)	Depth of Penetration in 2-ft-Thick Concrete Barrier (in.)
4-in. globe valve bonnet and motor	625	64	39,784	0.07
4-in. globe valve bonnet	250	99	38,078	0.84
1-in. globe valve bonnet	15	103	2473	0.24
6-in. gate valve bonnet and motor	925	102	149,557	0.24
Thermowell	3	202	1902	0.36
RTD and thermowell	8	220	6017	0.36
1 1/2-in. diameter valve stem	20	132	5415	1.92
1 1/4-in. diameter valve stem	11	123	2586	1.32

TABLE 3.5-2

SELECTED MISSILES INSIDE PRIMARY CONTAINMENT

Missile Description	Mass ($lb\text{-}s^2/ft$)	Diameter (in.)	Velocity (ft/s)	Steel Thickness Just Perforated (in.)	Steel Thickness Not Perforated (in.)
Main steam relief valve bonnet	9.32	6.76	150	0.49	0.61
4-in. motor-operated valve bonnet and motor	19.40	22.0	77	0.10	0.13
4-in. valve bonnet	7.76	6.0	118	0.36	0.46
1-in. valve bonnet	0.47	3.0	103	0.09	0.11
6-in. motor-operated valve bonnet and motor	28.70	25.0	107	0.19	0.23
Thermowell	0.09	2.0	202	0.11	0.14
RTD	0.25	3.6	220	0.14	0.18
1 1/2-in. diameter valve stem	0.62	1.5	132	0.31	0.39
1 1/4-in. valve stem	0.34	1.25	123	0.22	0.28

TABLE 3.5-3

TORNADO/TURBINE MISSILE PROTECTIVE BARRIERS

System/Component	Location	Protective Barrier - Minimum Thickness (in.)
RCPB systems and components	Drywell	Walls = 60 Cover = 72
	Reactor building (el 228 ft 0 in. & below)	Walls = 18[a] Cover = 18[b]
Systems and components required to ensure capability to shut down reactor and maintain reactor in a safe condition (excluding RCPB systems and components)	Reactor building (el 228 ft 0 in. & below)	Walls = 18[a] Cover = 18[b]
	Diesel generator building	Walls = 30 Cover = 24
	Intake structure	Walls = 30 Cover = 30
	Control building	Walls = 24 Cover = 30[c]

a. Reactor building walls are also provided with exterior mounted, 8-in.-thick precast concrete panels.
b. This slab is fabricated of concrete of f_c' = 5000 psi.
c. Thickness applies to the roof of the main control room.

TABLE 3.5-4 (SHEET 1 OF 2)

TABULATION OF ITEMS AND DESCRIPTION OF PROTECTION AGAINST TORNADO MISSILES

Components	Description of Protection Provided
A. Liquid nitrogen storage tanks	The liquid nitrogen storage tanks are located on each side of the HNP-1 reactor building railway airlock and are used in HNP-2 as a source of motive gas for essential air-operated valves and instruments. Drawing no. H-16147 shows the configuration of tank placement with respect to the railway air lock.
B. Ventilation system air intakes	Drawing no. H-16249 shows a typical arrangement for missile protection of the ventilation system air intakes, specifically illustrating the main control room ventilation air intake tornado missile protection system. This system for tornado missile protection is provided for the main control room ventilation system air intake, the reactor building ventilation system air intake, and the refueling area ventilation system air intake. With the exception of the diesel generator building combined ventilation and combustion air intakes discussed in item D, all other ventilation air intakes do not form a part of essential structures and are therefore not provided with tornado missile protection systems.
C. Ventilation system exhausts	Exhaust air from the turbine building, the radwaste building, the main control room, the reactor building, and the refueling area ventilation systems is discharged through the reactor building exhaust air vent plenum. Although the reactor building exhaust air vent plenum is not a missile-proof structure, missile damage to the vent plenum would not cause penetration of any essential structure. The air discharge from the standby gas treatment system is routed from the reactor building to the main stack via underground piping. The diesel generator building exhaust air system consists of relatively low-profile, roof-mounted Seismic Category I ventilators that are protected by the diesel generator building parapet wall.

TABLE 3.5-4 (SHEET 2 OF 2)

Components	Description of Protection Provided
D. Diesel generator combustion air intake	Drawing no. H-12619 shows the arrangement for a typical diesel generator room, battery room, switchgear room, and oil storage room. Combustion air for diesel generator operation is supplied through the corridor and then through the individual diesel generator room combustion air louvers. Tornado missile protection is provided by the corridor exterior wall immediately opposite the combustion air louvers. Drawing no. H-12320 further illustrates the general arrangement of the diesel generator building, showing that the corridor itself is protected from tornado missiles by the labyrinth at each end of the diesel generator building.
E. Diesel generator engine exhaust	Each generator diesel drive exhausts through a Seismic Category I muffler located on the roof of the diesel generator building. The mufflers present a relatively low-profile target and are protected from missiles by the building roof parapet. In addition, the engine exhaust mufflers are separated from each adjacent muffler by ~ 30 ft.
F. Vents for safety-related tanks	The following tanks are considered to be safety-related: • Diesel fuel oil day tanks. • Diesel fuel oil storage tanks. Each diesel fuel oil day tank is vented to its enclosure. Each diesel fuel oil storage tank is a buried tank and is vented to the atmosphere via a single vent pipe below grade elevation. Missile damage would be highly unlikely.

3.6 PROTECTION AGAINST DYNAMIC EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING

3.6.1 SYSTEMS IN WHICH DESIGN BASIS PIPING BREAKS OCCUR

Piping breaks have been postulated in those portions of the following systems inside containment which are pressurized during normal operation and hot standby:

- Reactor recirculation.
- Main steam.
- Feedwater.
- Residual heat removal (RHR).
- Core spray (CS).
- High-pressure coolant injection (HPCI) (steam).
- Reactor core isolation cooling (RCIC) (steam).
- Reactor water cleanup (RWC).
- Main steam drainage.
- Standby liquid control (SLC).
- Vessel drain (2 in.).
- Head vent (2 in.).
- Equalizing column (1 1/2 in.) and level-sensing line to reactor pressure vessel (RPV) nozzles N11A and B.
- Recirculation drainage.

Piping breaks for systems outside containment are identified and discussed in supplement 15A.

3.6.2 DESIGN BASIS PIPING BREAK CRITERIA

3.6.2.1 Postulated Failure Characteristics

The following types of pipe failures are postulated to occur at the corresponding locations specified in paragraph 3.6.2.2:

A. Circumferential Break

A circumferential break is a complete severance of a high-energy pipe, perpendicular to the pipe axis, resulting in an instantaneous release of mechanical internal pipe forces across the break. The resulting dynamic forces are assumed to separate the piping axially with at least a one-diameter lateral displacement of the ruptured piping sections except for certain break locations in the main steam, feedwater, RHR, and HPCI lines where pipe restraints were added to restrain pipe movement. The design of restraints is described in subsection 3.6.3. For analyzing the containment pressure and temperature responses as the result of postulated pipe breaks, at least one-diameter lateral displacement was assumed for all cases except the recirculation outlet nozzle breaker. For the recirculation outlet line, the penetration through the sacrificial shield was redesigned to restrict pipe movement as a result of a postulated nozzle break. Figure 3.6-1 provides a diagram of the modified penetration. The effective cross-sectional flow area of the pipe is used in the jet discharge evaluation. Movement is assumed to occur in the plane defined by piping geometry and configuration and in the direction of the jet reaction.

B. Longitudinal Break

A longitudinal break is an opening in a high-energy pipe wall parallel to the pipe axis without pipe severance, having a length of two inside pipe diameters and a cross-section area of one inside the pipe flow area. Dynamic forces resulting from such breaks are assumed to cause lateral pipe movement in a direction normal to the pipe axis.

3.6.2.2 Location of Postulated High-Energy Piping Failures

The requirements of the pipe break criteria in a post IEB 79-14 context were reviewed utilizing the guidelines presented in the Standard Review Plan (SRP), section 3.6.2, revision 1, which was in effect at the time of the review. The guidelines presented in SRP section 3.6.2, revision 1, state that, as a result of piping reanalysis, the highest stress locations may be shifted; however, the initially determined intermediate break locations need not be changed unless one of the following conditions existed:

(i) Maximum stress ranges or cumulative usage factors exceed the threshold levels identified in Branch Technical Position (BTP) MEB 3-1, paragraph B.1.c(1)(b) or B.1.c(1)(c).

(ii) A change is required in pipe parameters such as major differences in pipe size, wall thickness, and routing.

(iii) Breaks at the new highest stress locations are significantly apart from the original locations and result in consequences to safety-related systems requiring additional safety protection.

To determine whether intermediate break locations required reanalysis, the guidance provided in items i and ii above were utilized. Item iii would have resulted in a complete reevaluation, unless the break locations had not changed at all, which would have required a massive engineering effort. In addition, IEB 79-14 requirements were primarily invoked to reconcile the as-built systems with the design. It was not the intent of the Bulletin to repostulate the breaks and design the plant for a set of new break locations. Furthermore, the entire concept of postulating two intermediate breaks, even if the stresses in the pipe are below the threshold levels, is totally arbitrary.

It is evident and recognized in BTP MEB 3-1, section A, that pipe breaks are, at best, only a remote possibility, whether at postulated locations or otherwise. In addition, the inservice inspection requirements in effect provide reasonable assurance of the system integrity on a continued basis. Thus, the locations of postulated high-energy piping failures, as presented below, are not revised for each stress calculation revision. A safety impact review and break location changes will be done only for the following cases:

A. The revised pipe stress or cumulative usage factor exceeds the threshold levels.

B. There is a major change in the pipe diameter or routing.

C. Terminal ends have changed.

The 0.1 cumulative usage factor (CUF) criterion in BTP MEB 3-1 represents a screening criterion so that a sufficient number of postulated break locations is developed. The screening criterion of 0.1 CUF is not tied to the plant operating license term as applied to the HNP-2 stress calculations.

3.6.2.2.1 ASME Section III, Class I Piping (Other Than Between Containment Isolation Valves)

For the ASME Code, Section III, Class 1 piping listed in subsection 3.6.1, (except replaced recirculation, RHR, and RWC piping) pipe breaks are postulated to occur at terminal ends and at all intermediate locations throughout a piping system where the following criteria are not met (paragraph 3.6.2.2.1.1):

A. The stress range S_n does not exceed 2.4 S_m.

B. The stress range S_n as calculated by equation 10 of Paragraph NB-3653 exceeds 2.4 S_m but is < 3.0 S_m, and the cumulative usage factor is < 0.1.

C. The stress range S_n exceeds 3.0 S_m, but the stresses computed by equations 12 and 13 of subparagraph NB-3653 are < 2.4 S_m and the usage factor is < 0.1.

Where the stresses calculated for a particular run of piping between terminal ends are everywhere less than the stress limits stated above, so that all intermediate pipe break locations are considered unlikely, a minimum of two locations is chosen based on highest stress and/or usage factor.

At each postulated break location, circumferential breaks are assumed to occur in pipes larger than 1 in., and longitudinal breaks are assumed to occur in pipes 4 in. and larger, except where detailed stress analysis at a particular postulated break location demonstrates that either:

A. The maximum stress is in the longitudinal direction and is a factor of 1.5 higher than the circumferential stress at that point on the cross-section, in which case only a circumferential break is postulated at that location.

B. The maximum stress is in the circumferential direction and is a factor of 1.5 higher than the longitudinal stress at that point on the cross-section, in which case only a longitudinal break is postulated at that location and is oriented around the circumference at the point of maximum stress.

Longitudinal breaks are not postulated at terminal ends if the pipe does not have a longitudinal weld.

Longitudinal breaks are not postulated at intermediate locations where the criterion for a minimum number of break locations must be satisfied.

3.6.2.2.1.1 **Criteria for Break Locations in Recirculation Pipe Replacement**. The replaced recirculation, RHR, and RWC piping were all modeled and dynamically analyzed as a common piping system. The criteria used for postulating break locations are as follows:

- Terminal ends.

- Intermediate locations where the maximum stress range between any two load sets (including zero load set) according to subarticle NB-3600, ASME Code, Section III, 1980 Edition with Addenda through Winter 1981 for Service Levels A and B (including an operating basis earthquake (OBE) event transient as calculated by equation 10 of the Code and either equation 12 or 13) exceeds 2.4 S_m.

- Intermediate location where the cumulative usage factor exceeds 0.1.

- If two or more intermediate locations cannot be determined by either of the two preceding criteria, a total of two intermediate locations, as a minimum, is identified based upon the highest stress calculated by equation 10.

3.6.2.2.1.2 **Criteria for Break Types**. For the replaced recirculation, RHR, and RWC piping, the following criteria have been used to determine break type:

A. Circumferential breaks are assumed at all terminal ends and at intermediate locations identified by the criteria in paragraph 3.6.2.2.1.1.

B. At each of the intermediate postulated break locations identified to exceed the stress and usage factor limits of the criteria in paragraph 3.6.2.2.1.1, either a circumferential or a longitudinal break or both is postulated per the following:

   1. Circumferential breaks are postulated at fitting joints.

   2. Longitudinal breaks are postulated in the center of the fitting at two diametrically opposed points (but not concurrently) located so that the reaction force is perpendicular to the plane of the piping and produces out-of-plane bending.

   3. Consideration shall be given to the occurrence of either a longitudinal or circumferential break. Examination of the stress state in the vicinity of the postulated break location will be used to identify the most probable type of break.

   4. At intermediate locations chosen to satisfy the minimum break location criteria, only the circumferential breaks are postulated.

3.6.2.2.2 ASME Code, Section III, Class 2 and 3 Piping (Other Than Between Containment Isolation Valves)

There is no high-energy Class 2 or 3 piping inside containment. Supplement 15A provides a discussion of high-energy lines outside containment.

3.6.2.2.3 Piping Penetrating Containment

All high-energy piping between containment isolation valves is ASME Code, Section III, Class 1.

Pipe breaks are not postulated in portions of high-energy piping extending from the containment penetration to the first inside and/or outside isolation valve provided the following requirements are met:

A. The following design stress and fatigue limits are not exceeded:

   For ASME Code, Section III, Class I Piping

   1. The stress range S_n does not exceed 2.4 S_m.

2. The stress range S_n as calculated by equation 10 in subparagraph NB-3653 exceeds 2.4 S_m but is < 3.0 S_m, and the cumulative usage factor is < 0.1.

3. The stress range S_n exceeds 3.0 S_m, but the stresses computed by equations 12 and 13 of subparagraph NB-3653 are < 2.4 S_m and the usage factor is < 0.1.

B. The pipe is anchored or restrained at the containment penetration so that the forces and moments associated with failure of piping beyond the outboard isolation valve are not transmitted through the pipe to the containment penetration or the inboard isolation valve; and the forces and moments associated with failure of piping beyond the inboard isolation valve are not transmitted through the pipe to the containment penetration or the outboard isolation valve.

C. The extent of piping run between isolation valves is reduced to the minimum length practical.

D. The design at points of pipe fixity; e.g., pipe anchors or welded connections at containment penetrations, do not require welding directly to the outer surface of the piping; e.g., flued integrally forged pipe fittings are acceptable designs, except where such welds are 100% volumetrically examinable in service to the maximum extent practicable without imposing design changes.

Stress analyses have been completed for ASME Class 1 piping between containment isolation valves. The results of the analyses indicate that there are no Class 1 pipes between containment isolation valves that have calculated stress levels and fatigue usage factors in excess of the limits specified.

3.6.2.3 Moderate-Energy Piping Failures

A. The remaining piping within the containment not listed in subsection 3.6.1 is considered moderate energy. Because the high-energy line breaks within containment are more severe than any crack in a moderate-energy line, cracks are not postulated in this piping.

B. Piping cracks in moderate-energy piping systems outside containment are discussed in supplement 15A.

3.6.3 DESIGN LOADING COMBINATIONS

3.6.3.1 Design of Pipe Whip Restraints

A. Design Loads

The magnitude of loads for the pipe restraint and support steel design is determined by the following formula:

$F = K_1 K_2 PA$ lb

where:

K_1 = thrust multiplication factor for the change in momentum due to a two-phase flow. A value of 2.0 is for cold (nonflashing) water. Unless it can be justified that pipe friction, flow restriction, and capacity of available energy reservoir reduce the thrust coefficient, a magnitude of 1.26 PA will be used for steam-water mixtures or saturated water. Figure 3.6-4 from American Society of Civil Engineers (ASCE) Structural Design of Nuclear Power Plants, Volume 1, 1973, will be used for steady state subcooled water (flashing) blowdown. Feedwater, although subcooled while within its pressure boundary, will flash to a steam-water mixture as a result of a line break. This phenomenon does not immediately occur, resulting in a situation where subcooled water is being expelled from the broken pipe. This initial force, however, is only PA. The steady state condition does not occur until the subcooled water in the line is expelled and the vessel stagnation pressure is reached. The broken-ended pipe toward the vessel experiences a maximum steady thrust of 1.26 PA because of the saturated water source in the vessel. The other end of the severed pipe is subjected to the feedwater pump head. If allowed, the flow and head would increase following a line break until the pump trips on overspeed. A feedwater break within the containment, however, would isolate the main steam lines and cut steam flow to the reactor feed pump turbines. The effect of this is a reduced pump head which results in thrust forces well below those on the vessel side. This reasoning is supported by F. J. Moody, Fluid Reaction and Impingement Loads, Conference on Structural Design of Nuclear Power Plant Facilities, ASCE, Chicago, December 1973 and by BN-TOP-2, Revision 2.

K_2 = dynamic load factor to account for the effects of rapidly applied load. All pipe whip restraints whose dynamic loadings include gap effects use a dynamic load factor (K_2) of 2.0. Design adequacy is then determined by using the energy balance methods specified in BN-TOP-2, Revision 2.

The anchors and restraints at the flued heads which normally contact the pipe are designed in accordance with ASME Section III, Appendix F, and the American Institute of Steel Construction (AISC) and use the ultimate strength of the pipe for design loads. Normal operating loads are also considered in accordance with ASME Section III, but these loadings are minor compared to the rupture loads.

P = operating pressure.

A = pipe internal area, in^2.

B. Design Stress

Restraints and supporting steel within the elastic range are designed in accordance with the AISC code, seventh edition, using a 50% increase in code allowable stresses using forces as described in A on the preceding page. Restraints are designed using American Society of Testing Materials (ASTM) A-36 steel. The design stress limits for main steam line restraints and anchors were limited to the yield point in all cases when a 50% increase in code allowable stresses was used. Since the yield point stress was not exceeded, this approach is more conservative than imposing a maximum design strain limit not exceeding 0.5 of the ultimate uniform strain for ASTM A-36 restraint material.

Restraints which deform plastically are analyzed using the displacement techniques explained in BN-TOP-2, which require the restraint strains to be below 50% of strain of ultimate stress.

The restraints used on the recirculation system are of a low clearance design with a structural restraint frame attached to the supporting structure and high-strength carbon steel wire ropes used to restrain the pipe. The criteria used to determine the adequacy of the restraint load-carrying capacity are as follows:

A. The permanent deformation in the carbon steel restraint frame is limited to a deflection corresponding to 0.5 ultimate uniform strain of the material. This is in compliance with the acceptance criteria.

B. For the high-strength carbon steel wire ropes, the maximum acceptable load was 90% of the load-carrying capacity of the cable in the restraint configuration. This corresponds to a design load limited to 75% of a minimum certified load-carrying capacity of the cable in tension. To demonstrate the adequacy of such cable pipe whip restraints, a comprehensive testing program was undertaken by General Electric (GE). The test results provide sufficient verification and a basis for the use of these design criteria which are reported in detail in sections 4.3 and 5 of reference 1. The overall conclusion that can be drawn from the results of these tests is that there is sufficient conservatism in the design concept of cable-type restraints used in HNP-2 and that these restraints are effective with sufficient margins to meet all the safety design requirements.

Although not as conservative as the acceptance criteria, the estimated margin on energy capacity for the worst case loading event, it is sufficient for the conservative assumptions employed. The direct application of a strain limit for the high-strength carbon steel cables was not employed in the design of this type of restraint. In each case, an evaluation of the potential for rebound has been made based on the actual predicted thrust-time loading for the breaks postulated. These breaks are delineated in HNP-2 stress calculations, as specified in subsection 3.6.4.

For the cable restraints, it has been shown that no rebound gap actually occurs on the 12-in. riser line. This can be attributed to the very high elastic force deflection characteristics of the cables that hold the pipe (and the low stored elastic energy that results due to the steep curve).

For the recirculation suction nozzle break, these restraints have stainless steel bars replacing the original cables, and their force deflection curve is somewhat softer and a rebound gap of 0.184 in. does result. However, reimposition of the load through this clearance does not result in further strain of the restraint material because the pipe is softer and absorbs the energy.

The description of the analytical methods used to evaluate the pipe restraint and blowdown calculations is delineated in reference 1.

3.6.3.2 Jet Impingement Forces

The magnitude of jet impingement forces is determined in accordance with methods indicated in BN-TOP-2, Revision 2. Jet thrust pressures at the target distance are determined by using the Moody development[(2)] for jet expansion, and impingement target effective areas are determined as described in BN-TOP-2, Revision 2. The resulting jet impingement force is then the calculated pressure times the effective target area.

3.6.4 DYNAMIC ANALYSES

Break locations for high-energy piping in the containment are provided in HNP-2 stress calculations which were reviewed and revised (if necessary) as part of the overall pipe stress reanalysis effort performed for NRC Inspection and Enforcement Bulletin (IEB) 79-14. These HNP-2 stress calculations also provide stress intensities and usage factors for the various data points analyzed on the high-energy piping. However, the stress intensities and usage factors are not reviewed for each stress calculation revision. These values will only be updated if the stress calculation revisions define new break locations as described in paragraph 3.6.2.2.

The pipe break criteria given in subsection 3.6.2 are satisfied as described below:

Main Steam Piping

Breakpoints and stress levels for the main steam piping are included in HNP-2 stress calculations. Stress intensities and usage factors are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2. Circumferential breaks have been postulated at

terminal ends, but because the pipe is seamless, longitudinal breaks are not considered. These postulated breaks are at the vessel nozzles and at the anchor outside the primary containment, which is discussed in supplement 15A. Branch lines above 2 in. in diameter have been modeled with the steam lines and dynamically analyzed together. All stress levels are sufficiently low so as not to exceed the specified criteria in paragraph 3.6.2.2.1. Therefore, two intermediate circumferential breaks were chosen, based on the highest stress intensity and/or usage factor. The breaks at the elbows have an opening equivalent to the cross-sectional area of the 24-in. main steam line. The sweepolet breaks are considered circumferential and occur at the base. The effective opening is equivalent to a 15-in.-diameter longitudinal break in the steam line.

Feedwater Piping

The two feedwater loops are symmetrical but located on opposite sides of the containment, and, therefore, the following discussion is limited to one side but is typical for both.

The feedwater penetrates the containment with an 18-in. diameter run which splits into two 12-in. runs. Because all piping has been modeled together in a dynamic analysis and the pipe diameters are reasonably close, the branch point is not considered a terminal end. Therefore, with the 18-in. run being anchored outside the containment, the only terminal ends in the feedwater piping inside the primary containment are at the vessel nozzles. As there are no seam welds in the piping, only circumferential breaks are postulated.

Intermediate circumferential and longitudinal breaks were chosen where the stress intensities calculated by equation 10 exceed 3.0 S_m and the cumulative usage factor exceeds 0.1. These points are indicated in HNP-2 stress calculations, which also provide the stress levels and cumulative usage factors. Stress intensities and usage factors are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2. A minimum of two breaks is postulated on each run from the vessel nozzle to the anchor outside the containment.

There are no breaks between isolation valves; because the cumulative usage factors are below 0.1, and the stress intensities calculated by equations 12 and 13 do not exceed 2.4 S_m.

HPCI Steam

The 10-in. HPCI seamless piping connection to main steam line C is considered a terminal end for the HPCI line, and thus a circumferential break is postulated. The other terminal end is at the anchor outside the containment. However, no break is assumed at this point, because it is between the containment isolation valves with the equation 10 stresses below 2.4 S_m.

Two intermediate breaks were chosen, based on the highest equation 10 stress intensity and cumulative usage factor. Because neither exceeds the specified limits, only circumferential breaks are postulated. These points and the calculated stresses are indicated in HNP-2 stress calculations. Stress intensities and usage factors are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2.

RCIC Steam

The 4-in. RCIC seamless pipe connection to main steam line A is considered a terminal end for the RCIC line, and thus a circumferential break is postulated. As was the case with the HPCI line, the other terminal end is the anchor at the flued head outside the containment. However, no break is assumed at this point because it is between isolation valves with the equation 10 stresses below 2.4 S_m.

Two intermediate breaks were chosen, based upon the highest equation 10 stress intensity and cumulative usage factor. Because neither exceeds the specified limits, only circumferential breaks are postulated. These points and the calculated stresses are indicated in HNP-2 stress calculations. Stress intensities and usage factors are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2.

RHR Return

The two RHR return lines inside the primary containment are symmetrical but located on opposite sides of the containment; therefore, the following discussion is limited to one side but is typical for both.

The RHR system is not used during normal plant operation. However, the drywell piping does experience reactor pressure and, therefore, is considered as high-energy. The 24-in. RHR return header has been dynamically analyzed with the 28-in. recirculation piping; therefore, the branch point is not considered to be a terminal end.

The analysis model terminates at the flued-head anchor outside the containment, but the terminal end and, thus, the circumferential break are assumed at the first normally closed valve, which is the inboard isolation check valve. Although the piping between isolation valves may become pressurized due to valve leakage, a break in this area would be inconsequential since the break is isolated from the energy source by the inboard isolation valve. The piping connected to the inboard isolation valve is seamless, and as a result, only a circumferential break is possible.

HNP-2 stress calculations specify the stress levels for this piping and indicate the postulated breakpoints for the entire model. Stress intensities and usage factors are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2. Because usage factors are below 0.1 and equations 12 and 13 stresses do not exceed 2.4 S_m, a minimum of two intermediate circumferential breaks are chosen points of highest stress. These occur on the RHR piping.

RHR Suction

The RHR suction line is similar to the RHR return lines in that it is analyzed with the recirculation piping and is considered high energy only up to the first normally closed valve, which is the inboard isolation valve.

A circumferential break is postulated at this terminal end. HNP-2 stress calculations specify stress levels for the piping and indicate breakpoints for the entire model. Stress intensities and usage factors are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2. The 20-in. RHR suction pipe was dynamically analyzed with the 28-in. recirculation piping; therefore, the branch point is not considered to be a terminal end. Stress levels do not exceed the specified criteria for postulating breaks, but a minimum of two intermediate circumferential breaks have been chosen at the points of highest stress.

Core Spray

The two CS lines have similar stress levels and identical breakpoints. These points are indicated in HNP-2 stress calculations and consist of the following:

A. A circumferential break at the vessel nozzle was assumed, because it is considered a terminal end. The other terminal end of the run is the first normally closed valve which, as was the case of the RHR, is the inboard isolation check valve. Because the pipe is seamless, only a circumferential break is postulated.

B. The two intermediate breaks on these lines occur at either end of the normally open, hand-operated valve closest to the vessel. Only circumferential breaks are postulated because the stresses calculated by equations 12 and 13 are within 2.4 S_m, and the cumulative usage factors are below 0.1.

Stress intensities and usage factors shown are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2.

RWC

HNP-2 stress calculations provide the equation 10 stresses for the RWC piping at the break locations. Stress intensities and usage factors are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2.

The 6-in. RWC piping was dynamically analyzed with the 20-in. RHR and recirculation piping system. Since the RWC piping is small relative to the 20-in. RHR piping, the branch point (sweepolet connection) is considered to be a terminal end.

The other end of the RWC piping is at the flued-head anchor outside the containment; however, a break point was not considered at this point since the stresses calculated by equation 10 are below 2.4 S_m.

A circumferential intermediate break is postulated at the elbow closest to the RHR and the inlet to the first valve, and a longitudinal break is postulated at the center of the same elbow. These breaks are postulated since the stress ratio calculated by equations 10 and 12 exceeds the 2.4 S_m stress limit.

Main Steam Condensate Drainage

The main steam condensate drainage piping in the drywell consists of 3-, 2-, 1 1/2-, and 1-in.-diameter piping. The 1-in.-diameter piping does not require evaluation in this section, and the others require only circumferential breaks since they are under 4-in. in diameter. The results of the analyses and the postulated breakpoints are summarized in HNP-2 stress calculations. Stress intensities and usage factors are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2.

This piping was analyzed separately from the main steam piping, and there are terminal ends at the connection to the steam lines and HPCI line. There is another terminal end at the first normally closed valve, which is the inboard isolation valve. Circumferential breaks are assumed at all these points. Because the piping was analyzed together and the relative sizes are similar, no terminal ends were assumed at branch points.

All usage factors on the piping are below 0.1, and equation 10 stresses are below 3 S_m. Therefore, only two intermediate circumferential breaks are postulated per run.

SLC, Vessel Drain, Head Vent Equalizing Column, Recirculation Loop Drains, and Pipe to RPV Nozzles N11A and B

The 1 1/2- and 2-in.-diameter lines inside the containment are not restrained, because the energy associated with their whipping is unable to damage any structure or component important to safety, except for other small-bore piping, valve actuators, or electrical conduit. The routing of these lines was reviewed to ensure that they cannot be damaged by jet impingement or whip into any of the small-bore piping, valve actuators, or electrical conduit required for safe shutdown following a small-bore piping break.

The postulated break locations are indicated in HNP-2 stress calculations. Stress intensities and usage factors are not revised for each stress calculation revision. The values will be updated only if the stress calculation revisions define new break locations as explained in paragraph 3.6.2.2.

3.6.4.1 Design Bases for GE Recirculation Loop Pipe Whip Restraints

The restraint design used on this plant is of the type used on a number of GE boiling water reactor (BWR) 4 and BWR 5 product line recirculation systems. The restraint uses a moderately low-clearance design with a frame attached to a support and either high-strength carbon steel wire ropes or stainless steel bars restraining the pipe.

The analytical methods used in the design are not dissimilar to those used on Fermi II and Duane Arnold recirculation piping. They have, however, been upgraded by applying the latest force-deflection data available on wire rope and using GE's preliminary design approval (PDA) code for the dynamic analysis. Load capacities for the restraint frames were developed by using the SAP code (a finite element structural analysis program), and were confirmed by a test series using slowly applied loading methods to determine restraint load-deflection data in the

tangential direction, that is, parallel to the restraint base. (Refer to section 5, reference 1 for test results.) The criteria used to determine the adequacy of the restraint load-carrying capacity are as follows:

A. For the high-strength carbon steel wire ropes, the maximum acceptable load was 90% of the load-carrying capacity of the cable in the restraint configuration. This limit takes into consideration efficiency reduction experienced when a cable is wrapped around a pipe. This means that the design load is limited to 75% of a minimum certified load-carrying capacity of the cable in tension.

B. The permanent deformation in the carbon steel restraint frame is limited to a deflection corresponding to 0.5 ultimate uniform strain of the material. This is in compliance with the acceptance criteria.

   Although not as conservative as the acceptance criteria (the estimated margin on energy capacity for the worst case loading event) it is sufficient for the conservative assumptions employed. The direct application of a strain limit for the high-strength carbon steel cables was not employed in the design of this type of restraint.

   To demonstrate the adequacy of such cable pipe whip restraints, a comprehensive testing program was undertaken by GE. The test results provide sufficient verification and a basis for the use of these design criteria which are reported in detail in sections 4.3 and 5 of reference 1. The overall conclusion that can be drawn from the results of these tests is that there is sufficient conservatism in the design concept of cable-type restraints used in HNP-2 and that these restraints are effective with sufficient margins to meet all the safety design requirements.

Reference 1 delineates restraint loads, configuration, and deflections pertinent to the HNP-2 application.

3.6.4.2 Dynamic Analysis (PDA Code)

An instantaneous circumferential or longitudinal break is the event which initiates the pipe/restraint system response. The instant the break occurs, and before any movement can take place, the broken pipe assumes the configuration shown in figure 3.6-2 (if the break is circumferential), while figure 3.6-3 is a typical configuration if the break is longitudinal. Several elements can be seen in these sketches. In either case the thrust load, F_t, is the forcing function which activates the system response. This load, which can vary with time, acts along a line perpendicular to the break area and is applied at the break.

The circumferential break pipe/restraint system will be described first because it is the simpler of the two. The break area in this system may be at the end of the pipe tail, which can be some distance from an elbow or significant change in direction. The pipe immediately upstream of the elbow is loaded as a cantilever whose point of fixity is the next elbow, the nonpiping component element such as a pump, vessel, or containment penetration. The weight of these pipes is small compared to the thrust load. Therefore, gravitational forces are neglected in the model. However, the inertial effects of these masses to the applied load cannot be neglected.

Therefore, the weight of the pipe tail and any fitting, valve, or other concentrated load which may be located within the tail is treated as a point mass applied to the end of the beam section.

The weight of the beam section is treated as a distributed load which includes the weight located between the point fixity, and the restraint is treated as an additional distributed mass. If the concentrated weight in the beam is between the restraint and the broken end, it is treated in the model as an additional point mass transferred to the end of the beam. The restraint closest to the broken end provides controlled deceleration of the pipe masses. Any other restraint along the beam section of pipe is neglected in the analysis. It may, however, be included as a guide or installed to protect against other potential breaks.

The break shown in figure 3.6-3 is a longitudinal break along the outside bend of the elbow. The model element is generally similar to those of the circumferential break. However, an additional element, the equivalent beam restraint, L_3, can be discerned in figure 3.6-3. This element shares the applied load with the beam element from the instant the break occurs.

The applied load in figure 3.6-3 has two components. The first, F_{BA}, acts parallel to the axis of the equivalent restraint beam and places it in compression. Unless the equivalent restraint beam ends in a true point of fixity, i.e., a vessel, containment penetration, etc., it would load some other combination of beams and equivalent restraint beam. The tail of the case presently being considered becomes one of the beam elements of this new system. The second component, F_{BB}, acts perpendicular to the equivalent restraint beam and to the beam. The pipe tail and beam are similar to the element previously defined for the circumferential break and will not be further discussed. The equivalent restraint beam is treated in the model as a beam spring whose force is directly opposite to the thrust load. However, its mass cannot be neglected. It is, therefore, treated along with any concentrated loads it may contain, and an additional equivalent point mass applied to the end of the beam section. The model makes two additional assumptions:

A. The break region has no bending resistance. Therefore, it acts as a pinned connection.

B. From assumption A, the linear displacement and velocities of the equivalent restraint beam end are equal to the total linear displacement and velocities of the end of the beam.

The model recognized the following pipe modes of response:

A. The first mode is the free movement of the pipe system before it contacts the restraint. In this mode, the energy which is not absorbed as deformation energy of the beam in the circumferential break and of the beam and equivalent beam restraint in the longitudinal break is stored as kinetic energy of the beam system.

B. The instant the pipe hits the restraint the system passes from the first response mode to the second response mode. This is the most complex mathematical model because the multilink response of the system required a Lagrangian transform solution for the acceleration of the various components of the system. In this mode the independent variable is a small time step interval. During this

interval the thrust force, restraint forces, and pipe bending resisting moments are considered constant. The accelerations, velocities, and displacements at the broken end of the pipe are computed. Then the displacement at the restraint is compared to its value during the previous time interval. If the current value is less than its previous value, it is assumed that the restraint has reached its maximum displacement and stopped. Therefore, the third mode of response is analyzed. If the current value of the restraint displacement is greater than its value in the previous time interval, the relative magnitude of the displacement of the end of the pipe is checked. If the current displacement of the free end of the pipe, relative to the bound section, is less than its value in the previous time interval, it is assumed that the free end has reached its maximum relative displacement. Therefore, the fourth mode of response is analyzed.

If the current free beam displacement relative to the bound beam displacement is greater than its previous value, new values of the forces and moments are calculated. Then the process is repeated for the next time interval.

C. In the third mode, the restraint and bound end of the pipe have stopped but the free end is still in motion. In this mode the independent variable is a small displacement step. During each displacement step the forces and moments of the various load elements are computed, then the energy balance is computed and checked to assure that the kinetic energy is positive. If it is positive, the velocities and displacement time interval are calculated and the process is repeated for the next displacement interval. If the kinetic energy is zero or negative, it is assumed that the free end is stopped.

D. In the fourth mode, the motion of the free end of the beam relative to the bound end is zero. The independent variable is a small displacement and the computation sequence is the same as in mode C.

E. The fifth mode, mode E, is the steady state response. The model compares the steady state load to the maximum allowable restraint load. A comparison is also made of the allowable restraint deflection to the actual restraint deflection. If the actual load and deflection are less than the maximum allowable, the requirements have been satisfied.

3.6.4.3 Bechtel-Designed Restraints

Restraints designed not to contact the pipe normally are designed according to the procedures outlined in BN-TOP-2, Revision 2. A circumferential or longitudinal break with a thrust force equal to K_1PA, as explained in paragraph 3.6.3.1, is assumed in the analysis. The thrust force is assumed constant with no rise time and is radically applied to the restraint as a concentrated load. Alternatively, nonlinear dynamic time-history analyses are performed.

Whip restraints at the flued heads are designed according to ASME Section III, Appendix F, and the AISC.

3.6.5 PROTECTIVE MEASURES

3.6.5.1 Pipe Restraints

The locations of pipe whip restraints for systems containing high-energy piping are identified on plant drawings controlled by the HNP configuration control management program. Additional information on whip restraint design in the main steam, feedwater, and reactor recirculation systems are as follows:

A. Reactor Recirculation System

The recirculation system piping loops are restrained against pipe movement in the event of a pipe break. Both circumferential and longitudinal-type pipe breaks are considered in the design of pipe restraints. The pipe breaks are assumed to occur anywhere in the system. The restraints are located and spaced, in accordance with the criteria of reference 1, so as to protect the primary containment pressure boundary, to assure that the design basis accident pipe break area is not exceeded, and to assure sufficient emergency core cooling capability for safe shutdown of the reactor.

B. Main Steam and Feedwater

A feasibility study was made to provide as many pipe restraints as possible on main steam and feedwater lines inside the drywell. The results of this study showed that these lines can be restrained only partially due to space and structural limitations.

Specifically, restraints are provided on the vertical risers of the main steam and feedwater lines where the sacrificial shield wall is available for anchoring of the restraints. These restraints serve to protect the CS injection lines from a rupture of the main steam lines and to protect the containment shell from a rupture of a main steam or feedwater line in this area. A typical restraint for main steam and feedwater is shown on drawing no. H-29026.

3.6.5.2 Protective Barriers

Circumferential pipe breaks at weld joints in all unrestrained pipes inside the drywell larger than 1 in. and forming a part of the reactor coolant pressure boundary were studied and their effects on the drywell were determined. All drywell areas where the broken pipe is postulated to contact the primary containment pressure boundary were then analyzed to determine if the broken pipe had sufficient energy to rupture the primary containment. Areas where the possibility of the primary containment rupture exists are then protected by providing barriers consisting of steel plates welded directly to the drywell.

The primary containment shell and the containment spray headers have been designed to withstand the jet forces resulting from a break in the largest pipe inside the containment.

Stiffened steel barriers mounted on a structural steel frame between redundant divisions of the plant service water (PSW) system pump motors provide protection from jet impingement on the PSW pump motors due to a critical crack in the PSW line.

3.6.5.3 Physical Separation

Essential equipment within the primary containment including components of the engineered safety features has been located so as to mitigate the consequences of blowdown jet forces and pipe whip.

The equipment associated with engineered safety systems such as the CS and the low-pressure coolant injection (LPCI) are segregated in such a manner that the failure of one cannot cause the failure of the other. CS lines enter at the upper cylindrical portion of the drywell whereas the equipment associated with the LPCI is located in the lower spherical portion of the drywell. Also, components of the various redundant engineered safety systems are physically separated so that any single failure in one system will not jeopardize the functioning of the other redundant system. The two CS injection lines are 180 degrees apart. The two LPCI injection lines are 29 ft 4 in. apart at the closest point.

The four nonautomatic depressurization system safety relief valves are designated as low-low set (LLS) valves and are required to function to mitigate the consequences of a small- or intermediate-break accident inside the drywell. Each of the four LLS valves, together with its air supply (pipe, accumulator, check valve, flex hose, etc.) and its power and control cables, constitutes one target. An evaluation assured that no postulated break, which is < 10 in. in diameter, can disable more than one LLS valve.

All the main steam lines are designed Seismic Category I out to the turbine stop valves. As indicated in HNP-2 stress calculations performed in response to NRC IEB 79-14 requirements (subsection 3.6.4), the only postulated breakpoint in that run of main steam piping located in close proximity to the cable spreading room and the switchgear room is breakpoint 888E. As stated in subsection 15A.3.2.D, the only equipment or instrumentation located in the turbine building proper which would obviate the ability to shut down the reactor safely from a main steam line high-energy line failure are cables routed in conduits which are protected by a steel barrier. Due to the configuration of the pipe and the location of the breakpoint (888E), the main steam pipe whip was evaluated and it was determined that the line will not whip into either the wall of the cable spreading room or the switchgear room.

Separation is provided between the safety relief valves and the associated pneumatic supply header and cables on one side of the drywell, the same for safety relief valves on the opposite side of the drywell, and the inboard RHR shutdown cooling valve and associated cables. No high-energy line break smaller than three safety relief valve port areas can damage more than one of the above targets at a time. If a single, active failure disables a second of the above targets, one path still remains available for long-term shutdown cooling of the reactor.

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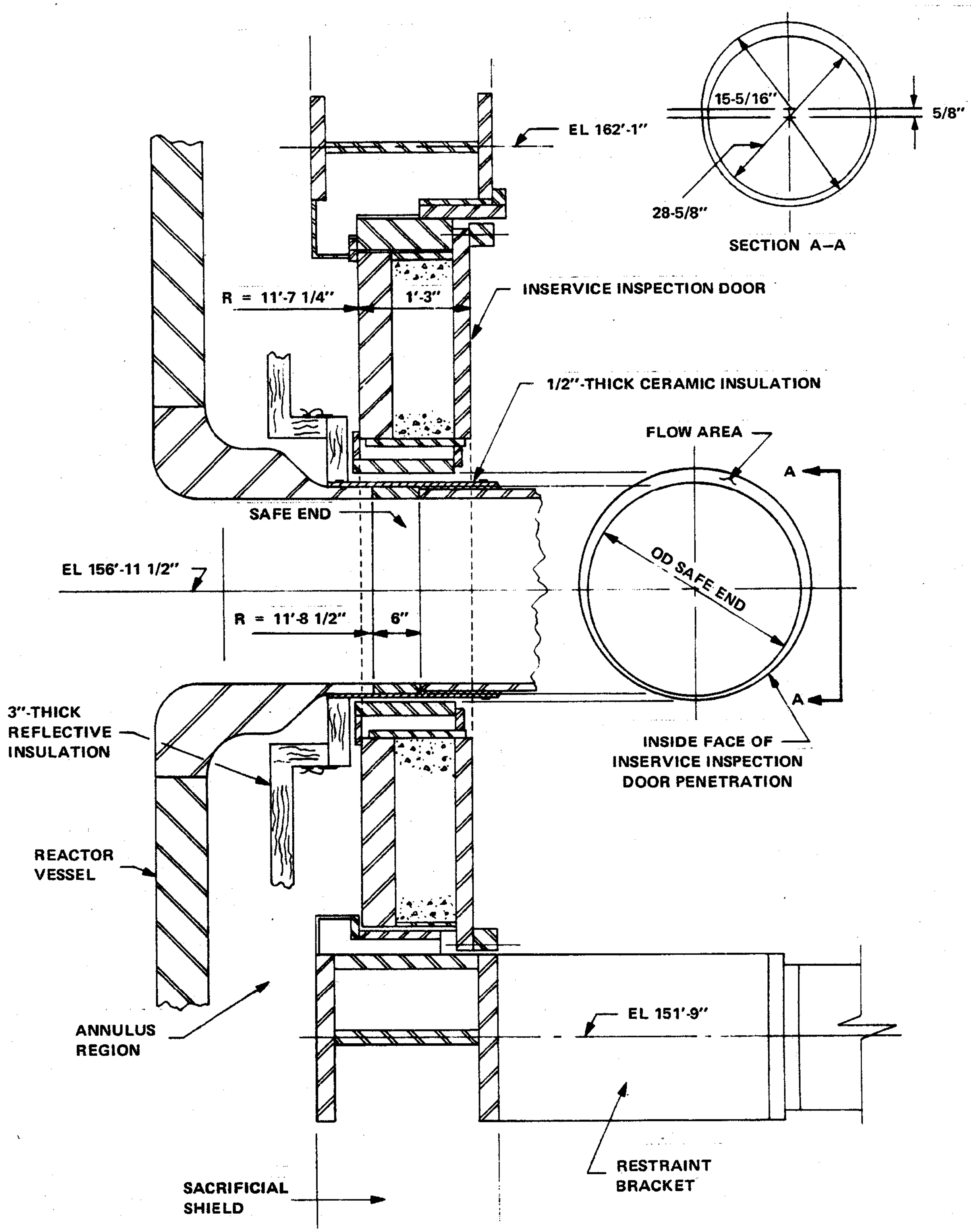
2. Moody, F. J., Fluid Reaction and Impingement Loads, Conference on Structural Design of Nuclear Power Plant Facilities, ASCE, Chicago, December 1973.

3. "Recirculation Piping System Design Report," 25A5746, Revision 0, and 25A5747, Revision 0, General Electric Company.

4. "Recirculation, RHR, and RWC Pipe Break Postulation," 23A1951, Revision 1, General Electric Company, August 17, 1984.



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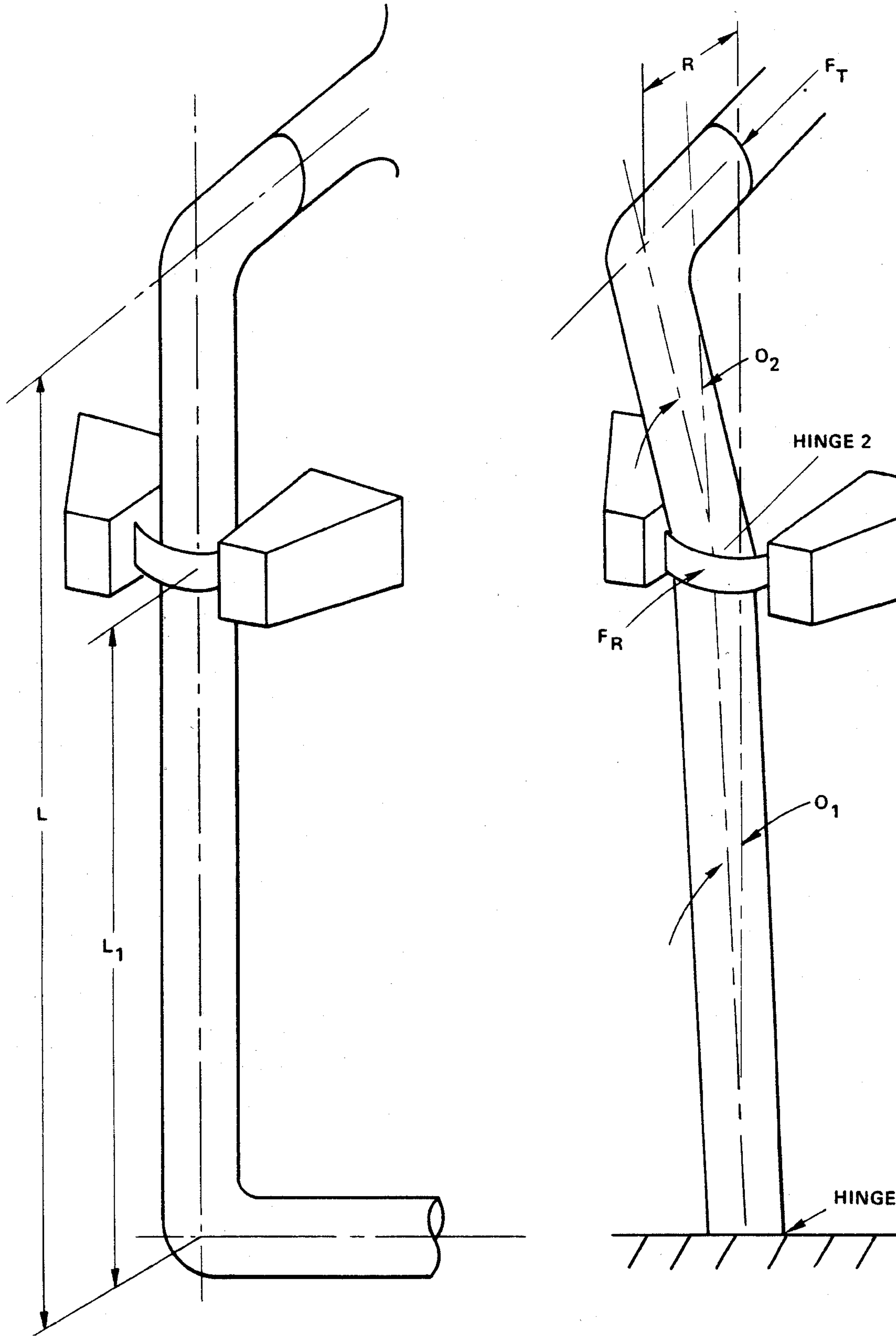
RECIRCULATION OUTLET
NOZZLE CONFIGURATION

FIGURE 3.6-1



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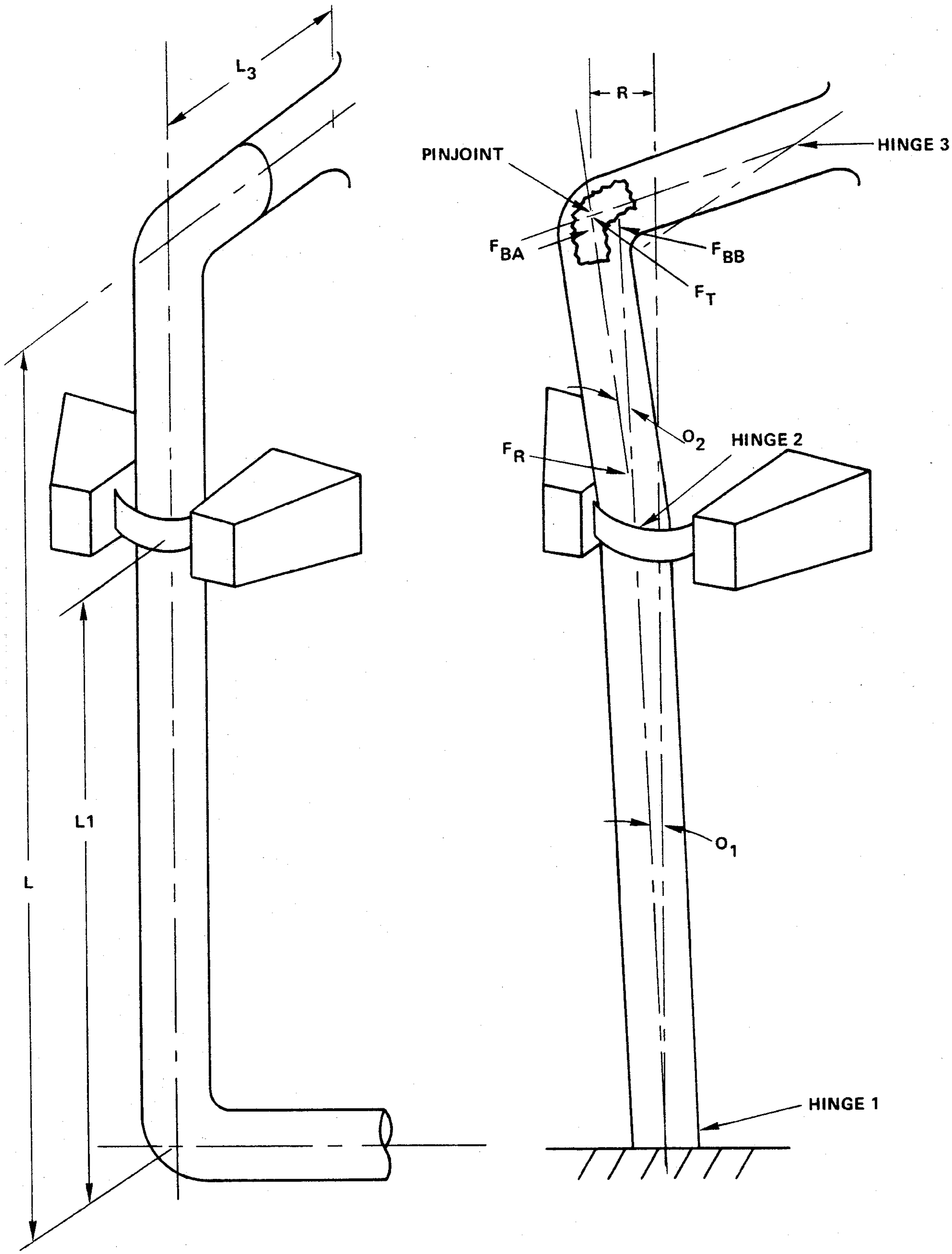
UNIT 2

RECIRCULATION SYSTEM
CIRCUMFERENTIAL BREAK MODEL
FOR GE-DESIGNED RESTRAINTS

FIGURE 3.6-2



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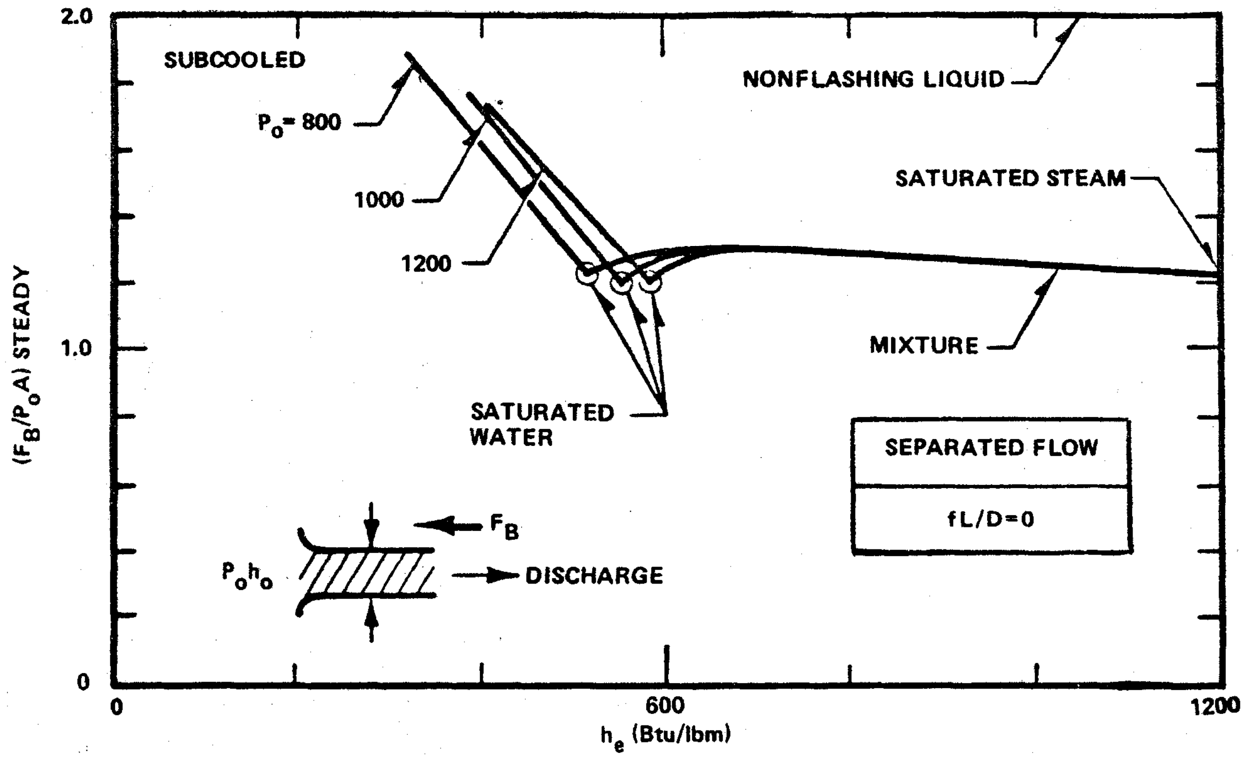
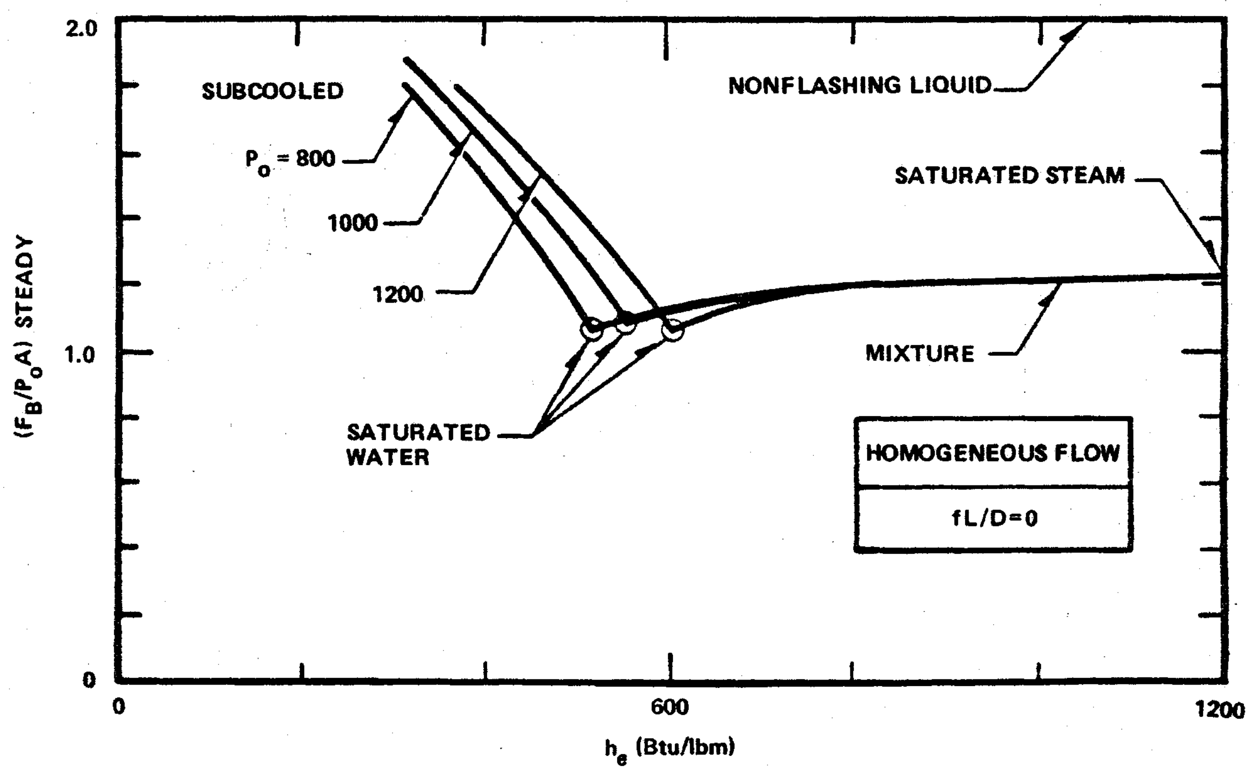
RECIRCULATION SYSTEM
LONGITUDINAL BREAK MODEL
FOR GE-DESIGNED RESTRAINTS

FIGURE 3.6-3





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STEADY BLOWDOWN FORCE

FIGURE 3.6-4

SUPPLEMENT 3.7A

SEISMIC DESIGN

Supplements 3.7A and 3.7B describe the seismic design requirements used to determine the seismic adequacy of mechanical and electrical equipment including cable and conduit raceway systems. The criteria for verifying the seismic adequacy of equipment in a general manner or for specific equipment applications remain valid and may continue to be used as described. However, as an alternative, the methodology based on earthquake experience data developed by the Seismic Qualification Utility Group and documented in the Generic Implementation Procedure (GIP), Revision 2, plus any addition to the GIP reviewed and accepted by the NRC, for resolving Unresolved Safety Issue A-46 in response to NRC Generic Letter 87-02 may be used to verify the seismic adequacy of currently installed equipment after the equipment has been walked down and any outliers resolved, as well as new and replacement mechanical and electrical equipment within the scope of the GIP. This alternative method of verifying the seismic adequacy of equipment used for modifications and replacement equipment assemblies, subassemblies, and devices that are a part of the assemblies is acceptable where no specific NRC commitment to use IEEE 344-1975 was made.

This section describes the seismic design requirements and methods used for Hatch Nuclear Plant-Unit 2 (HNP-2), and the seismic design and analysis of nonnuclear steam supply system equipment. Non-NSSS Seismic Category I equipment installed at HNP-2 was seismically qualified in accordance with either IEEE 344-1971, as amended by this supplement and supplement 3.7A.A, or IEEE 344-1975. In addition, the qualification of some non-NSSS equipment was established using the SQUG criteria discussed above. Seismic design of nuclear steam supply system (NSSS) equipment is described in supplement 3.7B.

In response to Generic Letter (GL) 87-02, Supplement 1, Edwin I. Hatch Nuclear Plant submitted a USI A-46 summary report as a response to the NRC request for information per 10 CFR 50.54(f). The GIP, Revision 2 and the staff's Supplemental Safety Evaluation Report No. 2 for resolution of USI A-46 formed the basis for developing the Plant Hatch response. The NRC's safety evaluation of the Plant Hatch A-46 program, dated September 24, 1998, concluded Plant Hatch had adequately addressed the purpose of the 10 CFR 50.54(f) request for information and had provided sufficient basis to close the USI A-46 review at the facility.

3.7A.1 SEISMIC INPUT

The two types of seismic inputs used in the seismic analyses were the ground design spectra and the associated synthetic accelerogram.

3.7A.1.1 DESIGN RESPONSE SPECTRA

Ground design spectra were established through extensive investigations on the geological conditions of the plant site and past seismological history of the neighborhood areas. The details of these investigations and the resulting recommendations are presented in section 2.5.

The recommendations were given in the form of maximum horizontal acceleration values of the ground, 0.08 g and 0.15 g for operating basis earthquake (OBE) and design basis earthquake (DBE), respectively. The modified Newmark design spectra associated with these acceleration levels were adopted and are shown in figures 3.7A-1 and 3.7A-2. They are characterized by a maximum amplification factor of 3.5 for 2% of critical damping and no amplification for frequencies beyond 30 Hz.

3.7A.1.2 SYNTHETIC TIME HISTORIES

3.7A.1.2.1 <u>Modified TAFT Time History</u>

The synthetic acceleration time history shown in figure 3.7A-3 was developed for use as input to the time history analyses that resulted in the generation of the floor response spectra (FRS) used to seismically qualify subsystems until April 4, 1985.

In developing this synthetic accelerogram, the first 20 s of the TAFT 1952 horizontal earthquake component was selected as the input motion. It was then modified using spectrum suppressing and spectrum raising techniques[1] such that its response spectra enveloped the corresponding design spectra at all but a few frequencies. At the few points where the design spectra were not enveloped, the calculated response spectra were within 10% of the design spectra. Figures 3.7A-4 and 3.7A-5 show comparisons of response spectra for the modified TAFT earthquake time history with the ground design spectra.

The spectra of the time history were computed at the following 71 frequencies (in Hz):

> 0.2 . . . (increment = 0.1 Hz) . . . 3.0, 3.15, 3.3, 3.45, 3.6, 3.8, 4.0, 4.2, 4.4, 4.7, 5.0, 5.25, 5.5, 5.75, 6.0, 6.25, 6.5, 6.75, 7.0, 7.3, 7.6, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5, 12.0, 12.5, 13.0, 13.5, 14.0, 14.5, 15.0, 16.5, 18.0, 20.0, 22.0, 25.0, 28.0, and 33.0.

These frequencies were chosen so that most of the increments do not exceed 5% within the range of 1 to 15 Hz.

3.7A.1.2.2 <u>Synthetic Time Histories (1984)</u>

A review was performed in 1984 to address the FSAR peak-broadening requirements for FRS, and it was concluded that no significant safety issue exists with the subsystems that were seismically qualified using the existing FRS. As a part of the review, two updated (1984) acceleration time histories were developed for use in generating new FRS. The two time histories developed (one for use in OBE analyses and one for use in DBE analyses) are shown in figures 3.7A-18 and 3.7A-19. Figure 3.7A-20 presents a plot comparing the 3% damped spectrum for the OBE time history with the corresponding design spectrum. Similarly, figure 3.7A-21 presents a plot comparing the 5% damped response spectrum for the DBE time history with the corresponding design spectrum. The calculated spectra shown in both figures were computed at the 71 frequencies defined in paragraph 3.7A.1.2.1. Comparison of these two figures with the corresponding figures for the original time history (i.e., figures 3.7A-4 and

3.7A-5) demonstrates that the two updated time histories provide a more realistic representation of the design response spectra than does the original time history discussed in paragraph 3.7A.1.2.1.

The new (1984) FRS were developed during the seismic review to reflect the as-built conditions of the structures and to provide a more realistic representation of the specified seismic design environment. These new spectra, which were developed using the updated time histories in conjunction with the applicable methodology defined in the balance of this section, are used, as of April 4, 1985, to seismically qualify subsystems.

3.7A.1.3 DAMPING VALUES

Energy dissipation in structures is generally represented by equivalent viscous damping. Evaluation of damping coefficients is based on the material, the predicted stress and strain level, and the type of connections used in the structural system. Table 3.7A-1 summarizes the damping values used in the seismic analyses. The values listed for structures, assemblies, and piping were adopted from Newmark's paper.[(2)] As noted in table 3.7A-1, in lieu of using the soil damping values presented in the table, the equations in table 3.7A-2 could be used to calculate the soil damping coefficients. As of April 4, 1985, damping per figures 3.7A-22 and 3.7A-23 for piping systems and cable tray supports, respectively, is used for all new and replacement systems and load reconciliation work.

3.7A.1.4 BASES FOR SITE DEPENDENT ANALYSIS

Site dependent analysis is not used. Subsection 2.5.2 describes the bases for specifying the vibratory ground motion for design use.

3.7A.1.5 SOIL-SUPPORTED SEISMIC CATEGORY I STRUCTURES

Except for the main stack, which is supported on piles, the Seismic Category I structures are supported on soil. The soil underlying the structures extends to a depth of at least 4000 ft before bedrock is encountered.

3.7A.1.6 SOIL-STRUCTURE INTERACTION

The lumped representation and equivalent soil springs and dampers were used to account for soil-structure interaction in the mathematical model for all Seismic Category I structures. The lumped representation is derived from analyzing a model composed of a rigid plate resting on the surface of an elastic half-space. The resulting foundation compliance is frequency dependent but can be approximated by a constant compliance for engineering application.[(3)] The foundation compliance is a function of the mass and dimensions of the foundation mat and the properties of the foundation medium. As a mechanical analog this compliance function can be represented by equivalent springs and dampers. Expressions for the equivalent soil spring

constants used in the seismic analyses are defined in table 3.7A-2. The soil-damping values used are described in subsection 3.7A.1.3.

3.7A.2 SEISMIC SYSTEM ANALYSIS

3.7A.2.1 SEISMIC ANALYSIS METHODS

Response of Seismic Category I structures, systems, and components was determined analytically using the methods described in the following sections. Where the analytical method of analysis cannot ensure the functional integrity of a structure, system, or component, dynamic testing was employed. The procedures of dynamic testing are described in paragraph 3.7A.2.1.2.

3.7A.2.1.1 Modal Superposition

The method of modal superposition was used for the seismic analysis of all Seismic Category I structures, systems, and components. The mathematical model of each of the structures, systems, and components consists of lumped masses and weightless members and was represented by natural frequencies and the associated natural modes. A typical modal equation is given as follows:

$$\ddot{q}_j + 2\beta_j\omega_j\dot{q}_j + \omega_j^2 q_j = -\Gamma_j\ddot{u} \qquad (1)$$

where:

q_j = jth displacement coordinate.

ω_j = jth circular natural frequency.

β_j = jth modal damping ratio.

Γ_j = jth modal participation factor.

$\ddot{u}$ = base motion expressed in terms of acceleration.

For engineering purposes, all those modes with frequencies lower than 33 Hz were considered in the analysis. The mathematical models of the Seismic Category I structures are shown in figures 3.7A-8 through 3.7A-17.

Depending on the form of earthquake inputs and the information required, two different techniques, response spectrum technique and time-history analysis, were engaged in the computation.

A. Response Spectrum Technique

With the input given in terms of design spectra, the modal displacement response is determined by:

$$q_{j,max} = \Gamma_j (SA)_j / \omega_j^2 \quad (2)$$

where:

(SA) = the value of spectral acceleration at the frequency $f_j (f_j = \omega_j / 2\pi)$ and for damping β_i. The displacement response per mode at any mass point, i, is:

$$\chi_{ij\ max} = \phi_{ij} q_{ij,max}$$

where: ϕ_{ij} = modal coordinate

Other structural response quantities per mode, such as shears and moments, can be obtained from $\chi_{ij\ max}$ by making use of the stiffness properties of the structural members.

With the modal responses determined, the total response is computed according to the criterion of "the square root of the sum of the squares (SRSSs) of individual modal responses." When modes are closely spaced, they are first divided into groups such that in each group the deviation in frequency between the first and the last modes does not exceed 10% of the lower frequency. The criterion of "the sum of absolute values" is then applied to each group and the results from all the groups and the remaining modal responses are combined according to the criterion of SRSSs.

B. Time History Analysis

With the input given in the form of an acceleration time-history, the modal responses are evaluated by a step-by-step integration process using equation 1. The total response of interest is then determined by directly superimposing the modal responses in the time domain.

3.7A.2.1.2 <u>Testing Procedures</u>

For certain Seismic Category I equipment and components where dynamic testing was required to demonstrate functional integrity during and after specified seismic conditions, one of the following approaches was used to satisfy the requirements:

A. Performance data of equipment which, under the specified conditions, were subjected to equal or greater dynamic loads than those to be experienced under the specified seismic conditions.

B. Test data from previously tested comparable equipment which, under similar conditions, were subjected to equal or greater dynamic loads than those specified.

C. Actual dynamic testing was in accordance with subsection 3.7A.A.3.2.

D. Alternate test procedures that satisfied the requirements are specified in subsection 3.7A.A.3.2.

3.7A.2.2 NATURAL FREQUENCIES AND RESPONSE LOADS

Table 3.7A-3 presents the first five frequencies for Seismic Category I structures. In addition, the SRSSs response loads for the reactor building are also presented. The mathematical models of the major Seismic Category I structures whose natural frequencies appear in table 3.7A-3 are shown in figures 3.7A-8 through 3.7A-17.

For Seismic Category I structures, the response spectra for different damping values were generated at all mass points in the mathematical model on which the equipment is supported.

3.7A.2.3 PROCEDURES USED TO LUMP MASSES

A structure is modeled as a discrete mass system by lumping the mass of the structure, equipment, and components at various locations of high-mass concentration such as floors and/or locations of Seismic Category I equipment. In general, the weight of any one member together with the loads acting on it were equally lumped at two adjacent points where the member was connected. An equipment, component, or system was usually lumped into the supporting structure mass if its estimated weight was less than one-tenth that of the supporting mass; otherwise, the equipment, component, or system would be itself a mass point. In any case, the number of lumped masses was at least twice the number of the highest mode used in the analysis unless the seismic behavior of the structure was adequately described using a lesser number.

3.7A.2.4 ROCKING AND TRANSLATIONAL RESPONSE SUMMARY

A lumped representation to account for the soil-structure interaction effect was assumed for all Seismic Category I structures and is described in subsection 3.7A.1.6.

3.7A.2.5 METHODS USED TO COUPLE SOIL WITH SEISMIC-SYSTEM STRUCTURES

Finite element analyses were not used for HNP-2.

3.7A.2.6 DEVELOPMENT OF FLOOR RESPONSE SPECTRA

The multi-mass time history method was used to develop the FRS. The spectra were generated at various floors or other locations of concern based on the time history motions obtained from the time history analysis of the structures as described in paragraph 3.7A.2.1.1B. The spectra were calculated at the structural frequencies as well as at additional selected frequencies such that the frequency interval between consecutive frequencies typically did not exceed 10% and in no case exceeded 13% of the lower frequency for the frequency range from 1 to 22 Hz. For example, the 1984 spectra were calculated at the following 124 frequencies (Hz) in addition to the structural frequencies:

> 0.1, 0.15, . . . (increments = 0.05 Hz) . . . 1.0, 1.1, (increments = 0.1 Hz) . . . 10.0, 11.0, 12.0, . . .(increments = 1.0 Hz) . . . 25.0.

3.7A.2.7 DIFFERENTIAL SEISMIC MOVEMENT OF INTERCONNECTED COMPONENTS

The method of analysis discussed in paragraph 3.7A.2.1.1A was used to compute stresses for any interconnected components between floors. The input floor response spectrum is the envelope of the spectra for the floors to which the components are connected.

3.7A.2.8 EFFECTS OF VARIATIONS ON FLOOR RESPONSE SPECTRA

To account for the effect of possible variations in structural frequencies and subsequently the FRS due to the uncertainties in the material properties of the structure and soil, the computed FRS were smoothed, and peaks associated with the structural frequencies were widened by ±10%.

3.7A.2.9 USE OF CONSTANT VERTICAL LOAD FACTORS

No constant vertical load factors were used for Seismic Category I structures. The same method of analysis described in subsection 3.7A.2.1 was also used for the vertical direction. Two-thirds of the horizontal ground spectrum and the horizontal modified accelerogram were used as the minimum vertical input for analysis.

3.7A.2.10 METHODS USED TO ACCOUNT FOR TORSIONAL EFFECTS

For those Seismic Category I structures which are nearly symmetric and have torsional frequencies much higher than the corresponding translational frequencies, the slight eccentricity between the center of mass and rigidity is unlikely to cause any significant effect on the total response. Therefore, the torsional coupling was neglected in the mathematical model of these structures. Static torsional moments were computed, however, to ensure the adequacy of the design.

For those Seismic Category I structures and components which are unsymmetric in nature, including all Class 1 piping systems, torsional coupling was included in the multimass model for computing coupled dynamic response.

3.7A.2.11 COMPARISON OF RESPONSES

Table 3.7A-4 shows the comparison of responses at selected points in the Seismic Category I structures.

3.7A.2.12 METHODS FOR SEISMIC ANALYSIS OF DAMS

Dams were not constructed to impound bodies of water to serve as heat sinks.

3.7A.2.13 METHODS TO DETERMINE SEISMIC CATEGORY I STRUCTURE OVERTURNING MOMENT

The overturning moments of the Seismic Category I structures were calculated by the response spectrum method. The stability of the structures is checked by combining the overturning moment, dead load of the structure, and vertical acceleration. The soil reaction under the containment is obtained by considering the linear stress distribution under a rigid base mat subjected to the worst combined effects of overturning moment, dead load, and vertical acceleration.

3.7A.2.14 ANALYSIS PROCEDURE FOR DAMPING

For structures composed of major subsystems that are made of different materials, the composite modal damping was computed using either the mass proportional, stiffness proportional, modal weighting, or Tsai method. A description of the mass proportional method is illustrated below; the first step involves the formation of the following matrix:

$$[C] = [\phi]^T \left[\diagdown \beta \diagdown \right] [M] [\phi] \tag{3}$$

where:

$[\phi]$ = the modal matrix.

$[M]$ = the mass matrix.

$\left[\diagdown \beta \diagdown \right]$ = a diagonal matrix made up of the damping value specified for the subsystems.

The composite damping is then obtained from [C] by using the diagonal terms after they are divided by the generalized mass of the corresponding mode where the generalized mass is defined by $\overline{M}_j$ as follows:

$$\left[\overline{M}_j \right] = [\phi]^T [M][\phi] \qquad (4)$$

3.7A.3 SEISMIC SUBSYSTEM ANALYSIS

3.7A.3.1 DETERMINATION OF NUMBER OF EARTHQUAKE CYCLES

3.7A.3.1.1 Seismic Category I Structures

The number of maximum amplitude cycles is not a consideration for Seismic Category I structures.

3.7A.3.1.2 Piping and Other Systems and Components

During the 20- to 30-s duration of an earthquake event, strong motion is typically experienced for 4 to 6 s. Frequencies of vibration for which the response is significant are mostly in the range from 1 to 20 Hz with the highest responses occurring within a more narrow range, usually 3 to 8 Hz. One DBE and two OBEs are considered in the design.

The number of cycles for the DBE then can be estimated by multiplying 20 Hz by 6 s by one earthquake which yields 120 cycles. Similarly, the number of cycles for the OBE can be estimated by multiplying 20 Hz by 6 s by two earthquakes which yields 240 cycles. To be conservative, the following total number of loading cycles were used in the design:

- DBE - 300 cycles.
- OBE - 600 cycles.

3.7A.3.2 BASIS FOR SELECTION OF FORCING FREQUENCIES

The methods used to analyze subsystems for dynamic loadings can be either the time history method or the response-spectrum technique. In general, these loadings are in the form of acceleration, velocity, or displacement time histories, or they may be in the form of FRS.

In both of these methods of describing the seismic environment, the structural amplifications are reflected. Therefore, when these loads are used as inputs to the subsystems, each mode responds according to the amplification that was predetermined in the time history analysis of the supporting structure.

It is considered good practice to avoid the regions of load amplification with any system being designed. This is easily identified by observing the frequencies of all predominant modes which lie near the region of spectral amplification; however, it is sometimes found to be impractical or impossible. In these cases, the subsystem is analyzed and designed for the amplified loadings.

3.7A.3.3 ROOT MEAN SQUARE BASIS

The term "root mean square basis" is not used in describing the procedure for the combination of modal responses for HNP-2.

3.7A.3.4 PROCEDURES FOR COMBINING MODAL RESPONSES

The discussion of the procedures for combining modal responses is referred to in paragraph 3.7A.2.1.1A.

3.7A.3.5 SIGNIFICANT DYNAMIC RESPONSE MODES

IEEE 344-1971, as amended by supplement 3.7A.A, and IEEE 344-1975 describe the analysis techniques to be used if the peak of the spectra is used by equipment suppliers. The design and analysis of instrumentation and electrical equipment are described in section 3.10.

3.7A.3.6 DESIGN CRITERIA AND ANALYTICAL PROCEDURES FOR PIPING

Piping systems are anchored and restrained to floors and walls of buildings. The relative seismic displacements between buildings, between floors in buildings, and between major components are applied to the piping, anchors, and restraints in a rational and conservative manner. Seismic movements are always considered to be out of phase between independent structures so that maximum relative displacements are used. The resulting stresses are classified as secondary and are combined with other secondary stresses. The sum of secondary stresses is held within the limits of the applicable piping code.

The seismic inputs to the original OBE and DBE piping systems analyses were defined using the 0.5% and 1.0% damped FRS, respectively. As of April 4, 1985, damping per figure 3.7A-22 is used in response spectrum analyses performed for all new and replacement systems and load reconciliation work. If as a result of using these damping values, piping supports are removed, modified or eliminated, the expected increased piping displacements due to greater piping flexibility will be checked to assure that they can be accommodated and that there will be no adverse interaction with adjacent structures, components, and equipment. The damping criteria established by this figure are consistent with the frequency-dependent approach established by the Pressure Vessel Research Council Technical Committee on Piping Systems.[5]

3.7A.3.7 BASES FOR COMPUTING COMBINED RESPONSE

The basis for combining the modal responses, i.e., displacements, effective inertia forces and accelerations, internal forces and moments, and support reactions, is the SRSS method. To obtain conservative results, the three directional (one vertical and two horizontal) responses obtained by the modal combination of each direction are combined by the SRSSs method, or by the absolute sum of the worst horizontal with the vertical.

Having the total internal moments computed by either of the above procedures, stresses were then calculated and combined with the stresses due to other loadings. The combined stresses are held within the stress limits of the applicable code.

3.7A.3.8 AMPLIFIED SEISMIC RESPONSES

A constant vertical load factor is not used for seismic design of Seismic Category I structures, components, or equipment.

3.7A.3.9 USE OF SIMPLIFIED DYNAMIC ANALYSIS

Simplified dynamic analysis is not used for Seismic Category I structures and is normally applied to field-routed, 2-in. and under piping and some subsystems.

To perform a simplified dynamic analysis on a system, it must have a first mode natural frequency in the rigid range of the response spectrum. The rigid range of the response spectrum curve is defined as that portion in which there is no significant change in spectral acceleration with increasing frequencies. (See point "A" on figure 3.7A-6.) If piping is supported and restrained so that the first mode of vibration occurs in this range, it is classified as rigid.

Rigid piping systems are analyzed with static equivalent loads corresponding to the acceleration in the rigid range of the response spectrum curves for the applicable floor elevations. Both horizontal and vertical static equivalent loads are applied to the rigid piping systems. The response of the component for two horizontal and one vertical direction is combined on a SRSSs basis. The stresses are then computed in accordance with American Society for Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components, including 1971 Winter Addenda. The rigid range is dependent on building response and as such is determined on a case basis. The rigid range of floor spectrum typically begins at ~ 20 Hz.

Classification of a specific piping system may be made in either of the following ways:

A. Restraints are located such that no span between rigid restraints exceeds the length of a simple support beam with a rigid range frequency. In addition, restraints are located at changes in direction, concentrated masses, and extended masses.

B. A dynamic analysis is run to obtain the mode shapes of the piping system. If the first mode frequency is found to be in the rigid range, the system can be assumed rigid.

A summary of typical results comparing the simplified dynamic methods and the response spectrum modal analysis method is contained in Appendix D of BP-TOP-1, Revision 1.[4]

When piping is analyzed by simplified methods all supports and components attached to the piping are required to be in the rigid range so that no amplification of seismic motion exists.

3.7A.3.10 MODAL PERIOD VARIATION

The procedures used to account for modal period variation in models of Seismic Category I structures are discussed in subsection 3.7A.2.8.

3.7A.3.11 TORSIONAL EFFECTS OF ECCENTRIC PIPING

The seismic mass model accounts for the effect of masses that are offset from the pipe centerline. Components with eccentric masses are modeled by placing the component's mass at the component's calculated center of gravity and connecting this mass to the pipe centerline with a rigid connection thereby accounting for its torsional effects.

3.7A.3.12 PIPING OUTSIDE CONTAINMENT

Applicable subsections of sections 3.7A.2 and 3.7A.3 are used for design and analysis of Seismic Category I piping inside and outside containment.

The techniques and criteria used to analyze structural stresses in buried Seismic Category I piping and electrical ducts are presented in supplement 3.7A.B.

3.7A.3.13 INTERACTION OF OTHER PIPING WITH SEISMIC CATEGORY I PIPING

The interface between Seismic Category I piping and non-Category I piping is always an anchor. The anchor is designed to prevent interaction between seismic and nonseismic piping under the most conservative combination of thermal, weight, and seismic loads.

3.7A.3.14 LOCATION OF SUPPORTS AND RESTRAINTS

Seismic supports and restraints for Seismic Category I piping are located so that the stresses, as determined by the dynamic analysis, are less than the appropriate code allowable limits. When rigid seismic supports result in excessive thermal loads on piping or equipment, snubbers or dampers are used.

The pipe support contractors' pipe restraint locations and detailed support drawings are reviewed by pipe stress engineers to ensure that they conform to requirements. In addition, a field inspection of the pipe supports is made by stress engineers to ensure that supports have been installed properly and meet design requirements.

For 2-in. and under Seismic Category I piping, a Bechtel field installation manual is provided so that field engineers can properly design and locate pipe supports and restraints. When the field engineers have completed their designs, they are reviewed by pipe stress engineers.

3.7A.3.15 SEISMIC ANALYSIS FOR FUEL ELEMENTS, CONTROL ROD ASSEMBLIES, AND CONTROL ROD DRIVES

The seismic analysis for fuel elements, control rod assemblies, and control rod drives is discussed in paragraph 3.7B.2.1.6.3.

3.7A.3.16 SEISMIC ANALYSIS OF CABLE TRAY SUPPORTS

Cable tray supports are designed to withstand the calculated seismic loads using the FRS corresponding to the locations where the supports are attached. The simultaneous application of the horizontal and vertical earthquake components, which create the highest stresses, is used to design the cable tray supports. Stresses are limited to the allowables specified in paragraph 3.10.2.1.1.

In the original cable tray support analyses, the applicable damping values were established, based upon the supports' type of construction, using the values specified in table 3.7A-1. As of April 4, 1985, damping per figure 3.7A-23 is used for all new and replacement systems and load reconciliation work. The damping criteria specified in figure 3.7A-23 provide a conservative estimate of damping for cable tray supports based upon a test program.(6) As an alternative, the Seismic Qualification Utility Group (SQUG) Generic Implementation Procedure (GIP) criteria, discussed in the beginning of this supplement, may also be applied to existing, new, and replacement cable and conduit raceway systems.

3.7A.4 <u>SEISMIC INSTRUMENTATION PROGRAM</u>

3.7A.4.1 COMPARISON WITH NUCLEAR REGULATORY COMMISSION GUIDE 1.12

The seismic instrumentation program complies with the requirements of Regulatory Guide 1.12 (April 1974), with the following additional explanations:

A. Response spectrum recorders at locations other than the containment foundation, which are required by subsection C.1.c of the Guide, are not supplied as discrete instruments. Data from these instruments is not required immediately following the earthquake and is used only in the subsequent post-earthquake

analysis. Time-history strong-motion accelerometers are provided at the locations specified for response-spectrum recorders.

B. Ranges, set points, damping values, and recording times for the instruments are based on the site seismicity and estimated structural response, and satisfy the guide as far as commercial availability permits.

3.7A.4.2 LOCATION AND DESCRIPTION OF INSTRUMENTATION

The following instrumentation is used to measure plant response to earthquake motion:

A. Four strong-motion triaxial time-history accelerographs are installed at appropriate locations to provide data on the frequency, amplitude, and phase relationship of the seismic response of the reactor building structure and to provide data on the seismic input to other Seismic Category I structures, systems, and components.

These accelerographs are activated on a common time base by a seismic trigger, which is shared with HNP-1, located in the free field. They are rigidly mounted and located so that they are accessible for servicing. The output from these accelerograph sensors are recorded on magnetic tape which are available for playback following an earthquake. These records are the primary means of determining the severity of any earthquake which may be experienced. A visual signal in the control room alerts the operator that a recording has been made.

1. One strong-motion accelerograph is installed in the switchyard to measure the free-field-ground acceleration. The accelerograph is used for HNP-1 also.

2. Two strong-motion accelerographs are located in the reactor building: one on the east side of the reactor building drywell pedestal at el 87 ft and the other inside the drywell on the feedwater discharge line to the reactor pressure vessel (RPV). These two strong-motion accelerographs in the reactor building are oriented so that the three axes of the sensors on one accelerograph are pointing in the same directions as the three axes of the other accelerograph. These provide data on the frequency, amplitude, and phase relationship of the seismic response of the containment structure.

3. One strong-motion accelerograph is installed in the diesel generator building at el 130 ft. This accelerograph is used for HNP-1 also.

B. A magnetic tape recorder and playback system is located in the main control room and receives all the output signals from the strong-motion accelerograph from both HNP-1 and HNP-2.

C. Two seismic switches are provided with a strong-motion accelerograph as described in A.2 above. The seismic switches are set to actuate at the OBE level of estimated response. These two switches annunciate in the HNP-2 main control room (MCR).

D. Peak recording accelerometers are provided to record the actual peak response of the locations listed below. (All except items 5, 6, and 7 are shared with HNP-1.)

1. Diesel generator base support.
2. Intake structure.
3. Control building floor el 112 ft.
4. Control building - main control room floor.
5. Reactor building refueling floor.
6. Inside biological shield on reactor pedestal.
7. Reactor piping - feedwater discharge line to the RPV.

E. Triaxial peak acceleration recorders are installed inside the drywell in the HNP-2 reactor building. The locations of the peak acceleration recorders are provided at known points of amplified response which permit an evaluation of the actual amplification factors to the design values.

The triaxial acceleration recorder consists of metal plates on which scratch lengths are mechanically etched in the three axes by diamond styli as the component or structure moves. The zero reference line is established when the plates are inserted and removed. When the component or structure experiences any movement, a record of the displacement is scratched permanently on the metal plate. Normally, the maximum displacement from the zero line, regardless of direction, is recorded. The length etched on each plate is measured by placing it under a calibrated microscope. Each scratch length (displacement) is then multiplied by its acceleration sensitivity (a constant supplied by the vendor) to convert it to acceleration.

F. One triaxial peak acceleration spectrum recording unit is provided on the HNP-1 containment foundation for measuring both horizontal motions and the vertical motion. This recording unit is used also for HNP-2 since the seismic response for HNP-2 is essentially the same as that for HNP-1. This instrument is electrically connected to an annunciator unit in the main control room to provide an alarm indicating that the specified preset response acceleration has been exceeded.

3.7A.4.3 MCR OPERATOR NOTIFICATION

The strong-motion accelerographs record their signals on magnetic tape in the MCR. An audio/visual signal in the MCR alerts the operator that the recording system is in operation.

Immediately after the seismic occurrence, the output data from the sensors is processed and played back graphically in a way that makes possible a comparison with calculated design spectra of Seismic Category I structures and other components.

The two seismic switches in the reactor building are electrically connected to the annunciator for immediate indication that specific preset response accelerations have been exceeded.

MCR indication provides immediate information which could provide a basis for plant shutdown if an OBE should be exceeded. It also provides a permanent record of data for analysis of design parameters.

3.7A.4.4 COMPARISON OF MEASURED AND PREDICTED RESPONSES

Plant operators are provided with seismic criteria and procedures to follow after a seismic event to determine whether the plant can continue to operate or if it must be shut down. An outline of the order of actions to be taken after a seismic event is provided in figure 3.7A-7. Evaluations of whether the ground motion has exceeded the OBE input are based on recordings from strong motion accelerographs described in subsection 3.7A.4.2.

3.7A.5 SEISMIC DESIGN CONTROL

The primary design organizations involved in the seismic design of the various structures, systems, and components are General Electric Company, Bechtel Power Corporation, and Southern Company Services, Inc.

Components designed by others which fall under one of the three primary areas of responsibility are designed to the overall seismic requirements and checked by one of these organizations.

The original seismic design responsibilities are summarized below:

A. General Electric Company

The General Electric Company was responsible for design of the NSSS. This includes the reactor vessel, recirculation system, main steam line piping up to the second isolation valve, and equipment for safety-related systems.

B. Bechtel Power Corporation

The Bechtel Power Corporation was responsible for design of the containment, reactor building, main stack, parts of the turbine building, radwaste building, and associated piping for safety-related systems. In addition, Bechtel had

responsibility for reviewing seismic designs originated by Southern Company Services, Inc.

C. Southern Company Services, Inc.

Southern Company Services, Inc. had responsibility for basic design criteria and detail design responsibility for the control and diesel generator buildings and intake structure.

Subsequent to commercial operation, the responsibility for design control was changed. Primary design responsibility was assigned to Southern Company Services, Inc. Bechtel Power Corporation and General Electric Company perform designs as directed by Southern Company Services, Inc. and Georgia Power Company. Effective March 22, 1997, SNC is the exclusive operating licensee and has accepted the assignment of the primary design responsibility to Southern Company Services, Inc.

3.7A.5.1 GENERAL ELECTRIC-SUPPLIED EQUIPMENT AND COMPONENTS

For General Electric-supplied equipment and components, see section 3.7B.5.

3.7A.5.2 BECHTEL POWER CORPORATION SPECIFIED EQUIPMENT AND COMPONENTS

Safety-related systems, structures, and components are seismically designed and checked using design control measures within the organization. Although not specifically identified as seismic quality assurance provisions, the quality assurance provisions of chapter 17 ensure the adequacy of Seismic Category I components to perform their intended functions during and after seismic disturbance.

Equipment and Component Specifications

Specifications for Seismic Category I equipment incorporate a section on seismic design criteria. This section includes seismic-response spectra, generated by a time history, which were developed for the particular equipment location and a list of damping factors. This specification requires one of the following:

- Perform a seismic analysis based on the appropriate damping factor and response spectrum as well as the natural frequency of the equipment.

- If it is not practical to calculate the natural frequency of the equipment, use the maximum acceleration of the spectrum curve for the seismic analysis.

- Subject prototype equipment to a test demonstrating its ability to perform its intended function during and after seismic disturbance.

Certification that this equipment functions during and after seismic disturbance is required from each vendor. This certification may consist of calculations checked by an engineer knowledgeable in the design of such equipment or it may consist of a written certification acknowledging the equipment has successfully passed tests of forces equal to or higher than those stated in the seismic requirement and has been exposed to these severe vibration requirements. The method of analysis, calculation, or testing is reviewed and approved by the responsible engineer.

Inspection plans require that a certification of calculations be delivered with each Seismic Category I component. This is required on the applicable documentation checklist which is included in the inspection plan approved by engineering. Release for shipment is not provided without all documentation being provided.

3.7A.5.3 SOUTHERN COMPANY SERVICES, INC., SPECIFIED EQUIPMENT AND COMPONENTS

Equipment and Component Specifications

Seismic-response spectra for each location in the plant is developed for use by the design engineer. The design engineer is responsible for including the appropriate seismic-response spectra in the equipment purchase specification in a form that is meaningful to the vendor. All purchase specifications are reviewed by engineers competent in seismic analysis and testing for verification acknowledging complete and correct seismic requirements have been included.

Vendor analyses and/or test data are submitted to the responsible design engineer as agreed upon as part of the purchase specification. The responsible design engineer agrees with the submitted material in writing only after he is satisfied that it meets the design specification requirements. Guidance and counsel of engineers competent in the applicable discipline are made available to the responsible design engineer in the course of such reviews.

The quality assurance program is described in chapter 17 and provides a description of the review and approval of purchase specifications and vendor documents by competent engineering personnel.

REFERENCES

1. Tsai, N.C., "Spectrum - Compatible Motions for Design Purposes," Journal of Engineering Mechanics Division, ASCE, Vol. 98, No. EM2, April 1972.

2. Newmark, N.M., Design Criteria for Nuclear Reactors Subjected to Earthquake Hazards, Proc. I AEA Panel on Aseismic Design and Testing of Nuclear Facilities, Japan Earthquake Engineering Promotion Society, Tokyo, Japan, 1967.

3. Richart, Jr., F.E., Hall, Jr., J.R., and Woods, R.D., Vibrations of Soil and Foundations, Prentice Hall, Inc., New Jersey, 1970.

4. "Seismic Analysis Piping System," BP-TOP-1, Revision 1, February 1974.

5. "Welding Research Council Bulletin Number 300: Technical Position on Criteria Establishment; Technical Position on Damping Values for Piping-Interim Summary Report; Technical Position on Response Spectra Broadening; and Technical Position on Industry Practice," 1984.

6. Cable Tray and Conduit Raceway Seismic Test Program, Release 4, Report 1053-21.1-4, ANCO Engineers, Inc., December 15, 1978.

TABLE 3.7A-1

DAMPING FACTORS FOR SEISMIC ANALYSIS IN PERCENT OF CRITICAL DAMPING[a]

	OBE	DBE
Reinforced concrete structures	3.0	5.0
Steel frame structures	3.0	5.0
Bolted and riveted assemblies	3.0	5.0
Welded assemblies	2.0	3.0
Vital piping	0.5	1.0
Translation and rotation of foundation soil[b]	4.0	5.0

a. As of April 4, 1985, damping per figures 3.7A-22 and 3.7A-23 for piping systems and cable tray supports, respectively, is used for all new and replacement systems and load reconciliation work.
b. In lieu of using the soil damping values specified in this table, the equations in table 3.7A-2 may be used to calculate soil damping coefficients.

TABLE 3.7A-2 (SHEET 1 OF 2)

FORMULAS FOR EQUIVALENT FOUNDATION SPRING CONSTANTS AND DAMPING COEFFICIENTS (RECTANGULAR BASE)

Motion	Equivalent Spring Constant	Equivalent Damping Coefficient
Horizontal	$k'_x = 2(1+\nu)\,G\beta_x\,\sqrt{BL}$	$c'_x = 0.576\,k_x R\,\sqrt{\rho/G}$
Rocking	$k'_\psi = \dfrac{G}{1-\nu}\,\beta_\psi B^2 L$	$c'_\psi = \dfrac{0.30}{1+B_\psi}\,k_\psi R\,\sqrt{\rho/G}$
Vertical	$k'_x = \dfrac{G}{1-\nu}\,\beta_z\,\sqrt{BL}$	$c'_x = 0.85\,k_z\,R\,\sqrt{\rho/G}$

where:

B = width of the base mat in the plane of horizontal excitation.

L = length of the base mat perpendicular to the plane of horizontal excitation.

ν = Poisson's ratio of foundation medium.

G = shear modulus of foundation medium.

ρ = density of foundation medium.

R = equivalent radius of the base mat as defined below.

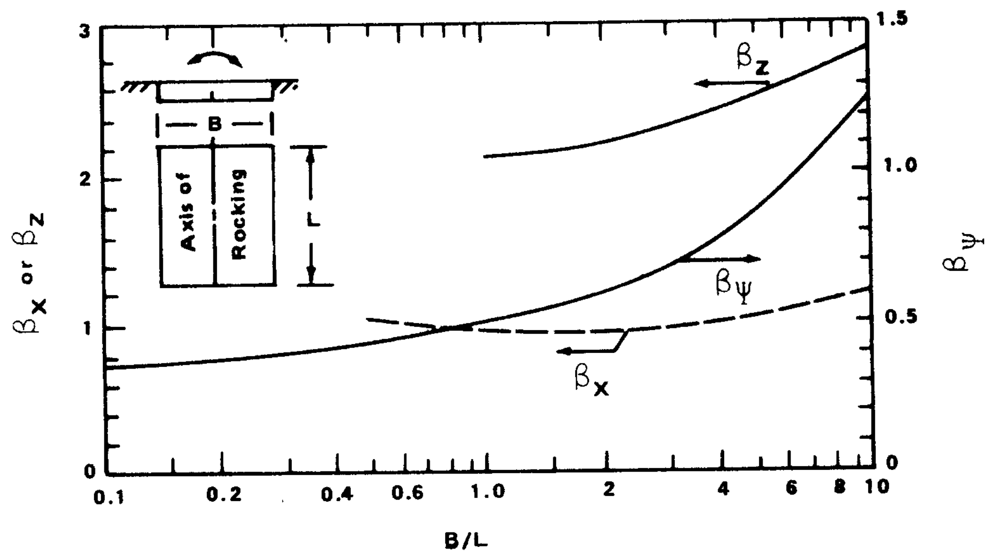
β_x, β_ψ, β_z = constants that are functions of the dimensional ratio, B/L (from figure 10-16 in reference 3).

$$B_\psi = \frac{3\,(1-\nu)\,I_O}{8\rho R^5}$$

where:

I_O = total mass moment of inertia of structure and base mat about the rocking axis at the base.

TABLE 3.7A-2 (SHEET 2 OF 2)



Constants β_x, β_ψ, and β_z for Rectangular Bases

(EQUIVALENT RADIUS FOR RECTANGULAR BASE)

For a rectangular base having a dimension of B x L (B = width of base in the plane of horizontal vibration), the equivalent radius, R, is taken to be the smallest of parameters, R_x, R_ψ, and R_z, defined below:

$$R_x = \frac{(1+\nu)(7-8\nu)\beta_x\sqrt{BL}}{16(1-\nu)}$$

$$R_\psi = \sqrt[3]{3\beta_\psi B^2 L/8}$$

$$R_z = \beta_z\sqrt{BL/4}$$

TABLE 3.7A-3 (SHEET 1 OF 2)

SUMMARY OF FREQUENCY AND RESPONSE LOADS

	Reactor Bldg		Control Bldg		Diesel Generator Bldg[a]		Intake Structure		Main Stack	
	E-W (Hz)	Vert (Hz)	E-W (Hz)	Vert (Hz)	E-W (Hz)	Vert (Hz)	E-W (Hz)	Vert (Hz)	E-W (Hz)	Vert (Hz)
Freq No. 1	1.61	6.45	1.01	2.37	4.12	4.59	7.04	14.60	0.60	8.64
Freq No. 2	3.73	20.41	5.38	9.44	7.76	83.25	21.13	66.27	2.24	18.02
Freq No. 3	8.37	26.26	7.00	13.71	36.20	NA	35.32	106.73	4.88	24.60
Freq No. 4	9.39	29.00	11.07	37.87	NA	NA	44.41	136.26	8.14	34.74
Freq No. 5	9.74	NA	15.27	49.11	NA	NA	53.74	178.87	11.66	43.98

a. The diesel generator building natural frequencies specified are those associated with the mean soil properties for this building.

TABLE 3.7A-3 (SHEET 2 OF 2)

REACTOR BUILDING SRSSs RESPONSES[(a)]

Elevation	87 ft		130 ft		158 ft		185 ft	
	E-W DBE	Vert DBE	E-W DBE	Vert DBE	E-W DBE	Vert DBE	E-W DBE	Vert DBE
Accel (g)	0.15	0.10	0.19	0.10	0.26	0.11	0.33	0.12
Disp (ft 10^{-4})	48.9	17.4	99.5	19.3	146.6	21.4	190.8	22.6
Force (kips 10^3)	42.1	16.4	40.3	15.7	34.2	12.2	27.5	9.2
Moment (K-ft 10^4)	414.6	NA	228.7	NA	133.5	NA	60.1	NA

Elevation	203 ft		228 ft		256 ft		280 ft	
	E-W DBE	Vert DBE	E-W DBE	Vert DBE	E-W DBE	Vert DBE	E-W DBE	Vert DBE
Accel (g)	0.38	0.12	0.45	0.12	0.60	0.14	0.67	0.14
Disp (ft 10^{-4})	217.2	23.1	253.6	23.5	621.4	25.0	1692.3	25.4
Force (kips 10^3)	19.1	6.2	10.2	3.4	2.0	0.7	1.4	0.3
Moment (K-ft 10^4)	26.9	NA	7.6	NA	3.3	NA	0.0	NA

a. These responses were not updated to reflect the 1984 analysis discussed in paragraph 3.7A.1.2.2.

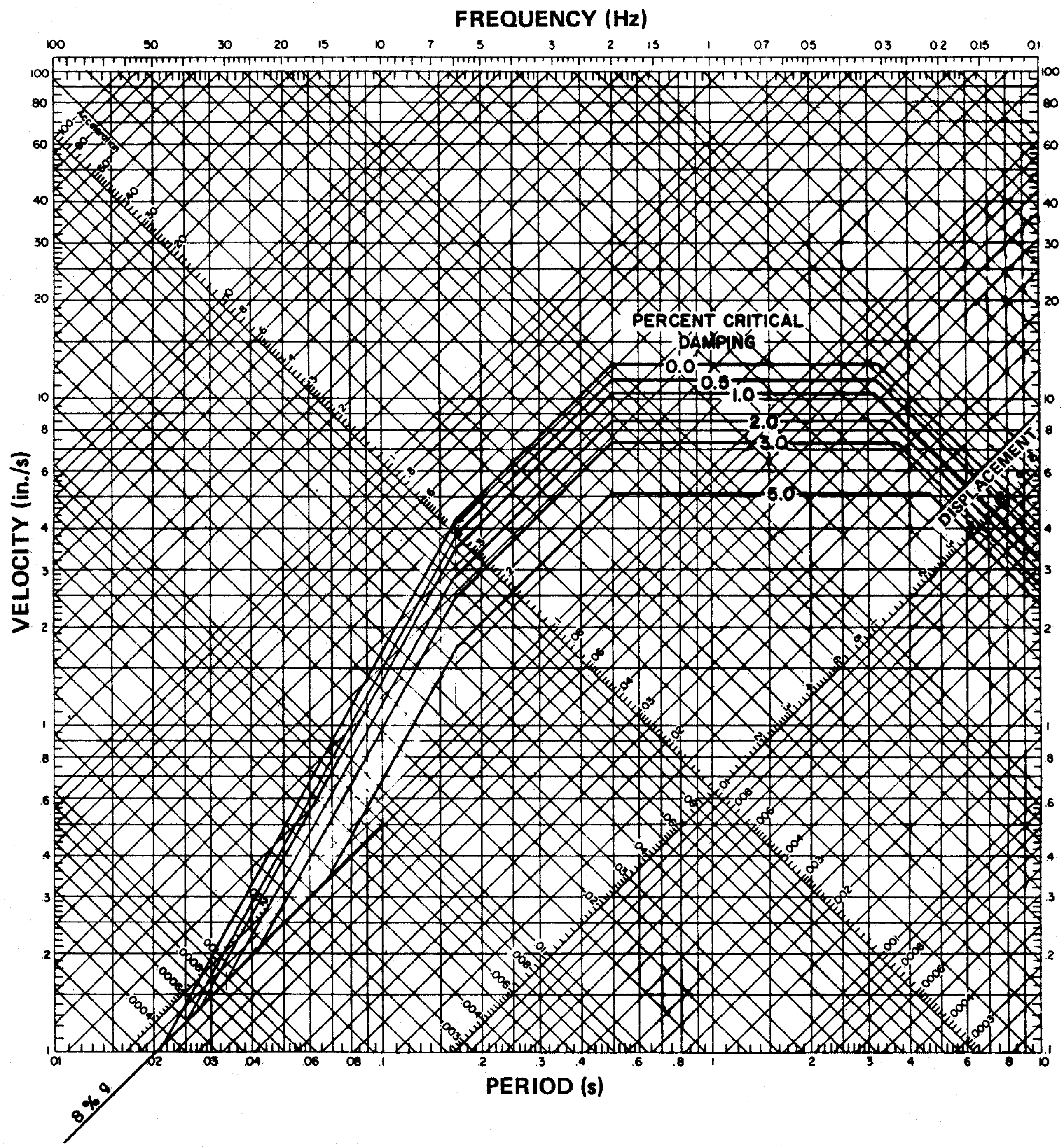
TABLE 3.7A-4

COMPARISON OF RESPONSES[a]

	Acceleration (g)				Displacement (ft 10^{-4})			
Mass	E-W		Vert		E-W		Vert	
Point	SRSS	TH	SRSS	TH	SRSS	TH	SRSS	TH
				Reactor Building (OBE)				
1	0.08	0.09	0.05	0.06	26.6	29.3	9.4	8.8
2	0.10	0.12	0.05	0.06	54.3	56.0	10.5	9.8
3	0.14	0.15	0.06	0.06	80.0	80.2	11.6	10.7
4	0.18	0.18	0.06	0.06	104.2	101.9	12.2	11.2
5	0.21	0.19	0.06	0.06	118.6	115.9	12.5	11.5
				Control Building (OBE)				
1	0.11	0.13	0.08	0.10	16.9	20.0	7.2	9.0
2	0.16	0.19	0.09	0.11	26.8	30.0	8.1	10.0
3	0.20	0.22	0.09	0.12	32.8	35.0	8.7	11.0
4	0.23	0.24	0.10	0.12	37.6	40.0	9.1	11.0
5	0.19	0.19	0.18	0.21	721.3	770.0	15.7	19.0
				Diesel Generator Building (OBE)[b]				
1	0.20	0.23	0.13	0.15	94.6	110.0	49.9	58.1
2	0.21	0.24	0.13	0.16	100.7	117.0	50.2	58.4
				Intake Structure (OBE)				
1	0.06	0.09	0.06	0.08	7.0	7.0	2.0	3.0
2	0.10	0.12	0.06	0.08	15.0	16.0	2.0	3.0
3	0.16	0.17	0.07	0.09	25.0	26.0	2.0	3.0
4	0.23	0.23	0.07	0.09	36.0	38.0	3.0	3.0
5	0.29	0.29	0.07	0.09	44.0	46.0	3.0	3.0

a. These responses were not updated to reflect the 1984 analysis discussed in paragraph 3.7A.1.2.2

b. Responses are those associated with the mean soil properties for this building.



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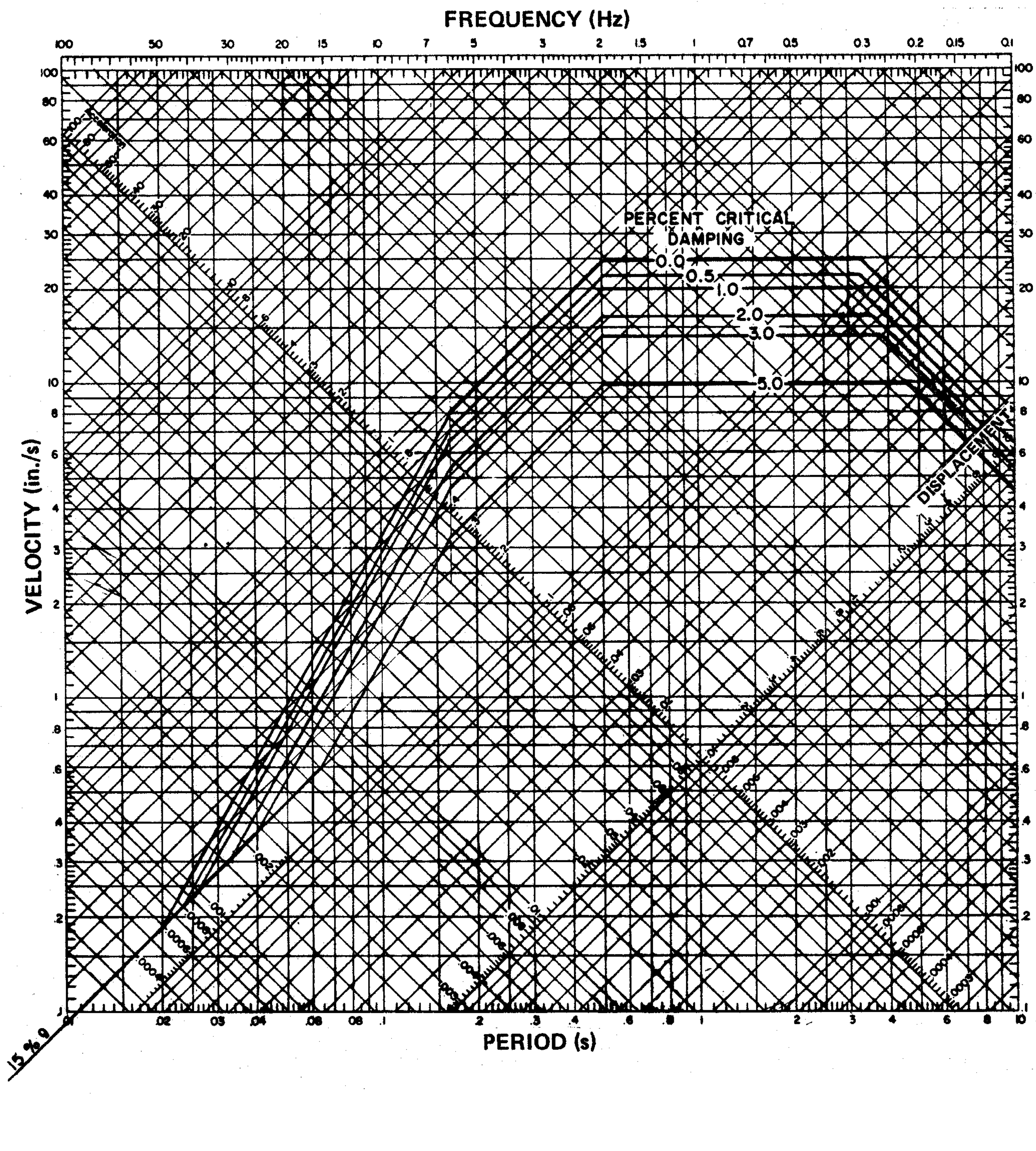
DESIGN SPECTRUM FOR OBE

FIGURE 3.7A-1



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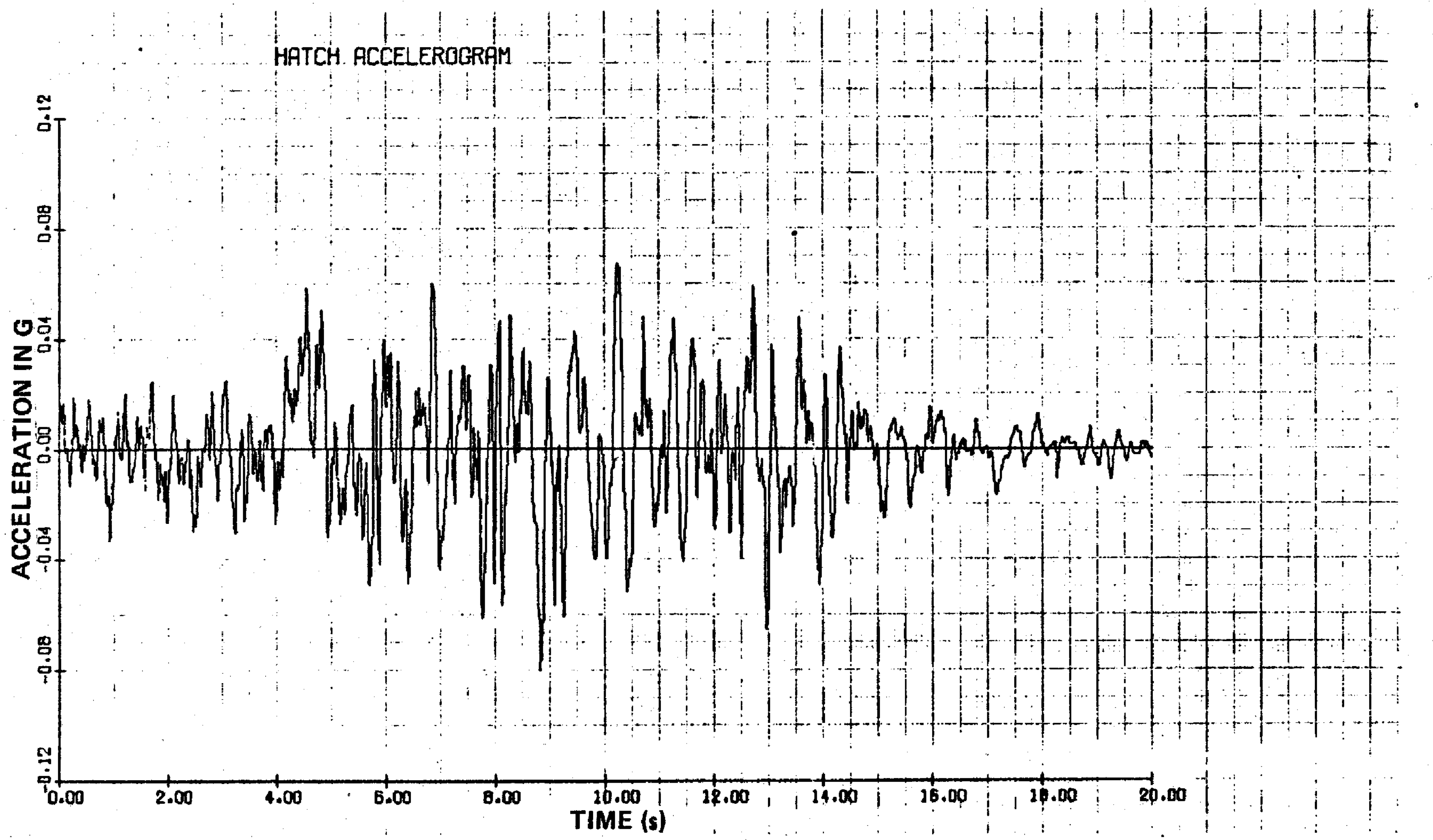
DESIGN SPECTRUM FOR DBE

FIGURE 3.7A-2



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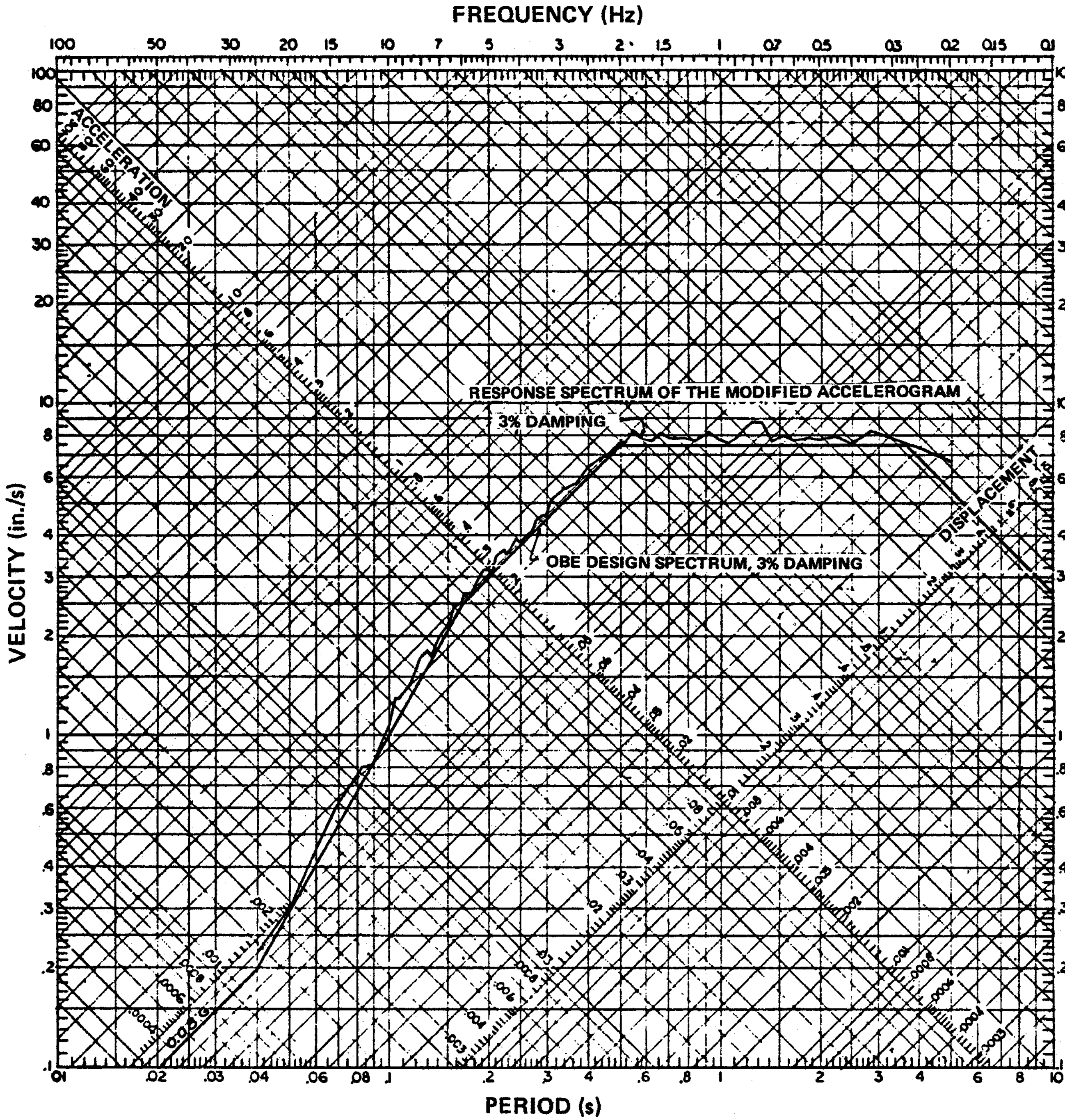
MODIFIED ACCELEROGRAM

FIGURE 3.7A-3



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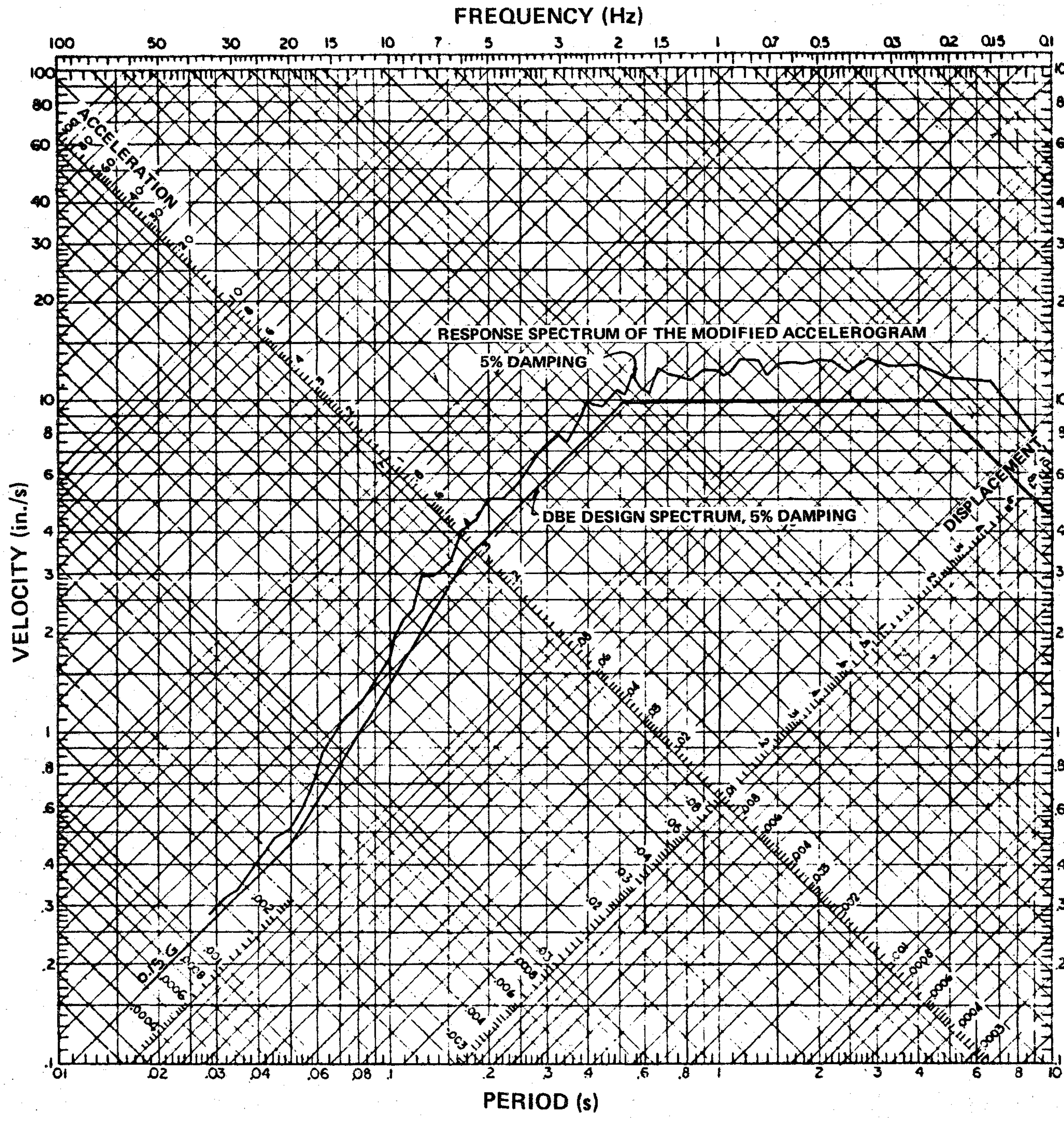
DESIGN SPECTRUM COMPARED WITH RESPONSE SPECTRUM OF MODIFIED ACCELEROGRAM FOR OBE WITH 3% DAMPING

FIGURE 3.7A-4



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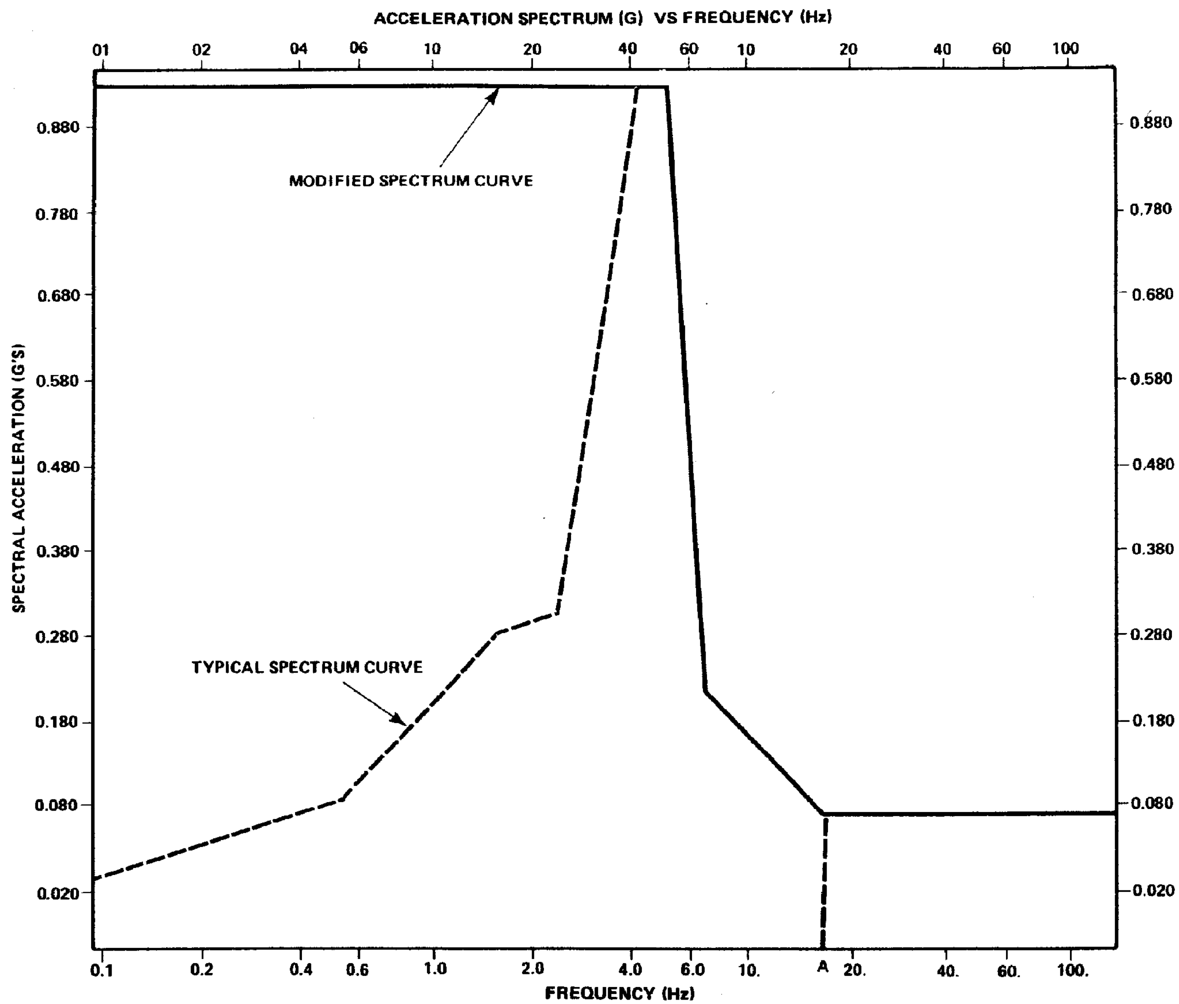
DESIGN SPECTRUM COMPARED WITH RESPONSE SPECTRUM OF MODIFIED ACCELEROGRAM FOR DBE WITH 5% DAMPING

FIGURE 3.7A-5



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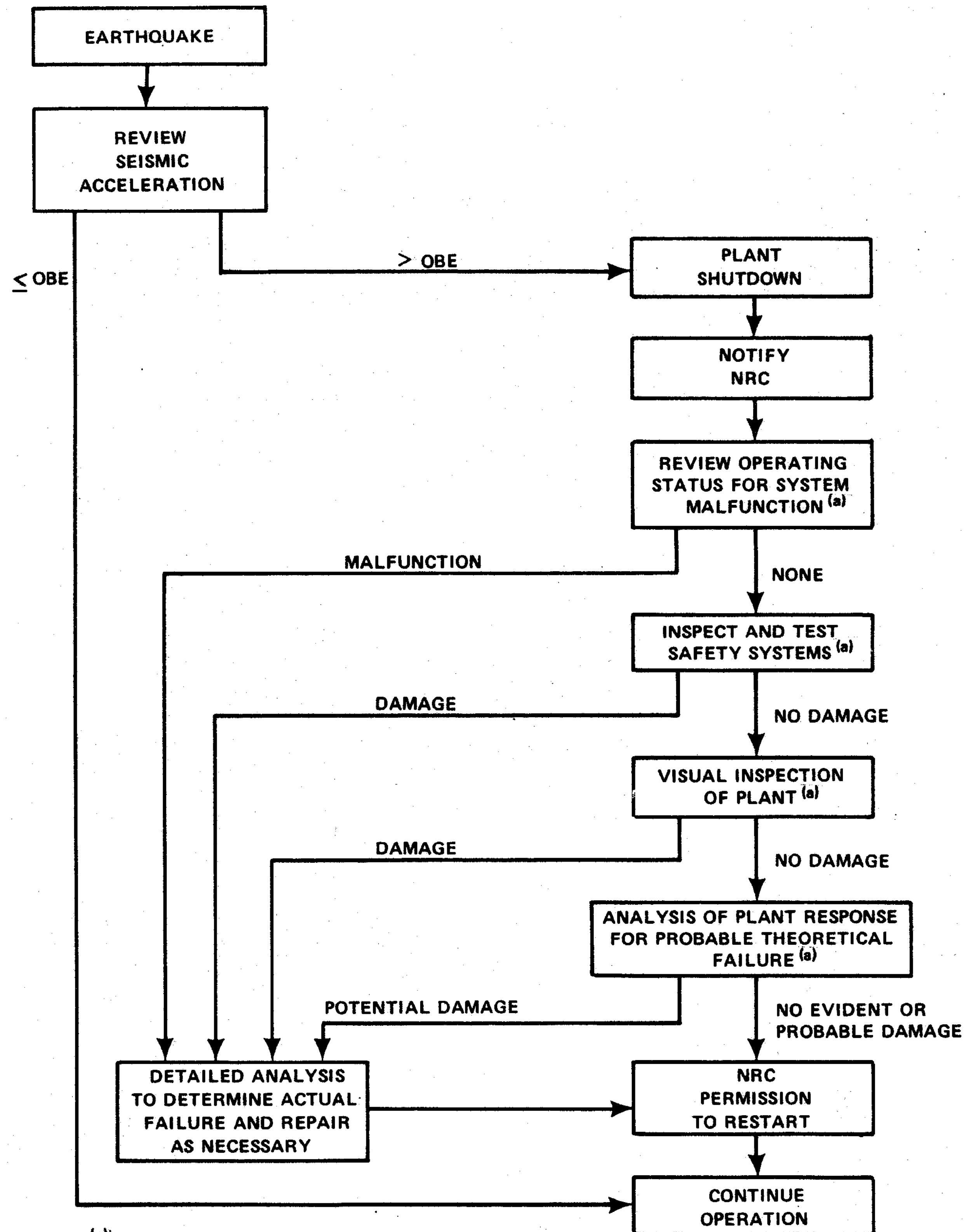
MODIFIED RESPONSE SPECTRUM CURVE

FIGURE 3.7A-6



(a) REPORT RESULTS TO NRC

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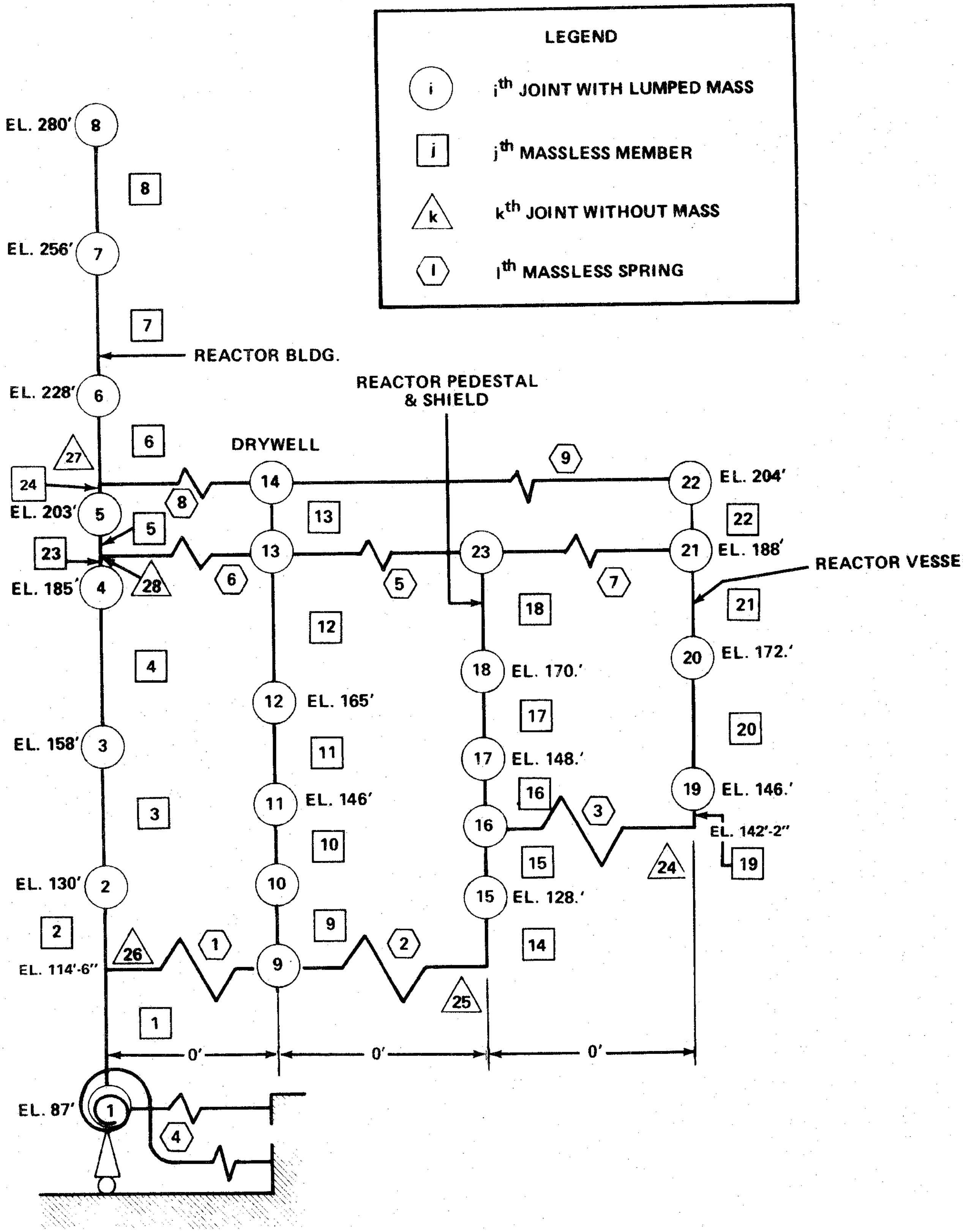
POST-SEISMIC EVENT PLANT PROCEDURES

FIGURE 3.7A-7



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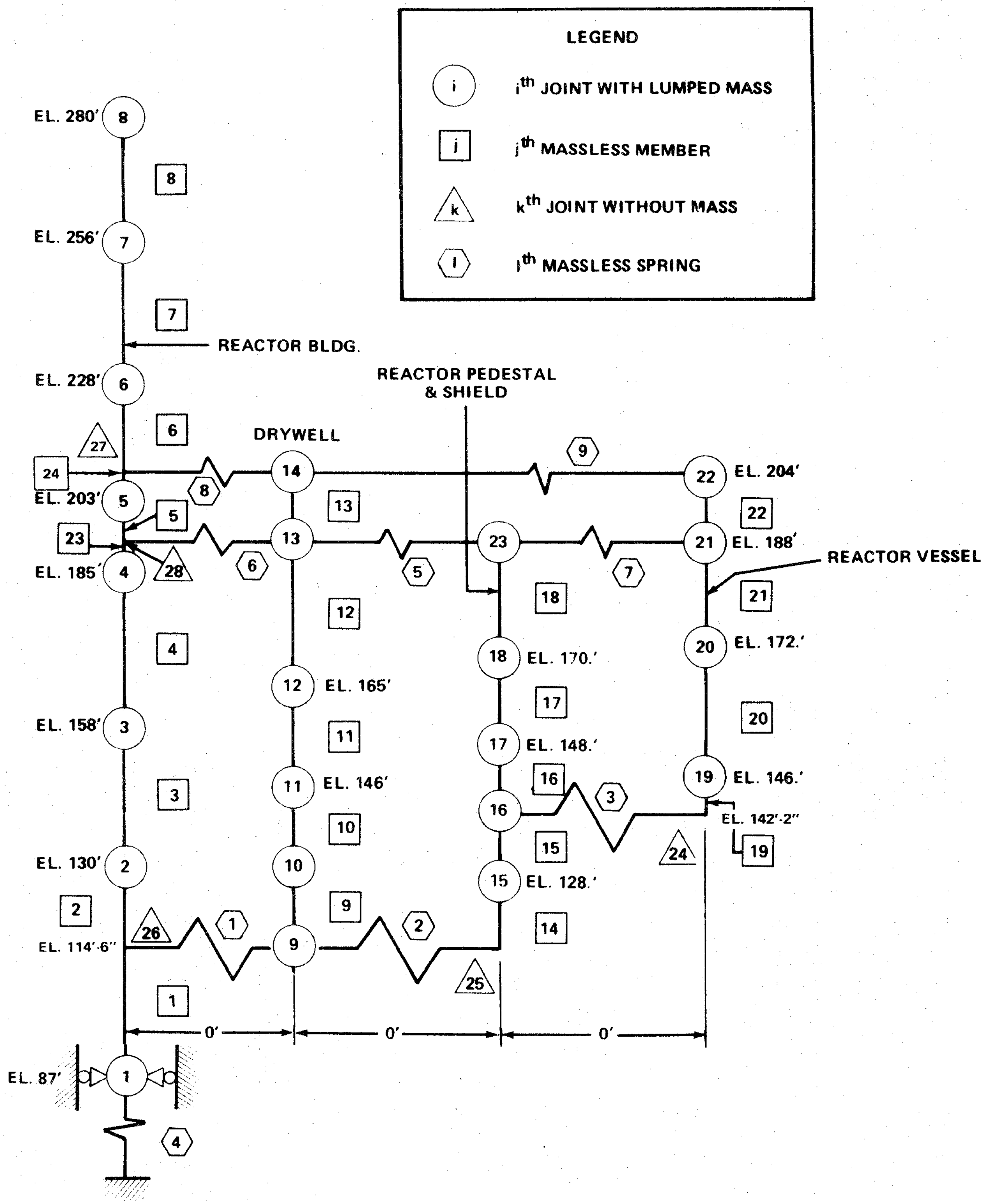
SEISMIC MODEL
REACTOR BLDG: LATERAL

FIGURE 3.7A-8



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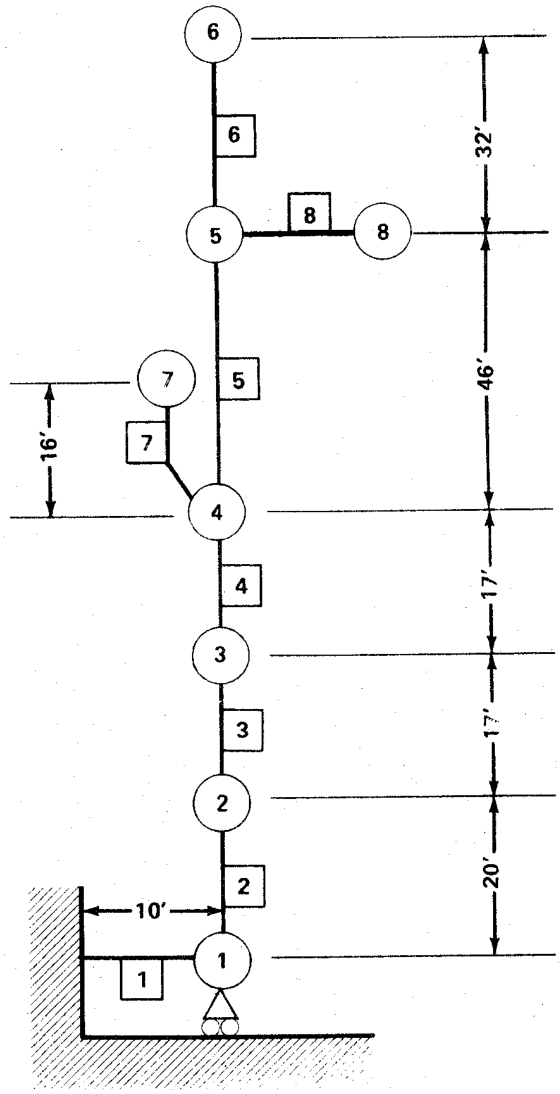
SEISMIC MODEL
REACTOR BLDG: VERTICAL

FIGURE 3.7A-9



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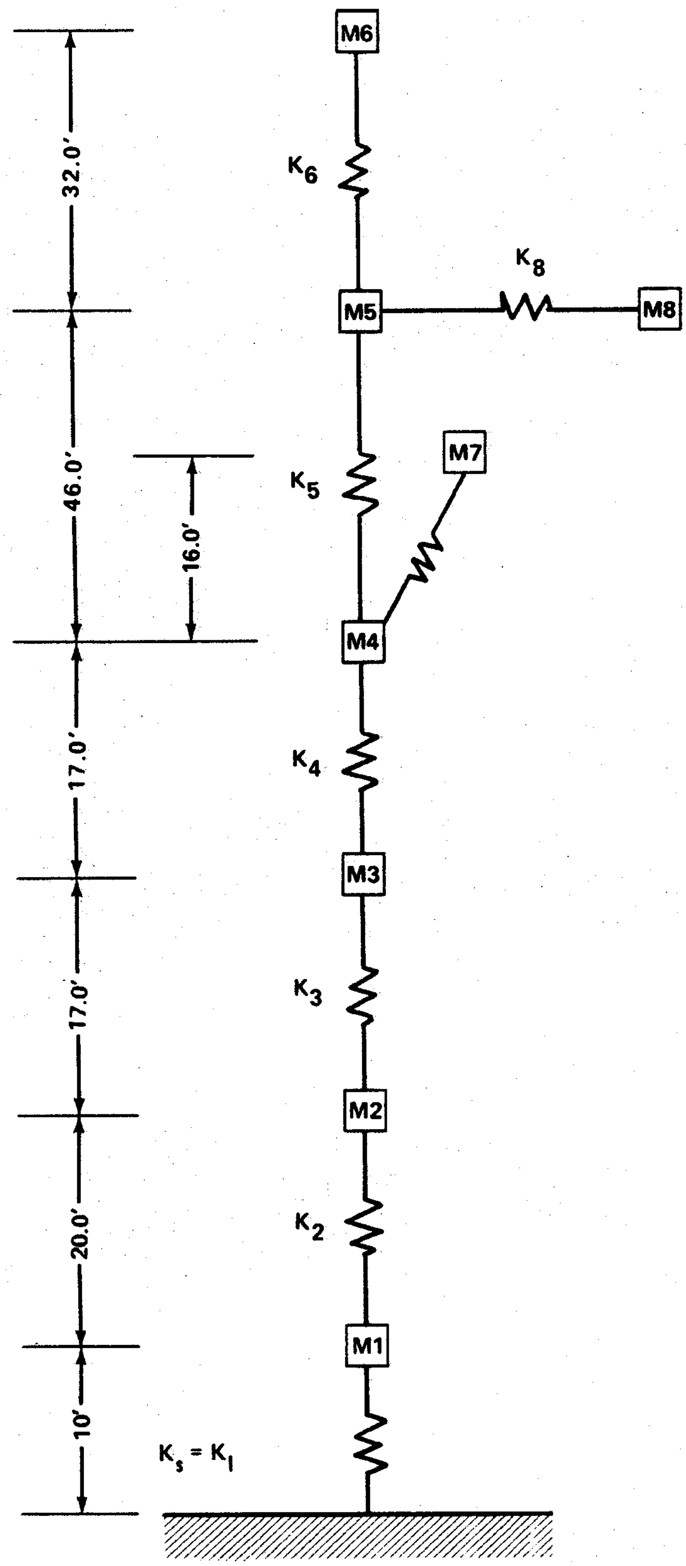
SEISMIC MODEL
CONTROL BLDG: LATERAL

FIGURE 3.7A-10



NOTES:

1) M_1 THROUGH M_8 = LUMPED MASSES
2) K_1 THROUGH K_8 = VERTICAL STRUCTURAL SPRINGS
3) K_s = VERT. SOIL SPRING

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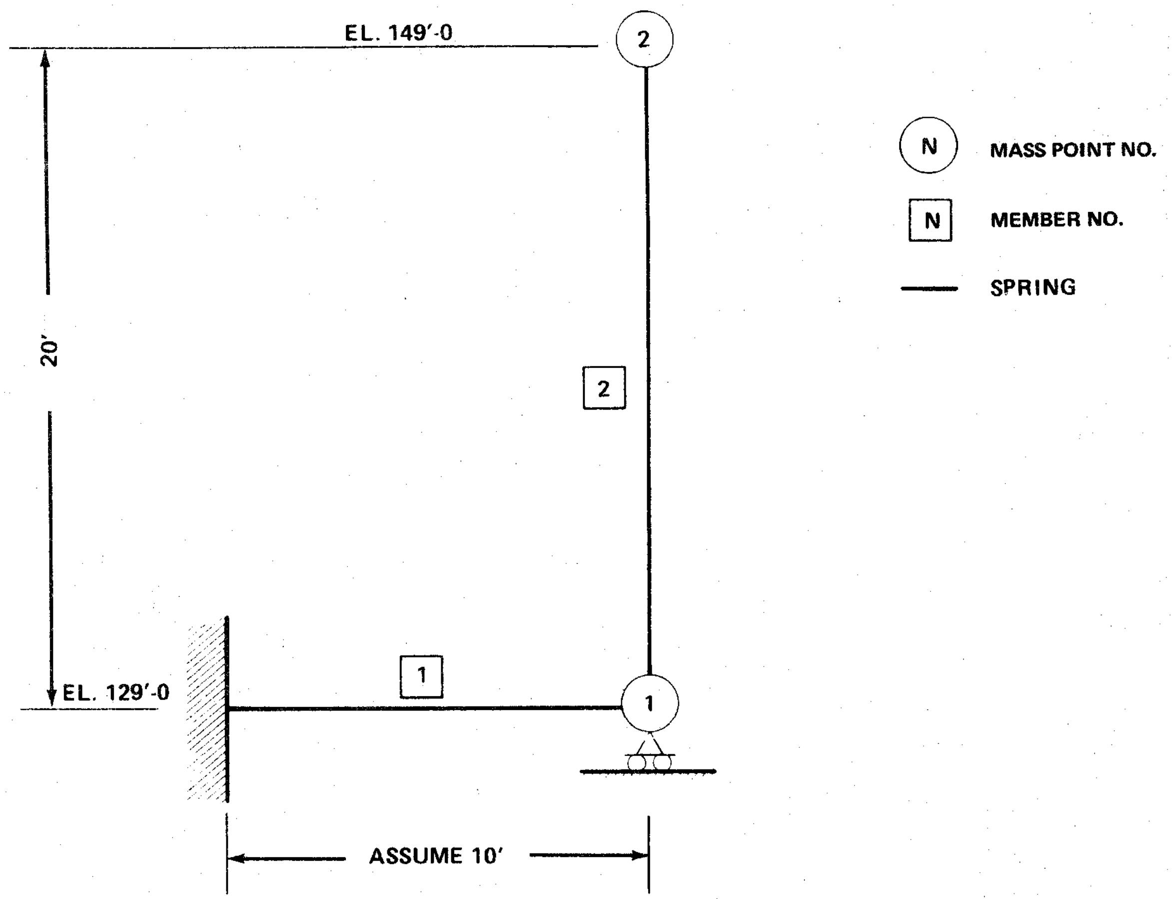
SEISMIC MODEL
CONTROL BLDG: VERTICAL

FIGURE 3.7A-11



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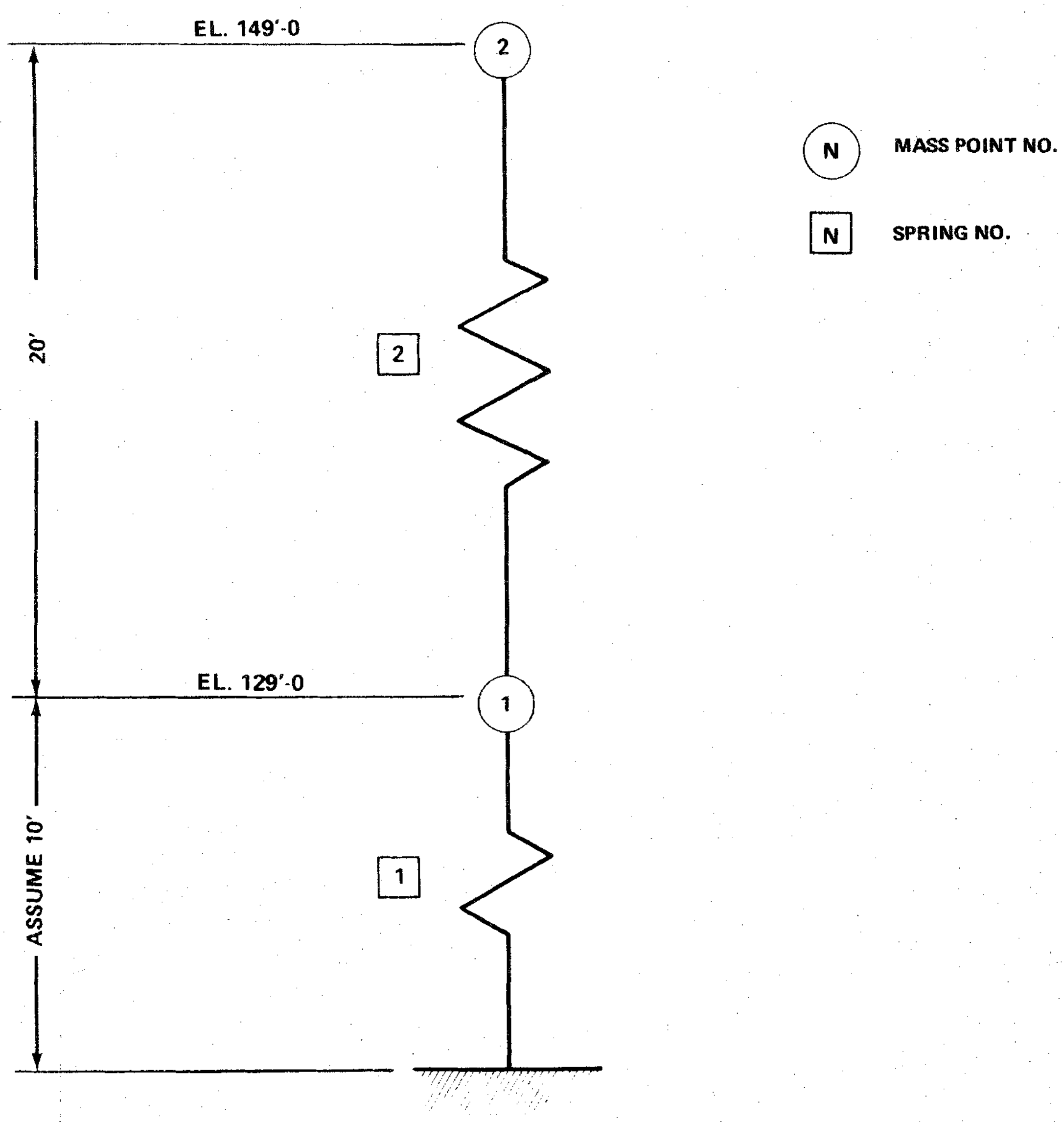
SEISMIC MODEL
DIESEL GENERATOR BLDG: LATERAL

FIGURE 3.7A-12



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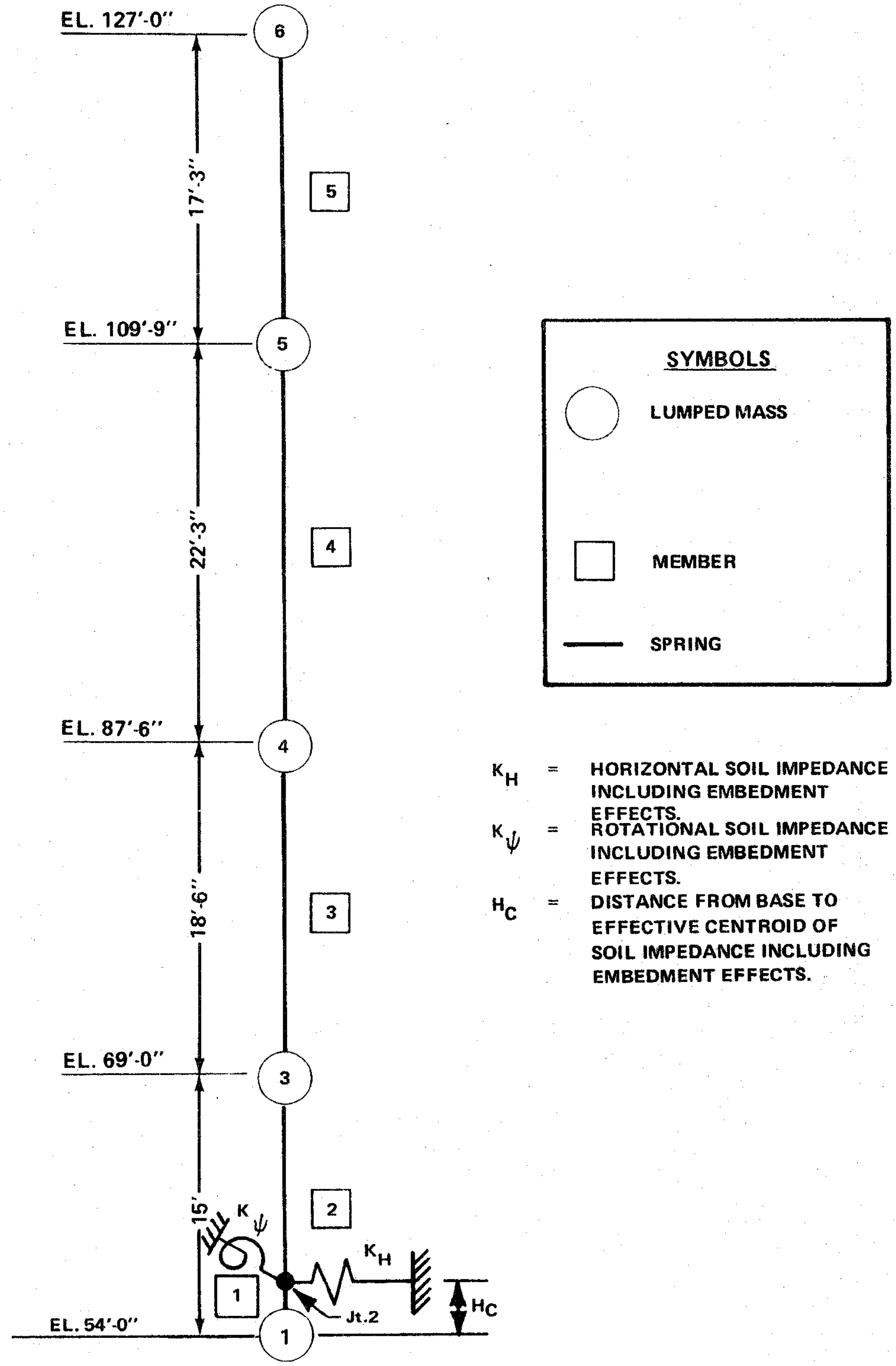
SEISMIC MODEL
DIESEL GENERATOR BLDG: VERTICAL

FIGURE 3.7A-13



K_H = HORIZONTAL SOIL IMPEDANCE INCLUDING EMBEDMENT EFFECTS.

K_ψ = ROTATIONAL SOIL IMPEDANCE INCLUDING EMBEDMENT EFFECTS.

H_C = DISTANCE FROM BASE TO EFFECTIVE CENTROID OF SOIL IMPEDANCE INCLUDING EMBEDMENT EFFECTS.

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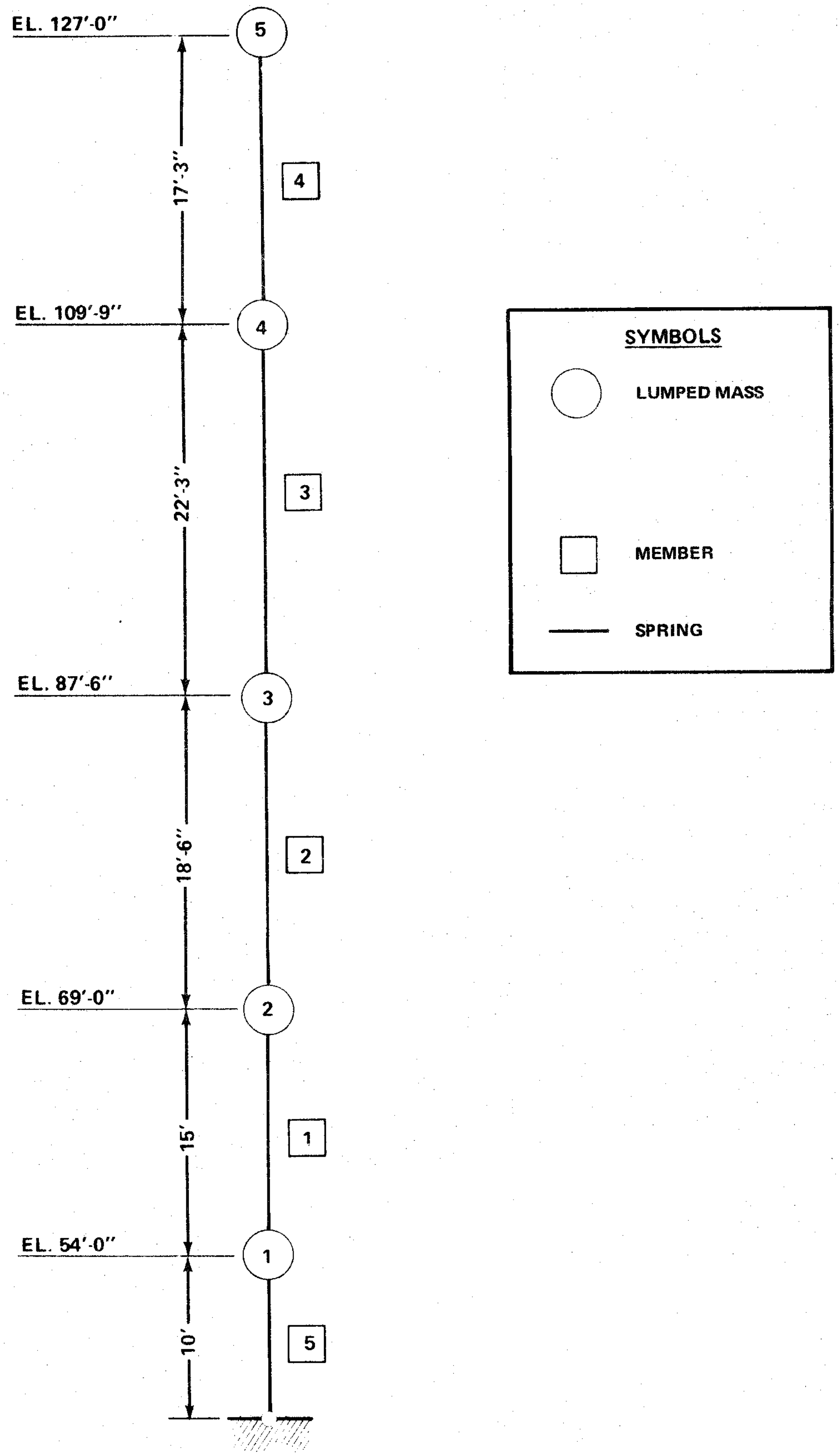
SEISMIC MODEL
INTAKE STRUCTURE: LATERAL

FIGURE 3.7A-14



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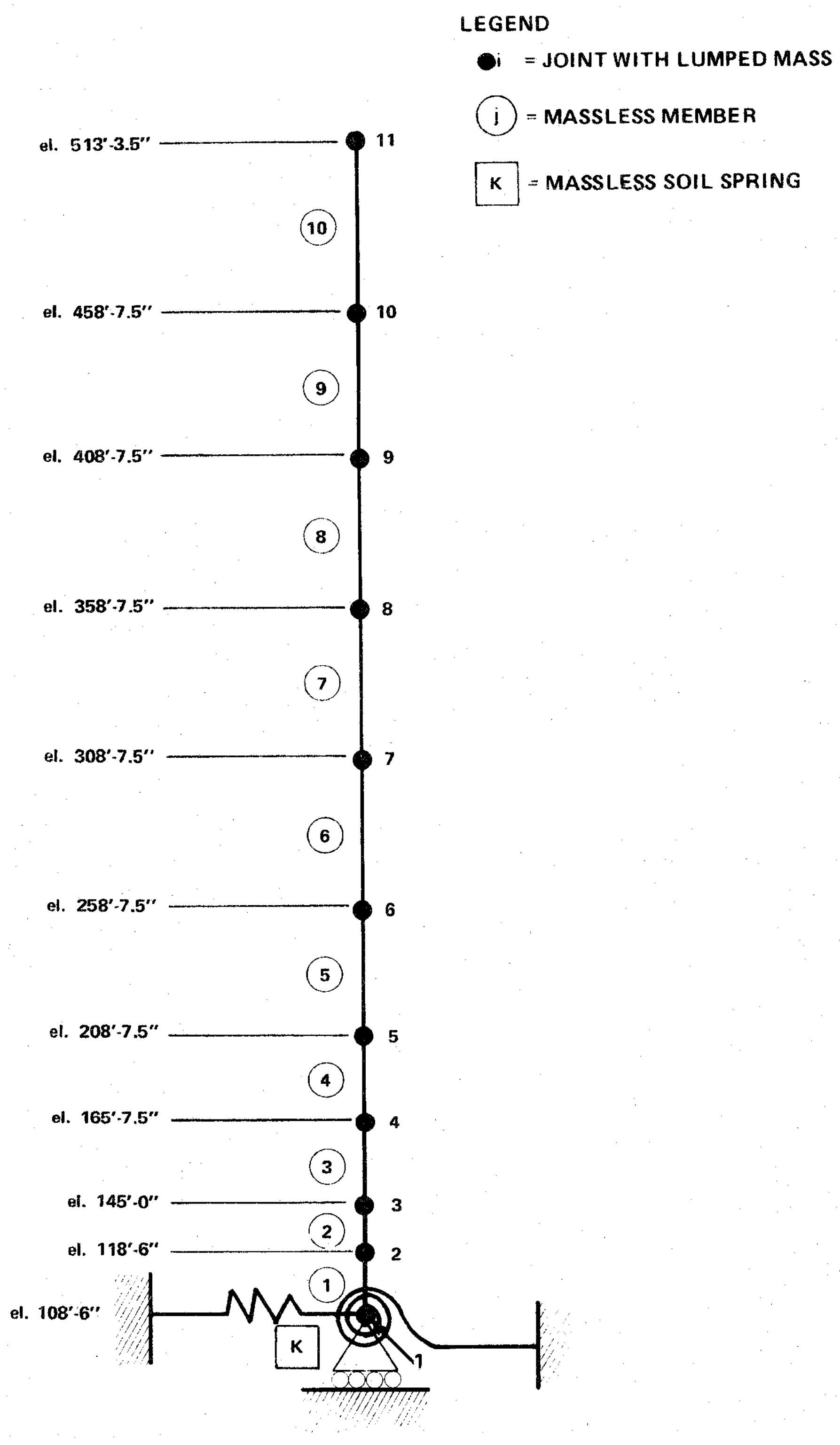
SEISMIC MODEL
INTAKE STRUCTURE: VERTICAL

FIGURE 3.7A-15



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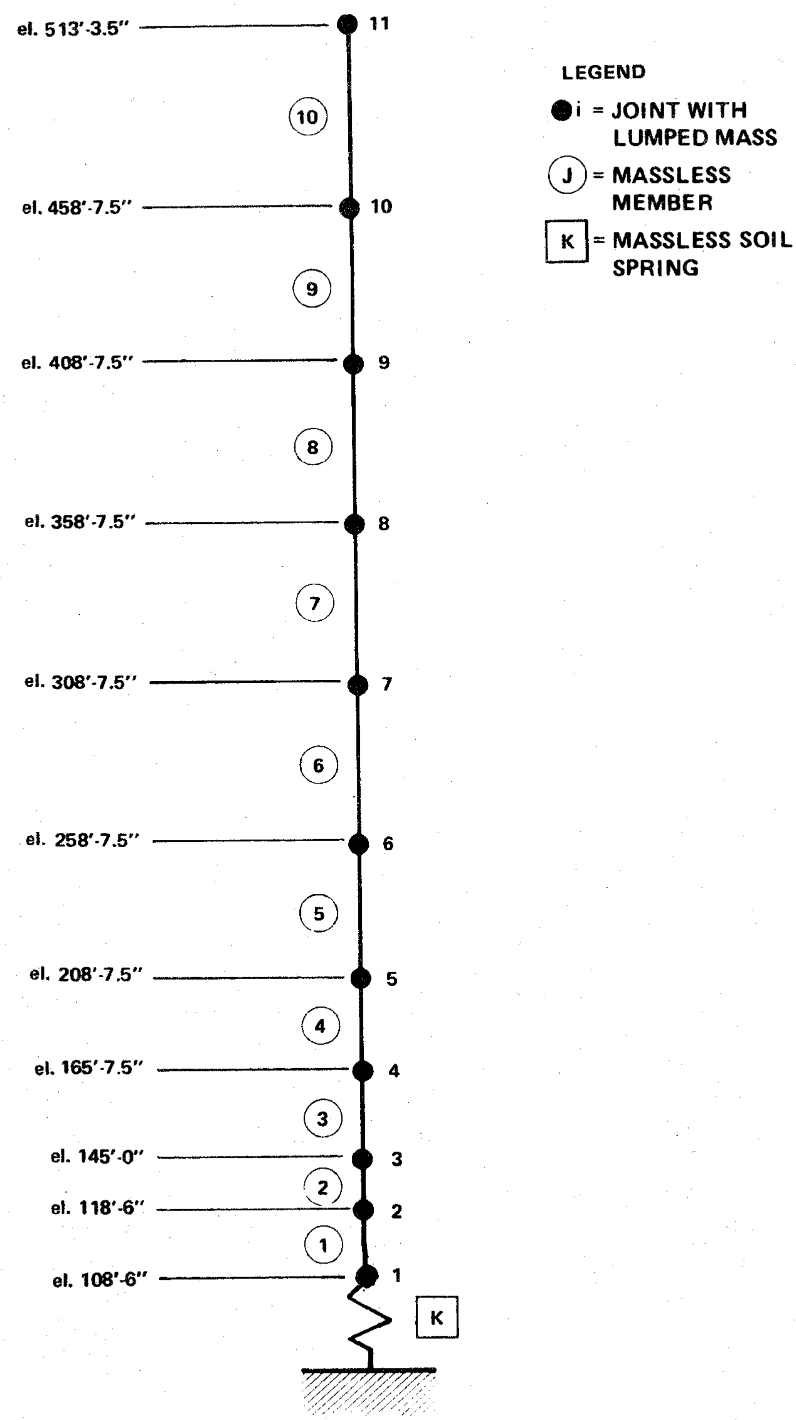
SEISMIC MODEL
MAIN STACK: LATERAL

FIGURE 3.7A-16



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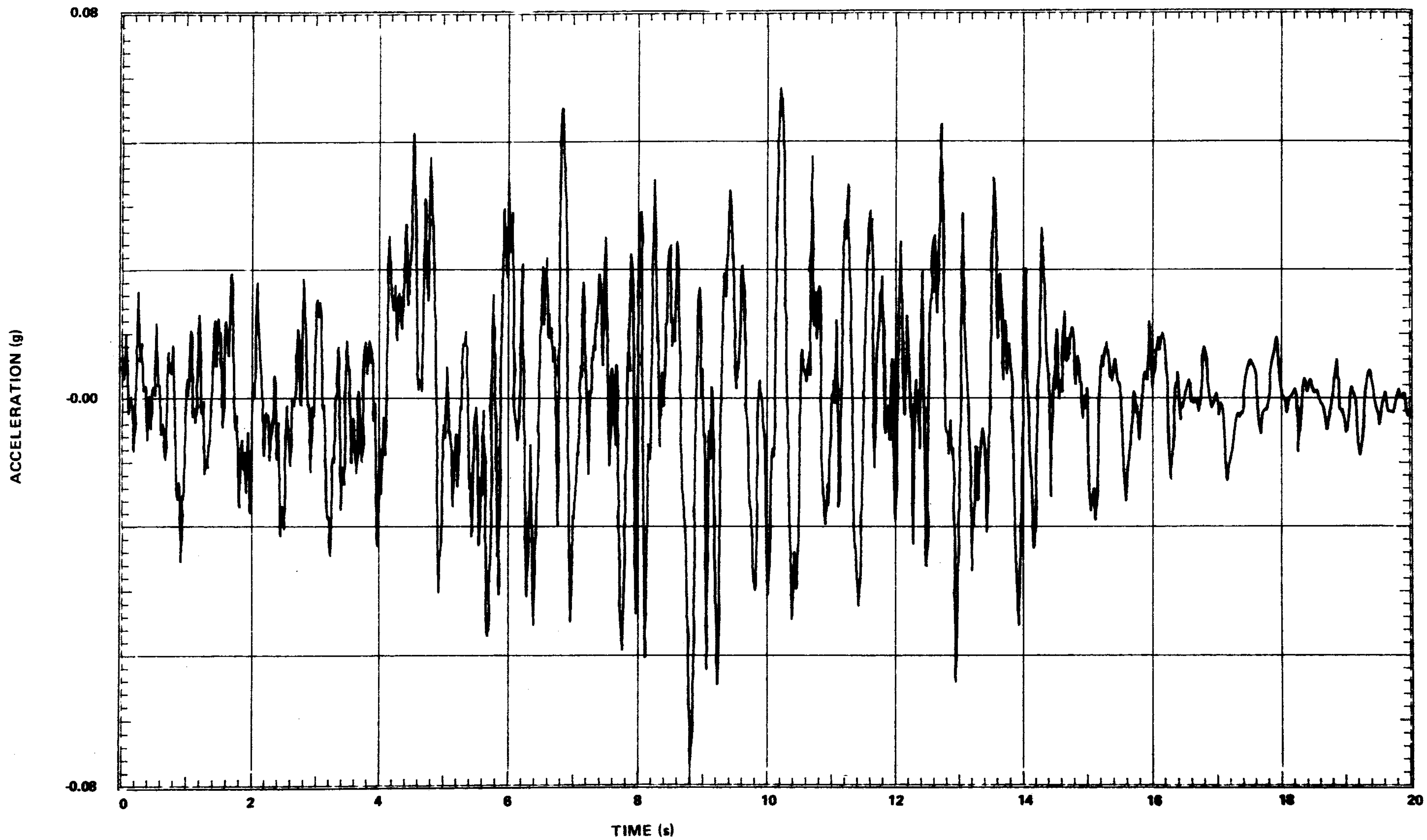
SEISMIC MODEL
MAIN STACK: VERTICAL

FIGURE 3.7A-17



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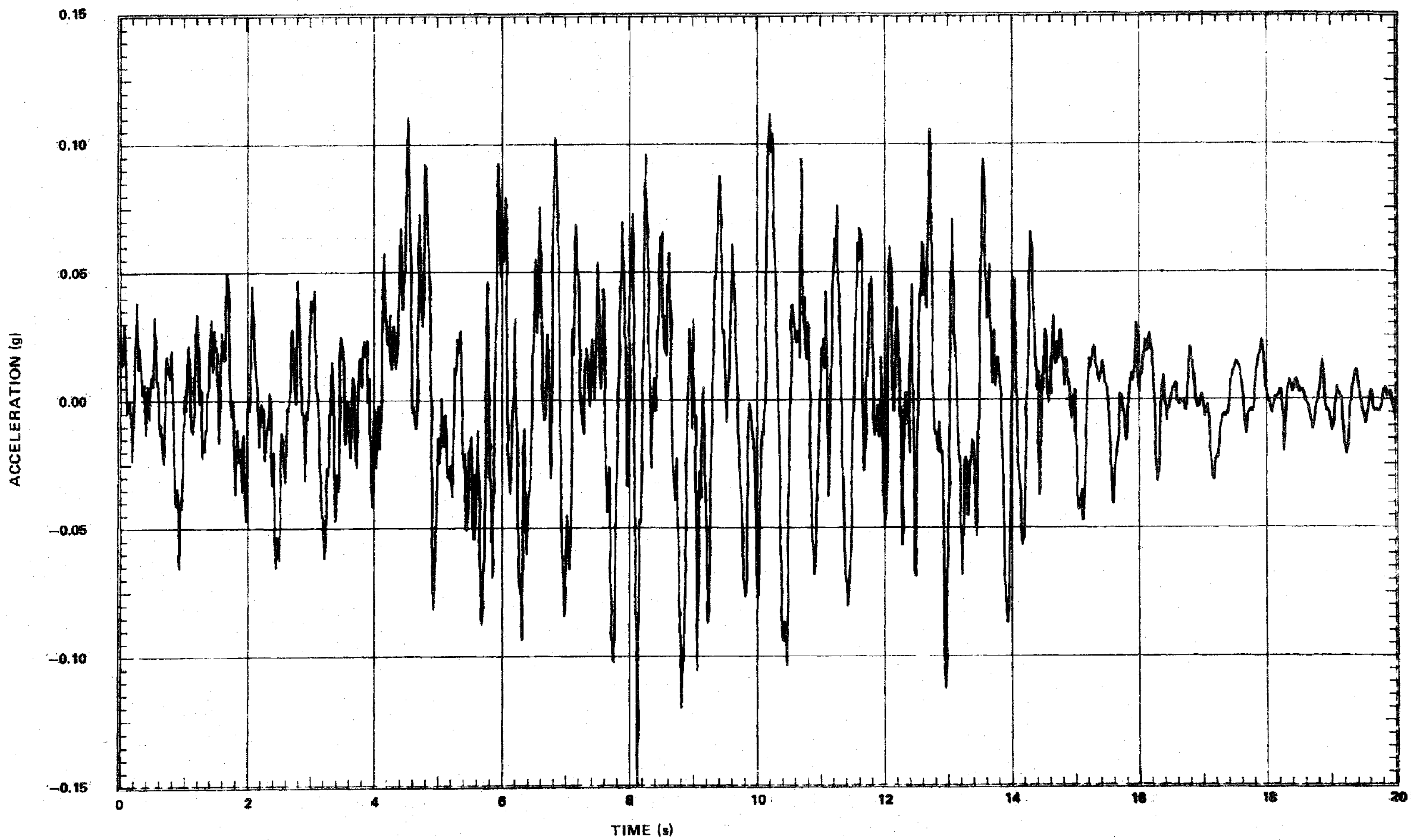
OBE SYNTHETIC ACCELEROGRAM (1984)

FIGURE 3.7A-18



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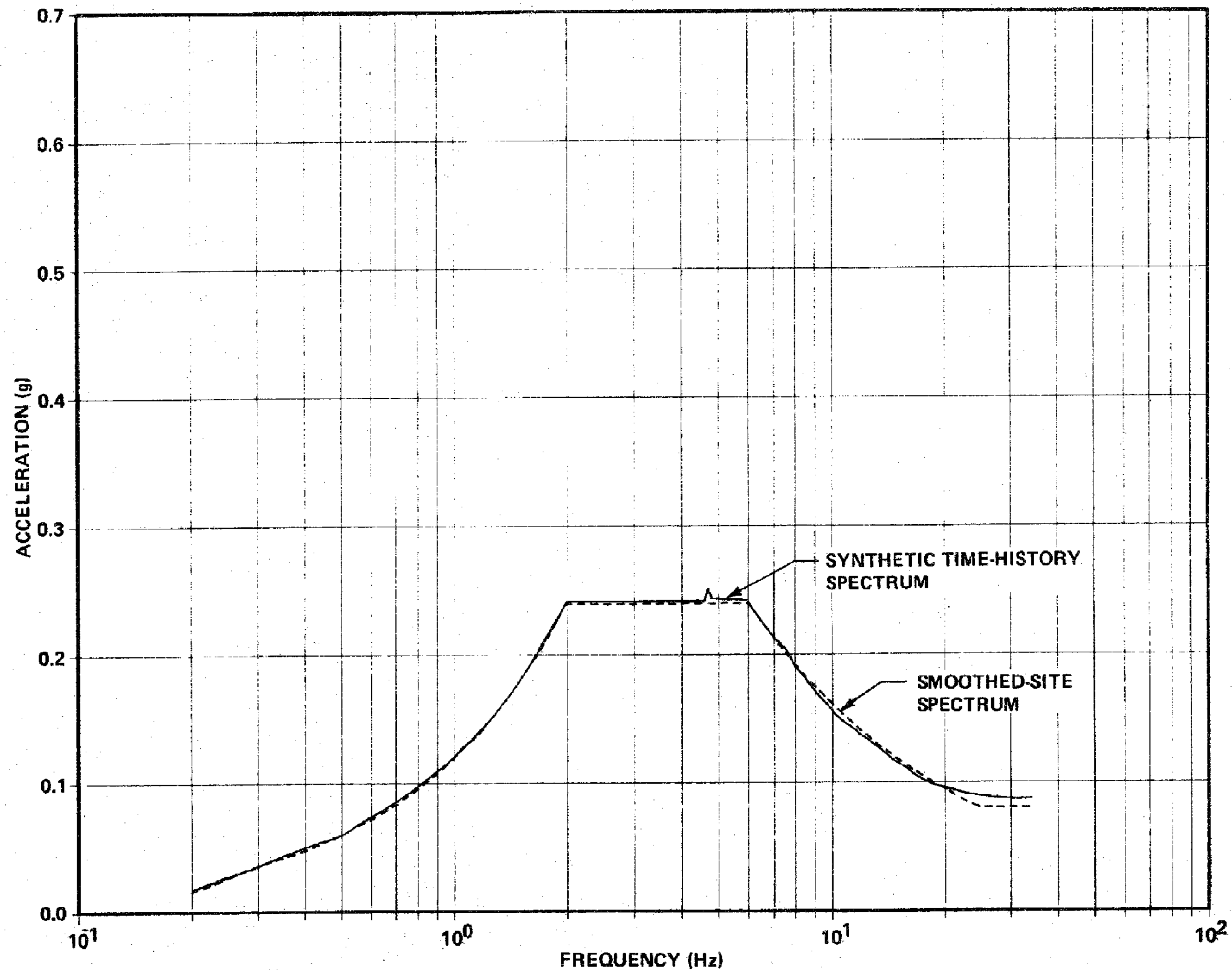
DBE SYNTHETIC ACCELEROGRAM (1984)

FIGURE 3.7A-19



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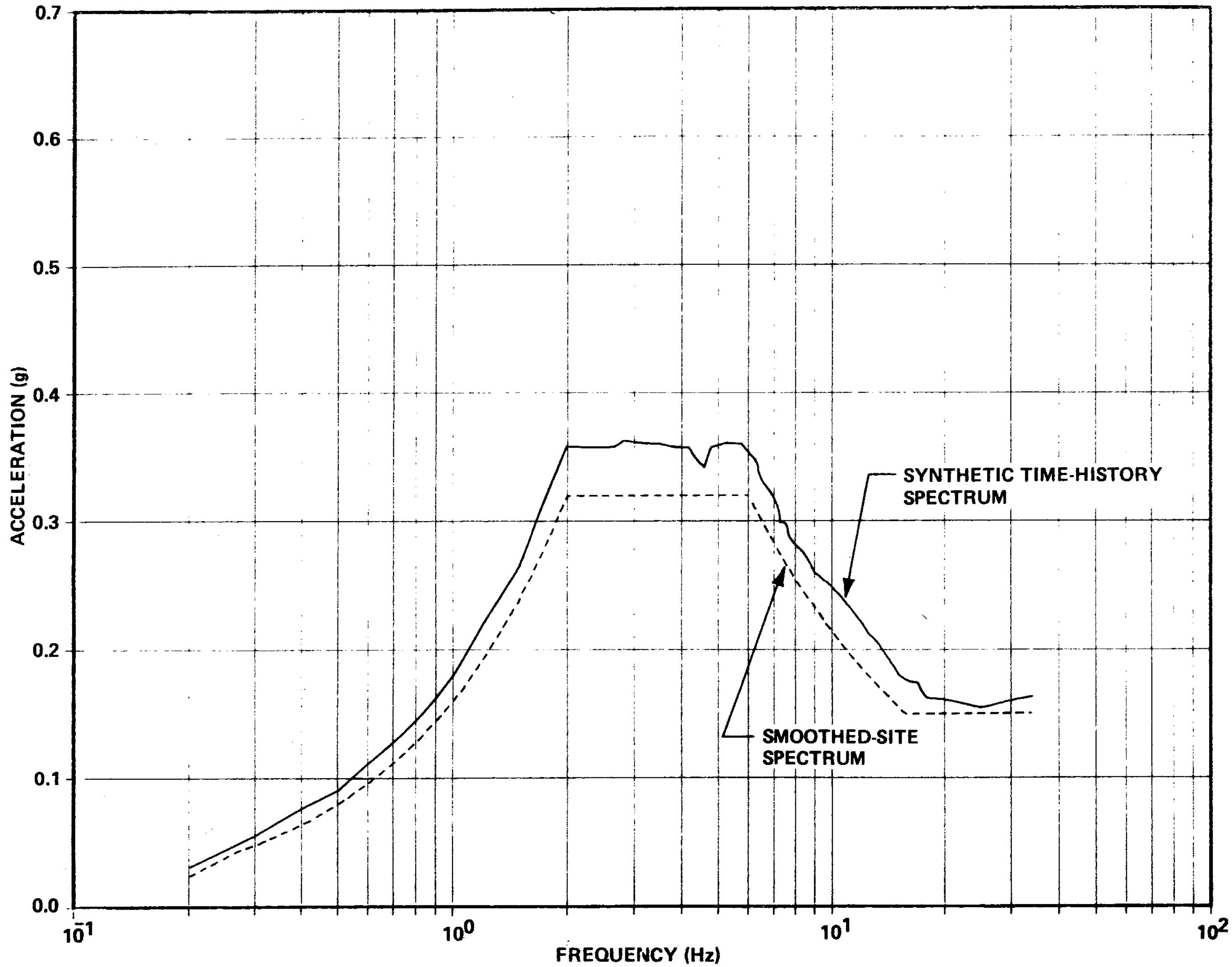
DESIGN SPECTRUM COMPARED WITH RESPONSE SPECTRUM OF OBE SYNTHETIC ACCELEROGRAM (1984) FOR 3% DAMPING

FIGURE 3.7A-20



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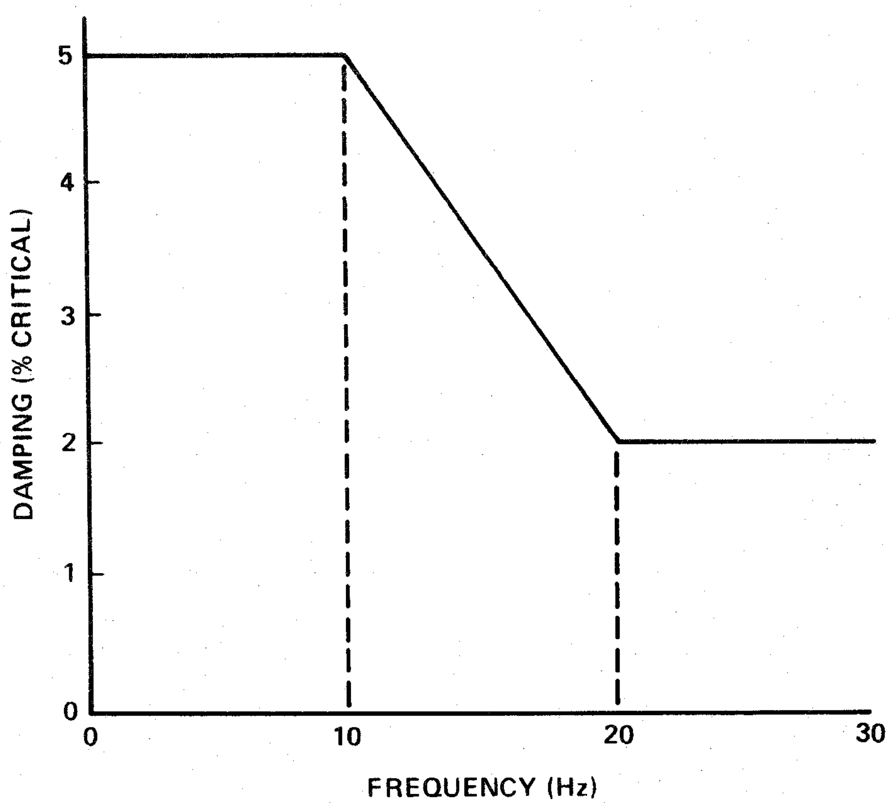
DESIGN SPECTRUM COMPARED WITH RESPONSE SPECTRUM OF DBE SYNTHETIC ACCELEROGRAM (1984) FOR 5% DAMPING

FIGURE 3.7A-21



NOTES

1. Applicable to both OBE and DBE, independent of pipe diameter.
2. As of April 4, 1985, damping per this figure is used for all new and replacement systems and load reconciliation work.

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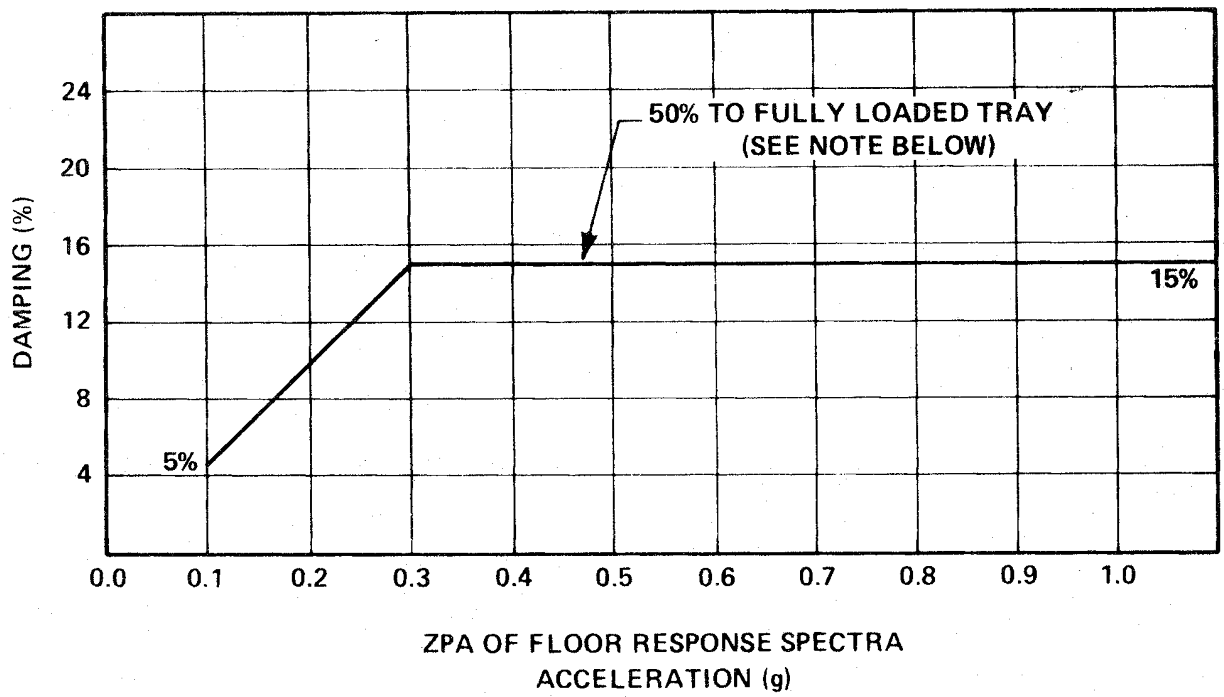
DAMPING CRITERIA FOR SEISMIC ANALYSIS OF PIPING SYSTEMS

FIGURE 3.7A-22



NOTES:

1. For unloaded tray, use damping values specified in Table 3.7A-1 for steel structures. For tray loaded less than 50% linear, interpolation is used to determine the applicable design damping value.

2. As of April 4, 1985, damping per this figure is used for all new and replacement systems and load reconciliation work.

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DAMPING CRITERIA FOR SEISMIC ANALYSIS OF CABLE TRAY SUPPORTS

FIGURE 3.7A-23

SUPPLEMENT 3.7A.A

CRITERIA FOR SEISMIC QUALIFICATION OF SEISMIC CATEGORY I EQUIPMENT AND PIPING

3.7A.A.1 SCOPE

All Seismic Category I systems and equipment (assemblies and devices) supplied must withstand the postulated seismic occurrence as specified below. This supplement contains criteria that were used, in conjunction with IEEE 344-1971, to define the methods and procedures to be used in establishing the seismic qualification of the non-NSSS Seismic Category I equipment installed originally at HNP-2. For piping system analysis, the techniques in Bechtel Topical Report BP-TOP-1 were utilized. For loading combinations and allowable stress levels for seismic events, refer to section 3.9.

The maximum values of the codirectional responses caused by each of the components of earthquake are combined either by the summation of absolute values or by the square-root-of-the-sum-of-the-squares.

The summation of the codirectional inter-modal responses is by the square-root-of-the-sum-of-the-squares.

The equipment supplier is responsible for ensuring safe operation of the equipment and systems under the seismic conditions specified below. The supplier shall verify that the equipment will meet the stated functional requirements for continued operation without malfunction or loss of function during and after a postulated seismic event.

The "Institute of Electrical and Electronic Engineers (IEEE) Guide for Seismic Qualification of Class 1 Electric Equipment for Nuclear Power Generating Stations," IEEE Standard 344-1971 is used except as amended herein. The amendments listed below define and provide minimal values needed to verify the equipment capability. The term "electrical equipment" used throughout the Guide refers to all types of Seismic Category I equipment. Complete qualification procedures and monitoring techniques shall be presented by the equipment supplier to the buyer for review prior to the actual start of qualification work.

3.7A.A.2 DEFINITION

Add the following new paragraphs to IEEE Standard 344 as numbered below:

2.8 Operating Basis Earthquake (OBE)

The OBE is the largest earthquake which could reasonably be expected to occur at the site during the life of the plant and for which the equipment must remain operational or be able to shut down and start up again.

2.9 Fluid Systems

Those systems or equipment, such as pipes, pumps, valves, vessels, and tanks, that are part of a fluid-containing barrier. The support structure for a fluid system is an integral part of that system.

2.10 Malfunction or Functional Impairment

Equipment malfunction or functional impairment is the failure of equipment to perform its function in the same manner in which it would have in the absence of a seismic disturbance. For protective systems, malfunction is the loss of capability to initiate or sustain a protective action and not to initiate an action spuriously.

3.7A.A.3 PROCEDURE

Add the following to the end of paragraph 3 of IEEE Standard 344:

When the malfunctioning of Class I equipment is considered, testing is the method recommended to verify the functional requirements.

3.7A.A.3.1 ANALYSIS

3.7A.A.3.1.1 Add the following to the end of paragraph 3.1.1 Standard 344:

The number of masses shall be sufficient to define the dynamic behavior of the equipment (the mathematical model shall be shown even for a single degree of freedom system).

3.7A.A.3.1.2 Add the following to the end of paragraph 3.1.2 of IEEE Standard 344:

The equipment natural frequencies as determined shall be assumed to have a minimum variation of ±10 percent. The actual variation shall depend on the expected accuracy of the calculations.

3.7A.A.3.1.3 Add the following sentence to the end of paragraph 3.1.5 of IEEE Standard 344:

Further, if the equipment is part of a fluid system, then the liquid should be considered in the analysis. Fluctuation of pressures due to acceleration, sloshing, compression waves, breathing modes, hydraulic transients, etc., shall be considered.

3.7A.A.3.1.4 Add the following new paragraph as 3.1.6 to IEEE Standard 344:

Seismic Category I equipment shall be designed for gravity loads, normal operating loads, operating temperature loads, and other loads that are included in the specification, combined with appropriate seismic loads. The seismic load shall include both the vertical and horizontal components acting simultaneously. The loading combination that will produce the maximum stress shall be considered.

The combined normal operating primary stresses and the primary stress due to the OBE shall be maintained equal to or below allowable working stress limits that are accepted as good practice and set forth in appropriate design standards and codes. However, no increase in the allowable working stress will be permitted because of dynamic loads except when permitted by the standards and codes for nuclear service. Local, primary, and self-limiting secondary stresses shall conform to the allowable values permitted by the appropriate code.

The normal operating primary stresses combined with the design basis earthquake (DBE) shall not exceed 90 percent of the minimum guaranteed yield strength[*] of the material as stated in the American Society for Testing and Materials ASTM standards with applicable reduction due to temperature of stability. For mechanical equipment the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code shall be used where specified. Local, primary, and self-limiting secondary stresses may exceed yield stress levels to the extent permitted by the appropriate codes as long as malfunction is prevented.

3.7A.A.3.2 TESTING

3.7A.A.3.2.1 Add the following to the end of paragraph 3.2.2.1 of IEEE Standard 344:

Biaxial testing in the vertical and horizontal directions simultaneously is allowed and preferred.

3.7A.A.3.2.2 Add the following sentence to the end of paragraph 3.2.2.3.1 of IEEE Standard 344:

The minimum frequency range shall be from 1 to 35 cps; however, an extended frequency range shall be used where it is necessary because of special conditions.

---

* In no case shall the algebraic difference between the maximum and minimum principal stresses be greater than 90 percent of minimum guaranteed yield stress.

3.7A.A.3.2.3 Add the following sentence to the end of paragraph 3.2.2.4.1 of IEEE Standard 344:

The minimum time duration for each condition and direction shall be 20 seconds and in no case less than that required to produce the desired amplification. If the location of the device necessitates a longer duration for a conservative test, then the above duration should be increased accordingly.

3.7A.A.3.2.4 Delete the fourth sentence of paragraph 3.2.2.4.2 of IEEE Standard 344 in its entirety and replace with the following:

For a test at any frequency, five beats are normally used; however, enough additional beats shall be added to make a total excitation duration of 5 seconds in any axis. There shall be a pause between the beats such that there results no significant superposition of motion.

3.7A.A.3.2.5 Add the following sentence to the end of paragraph 3.2.2.4.3 of IEEE Standard 344:

Before other tests are used, the test procedures and justifications shall be submitted to the buyer for review. If the buyer concurs with the proposed tests and procedures, then they may be used.

3.7A.A.3.2.6 Add the following to the end of paragraph 3.2.3.1 of IEEE Standard 344:

Biaxial testing in the vertical and horizontal directions simultaneously is allowed and preferred. The majority of Seismic Category I equipment is single-axis tested and qualified in accordance with the requirements of IEEE 344-1971.

3.7A.A.3.2.7 The following sentence shall be added to the end of paragraph 3.2.3.4 of IEEE Standard 344:

The minimum frequency range shall be from 1 to 35 cps; however, an extended frequency range shall be used where it is necessary because of special conditions.

3.7A.A.3.2.8 Add the following new paragraph as 3.2.3.5 to IEEE Standard 344:

The magnitude of the test input acceleration shall be determined from the appropriate DBE response spectra, the method of testing, the duration of excitation, and the damping of the equipment. The theoretical correlation of the response spectra and the test input is presented in figure 3.7A.A-1. The curves are based upon analysis of a linear single-degree-of-freedom mass-spring-damper model with the designated base input. The vibration magnification curves are calculated over a range of damping values (percent of critical).

NOTE: Conservative values of damping shall be used when the actual value is not known. In testing, this is usually the higher value of damping. The table input should include the necessary factor to account for other mode contribution for multi-degree-of-freedom systems. It is suggested that a factor of 1.5 be used until future evidence indicates other values.

The damping values referred to are the ones used to compute a response spectrum curve from random time-history motion. The amplification factor associated with random time-history motion, which is the ratio of peak response to floor response, is less than the amplification factor associated with sinusoidal motion, which is generally used as input for equipment testing. A method for obtaining the required peak input motion from a response spectrum curve is to take the peak response and divide that by the proper amplification factor. Therefore, by using a response spectrum curve with the higher damping factor, B_n, and dividing that by the amplification factor with testing motion for the same damping factor, B_n, one would obtain a higher peak input response.

This 50 percent increase in input to the shaker table input motion is analogous to the recommendations for the static load method of analysis presented in paragraph II.1.b.3 of Nuclear Regulatory Commission Standard Review Plan 3.7.2.

3.7A.A.3.2.9 Add the following new paragraphs as 3.2.4, 3.2.5, and 3.2.6 to IEEE Standard 344:

3.2.4 Post-Test Inspection

The tested item shall be thoroughly inspected for any damage sustained during testing. A detailed description of damage and repair and/or replacement shall be included in the test report.

3.2.5 Equipment Malfunction

If the equipment fails, malfunctions (change in status due to dynamic motion), or will not operate after the test, the supplier shall redesign the system and resubmit drawings and data for approval. A new test shall be conducted on the redesigned equipment to show compliance with the specification at no additional expense to the buyer.

3.2.6 Schedules and Delivery

Modifications of testing procedures, reanalysis, redesign, or resubmittals to satisfy these criteria in obtaining the engineer's concurrence shall not be the basis for late delivery of systems, equipment, and components. The supplier is invited to submit his proposed seismic analysis, design, and testing program prior to actual implementation to minimize such delays. Such delays shall remain the supplier's responsibility. Any changes that may be required due to the seismic analysis or testing shall automatically void prior approval or concurrence of drawings that have been submitted earlier.

3.7A.A.4 DOCUMENTATION

3.7A.A.4.1 ADD THE FOLLOWING TO THE END OF PARAGRAPH 4.1 OF IEEE STANDARD 344:

This documentation shall be submitted in report form to the buyer.

3.7A.A.4.2 ADD THE FOLLOWING TO THE END OF PARAGRAPH 4.2 OF IEEE STANDARD 344:

It shall also include a summary and conclusion with reference to the analysis where information for the conclusion can be reviewed.

3.7A.A.4.3 DELETE PARAGRAPH 4.3(5) AND ADD NEW PARAGRAPH 4.3(5) TO IEEE STANDARD 344 AS FOLLOWS:

(5) Test data shall include natural frequencies, response accelerations, stresses, calibration history of test equipment, damping values, reactions, and mounting details.

3.7A.A.4.4 DELETE PARAGRAPH 4.3 (6) AND ADD NEW PARAGRAPH 4.3 (6) TO IEEE STANDARD 344 AS FOLLOWS:

(6) Data analysis and evaluation (including the resultant response spectra for the surface upon which the equipment was mounted when tested).

3.7A.A.4.5 ADD THE FOLLOWING NEW PARAGRAPH AS 4.4 TO IEEE STANDARD 344:

Certification of Compliance

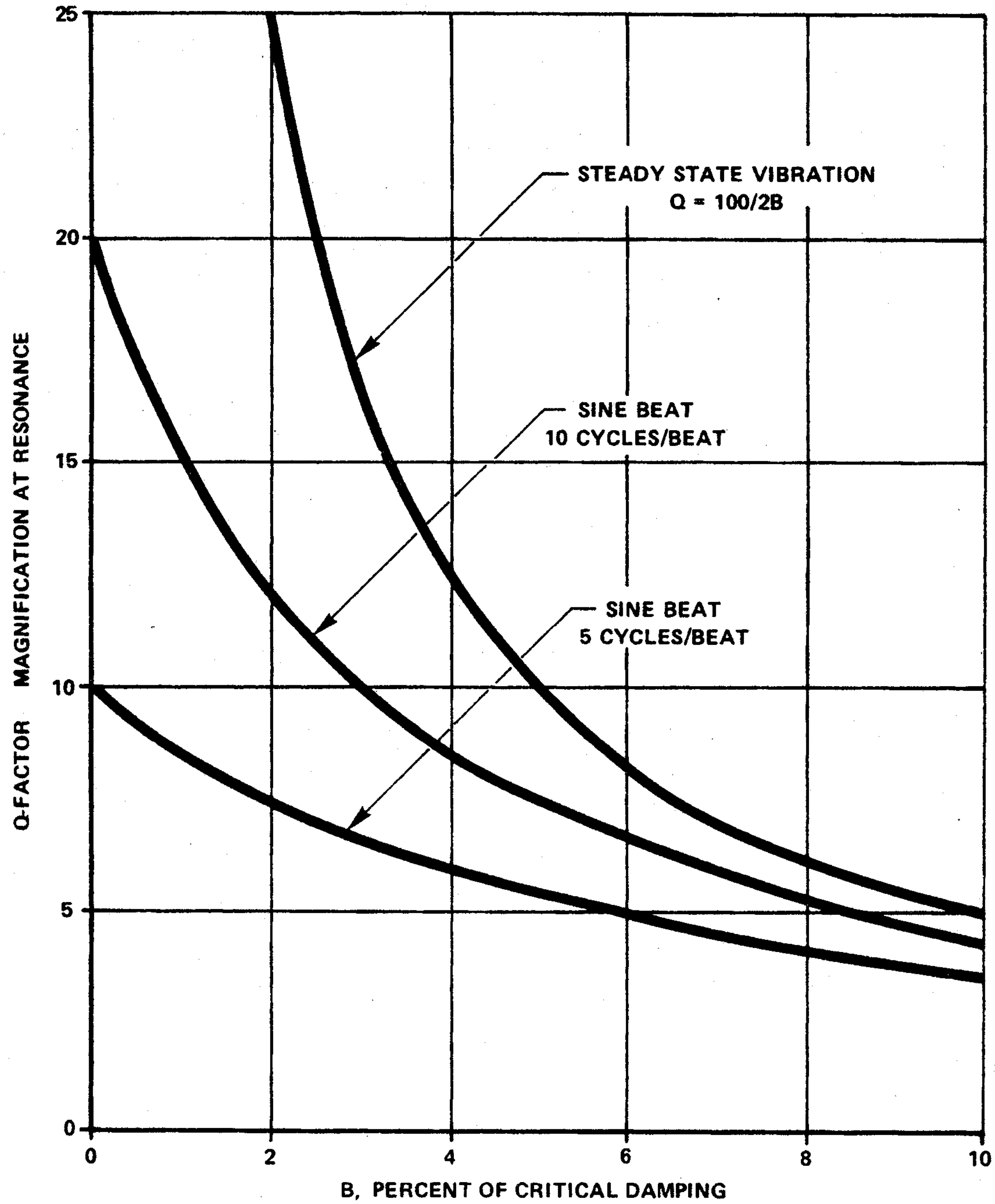
All the test data submitted by the supplier to satisfy the requirements of this specification shall be supervised, witnessed, and reviewed by a supplier's competent engineer. The test data, design calculations, and the certification submitted shall be signed and approved for compliance to the specification under the seal of a registered professional engineer and the supplier. These documents must be submitted for the buyer's approval prior to the release of shipment of the equipment.

3.7A.A.5 ADD THE FOLLOWING NEW PARAGRAPH AS 6.0 TO IEEE STANDARD 344:

Design Response Spectra

The attached operating basis earthquake and DBE horizontal and vertical floor-response spectra reflect the instructure floor accelerations resulting from the dynamic analysis.

Equipment location, special orientation, and appropriate response spectra are supplied to the suppliers for the seismic qualification of their equipment.



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VIBRATION MAGNIFICATION
AT RESONANCE

FIGURE 3.7A.A-1

SUPPLEMENT 3.7A.B

ANALYSIS OF LONG-BURIED STRUCTURES

3.7A.B.1 INTRODUCTION

This section outlines the methods used for seismic analysis of buried Seismic Category I piping and electrical ducts. It was assumed that the soil does not lose its integrity during an earthquake. Pipes and electrical ducts were assumed to move with the soil as the seismic wave propagates across them. The effect of soil-pipe and soil-duct interactions was neglected in the analysis. This assumption is necessary because of the present level of analytical techniques; however, this assumption is considered justifiable as discussed in a previous study,[1] particularly if the size of the pipe or the duct is small compared to the other parameters of the problem. It was further assumed that the earthquake wave would come only in one direction parallel to the longitudinal axis of the buried structures and that it would have no change in shape.

3.7A.B.1.1 METHOD OF ANALYSIS

Two separate approaches are employed in the analysis of the problem. The following are brief formulations of both approaches.

3.7A.B.1.1.1 Free-Field Case

For the portion of a pipe or duct far from two ends, free of any external barrier except the surrounding soil, it is reasonable to assume that this portion of the pipe or duct will move together with the soil as the seismic wave propagates. The effect of interaction between the buried member and soil will probably be negligible if its size is relatively small in relationship to the other dimensions. In this regard, Newmark[2] first proposed two equations using the wave propagation approach. The two equations which expressed the strains of the soil in terms of the velocity of the acceleration of the incoming seismic wave are the basis of this portion of the study.

Consider two points, points A and B, at a distance, d, apart, as shown in figure 3.7A.B-1. u is the displacement at A, and u plus an increment as shown is the displacement at B. It is noted that the second derivative of u with respect to x is significant only if d is very large. Now, consider a wave propagating from A towards B, with a displacement in the form of:

$$u = f(x - ct) \tag{1}$$

where: c = the velocity of wave propagation and t is the time.

Differentiating equation 1 with respect to x and t, respectively, one has:

$$\frac{\partial u}{\partial x} = f'(x - ct) \tag{2}$$

$$\frac{\partial u}{\partial x} = -cf'(x - ct) \tag{3}$$

From equations 2 and 3, it follows that:

$$\frac{\partial u}{\partial x} = -\frac{1}{c}\frac{\partial u}{\partial x} \tag{4}$$

In the case where u is in the direction of x, equation 4 leads to:

$$\varepsilon_m = -\frac{\dot{u}_m}{c} \tag{5}$$

where: ε_m = the maximum strain at point A, and $\dot{u}_m$ the maximum particle velocity at point A.

In the case where u is perpendicular to the direction of x, either horizontally or vertically, one may differentiate equations 2 and 3 to obtain the expression for maximum curvature. It then follows:

$$\frac{\partial^2 u}{\partial x^2} = f''(x - ct) \tag{6}$$

$$\frac{\partial^2 u}{\partial t^2} = c^2 f''(x - ct) \tag{7}$$

They lead to:

$$\frac{\partial^2 u}{\partial x^2} = \frac{1}{c^2}\frac{\partial^2 u}{\partial t^2} \tag{8}$$

Thus,

$$\xi_m = \frac{\ddot{u}_m}{c^2} \tag{9}$$

where: ξ_m = the maximum curvature at point A, and $\ddot{u}_m$ = the maximum particle acceleration at A.

Having the maximum strains obtained from equations 5 and 9, the maximum stresses experienced by pipes or ducts may then be determined by the simple stress-strain relationship; i.e.,

$$S_{max} = (\varepsilon_m + \xi_m r)E$$

where: S_{max} = the maximum stress and E is the modulus of elasticity. r = the radius in case of a pipe and the distance of the extreme fiber to the neutral axis of the cross section in case of a duct.

3.7A.B.1.1.2 End-Connection Case

For the portion of a pipe or duct connected to a building, the behavior is a little different due to the fact that the mass of the structure is significant as compared to the surrounding soil, and there will be a relative movement of the structure to the surrounding soil. Two considerations may be undertaken:

- The case where the relative movement of the building is in the direction of the pipe or duct.

- The case where the relative movement of the building is perpendicular to the direction of the pipe or duct.

For the first case, methods are developed for straight and bent members, while for the second case, the method of beams on an elastic foundation is adopted.

3.7A.B.1.1.2.1 Relative Movement in the Direction of the Pipe or Duct

A. Straight Members

Consider a straight member connected to a building as shown in figure 3.7A.B-2. The stress, s, at a point with a distance, x, from the building is equal to:

$$s(x) = (P - FX)/A \quad \text{for } d < P/F$$

$$F = \gamma H \varphi \tan \alpha$$

where:

P = the end force.

F = the frictional force per unit length.

A = the cross-sectional area.

γ = the unit weight of soil.

H = the height of overburden soil.

ϕ = the perimeter of the member.

α = the frictional angle of the soil.

The total deformation of the member through the length, d, is then given by

$$\delta = \int_0^d \frac{s(x)}{E} dx = \int_0^d \frac{(P - Fx)}{EA} dx$$

$$\delta = \left(Pd - \frac{Fd^2}{2} \right) \Big/ EA$$

But, d = P/F and $\delta = \Delta x$

Thus, $P = \sqrt{2FEA\Delta x}$ (10)

Where: Δx = the relative movement of the building in the direction of member.

B. Bent Members

Consider a bent member connected to a building as shown in figure 3.7A.B-3. The total deformation of the member through a length, L, is given by:

$$\Delta L = \left(PL - FL^2/2\right)/EA$$

and R = P - FL

The net displacement of point A is equal to:

$$\delta = \Delta x - \frac{1}{EA}\left(PL - \frac{FL^2}{2} \right) \quad (11)$$

Now consider a beam of finite length with free ends on both sides surrounded by soil. The displacement at one end induced by a concentrated load P* acting at that end is equal to:

$$y = \frac{2P^* \lambda}{k} C \quad (12)$$

where:

$$C = \frac{\sinh\lambda h \cosh\lambda h - \sin\lambda h \cos\lambda h}{\sin^2 \lambda h - \sin^2 \lambda h}$$

k = bk_o for vertical movement.

= 0.5 bk_o for horizontal movement.[(4)]

K_o = modulus of subgrade reaction.

b = dimension of the contact width of the member.

h = length of the member.

λ = $\sqrt[4]{\frac{k}{4EI}}$

EI = flexural rigidity of the member.

To keep point A in equilibrium, it is necessary that the force

$$P^* = R = P - FL \qquad (13)$$

and the compatibility relationship at point A leads to:

$$y = \delta = \Delta x - \frac{1}{EA}\left(PL - \frac{FL^2}{2}\right) \qquad (14)$$

From equations 13 and 14, one obtains:

$$P = \frac{2EAk\Delta x + FL^2k + 4FL\lambda CEA}{2(2\lambda CEA + Lk)} \qquad (15)$$

It is noted that the above derivation involves a certain degree of approximation because of the assumption made in using equation 12. It is believed, however, that the effect of this approximation will have little significance in the result.

3.7A.B.1.1.2.2 Relative Movement Perpendicular to the Pipe or the Duct

Consider a member subjected to an end movement as shown in figure 3.7A.B-4. While the solution of this problem may be found in reference 3, a brief description is given below. Since all pipes are welded to the connections and flexible joints are inserted between the conduit inside the electrical ducts at connections, two end conditions are considered.

A. Fixed End

The end-conditioning force induced by a movement of fixed end is equal to:

$$P_O = -\frac{2k}{y}\Delta y \qquad (16)$$

Substituting this force to the solution of an infinite beam, one obtains:

Moment at any point of x

$$M(x) = \frac{k}{2\lambda^2}\Delta y\, C_{\lambda x} \qquad (17)$$

Shear at any point of x

$$Q(x) = -\frac{k}{\lambda}\Delta y D_{\lambda x} \qquad (18)$$

where:

$$C_{\lambda x} = e^{-\lambda x}(\cos\lambda x - \sin\lambda x)$$

$$D_{\lambda x} = e^{-\lambda x}\cos\lambda x$$

$C_{\lambda x}$ and $D_{\lambda x}$ are maximum at x = 0. Thus,

$$M_{max} = \frac{k}{2\lambda^2}\Delta y$$

$$Q_{max} = -\frac{k}{\lambda}\Delta y \qquad (19)$$

B. Hinged End

The end-conditioning force induced by a movement of hinged end is given by:

$$P_O = -\frac{2k}{\lambda}\Delta y$$

$$M_O = \frac{K}{\lambda^2}\Delta y \qquad (20)$$

Similarly, these forces will lead to a solution

$$M(x) = \frac{k}{2\lambda^2}\Delta y\left(D_{\lambda x} - C_{\lambda x}\right)$$

$$Q(x) = \frac{k}{\lambda}\Delta y\left(D_{\lambda x} - \frac{A_{\lambda x}}{2}\right) \tag{21}$$

where: $A_{\lambda x} = e^{-\lambda x}\left(\cos\lambda x + \sin\lambda x\right)$

The maximum values of M(x) and Q(x) are at

$\lambda x = \frac{\pi}{4}$ and $\lambda x = 0$, respectively. Thus,

$$M_{max} = 0.3224\frac{k}{2\lambda^2}\Delta y$$

$$Q_{max} = \frac{k}{2\lambda}\Delta y \tag{22}$$

The dynamic analysis of all Seismic Category I structures has been performed, and the relative displacement of the reactor building, control building, intake structure, and the diesel generator building obtained from the seismic analysis of these structures is presented in table 3.7A.B-1. Carbon steel pipe sleeves, 4 in. greater in diameter than the process pipes, are provided on the exterior walls of the Seismic Category I structures for each system piping connection. The piping passes through these penetrations at specified elevations which are at the centerline elevation of the sleeves. Before fuel loading, all of these penetrations will be sealed as shown on figure 3.7A.B-5. At the time of fuel loading, the major portion of the predicted settlement will have occurred. The remaining estimated consolidation of ~ 1/2 in. will take place over a period of years. The sand bedding below the piping in the vicinity of the penetration will minimize stresses in the seal welds. Using the method of analysis described above and the equation

$$S = \frac{P}{A} \pm \frac{Mr}{I}, \tag{23}$$

the stresses were computed for ducts and pipes in the free field and at the end of pipes when they were connected to the structures.

Computed stress intensities are shown in tables 3.7A.B-2 through 3.7A.B-4.

It is noted that in the case of hinged end, the maximum moment does not occur at the end. But equation 23 is still used in computing the stress simply because the axial force at the point of maximum moment is not expected to differ significantly in magnitude due to the frictional force.

It is also noted that for pipes, stresses obtained in both paragraphs 3.7A.B.1.1.1 and 3.7A.B.1.1.2 have to be added to the stress due to the internal pressure. The magnitude of this stress = pr/2t, where: p = the internal pressure, r = the radius of the pipe, and t = the thickness of the pipe.

3.7A.B.2 REVISED STRESS ANALYSIS OF INTAKE STRUCTURE BURIED PIPING AND CONCRETE DUCTS

The original static, thermal, internal pressure, and seismic analyses performed for the intake structure buried piping and concrete ducts were recalculated to reflect the structural properties of the new backfill material described in paragraph 2A.9.2.6. A finite element computer program[5] was used to obtain the static and thermal stresses. The methodology specified in subsection 3.7A.B.1 was followed for the seismic analysis. The structural properties of the K-Krete backfill material and the intake structure relative displacements (table 3.7A.B-1) were used to calculate seismic pipe and duct stresses at the intake structure wall pipe penetrations and duct end connections. The resulting seismic stresses for the relative displacement cases are given in table 3.7A.B-5. For the K-Krete covered portions of pipes and ducts far from the intake structure wall, free-field case analyses were performed. The resulting seismic stresses are given in table 3.7A.B-5. The maximum stresses obtained for all load cases are summarized in table 3.7A.B-6. The maximum combined stresses as shown in the same table are less than the allowable stresses specified in Section III of the American Society of Mechanical Engineers Code.

REFERENCES

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2. Newmark, N.M., "Problems in Wave Propagation in Soil and Rock," International Symposium on Wave Propagation and Dynamic Properties of Earth Material, Albuquerque, New Mexico, 1967.

3. Hetenyi, M., Beams on Elastic Foundation, The University of Michigan Press, 1946.

4. Richart, Jr., F.E., Hall, Jr., J.R., and Woods, R.D., Vibration of Soils and Foundations, Prentice-Hall, New Jersey, 1970.

5. Bechtel Structural Analysis Program, BSAR-User's Manual Version D, Bechtel Power Corporation, December 1979.

TABLE 3.7A.B-1

RELATIVE MOVEMENTS OF VARIOUS BUILDINGS

Building	Horizontal Movement (ft)	Vertical Movement (ft)
Reactor building at el 130 ft	0.0065	0.0053
Control building at el 130 ft	0.00272	0.00081
Intake structure at el 109 ft 9 in.	0.0036	0.00025
Diesel generator building at el 129 ft	0.009928	0.004993

TABLE 3.7A.B-2

STRESSES IN DUCTS BURIED IN THE FREE FIELD

Duct Size	σ_1(psi)	σ_2(psi)	σ_3(psi)
2 ft 8 1/4 in. x 2 ft 1 1/2 in.	500	468	4083
4 ft 4 1/2 in. x 2 ft 8 1/4 in.	510	459	4163
3 ft 9 3/4 in. x 2 ft 8 1/4 in.	507	462	4137
1 ft 11 in. x 1 ft 5 1/2 in.	496	473	4045
7 ft 4 1/4 in. x 4 ft 1/2 in.	533	436	4350
7 ft 7 in. x 5 ft 3 1/4 in.	532	437	4344
7 ft 4 1/4 in. x 5 ft 1 1/4 in.	536	432	4381

σ_1 = maximum concrete extreme fiber stress.

σ_2 = minimum concrete extreme fiber stress.

σ_3 = maximum steel stress.

TABLE 3.7A.B-3

STRESSES IN DUCTS CONNECTED TO BUILDING STRUCTURES

Duct Size	Connected to	σ_1(psi)	σ_2(psi)	σ_3(psi)	σ_4(psi)
7 ft 8 in. x 1 ft 6 in.	Reactor building	405	69	4345	10
6 ft 10 in. x 2 ft 2 in.	Control building	153	-5	1861	2
8 ft 6 in. x 2 ft 2 in.	Control building	113	-3	1470	2
10 ft 2 in. x 2 ft 2 in.	Control building	112	-2	1414	2
1 ft 2 in. x 1 ft 11 in.	Diesel generator building	554	-73	18,734	12
5 ft 6 in. x 1 ft 11 in.	Diesel generator building	424	16	20,588	9
1 ft 10 in. x 1 ft 11 in.	Diesel generator building	499	-43	19,679	11
4 ft 6 in. x 1 ft 11 in.	Diesel generator building	438	7	19,774	9
3 ft 6 in. x 1 ft 11 in.	Diesel generator building	446	-4	19,645	9
6 ft 6 in. x 1 ft 11 in.	Diesel generator building	421	23	21,215	9
13 ft 9 in. x 1 ft 10 in.	Diesel generator building	411	47	20,270	10
19 ft 1/2 in. x 1 ft 10 in.	Diesel generator building	401	55	20,698	9

σ_1 = maximum concrete extreme fiber stress.

σ_2 = minimum concrete extreme fiber stress.

σ_3 = maximum steel stress.

σ_4 = maximum shear stress.

TABLE 3.7A.B-4

STRESSES IN BURIED PIPE

Piping System	Pipe Size (in.)	Stresses in Free Field σ_1(psi)	σ_2(psi)	Stresses at End σ_1(psi)	σ_2(psi)	σ_3(psi)
To high-pressure coolant injection pump	16 ϕ x 0.188	4577	4450	8996	-2849	442
Residual heat removal service water (RHRSW)	18 ϕ x 0.5	7788	7645	11,713	4104	225
Plant service water (PSW)	6 ϕ x 0.28	-	-	11,311	1144	211
PSW	10 ϕ x 0.356	4834	4749	-	-	-
PSW	8 ϕ x 0.322	-	-	10,677	1197	217
Conduit to reactor core isolation cooling	6 ϕ x 0.134	4317	4264	15,719	1022	367
Standby gas	18 ϕ x 0.375	4353	4210	6293	-2576	279
Service water pipe to diesel generator building	8 ϕ x 0.322	4752	4684	15,783	-588	241
RHRSW to intake structure	30 ϕ x 0.375	6301	6062	4347	810	146
RHRSW to intake structure	18 ϕ x 0.5	7788	7645	8045	2531	91

σ_1 = maximum fiber stress.

σ_2 = minimum fiber stress.

σ_3 = maximum shear stress.

TABLE 3.7A.B-5

MAXIMUM STRESSES IN BURIED PIPES AND DUCTS DUE TO SEISMIC LOAD

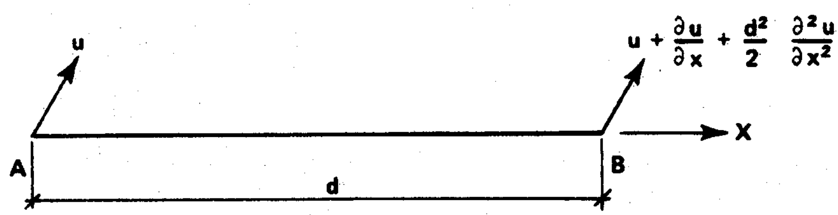
		Stresses At End (psi)	
Pipe/Duct Size	Stresses In Free Field (psi)	In Direction of Pipe/Duct Movement	Perpendicular To Pipe/Duct Movement
30 in. § pipe x 0.375 in.	5051	5150	425
18 in. § pipe x 0.500 in.	508	8940	1126
12 in. § pipe x 0.375 in.	5092	10,030	1260
2 in. § pipe x 0.218 in.	5115	12,860	1450
12 ft 3 in. x 3 ft duct	630	600	58

TABLE 3.7A.B-6

MAXIMUM STRESSES IN BURIED PIPES (K-KRETE BACKFILL)

	Location	A	B	C	D	E	Combination of Stresses[a]		
Pipe Size (in.)	Feet From Intake Structure	DL Stress + Shear Stress (psi)	Stress Due to Internal Pressure (psi)	Thermal Stress (psi)	Seismic End (psi)	Stress Free Field (psi)	A+B 15,000 (psi)	A+B+(D or E) 18,000 (psi)	A+B+C+ (D or E) 37,500 (psi)
30 x 0.375	0.0 In structure	30 + 502	2200	2960	5150	-	2732	7882	10,842
30 x 0.375	0.1 In K-Krete	18.5 + 502	2200	4875	5150	-	2721	7871	12,746
30 x 0.375	10.75 In K-Krete	650	2200	4630	-	5051	2850	7901	12,531
18 x 0.500	0.1 In K-Krete	7.2 + 314.3	3735	4875	8940	-	4056	12,996	17,871
12 x 0.375	0.1 In K-Krete	16.3 + 314	1190	4875	10,030	-	1520	11,550	16,425
2 x 0.218	0.1 In K-Krete	7.2 + 190	340	4875	12,860	-	537	13,400	18,275

a. From Section III, ASME Code.



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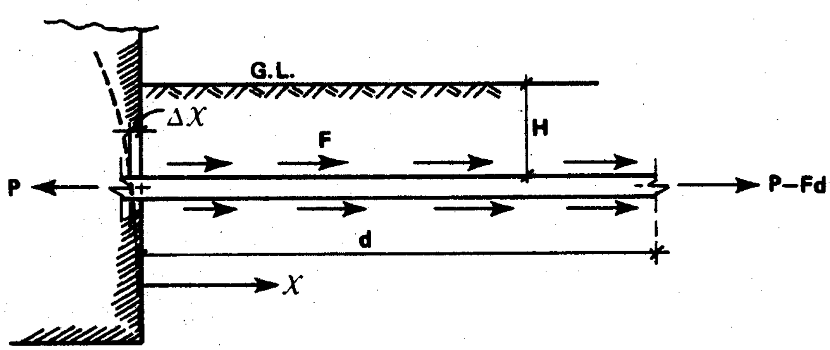
RELATIVE DISPLACEMENT

FIGURE 3.7A.B-1



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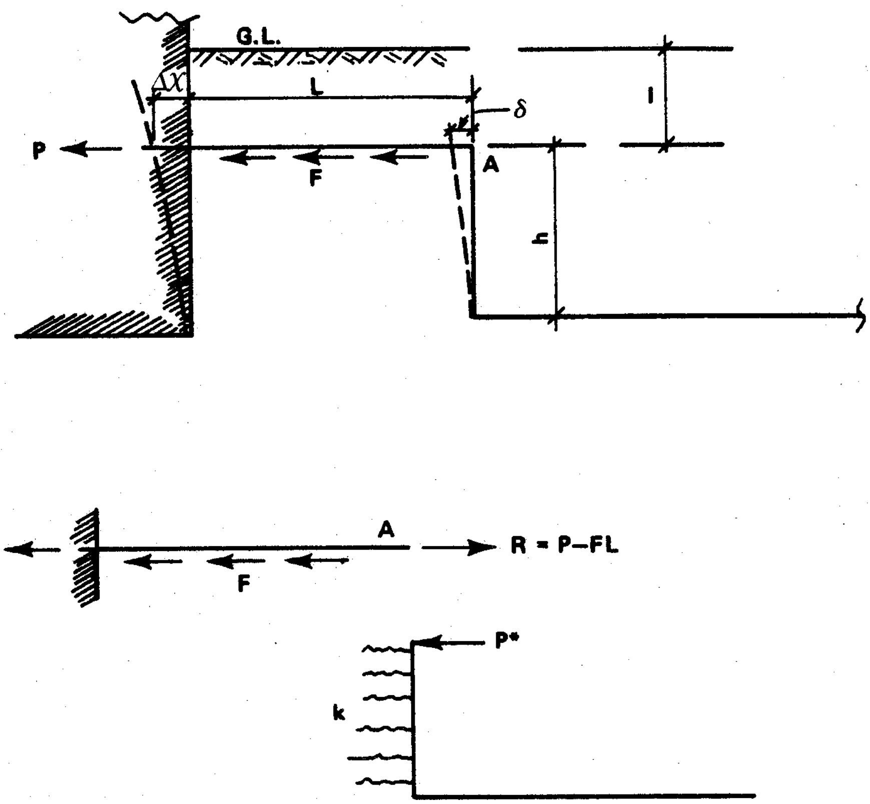
END MOVEMENT IN DIRECTION
OF MEMBER-STRAIGHT BAR

FIGURE 3.7A.B-2



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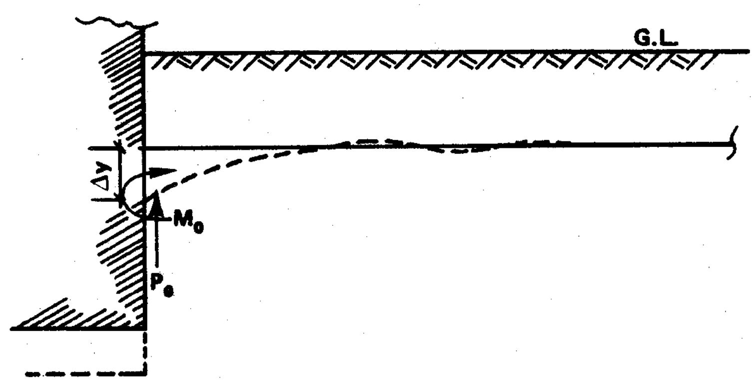
END MOVEMENT IN DIRECTION
OF MEMBER-BENT BAR

FIGURE 3.7A.B-3



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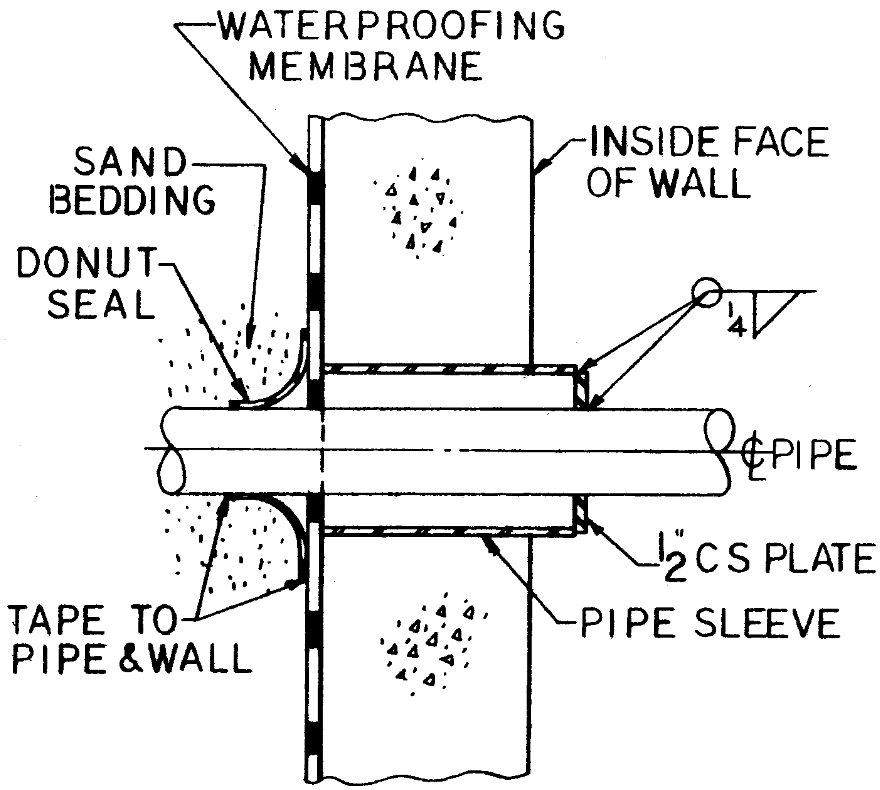
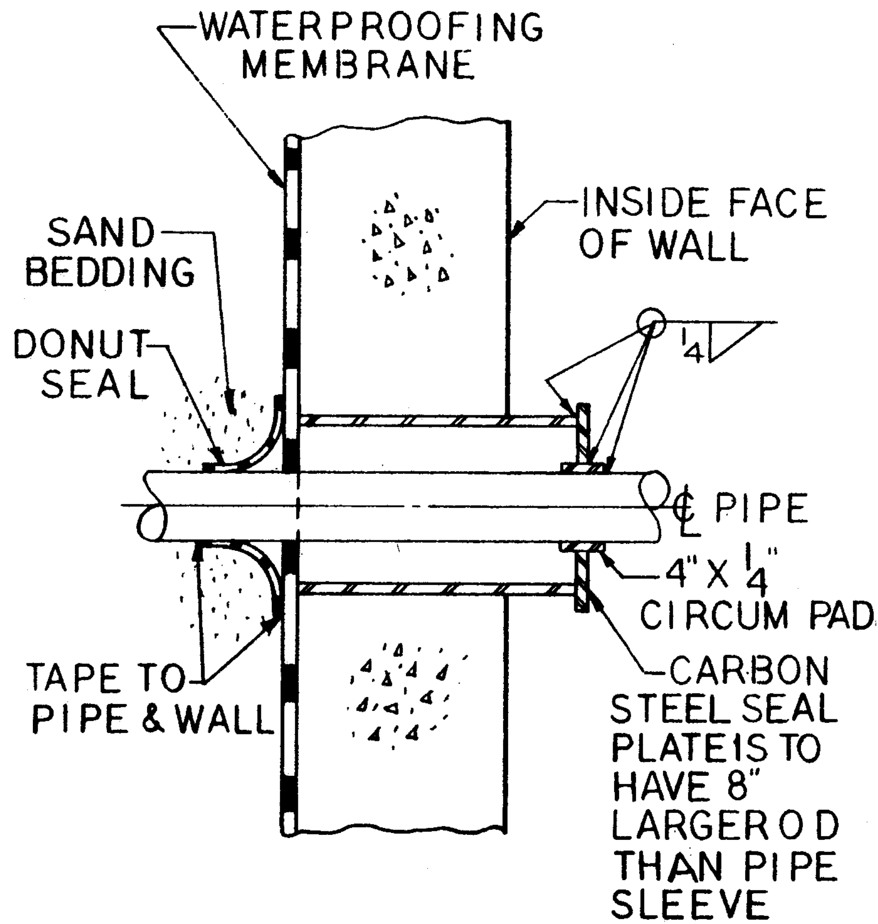
END MOVEMENT PERPENDICULAR
TO MEMBER

FIGURE 3.7A.B-4





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TYPICAL PIPE CONNECTION
TO CATEGORY I STRUCTURES

FIGURE 3.7A.B-5

SUPPLEMENT 3.7B

SEISMIC DESIGN - NUCLEAR STEAM SUPPLY SYSTEM

The seismic design of systems, components, and structures within the nuclear steam supply system (NSSS) scope of responsibility is presented in the following pages. The information presented in this supplement is intended to add to the information presented in supplement 3.7A in order to better differentiate responsibilities in the seismic design of Edwin I. Hatch Nuclear Plant-Unit 2 (HNP-2). As a result, not all subsections have a response but rather refer to the corresponding subsection in 3.7A.

3.7B.1 SEISMIC INPUT

3.7B.1.1 DESIGN RESPONSE SPECTRA

This subsection is covered in subsection 3.7A.1.1.

3.7B.1.2 DESIGN RESPONSE SPECTRA DEVIATION

This subsection is covered in subsection 3.7A.1.2.

3.7B.1.3 CRITICAL DAMPING VALUES

The damping factors indicated in table 3.7B-1 were used in the response analysis of various structures and systems and in preparation of floor response spectra used as forcing inputs for piping and equipment analysis or testing.

3.7B.1.4 BASES FOR SITE-DEPENDENT ANALYSIS

This subsection is covered in subsection 3.7A.1.4.

3.7B.1.5 SOIL-SUPPORTED SEISMIC CATEGORY I STRUCTURE

This subsection is covered in subsection 3.7A.1.5.

3.7B.1.6 SOIL-STRUCTURE INTERACTIONS

This subsection is covered in subsection 3.7A.1.6.

3.7B.2 SEISMIC SYSTEM ANALYSIS

3.7B.2.1 SEISMIC ANALYSIS METHODS

3.7B.2.1.1 Introduction

The modal-superposition method is used for the reactor vessel and internals, except as noted in paragraph 3.7B.2.1.4. This method involves two steps:

- The solution of the characteristic value problem represented by the free vibration response of the system.
- The transformation to normal coordinates utilizing the mode shapes of the system.

This procedure uncouples the equations of motion so that the response of the system in each individual mode may be evaluated independently.

The stress, strain, and deformation criteria are described in sections 3.8, 3.9, and 3.10.

3.7B.2.1.2 Equations of Dynamic Equilibrium

Assuming velocity proportional damping, the dynamic equilibrium equations for a lumped mass, distributed stiffness system are expressed in matrix form as:

$$[M]\{\ddot{u}(t)\} + [C]\{\dot{u}(t)\} + [K]\{u(t)\} = \{P(t)\} \qquad (1)$$

where:

$u(t)$ = time-dependent displacement of nonsupport points relative to the supports.

$\dot{u}(t)$ = time-dependent velocity of nonsupport points relative to the supports.

$\ddot{u}(t)$ = time-dependent acceleration of nonsupport points relative to the supports.

{M} = diagonal matrix of lumped masses.

[C] = damping matrix.

[K] = stiffness matrix.

P(t) = time-dependent inertial forces acting at nonsupport points.

The manner in which a distributed mass, distributed stiffness system is idealized into a lumped mass, distributed stiffness system of the building is described in supplement 3.7A, along with a

schematic representation of relative acceleration $\ddot{u}(t)$, support acceleration $\ddot{u}_s(t)$, and total acceleration $\ddot{u}_t(t)$.

3.7.B.2.1.2.1 Equations of Dynamic Equilibrium for Multi-Support Excitations of Piping, Systems, Components, and Equipment

Analytical procedures for obtaining force and displacement responses engendered by time-dependent base support excitation are discussed in the preceding sections. In a multi-support system, the relative motion among the individual multi-support points gives rise to time varying displacements at the nonsupport points.

The governing equations of motion of a multi-supported piping system, component, or equipment undergoing individual multi-support excitations may be expressed in the following matrix form:

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=\{F\} \qquad (2)$$

where:

$\{u\}$ = $\{u(t)\}$ = the corresponding dynamic model nodal displacement vector of absolute displacements.

The general case is considered in which k of the total n degrees-of-freedom corresponds to the individual multi-support points which undergo known time-history motions. The nodal displacement vector of absolute displacements can be partitioned and written as:

$$\{u\}=\left\{\frac{u_a}{\bar{u}_s}\right\}=\left\{\frac{u_a^d+u_a^s}{\bar{u}_s}\right\} \qquad (3)$$

where:

$\{u_a\}$ = absolute displacement vector of the active (unsupported) degrees-of-freedom.

$\{\bar{u}_s\}$ = known absolute displacement vector corresponding to the multi-supported degrees-of-freedom.

The vector $\{u_a\}$ in equation 3 was further separated into a dynamic part and a pseudo-static part

where:

$\left\{u_a^d\right\}$ = dynamic part of $\left\{u_a\right\}$

$\left\{u_a^d\right\}$ = pseudo-static part of $\left\{u_a\right\}$

Multi-support excitation may require the use of all modes which span the $\left\{u_a\right\}$ space of active (unsupported) degrees-of-freedom in the modal superposition to obtain reliable solutions of equation 2. Substitution of equation 3 enables the circumventing of that very costly requirement. Only the dynamic part $\left\{u_a^d\right\}$ is obtained by modal superposition which does not require all modes. The pseudo-static part $\left\{u_a^s\right\}$ is obtained from the known multi-support excitation.

The partition equations of motion are obtained by substituting equation 3 into equation 2 to yield:

$$\left[\begin{array}{c|c} M_a & 0 \\ \hline 0 & M_s \end{array}\right]\left\{\begin{array}{c} \ddot{u}_a^d+\ddot{u}_a^s \\ \hline \ddot{u}_s \end{array}\right\}+\left[\begin{array}{c|c} C_{aa} & C_{as} \\ \hline C_{sa} & C_{ss} \end{array}\right]\left\{\begin{array}{c} \dot{u}_a^d+\dot{u}_a^s \\ \hline \dot{u}_s \end{array}\right\}+\left[\begin{array}{c|c} K_{aa} & K_{as} \\ \hline K_{sa} & K_{ss} \end{array}\right]\left\{\begin{array}{c} u_a^d+u_a^s \\ \hline \overline{u} \end{array}\right\}=\left\{\begin{array}{c} F_a \\ \hline F_s \end{array}\right\} \qquad (4)$$

where:

$\left\{u_a^d\right\}$ = dynamic part (as defined by equation 3) of the absolute displacement vector of the active (unsupported) degree-of-freedom.

$\left\{u_a^s\right\}$ = pseudo-static part (as defined by equation 3 of the absolute displacement vector of the active (unsupported) degree-of-freedom.

$\left[M_a\right]$ and $\left[M_s\right]$ = lumped diagonal mass matrices associated with the active degrees-of-freedom and the multi-support points, respectively.

$\left[C_{aa}\right]$ and $\left[K_{aa}\right]$ = damping matrix and elastic stiffness matrix, respectively, relating the forces developed in the active degrees-of-freedom to the motion of the active degrees-of-freedom.

$\left[C_{ss}\right]$ and $\left[K_{ss}\right]$ = support forces due to unit velocities and displacements, respectively, of the multi-support points.

$\left[C_{as}\right]$ and $\left[K_{as}\right]$ = damping and stiffness matrices denoting the coupling forces developed in the active degrees-of-freedom due to the motion of the supports, vice versa.

$\{F_a\}$ = prescribed time-dependent applied load vector corresponding to the active degrees-of-freedom.

$\{\bar{F}_s\}$ = reaction force vector corresponding to the system multi-support points.

$(\dot{})$ = dot appearing over a time-varying variable indicates total differentiation with respect to time.

The procedure used to construct the damping matrix is discussed in paragraph 3.7.2.15(B). The mass matrix and elastic stiffness matrix are formulated by standard procedure.

Since the components of $\{\ddot{u}_s\}$, hence of $\{\dot{u}_s\}$ and $\{u_s\}$, are known functions of time, only the first partitioned equation of 4 is of interest.

$$[M_a]\{\ddot{u}_a^d\}+[M_a]\{\ddot{u}_a^s\}+[C_{aa}]\{\dot{u}_a^d\}+\boxed{\cancelto{0}{[C_{aa}]\{\dot{u}_a^s\}+[C_{as}]\{\dot{\bar{u}}_s\}}}+[K_{aa}]\{u_a^d\}+\boxed{\cancelto{0}{[K_{as}]\{u_a^s\}+[K_{as}]\{\bar{u}_s\}}}=\{F_a\} \qquad (5)$$

The pseudo-static displacement vector is written in terms of the multi-support displacement vector by taking

$$[K_{aa}]\{u_a^s\}+[K_{as}]\{\bar{u}_s\}=\{0\} \qquad (6)$$

Therefore,

$$\{u_a^s\}=-[K_{aa}]^{-1}[K_{as}]\{u_s\} \qquad (7)$$

It follows from equation 6 that

$$[C_{aa}]\{\dot{u}_a^s\}+[C_{as}]\{\dot{\bar{u}}_s\}=\{0\} \qquad (8)$$

The partitioned equation is reduced to its final form by substituting equations 6, 7, and 8 into equation 5 to yield:

$$[M_a]\{\ddot{u}_a^d\}+[C_{aa}]\{\dot{u}_a^d\}+[K_{as}]\{u_a^d\}=\{F_a\}+[M_a][K_{aa}]^{-1}[K_{as}]\{\ddot{\bar{u}}_s\} \qquad (9)$$

The solution in time of equation 9 for $\{u_a^d\}$ is readily obtained by the standard normal mode solution methodology. Once $\{u_a^d\}$ is obtained, the total solution for the absolute displacement vector $\{u_a\}$, corresponding to the active degrees-of-freedom, is given by substituting $\{u_a^d\}$ from equation 9 and $\{u_a^s\}$ from equation 7 into equation 3.

After obtaining the absolute displacement vector response of the active degrees-of-freedom, $\{u_a\}$, the second partitioned equation of 4 can be used to calculate the reaction force vector $\{\overline{F}_s\}$ corresponding to the multi-support degrees-of-freedom. That is,

$$\{\overline{F}_s\} = [M_s]\{\ddot{\overline{u}}_s\} + [C_{sa}]\{\dot{u}_a\} + [C_{ss}]\{\dot{\overline{u}}_s\} + [K_{sa}]\{u_a\} + [K_{ss}]\{\overline{u}_s\} \qquad (10)$$

Note that $\{\overline{F}_s\}$ is the total external force vector applied to the multi-support degrees-of-freedom required to produce the given multi-support excitation $\{\ddot{\overline{u}}_s\}$. The interaction force vector $\{F_s\}$ corresponding to the reaction of the active degrees-of-freedom portion of the dynamic model on the multi-support points is given by:

$$\{F_s\} = [C_{sa}]\{\dot{u}_a\} + [K_{sa}]\{u_a\} \qquad (11)$$

The interaction for vector $\{F_s\}$ can also be expressed in terms of the multi-support excitation input motion $\{\overline{u}_s\}$ by substituting equations 7 and 3 into equation 11 to yield:

$$\{F_s\} = [C_{sa}]\{\dot{u}_a^d\} + [K_{sa}]\{u_a^d\} - [C_{sa}][K_{aa}]^{-1}[K_{as}]\{\dot{\overline{u}}_s\} - [K_{sa}][K_{aa}]^{-1}[K_{as}]\{\overline{u}_s\} \qquad (12)$$

3.7B.2.1.3 <u>Solution of the Equations of Motion by Mode Superposition</u>

The second technique used for the solution of the equations of motion is the method of modal superposition.

The set of homogeneous equations represented by the undamped free vibration of the system is:

$$[M]\{\ddot{u}(t)\} + [K]\{u(t)\} = 0 \qquad (13)$$

Since the free oscillations are assumed to be harmonic, the displacements can be written as:

$$\{u(t)\} = \{\phi\}e^{i\omega t} \qquad (14)$$

where:

$\{\phi\}$ = column matrix of the amplitude of displacements $\{u\}$.

ω = circular frequency of oscillation.

t = time.

Substituting equation 14 and its derivatives in equation 13 and noting that $e^{i\omega t}$ is not necessarily zero for all values of ωt yields:

$$\left[-\omega^2[M]+[K]\right]\{\phi\}=\{0\} \tag{15}$$

Equation 15 is the classical algebraic eigenvalue problem wherein the eigenvalues yield the frequencies of vibration, ω_i, and the eigenvectors are the mode shapes, $\{\phi\}_i$.

For each frequency ω_i there is a corresponding solution vector $\{\phi\}_i$. It can be shown that the mode shape vectors are orthogonal with respect to the weighting matrix [K] in the n-dimensional vector space.

The mode shape vectors are also orthogonal with respect to the mass matrix [M].

The orthogonality of the mode shapes is used to effect a coordinate transformation of the displacements, velocities, and accelerations so that the response in each mode is independent of the response of the system in any other mode. Thus, the problem becomes one of solving n independent differential equations rather than n simultaneous differential equations; and, since the system is linear, the principle of superposition holds, and the total response of the system oscillating simultaneously in n modes is determined by direct addition of the responses in the individual modes.

3.7B.2.1.4 Analysis by Response Spectrum

As an alternative to the step-by-step mode superposition method described in paragraph 3.7B.2.1.3, the response spectrum method is used. The response spectrum method is based on the fact that the modal responses can be expressed as a set of integral equations rather than as a set of differential equations. The advantage of this form of solution is that for a given ground motion the only variables under the integral are the damping factor and the frequency. Thus, for a specified damping factor, it is possible to construct a curve that gives a maximum value of the integral as a function of frequency. This curve is called a response spectrum for the particular input motion and the specified damping factor.

Using the calculated natural frequencies of vibration of the system, the maximum values of the modal responses are determined directly from the appropriate response spectrum. The modal maxima are then combined as discussed in subsection 3.7B.3.4.

The calculated maximum responses due to one horizontal directional earthquake excitation are combined with the responses due to the vertical earthquake by the sum of the absolute values method. The maximum responses due to another perpendicular horizontal earthquake are also combined with the responses due to the vertical earthquake in the same manner. The larger of the two values is used for design. The basis of combining loads is discussed in subsection 3.7B.3.7.

Contributions due to the three spatial components of seismic excitation are combined, as described in paragraph 3.7B.2.1.7.2, for replacement recirculation piping.

3.7B.2.1.5 Support Displacements in Multisupported Structures

The preceding sections have discussed analysis procedures for forces and displacement induced by time-dependent support accelerations. In a multisupported structure there are, in addition, time-dependent support displacements, which produce additional displacements at nonsupport points and pseudostatic forces at both support and nonsupport points. The total force vector due to both support accelerations and support displacements are given by:

$$\begin{Bmatrix} F(t) \\ F_s(t) \end{Bmatrix} = \left[\overline{K}\right] \begin{Bmatrix} u(t) \\ u_s(t) \end{Bmatrix} \tag{16}$$

where:

$F(t)$ = time-dependent forces at nonsupport points.

$F_s(t)$ = time-dependent forces at support points (reactions).

$u(t)$ = time-dependent displacements at nonsupport points due to support accelerations.

$u_s(t)$ = time-dependent displacements at support points.

$\left[\overline{K}\right]$ = stiffness matrix of the free structure, i.e., a singular matrix, and it is built up from the static stiffness coefficients of each element without the application of displacement boundary conditions.

Similarly, the total or absolute displacement of nonsupport points is given by:

$$\{u_t(t)\} = \{u(t)\} + [R]\{u_s(t)\} \tag{17}$$

where:

$\{u_t(t)\}$ = total displacement.

$[R]$ = transformation matrix that relates displacements at nonsupport points due to unit displacements at support points.

3.7B.2.1.6 <u>Modeling Techniques for Seismic Category I Structures, Systems, and Components</u>

An important step in the seismic analysis of Seismic Category I systems or structures is the procedure used for modeling. The techniques currently being used are represented by lumped masses and a set of spring dashpots idealizing both the inertia and stiffness properties of the system. The details of the mathematical models are determined by the complexity of the actual structures and the information required for the analysis. The input data of the building is provided in supplement 3.7A.

3.7B.2.1.6.1 Modeling of Piping Systems

The continuous piping system is modeled as an assemblage of beams. The mass of each beam is lumped at the nodes connected by a weightless elastic member, representing the physical properties of each segment. The pipe lengths between mass points are no greater than the length that would have a natural frequency of 33 Hz when calculated as a simply supported beam with uniformly distributed mass. All concentrated weights on the piping system such as main valves, relief valves, pumps, and motors are modeled as lumped masses. The torsional effects of the valve operators and other equipment with offset centers of gravity with respect to centerline of the pipe is included in the analytical model. If the torsional effect is expected to cause pipe stresses < 500 psi, this effect may be neglected.

3.7B.2.1.6.2 Modeling of Equipment

For dynamic analysis, Seismic Category I equipment is represented by lumped mass systems, which consist of discrete masses connected by weightless springs. The criteria used to lump masses are as follows:

A. The number of modes of a dynamic system is controlled by the number of masses used. Therefore, the number of masses is chosen so that all significant modes are included. The modes are considered as significant if the corresponding natural frequencies are < 33 Hz and the stresses calculated from these modes are > 10% of the total stresses obtained from lower modes.

B. Mass is lumped at any point where a significant concentrated weight is located. Examples are the motor in the analysis of pump motor stand, the impeller in the analysis of pump shaft, etc.

C. If the equipment has a free-end overhang span whose flexibility is significant compared to the center span, a mass is lumped at the overhang span.

D. When a mass is lumped between two supports, it is located at a point where the maximum displacement is expected to occur. This tends to conservatively lower the natural frequencies of the equipment. Similarly, in the case of live loads (mobile) and a variable support stiffness, the location of the load and the magnitude of support stiffness are chosen so as to yield the lowest frequency

content for the system. This is to ensure conservative dynamic loads since equipment frequencies are such that the floor spectra peak is in the lower frequency range. If this is not the case, the model is adjusted to give more conservative results.

3.7B.2.1.6.3 Modeling of Reactor Pressure Vessel and Internals

The seismic loads on the reactor pressure vessel (RPV) and internals are based on a dynamic analysis of an entire RPV building complex with the appropriate forcing function supplied at ground level. The seismic model of the RPV and internals is given in figure 3.7B-1.

This mathematical model consists of lumped masses connected by elastic (linear) members. Using the elastic properties of the structural components, the stiffness properties of the model are determined. This includes the effects of both bending and shear. To facilitate hydrodynamic mass calculations, several mass points (fuel, shroud, vessel) are selected at the same elevation. The various lengths of control rod drive (CRD) housings are grouped into the two representative lengths shown. These lengths represent the longest and shortest housings in order to adequately represent the full range of frequency response of the housings. The high fundamental natural frequencies of the CRD housings result in very small seismic loads. Furthermore, the small frequency differences between the various housings due to the length differences result in negligible differences in dynamic response. Hence, the modeling of intermediate length members becomes unnecessary. Not included in the mathematical model are light components such as jet pumps, incore guide tubes and housings, spargers, and their supply headers. This is done to reduce the complexity of the dynamic model. If the seismic responses of these components are needed, they can be determined after the system response has been found.

The presence of a fluid and other structural components; e.g., fuel within the RPV, introduces a dynamic coupling effect. Dynamic effects of water enclosed by the RPV are accounted for by introduction of a hydrodynamic mass matrix, which serves to link the acceleration terms of the equations of motion of points at the same elevation in concentric cylinders with a fluid entrapped in the annulus. The seismic model of the RPV and internals has two horizontal coordinates for each mass point considered in the analysis. The remaining translational coordinate (vertical) is excluded, because the vertical frequencies of RPV and internals are well above the significant horizontal frequencies. Furthermore, all support structures, building and containment walls, have a common centerline, and, hence, the coupling effects are negligible. A separate vertical analysis is performed as discussed in sections 3.7B.2 and 3.7B.3. Dynamic loads due to vertical motion are added to or subtracted from the static weight of components, whichever is the more conservative. The two rotational coordinates about each node point are excluded because the moment contribution of rotary inertia from surrounding nodes is small. Since all deflections are assumed to be within the elastic range, the rigidity of some components may be accounted for by equivalent linear springs.

The shroud support plate is loaded in its own plane during a seismic event and, hence, is extremely stiff and may be modeled as a rigid link in the translational direction. The shroud support gussets and the local flexibilities of the RPV and shroud contribute to the rotational flexibilities and are modeled as an equivalent torsional spring.

The RPV system model assumed use of 100-mil-thick fuel assembly channels. The effects of using 80-mil channels on RPV internal loadings were evaluated[1] and found to be negligible.

3.7B.2.1.6.4 Vertical Seismic Analysis

The seismic loads acting on the structures within the RPV are based on a vertical dynamic analysis of a model shown in figure 3.7B-1.

The mathematical model represents the RPV, RPV internals, pedestal, and the shield wall. The system is represented by lumped masses and a set of springs idealizing both the inertial and stiffness properties of the system. Between mass points, the structural properties are reduced to uniform beam segments of cross-sectional area, effective shear area, and moment of inertia. The base is considered to be fixed. The effect of the surrounding water inside the RPV is included by applying concentrated mass unit to the node points in the mathematical model.

Seismic analysis is performed to determine the system natural frequencies and mode shapes. The relative displacement, acceleration, and load response are then determined for each node of interest and for each mode of vibration. The square-root-of-the-sum-of-the-squares (SRSSs) of these responses is then used for design calculations.

As shown in table 3.9-4, vertical seismic loads are applied to the reactor internals as a function of component weight statically applied. Because the fuel assembly with 80-mil channels weighs ~ 2% less than with 100-mil channels, the loads reported in table 3.9-4 conservatively bound the vertical seismic loads that would result if 80-mil channels were used.

3.7B.2.1.7 Dynamic Analysis of Seismic Category I Structures, Systems, and Components

Time-history techniques and the response spectrum technique are used as applicable for the dynamic analysis of Seismic Category I structures, systems, and components that are sensitive to dynamic seismic events.

3.7B.2.1.7.1 Dynamic Analysis of Piping Systems

Each pipeline is idealized as a mathematical model consisting of lumped masses connected by elastic members. The stiffness matrix for the piping system is determined using the elastic properties of the pipe. This includes the effects of torsional, bending, shear, and axial deformations as well as change in stiffness due to curved members. Next the mode shapes and the undamped natural frequencies are obtained. The dynamic response of the system is calculated by using the response spectrum method of analysis. When the piping system is being anchored and supported at points with different excitation, the response spectrum analysis is performed using the response spectrum above the center of mass of the piping system.

The relative displacement between anchors is determined from the dynamic analysis of the structures. The results of the relative anchor point displacement are used for a static analysis to determine the additional stresses due to relative anchor point displacements.

The dynamic model of the steam line piping system is given in figure 3.7B-2.

3.7B.2.1.7.2 Dynamic Analysis of Replacement Recirculation Piping

A. The replacement recirculation piping seismic design adequacy evaluation is completed by applying the multi-support excitation response spectrum methodology associated with the theoretical development described in paragraph 3.7B.2.1.2.1.

B. The continuous piping system is modeled as an assemblage of one-dimensional straight- or curved-pipe elements. The mass of each pipe element is lumped at its end nodes. The mass nodes are interconnected by an assemblage of weightless pipe elements. The weightless pipe element section properties reflect the stiffness characteristics of the corresponding segments of the piping system. The pipe element lengths between mass points are no greater than that of a simply supported beam of uniformly distributed mass having a fundamental frequency of 33 Hz. In addition, mass nodes are located at ends of elbows, tees, and at all pipe-mounted components, such as valves, pumps, and motors. The rotational effects of valve operators and other equipment with offset centers of gravity are included in the analytical model.

C. To account for the effects of parameter variations on the primary structure (reactor building) calculated frequencies, the piping input response spectra are peak broadened ±10%.

D. Maximum colinear response contributions due to the three spatial components of seismic excitation are combined by the SRSSs method.

E. Peak modal responses are combined by the double sum method which accounts for the effects of closely spaced modes. This method is defined mathematically by:

$$R = \left[\sum_{k=1}^{N} \sum_{s=1}^{N} \left| R_k R_s \right| \in_{ks} \right]^{1/2} \qquad (18)$$

where:

R = representative maximum value of a particular response in a given element due to a given component of excitation.

R_k = peak value of the response of the element due to the k^{th} mode.

N = number of significant modes included in the modal superposition.

R_s = peak value of the element response contributed by the sth mode.

Also,

$$\in_{ks}\left[1+\left(\frac{\omega'_k-\omega'_s}{\beta_k\omega_k+\beta_s\omega_s}\right)^2\right]^{-1} \tag{19}$$

in which

$$\omega'_k=\omega_k\left(1-\beta_k^2\right) \tag{20}$$

and

$$\beta'_k=\beta_k+\frac{2}{t_{dk}} \tag{21}$$

The quantities ω_k and β_k are the modal frequency and modal damping coefficient, respectively, for the kth mode; t_d is the duration of the earthquake. If there are no closely spaced modes, the double sum and SRSSs methods yield identical results.

F. Piping structural damping, specified in table 3.7B-1, is 0.5% for operating basis earthquake (OBE) and 1.0% for design basis earthquake (DBE).

G. The cut-off frequency for the seismic analysis is 33 Hz which corresponds approximately to the zero period acceleration frequency of the floor response spectra. The seismic free-field input motion for OBE and DBE analyses performed for HNP-2 are defined in figures 3.7A-1 and 3.7A-2.

H. Combination of Primary and Secondary Stresses - The inertia and displacement effects due to differential support motion are dynamic in nature, and their peak values have a very low probability of occurring at the same instant in time. Therefore, a combination of the peak inertia and the differential anchor displacement responses is conservative. Moreover, anchor movement effects are computed from static analyses in which the displacements are applied in the most unfavorable manner possible to produce the most conservative responses. In view of this, inertia and displacement effects can be combined by the SRSSs method.

I. Multi-Support Response Spectrum Analysis - The theoretical basis for the multi-support excitation methodology is described in paragraph 3.7B.2.1.2.1. For the multi-support response spectrum method, all input motions corresponding to piping support points located at the same elevation of the same primary structure substructure (e.g., RPV inlet manifold) are assumed to be 100% correlated. Consequently, corresponding response contributions due to each of the input motions are algebraically combined in the computer analysis.

Piping input motions for supports located at different elevations on the same substructure or located on different substructures are assumed to be uncorrelated in the analysis. Associated response contributions for the uncorrelated input motions are combined in the analysis by the SRSSs method.

In the computer analysis, different support points with the same input motion designation number are treated as correlated. Corresponding responses due to each of these identical input motions are consequently algebraically combined. Support points with different input motion designation numbers are treated as uncorrelated, and corresponding responses are combined by the SRSSs method.

3.7B.2.1.7.3 Dynamic Analysis of Equipment

Equipment is idealized as a mathematical model consisting of lumped masses connected by elastic members or springs. Results for some selected large Seismic Category I equipment are given in table 3.7B-2.

Seismic loadings due to two orthogonal horizontal directions and the vertical are combined as detailed in paragraph 3.7B.2.1.4.

When the equipment is supported at more than two points located at different elevations in the building, the response spectra at the elevation near the center of gravity of the equipment is chosen as the design spectra.

The relative displacement between supports is determined from the dynamic analysis of the structure. The relative support point displacements are used for a static analysis to determine the additional stresses due to support displacements. Further details are given in subsection 3.7B.2.7.

3.7B.2.1.8 Seismic Qualification by Testing

For certain Seismic Category I equipment and components where dynamic testing is necessary to ensure functional integrity, test performance data and results reflect the following:

- Performance data of equipment that, under the specified conditions, has been subjected to dynamic loads equal to or greater than those to be experienced under the specified seismic conditions.

- Test data from previously tested comparable equipment that, under similar conditions, was subjected to dynamic loads equal to or greater than those specified.

- Actual testing of equipment in accordance with one of the methods described in sections 3.9 and 3.10.

Alternate test procedures that satisfy the requirements of these criteria are allowed.

3.7B.2.2 NATURAL FREQUENCIES AND RESPONSE LOADS

This subsection is covered in subsection 3.7A.2.2.

3.7B.2.3 PROCEDURES USED TO LUMP MASSES

Paragraph 3.7B.2.1.6.2 discusses criteria used by General Electric in lumping masses.

3.7B.2.4 ROCKING AND TRANSLATIONAL RESPONSE SUMMARY

This subsection is covered in subsection 3.7A.2.4.

3.7B.2.5 METHODS USED TO COUPLE SOIL WITH SEISMIC-SYSTEM STRUCTURES

This subsection is covered in subsection 3.7A.2.5.

3.7B.2.6 DEVELOPMENT OF FLOOR RESPONSE SPECTRA

This subsection is covered in subsection 3.7A.2.6.

3.7B.2.7 DIFFERENTIAL SEISMIC MOVEMENT OF INTERCONNECTED COMPONENTS

The procedure for considering differential displacements for equipment anchored and supported at points with different displacement excitation is discussed in the following paragraphs.

The relative displacements between the supporting points induce additional stresses in the equipment supported at these points. These stresses can be evaluated by performing a static analysis where each of the supporting points is displaced a prescribed amount. From the dynamic analysis of the complete structure, the time history of displacement at each supporting point is available. These displacements are used to calculate stresses. The time history of stresses thus obtained is a superposition of all modal displacements of the structure at each instant of time.

In the static calculation of the stresses due to relative displacements in the response spectrum method, the maximum value of the modal displacement is used. Therefore, the mathematical model of the equipment is subjected to a maximum displacement at its supporting points obtained from the modal displacements. This procedure is repeated for the significant modes (modes contributing most to the total displacement response at the supporting point) of the structure. The total stresses due to relative displacement are obtained by combining the modal results using the SRSSs method. Since the maximum misplacements for different modes do not occur at the same time, the SRSSs method is a realistic and practical method.

When a component is covered by the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, the stresses due to relative displacement as obtained above are treated as secondary stresses.

3.7B.2.8 EFFECTS OF VARIATIONS ON FLOOR RESPONSE SPECTRA

Except for replacement recirculation piping input spectra, this paragraph is covered in subsection 3.7A.2.8. Peak broadening for the replacement recirculation piping is covered in paragraph 3.7B.2.1.7.2.

3.7B.2.9 USE OF CONSTANT VERTICAL LOAD FACTORS

This subsection is covered in subsection 3.7A.2.9.

3.7B.2.10 METHODS USED TO ACCOUNT FOR TORSIONAL EFFECTS

This subsection is covered in subsection 3.7A.2.10.

3.7B.2.11 COMPARISON OF RESPONSES

The comparison between the calculated seismic load in the RPV and internals is given in table 3.7B-3.

3.7B.2.12 METHODS FOR SEISMIC ANALYSIS OF DAMS

This subsection is covered in subsection 3.7A.2.12.

3.7B.2.13 METHODS USED TO DETERMINE SEISMIC CATEGORY I STRUCTURE OVERTURNING MOVEMENTS

This subsection is covered in subsection 3.7A.2.13.

3.7B.2.14 ANALYSIS PROCEDURE FOR DAMPING

This subsection is covered in subsection 3.7A.2.14.

3.7B.3 SEISMIC SUBSYSTEM ANALYSIS

3.7B.3.1 DETERMINATION OF NUMBER OF EARTHQUAKE CYCLES

To evaluate the number of cycles that exist within a given earthquake, a typical boiling water reactor building-reactor dynamic model was excited by three different recorded time histories - May I8, I940, El Centro NS component 29.4 s; 1952, Taft, N 69° W component, 30 s; and March 1957, Golden Gate S80E component, 13.2 s. The modal response was truncated so that the response of three different frequency bandwidths could be studied: 0 to 10 Hz, 10 to 20 Hz, and 20 to 50 Hz. This was done to give a good approximation to the cyclic behavior expected from structures with different frequency content.

By enveloping the results from the three earthquakes and by averaging the results from several different points of the dynamic model, the cyclic behavior, as given in table 3.7B-4, was determined.

Independent of earthquake or component frequency, 99.5% of the stress reversals occur below 75% of the maximum stress level, and 95% of the reversals lie below 50% of the maximum stress level. This relationship is graphically shown in figure 3.7B-3.

In summary, the cyclic behavior number of fatigue cycles of a component during an earthquake is found in the following manner:

A. The fundamental frequency and peak seismic loads are found by a standard seismic analysis.

B. The number of cycles that the component experiences are found in table 3.7B-4 according to the frequency range within which the fundamental frequency lies.

C. For fatigue evaluation, 0.5% of these cycles is conservatively assumed to be at the peak load 4.5% at three-quarter peak. The remainder of the cycles will have negligible contribution to fatigue usage.

The DBE has the highest level of response. However, the encounter probability of the DBE is so small that it is not necessary to postulate the possibility of more than one DBE during the 40-year life of a plant. Fatigue evaluation due to the DBE is not necessary since it is a faulted condition and thus not required by ASME Code, Section III.

The OBE is an upset condition and, therefore, must be included in fatigue evaluations according to ASME Code, Section III. Investigation of seismic histories in preliminary safety analysis reports (PSARs) of many plants shows that during a 40-year life it is probable that five earthquakes with intensities one-tenth of the DBE intensity and one earthquake ~ 20% of the proposed DBE intensity will occur. Therefore, the probability of even an OBE is extremely low. To cover the combined effects of these earthquakes and the cumulative effects of even lesser earthquakes, one OBE intensity earthquake is postulated for fatigue evaluation.

3.7B.3.2 BASIS FOR SELECTION OF FORCING FREQUENCIES

All frequencies in the range of 0.25 Hz to 33 Hz are considered in the analysis and testing of structures, systems, and components.

3.7B.3.3 SQUARE-ROOT-OF-THE-SUM-OF-THE-SQUARES

The SRSSs combination of modal responses is defined mathematically as:

$$R = \sqrt{\sum_{i=1}^{n} (R_i)^2} \tag{22}$$

where:

R = combined response.

R_i = response in the i^{th} mode.

n = number of modes considered in the analysis.

3.7B.3.4 PROCEDURE FOR COMBINING MODAL RESPONSES

When the response spectra method of modal analysis is used, all modes are combined by the SRSSs method as described in paragraph 3.7B.2.1.4.

Modal responses for the replacement recirculation piping seismic analysis are combined by the double sum method as described in paragraph 3.7B.2.1.7.2.

3.7B.3.5 SIGNIFICANT DYNAMIC RESPONSE MODES

When the natural frequency of a structure or component is unknown, it may be analyzed by applying a static force at the center of mass. To conservatively account for the possibility of more than one significant dynamic mode, the static force is calculated as 1.5 times the mass times the maximum acceleration from the response spectra of the point of attachments of multi-span structures. For structures that be reasonably approximated by a single-degree-of-freedom model, the peak spectral acceleration is used.

3.7B.3.6 DESIGN CRITERIA AND ANALYTICAL PROCEDURES FOR PIPING

This subsection is covered in subsection 3.7A.3.6.

3.7B.3.7 BASIS FOR COMPUTING COMBINED RESPONSE

The two horizontal components and one vertical component of ground motion are accounted for by obtaining two sets of seismic results. First, the maximum value of the horizontal component of the earthquake is assumed to act in one horizontal direction simultaneous with the vertical component, and the loads are computed for this combination. Next, the maximum value of the horizontal component of the earthquake is assumed to act perpendicular to the direction previously assumed and simultaneous with the vertical component, and loads are computed for this combination. The larger of these two loads at each point in the system is used for design.

This method of analysis is based on the fact that the seismologist specifies the maximum resultant value of the horizontal component of the earthquake when specifying the horizontal component of the DBE. This method conservatively assumes that the horizontal and vertical components of the earthquake response occur simultaneously.

For the replacement recirculation piping seismic analysis, the corresponding colinear responses due to the three spatial components of seismic excitation are combined by the SRSSs method as described in paragraph 3.7B.2.1.7.2.

3.7B.3.8 AMPLIFIED SEISMIC RESPONSES

This subsection is covered in subsection 3.7A.3.8.

3.7B.3.9 USE OF SIMPLIFIED DYNAMIC ANALYSIS

For equipment and piping supplied or analyzed by the General Electric Company, a simplified dynamic analysis is not used.

3.7B.3.10 MODAL PERIOD VARIATION

This subsection is covered in subsection 3.7A.3.10.

3.7B.3.11 TORSIONAL EFFECTS OF ECCENTRIC MASSES

Torsional effects of eccentric masses are discussed in paragraph 3.7B.2.1.6.1.

When the torsional effect of an eccentric mass is likely to have a significant effect on the results of an analysis, the eccentric mass is included in the analytical mode. If the pipe stresses due to an eccentric mass are expected to be insignificant, the offset moment due to the eccentric mass is usually negligible.

3.7B.3.12 PIPING OUTSIDE CONTAINMENT STRUCTURE

This subsection is covered in subsection 3.7A.3.12.

3.7B.3.13 INTERACTION OF OTHER PIPING WITH SEISMIC CATEGORY I PIPING

When other piping is attached to Seismic Category I piping, the other piping is analytically simulated in a manner that does not significantly degrade the accuracy of the analysis of the Seismic Category I piping. Furthermore, the other piping is designed to withstand the DBE without failing in a manner that would cause the Seismic Category I piping to fail.

3.7B.3.14 FIELD LOCATION OF SUPPORTS AND RESTRAINTS

The field location of seismic supports and restraints for Seismic Category I piping and piping system components is selected to satisfy the following two conditions:

A. The location selected must furnish the required response to control strain within allowable limits.

B. Adequate building strength for attachment of the components must be available.

The final location of seismic supports and restraints for Seismic Category I piping, piping system components, and equipment, including the placement of snubbers, is checked against the drawings and instructions issued by the engineer. An additional examination of these supports and restraints devices by an engineer who is competent in the design of Seismic Category I systems and components is made to assure that the location and characteristics of these supports and restraining devices are consistent with the dynamic and static analyses of the systems.

3.7B.3.15 SEISMIC ANALYSIS FOR FUEL ASSEMBLIES, CONTROL RODS, AND CRDs

The seismic analysis of the reactor is described in paragraph 3.7B.2.1.6.3.

3.7B.4 SEISMIC INSTRUMENTATION PROGRAM

This section is covered in section 3.7A.4.

3.7B.5 SEISMIC DESIGN CONTROL

3.7B.5.1 SEISMIC INPUT DATA OF PURCHASE SPECIFICATION FOR SEISMIC CATEGORY I COMPONENTS AND EQUIPMENT

The seismic design specification includes criteria for application to purchase equipment. These criteria stipulate the minimum design requirements for Seismic Category I equipment attached to the structure in which it is located.

The seismic criteria are in the form of conservative static coefficients based on expected response spectra from seismic history data.

The responsible equipment design engineers include the appropriate static responses in the purchase specifications. Guidance and counsel of expert dynamicists are available to the design engineers. The applicable discipline chief engineer or expert dynamicist design review includes purchase specifications for proper inclusion and display of necessary data.

3.7B.5.2 PROGRAM FOR AUDITING VENDOR SEISMIC ANALYSIS AND TESTS OF SEISMIC CATEGORY I COMPONENTS AND EQUIPMENT

The responsible equipment design engineers ensure the adequacy and validity of the analyses and/or tests employed by the vendors of Seismic Category I components and equipment.

Vendor analyses and/or test plans and data are submitted to the responsible design engineer. The material to be submitted has been agreed upon as a part of the purchase specification. The responsible engineer agrees with the submitted material, in writing, only after he is satisfied that it meets the design specification requirements. Guidance and counsel of expert dynamicists from the applicable discipline chief engineer's staff are made available to the responsible design engineers in the course of such reviews.

3.7B.5.3 EQUIPMENT TESTING AND TEST EVALUATION

Seismic Category I equipment, which is difficult to represent in a mathematical model for calculations or which was required to demonstrate its ability to remain operating without changing the mode or its operation (such as level switch, which should not switch from on to off or vice versa during the earthquake) was subjected to actual vibration inputs on shake tables.

These shake tests were performed by qualified laboratories for the equipment suppliers.

The seismic qualification in the laboratory generally followed the same procedures, which consisted of the following:

A. The equipment was mounted on the shake table in such a manner as to represent its installed condition.

B. Sine sweep tests were performed, covering all practical frequency ranges with constant or variable acceleration levels to determine the resonance frequencies of the equipment. This procedure permits the determination of the predominant mode periods by monitoring the output response.

C. With the predominant period thus obtained, it was used for the determination of the necessary acceleration levels, which were obtained from the applicable floor response spectra developed for 1% damping.

D. With acceleration levels and predominant frequencies obtained, the full level of endurance tests was performed to establish the capability of equipment to withstand and to function during the effects of the accelerations corresponding to the resonance frequency. This is accomplished by one of the following methods:

1. Sine dwell test

   This test uses a sine wave function with one of the equipment natural frequencies and the corresponding acceleration levels as input vibrations. The test duration is generally 30 s, during which time the behavior of the equipment is observed and recorded.

2. Sine beat tests

   A sine beat function with number of beats and cycles per beat corresponding to the equipment natural frequency and with predetermined acceleration level is used as input motion to test and record the behavior of the equipment tested.

A slightly different procedure was also used by Wyle Laboratories in testing sensitive instrumentation and control equipment. In lieu of resonance search and endurance tests, a sine wave function, which when integrated resulted in the same shape and intensity as the given floor response spectra for 1%, was developed. The equipment was then tested using this function as an input. This approach simulates the actual conditions that the equipment would undergo during the actual specified earthquake. The behavior of the equipment was observed and recorded to ensure its capability to withstand the input vibrations.

In all cases the testing was performed for both horizontal and vertical vibrations separately.

3.7B.5.4 ACCEPTANCE

Where calculations were performed, using the seismic coefficients or floor response spectra curves, the equipment was generally modeled as a multi- or single-lumped mass model for frequency analysis.

All calculations, test procedures, and results supplied by equipment manufacturers or their laboratories were reviewed and accepted by qualified specialist engineers prior to release of equipment for shipment.

REFERENCES

1. Letter SLI-8305, R. E. Engel (S. Levy, Inc.) to L. T. Gucwa (GPC), "Edwin I. Hatch Nuclear Plant, Unit 2, Evaluation of 80-mil Fuel Assembly Channels," May 6, 1983.

TABLE 3.7B-1

CRITICAL DAMPING RATIOS FOR DIFFERENT MATERIALS

Item	Percent Critical Damping	
	OBE Condition	DBE Condition
Reinforced concrete structures	3.0	5.0
Welded structural assemblies (equipment and supports)	2.0	3.0
Bolted or riveted structural assemblies	3.0	5.0
Vital piping systems	0.5	1.0
Drywell - building (coupled)	3.0	5.0
Suppression chamber	2.0	3.0
RPV, support skirt, shroud head, separator, and guide tubes	2.0	3.0
Fuel	7.0	7.0
Steel frame structures	3.0	5.0
Translation and rotation of soil	4.0	5.0

1. Other values may be used if they are indicated to be reliable by experiment or study.
2. As of April 4, 1985, damping per figures 3.7A-22 and 3.7A-23 for piping systems and cable tray supports, respectively, is used for all new and replacement systems and load reconciliation work.

TABLE 3.7B-2

COMPARISON OF CALCULATED SEISMIC LOADS TO DESIGN SEISMIC LOADS OF SEISMIC CATEGORY I EQUIPMENT, DBE CONDITION

Equipment	Calculated Results Natural Frequency	Calculated Results Seismic Loads	Design Seismic Load
High-pressure coolant injection pump and turbine	> 33 Hz	0.43 g	1.5 g
Reactor core isolation cooling pump and turbine	> 33 Hz	0.43 g	1.5 g
Standby liquid control tank	> 33 Hz	0.80 g	1.5 g
Spent-fuel racks[c]	8.5-12.1 Hz[a][b]	0.46 g	1.5 g
Defective-fuel racks	8.5-12.1 Hz[a][b]	0.46 g	1.5 g
New-fuel racks	18.75 Hz[a]	0.22 g	1.5 g
Refueling platform	1.9 Hz	26,400 psi	36,000 psi
Control room panels[c]	-	-	-
Fuel prep machine	> 0.79 Hz	0.10 g	1.5 g
Residual heat removal heat exchanger	> 31 Hz	0.40 g	1.5 g
Hydraulic control unit	> 2.2 Hz	1.40 g	4.9 g

a. 2% damping calculated lowest natural frequency.
b. Function of mode design and arrangement.
c. Holtec spent-fuel storage racks in the contaminated equipment storage area were analyzed using nonlinear dynamic analysis. Natural frequency and static acceleration coefficients are not applicable.

TABLE 3.7B-3

COMPARISON OF THE MAXIMUM SEISMIC LOADS OF RPV AND INTERNALS

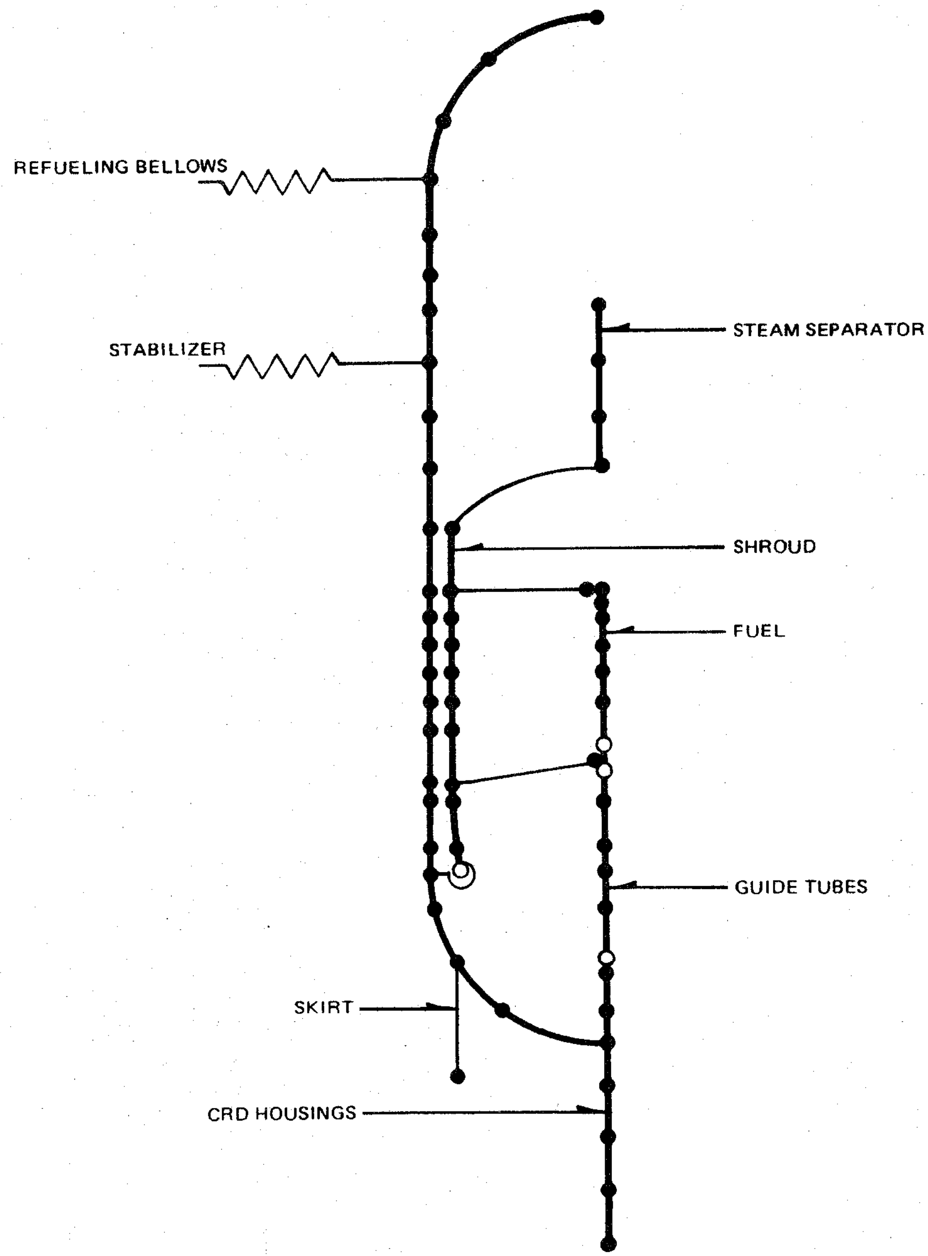
Location	DBE Seismic Loads[(a)(b)] N-S Excitation	E-W Excitation	Allowable Loads
Top guide shear (kips)	189	195	430
Core plate shear (kips)	180	186	430
Stabilizer force (kips)	514	916	1800
Shroud moment (in.-kips)	165×10^3	174×10^3	300×10^3
Shroud shear (kips)	723	755	1200
Vessel skirt moment (in.-kips)	243×10^3	277×10^3	900×10^3
Vessel skirt shear (kips)	843	970	2400

a. OBE moment is 8250 in.-lb/ bundle, and DBE moment is 16,500 in.-lb/ bundle.
b. Calculated loads shown assumed 100-mil-thick fuel channels.

TABLE 3.7B-4

NUMBER OF DYNAMIC RESPONSE CYCLES EXPECTED DURING A SEISMIC EVENT

Frequency band (Hz)	0^+ to 10	10 to 20	20 to 5
No. of seismic cycles	168	359	643



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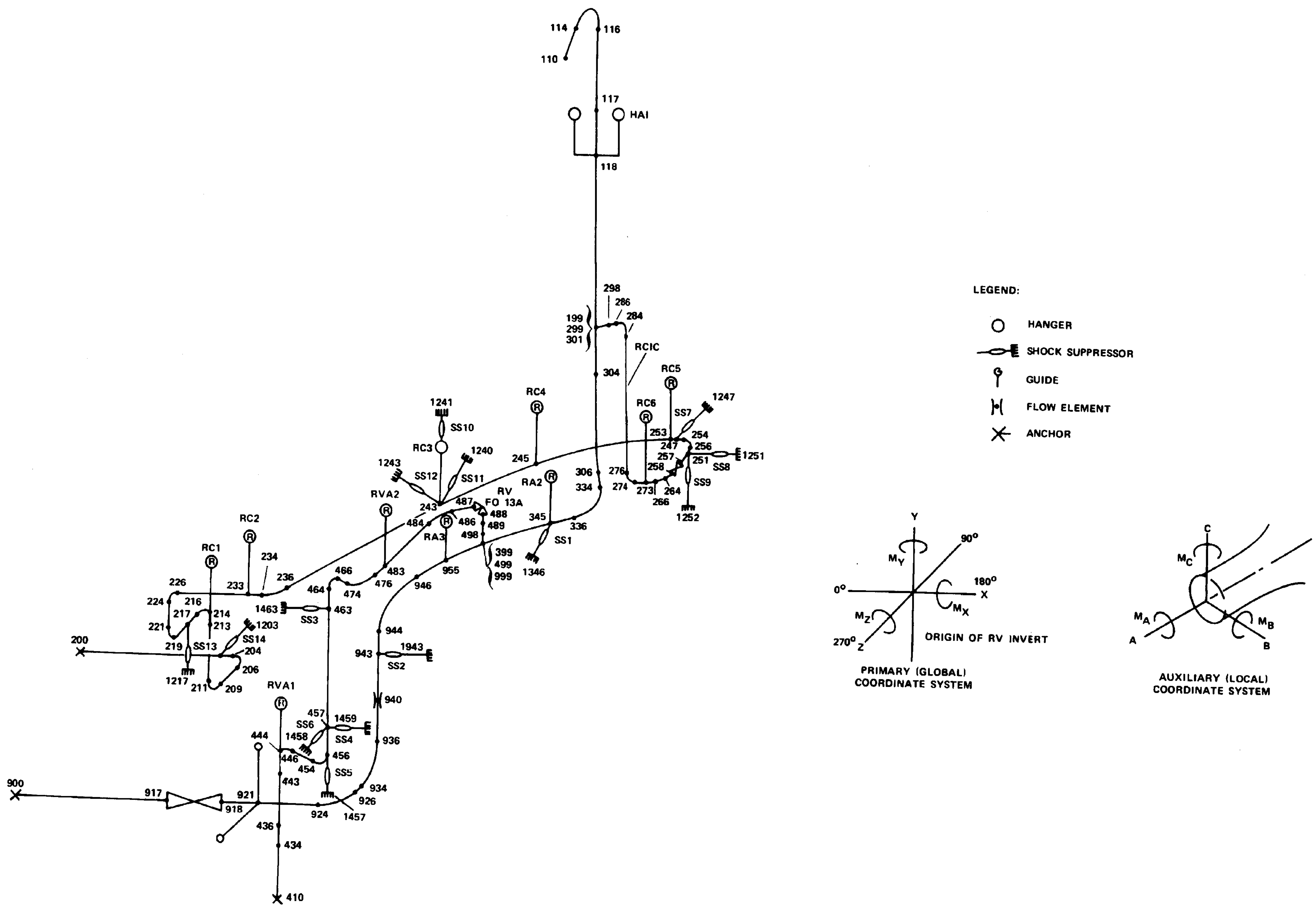
UNIT 2

MATHEMATICAL MODEL
RPV AND INTERNALS

FIGURE 3.7B-1



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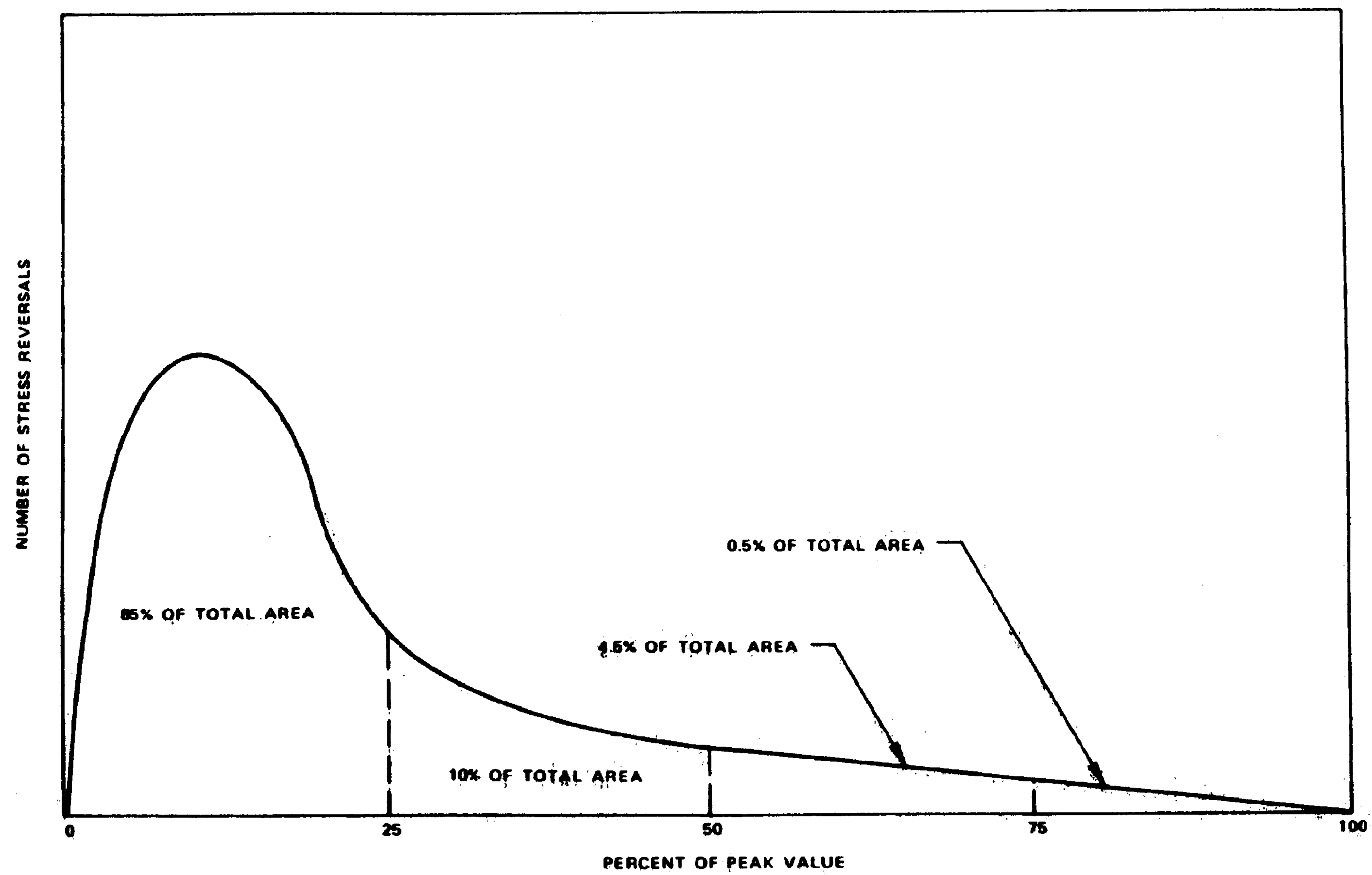
TYPICAL DYNAMIC MODEL OF
STEAM LINE PIPING SYSTEM

FIGURE 3.7B-2



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UNIT 2

DENSITY OF STRESS REVERSALS

FIGURE 3.7B-3

3.8 <u>DESIGN OF SEISMIC CATEGORY I STRUCTURES</u>

3.8.1 CONCRETE CONTAINMENT

A steel containment system was selected for the Hatch Nuclear Plant (HNP); therefore, this section does not apply to the design.

3.8.2 STEEL CONTAINMENT SYSTEM (AMERICAN SOCIETY OF MECHANICAL ENGINEERS (ASME) CLASS MC COMPONENTS)

The steel containment structure is designed to house the primary nuclear system and is referred to as the containment in the following sections. The steel containment structure is a part of the containment system whose functional requirement is the control of the release of radioactivity from the primary nuclear system. This section describes the structural design consideration for the containment. The following material discusses the information relative to the containment which provides the bases for design, construction, testing, and surveillance for the steel containment system.

The primary containment system houses the reactor pressure vessel (RPV), the reactor coolant recirculation system (RCRS), and other branch connections of the reactor coolant system (RCS). The primary containment is a pressure suppression system consisting of a drywell, pressure suppression chamber which stores a large volume of water, a connecting vent system between the drywell and pressure suppression chamber, isolation valves, a vacuum relief system, containment cooling systems, and other service equipment. The drywell is a steel pressure vessel in the shape of a light bulb, and the pressure suppression chamber is a torus-shaped steel pressure vessel located below and encircling the drywell. A vertical section of the drywell and suppression chamber is shown on drawing no. H-25000.

The primary containment system is designed to withstand the pressures resulting from a breach of the nuclear system process piping up to and including an instantaneous circumferential break of the reactor recirculation piping and provides a holdup for decay of any radioactive material released. The primary containment system also stores sufficient water to condense the steam released as a result of a breach in the nuclear system primary barrier and to supply the emergency core cooling system (ECCS).

3.8.2.1 <u>Description of the Containment</u>

3.8.2.1.1 Drywell

The drywell, shown on drawing no. H-25000, is a steel pressure vessel with a spherical lower portion 65 ft in diameter, a cylindrical upper portion 37 ft 1 in. in diameter, and an elliptical top head 30 ft 3 in. in diameter. The overall height is ~ 111 ft. The drywell rests on a concrete foundation and the inside is filled with concrete up to el 114 ft 6 in. Portions of the shell backed up by concrete resist local deformation and buckling. A sand pocket outside the drywell

between el 113 ft 2 in. and el 114 ft 6 in. provides a transition from fixed to free condition. The drywell is enclosed in a reinforced concrete structure for shielding purposes. Above the transition zone the drywell is separated from the reinforced concrete by an airgap of ~ 2 in. The airgap permits the containment vessel shell to deflect sufficiently, thereby allowing the jet load to be transferred to the surrounding concrete without rupturing the shell. Shielding over the top of the drywell is provided by removable, segmented, reinforced concrete shield plugs. In addition to the drywell head, one double-door airlock and two bolted equipment hatches are provided for access into the drywell.

The top portion of the drywell is removed during refueling operations. The head is held in place by bolts and is sealed with a double seal arrangement. The head is bolted closed when primary containment is required and is opened only when the primary coolant temperature is below 212°F and the pressure-suppression system is not required to be operational.

The double seal on the head flange provides a method for determining the leaktightness after the drywell head was replaced.

The design, fabrication, inspection, and testing of the drywell vessel comply with requirements of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components, Subsection NE, Requirements for Class MC Components, 1971 Edition, including 1971 Summer Addenda which pertain to containment vessels for nuclear power plants. The steel head and shell of the drywell are fabricated of SA-516 GR70 steel plate. Thermal stress in the steel shell due to temperature gradients is considered in the design.

Charpy V-notch impact tests were performed on specimens of all plate and forged materials.

Plates, forgings, and pipes of the drywell have an initial nil ductility transition temperature (NDTT) of ~ 0°F when tested in accordance with the appropriate code for these materials. It can be reasonably expected that the drywell is neither pressurized nor subjected to a substantial stress at temperatures below 30°F.

Provisions for protection of the drywell against missiles and pipe whip that can damage the primary containment are discussed in section 3.6.

3.8.2.1.2 Pressure Suppression Chamber and Vent System

The pressure suppression chamber is a steel torus-shaped pressure vessel located below and encircling the drywell, with a major diameter of 107 ft 1 in. and a cross-sectional inside diameter (ID) of 28 ft 1 in. The pressure suppression chamber contains the suppression pool and the airspace above the pool. Drawing no. H-25000 shows the section of the suppression chamber.

Large vent pipes connect the drywell and the pressure suppression chamber. A total of eight circular vent pipes are provided, each having an internal diameter of 6 ft 3 in. Jet deflectors are provided in the drywell at the entrance of each vent pipe to prevent possible damage to the vent pipes from jet forces which might accompany a pipe break in the drywell. The vent pipes are

provided with expansion joints which are enclosed within sleeves to accommodate differential motion between the drywell and suppression chamber.

The drywell vents are connected to a 4-ft-6-in.-diameter vent header in the form of a torus which is contained within the airspace of the suppression chamber. Projecting downward from the header are 80 downcomer pipes, 24 in. in diameter and terminating 4 ft 4 in. below the water surface of the pool.

The pressure suppression pool, which is contained in the pressure suppression chamber, initially serves as the heat sink for any postulated transient or accident condition in which the normal heat sink (main condenser or shutdown cooling system) is unavailable. Energy is transferred in the form of steam and water to the pressure suppression pool by either the discharge piping from the reactor pressure relief valves or the drywell vent system. The steam is condensed by the suppression pool. The condensed steam and any water carryover cause an increase in pool volume and temperature. Energy is removed from the suppression pool when the residual heat removal (RHR) system is operating in the suppression pool cooling mode. The pressure suppression pool also serves as a source of water for the ECCS.

Modifications made to the pressure suppression chamber and the drywell vent system due to hydrodynamic loads identified during the Mark I Containment Long-Term Program are presented in Supplement 3.8B.

3.8.2.1.3 Personnel and Equipment Access Locks

One personnel access lock is provided for access to the drywell. The lock has two gasketed doors in series. The doors are mechanically interlocked to ensure that at least one door is locked during times when primary containment is required. However, breakglass stations are provided inside the drywell as well as inside the airlock and a selector switch is provided inside the reactor building to defeat these interlocks in case of a threat to the safety of plant personnel. Breaking of the glass or operation of the selector switch is annunciated in the control room. The locking mechanisms are designed so that a tight seal is maintained when the doors are subjected to either internal or external pressure. The seals on this access opening are capable of being tested for leakage. A general arrangement of the personnel lock is shown on drawing no. S-26583.

A personnel access hatch with double, testable seals is provided in the drywell head. This hatch is bolted in place.

Two equipment access hatches with double, testable seals are also provided. These hatches are bolted in place.

Personnel and equipment hatches are sized and located with full consideration of service required; accessibility for maintenance and periodic testing programs. A 2-in. minimum gap is maintained around the barrel of the personnel and equipment hatches as they pass through or enter into the concrete shield wall.

A control rod drive (CRD) removal hatch with double, testable seals is provided. This hatch is bolted in place and permits extensive maintenance of the drive mechanism if required.

Access to the pressure suppression chamber is provided at two locations. These are two 4-ft-diameter manhole entrances with double-gasketed bolted covers connected to the chamber by 4-ft-diameter steel pipes. These access ports are bolted closed when primary containment is required and are opened only when the primary system temperature is below 212°F and the pressure suppression system is not required to be operational.

The drywell head personnel access hatch, the drywell equipment door assembly, the drywell CRD removal hatch, the suppression chamber manhole entrance, and a typical detail of the double, testable seal are shown in figure 3.8-1.

3.8.2.1.4 Penetrations

Two general types of pipe penetrations are provided: those which must accommodate thermal movement as illustrated by figure 3.8-2, and those which experience relatively little thermal stress as shown by figures 3.8-3 and 3.8-4. Figure 3.8-5 shows a typical instrument penetration. Figures 3.8-6 and 3.8-7 show typical electrical penetration structural components and assembly details. Figure 3.8-8 shows typical traversing incore probe (TIP) penetrations.

Some piping penetrations such as those used for the steam lines have special provisions for thermal movement. In these penetrations, the process line is enclosed in a guard pipe that is attached to the main steam line through a multiple-head fitting. This fitting is a one-piece forging with integral flues. The guard pipe and flued head are designed to the same pressure requirements as the process line. The process line penetration sleeve is welded to the drywell and extends through the biological shield where it is welded to a two-ply expansion bellows assembly which in turn is welded to the flued-head fitting. The pipe is guided through pipe supports at the end of the penetration assembly to allow steam line movement parallel to the penetration and to limit pipe reactions of the penetration to allowable stress levels.

The bellows assembly on the condensate drain line penetration X-8 is a single layer clamshell design welded over the existing bellows assembly as shown in figure 3.8-2, sheet 3. The bellows assembly for the reactor main feedwater line penetration X-9A consists of two layers of clamshell-design bellows longitudinally welded together as shown on figure 3.8-2, sheet 4. The modifications to the bellows assemblies for penetrations X-8 and X-9A utilized and comply with the guidance provided in USNRC Generic Letter 89-09.

Where necessary, the penetration assemblies are anchored outside the containment to limit the movement of the line relative to the containment. The bellows accommodates the relative movement between the pipe and the containment shell.

The cold piping, ventilation duct, and instrument line penetrations are generally welded directly to the sleeves. In some cases, where stress analyses indicate the need, double flued-head fittings are used. Bellows and guard pipes are not necessary in these designs since the thermal stresses are small and are accounted for in the design of the weld joint.

The electrical penetrations are hermetically sealed with provisions for periodic leak testing at design pressure. The penetration canisters are factory assembled and tested, with the number of field welds held to a minimum. These seals meet the intent of Section III of the ASME Code.

TIP guide tube penetrations pass from the reactor building through the primary containment. Penetration of the guide tubes through the primary containment is sealed by means of brazing, which meets the requirements of the ASME Boiler and Pressure Vessel Code, Section VIII. These seals also meet the intent of Section III of the ASME Code.

The designation and function of these penetrations are shown in ***Technical Requirements Manual (TRM) table T7.0-1 (incorporated by reference into the FSAR)***.

3.8.2.2 Applicable Codes, Standards, and Specifications

The following codes, standards, and specifications apply to the design, fabrication, erection, and testing of the containment:

A. Regulations

1. Title 10 Code of Federal Regulations (CFR) Part 50, Appendix J, "Primary Reactor Containment Leakage Testing for Water-Cooled Power Reactors."

2. 29 CFR 1910, "Occupational Safety and Health Standards."

B. Codes and Standard Specifications

1. ASME Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components, 1971 Edition including 1971 Summer Addenda.

2. Institute of Electrical and Electronics Engineers (IEEE) Standard No. 317-1971, "Standard for Electrical Penetration Assemblies in Containment Structure for Nuclear Fueled Power Generating Stations."

Code Classification

The steel containment vessel is classified Class MC in accordance with Subarticle NA-213O, Section III of the ASME Code.

Code Compliance

The steel cylindrical shell and dome of the steel containment vessel, including all penetrations and attachments within the code jurisdictional boundaries defined in paragraph 3.8.2.1.4, is designed and constructed in strict accordance with Subsection NE, Class MC Components, including the requirements for quality assurance of Article NA-4000, and inspection requirements of Article NA-5000 of Section III of the ASME Code.

The containment vessel is code-stamped in accordance with the ASME Boiler and Pressure Vessel Code, Section III, Paragraph N-161O.

The bellows assemblies for penetrations X-9A (figure 3.8-2, sheet 4) and X-8 (figure 3.8-2, sheet 3) were modified in 1989. Design and fabrication of the new bellows assemblies conform to the requirements of subsection NE, Class MC Components, of Section III of the ASME Code, except for materials and stamping. Replacement materials for the new bellows assemblies for penetrations X-9A and X-8 are not code stamped but have been procured under the guidelines of NRC Generic Letter 89-09.

C. Code Cases

1. ASME Code Case 1330-3, "Special Equipment Requirements," Section III, Approved by Council March 9, 1972.

2. ASME Code Case 1177-7, "Expansion Joints," Section VIII, Division 1, Approved by Council February 27, 1970.

3. ASME Code Case 1431, SA-350, Grade LF-2, for Class MC, Approved by Council August 8, 1969.

4. ASME Code Case 1443-1, "Radiography for Pipe," Section III, Approved by Council August 10, 1970.

5. ASME Code Case 1517, "Material Used in Pipe Fittings," Section III, Approved by Council March 9, 1972.

D. State and Local Building Codes

Southern Standard Building Code (SSBC), 1969 Edition

E. Nuclear Regulatory Commission (NRC), Regulatory Guides, General Design Criteria (GDC), Industry Standards and Specifications

1. NRC Regulatory Guides (Compliance is discussed in Appendix A.)

Regulatory Guide 1.11, "Instrument Lines Penetrating Primary Reactor Containment" (March 1971).

Regulatory Guide 1.29, "Seismic Design Classification" (August 1973).

Regulatory Guide 1.46, "Protection Against Pipe Whip Inside Containment" (May 1973).

Regulatory Guide 1.54, "Quality Assurance Requirements for Protective Coatings Applied to Water-Cooled Nuclear Power Plants" (June 1973).

Regulatory Guide 1.57, "Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components" (June 1973).

Regulatory Guide 1.59, "Design Basis Floods for Nuclear Power Plants" (August 1973).

Regulatory Guide 1.63, "Electric Penetration Assemblies in Containment Structures for Water-Cooled Nuclear Power Plants" (October 1973).

Regulatory Guide 1.64, "Quality Assurance Program Requirements for the Design of Nuclear Power Plants" (October 1973). This guidance has been superseded by that contained in ASME NQA-1-1994, as described in the SNC Quality Assurance Topical Report (QATR).

2. GDC of 10 CFR 50 (Compliance is discussed in section 3.1.)

   GDCs 1, 2, 3, 4, 16, 50, 52, and 53

3. Industry Standards

   Nationally recognized industry standards, such as those published by American Society for Testing and Materials (ASTM), are used whenever possible to describe material properties, testing procedures, fabrication, and construction methods.

   Applicable ASTM and American Welding Society (AWS) Material Standard Specifications permitted by Article NE-2000 of Section III of the ASME Code.

   Applicable ASTM Standard Specifications for nondestructive methods of examination referenced in Article X-3000 of Section III of the ASME Code.

4. Specifications

   The specification "For Furnishing and Delivery of Reactor Drywell and Suppression Chamber Containment Systems for Edwin I. Hatch Nuclear Plant - Unit 2," prepared in accordance with the requirements of paragraph NA-335O of the ASME Code, Section III, was used for the design, furnishing, fabrication, delivery, unloading, erection, painting, and testing of the primary containment. A summary of the structural and other design requirements from this specification is given in paragraph 3.8.2.2.1 and other applicable portions in this chapter.

5. Bechtel Power Corporation Topical Report BN-TOP-1, "Testing Criteria for Integrated Leak Rate Testing of Primary Containment Structures for Nuclear Power Plants," Rev 1, November 1972.

3.8.2.2.1 Structural Specifications

Structural specifications are prepared to cover the areas related to design and construction of the containment. These specifications emphasize important points of the industry standards for the design and construction of the containment and reduce options that otherwise would be permitted by the industry standards. The ASME Code cases listed in paragraph 3.8.2.2 were used to supplement the code requirements. The following areas are covered in the specifications and a summary of these requirements is described in paragraphs 3.8.2.3, 3.8.2.4, 3.8.2.5, 3.8.2.6:

- Design loads, loading combinations, and allowable stresses for the drywell, suppression chamber, ventlines, penetrations, and accessories.

- Materials for pressure retaining parts and appurtenances.

- Fabrication methods including welding requirements.

- Nondestructive examinations.

3.8.2.3 Loads and Loading Combinations

The containment is designed for all credible conditions of loadings, including normal loads, loads resulting from a loss-of-coolant accident (LOCA), test loads, and loads due to adverse environmental conditions.

Critical loading combinations are those caused by a postulated loss of reactor coolant, by a postulated earthquake, or by a pipe rupture in the containment.

Wind and tornado loads, flood design bases, and seismic loads are given in sections 3.3, 3.4, and 3.7, respectively. Missile effects and the postulated pipe rupture effects are discussed in sections 3.5 and 3.6.

Chapter 15 provides information on the design pressure load.

3.8.2.3.1 Loads

The following loads are considered: dead loads, live loads, LOCA loads, thermal loads, wind and tornado loads, hydrostatic loads, earthquake loads, jet impingement loads, test loads, and penetration loads.

A. Dead Loads (D)

   Structural dead loads consist of the weight of the containment shell, drywell concrete floor, welding pads, jet deflectors, vents, penetrations, pipe supports, supporting structures, platforms, handrails, spray headers, strainers, monorails,

drywell water seal assemblies, expansion bellows, suppression pool water, and accessories.

B. Live Loads (L)

Live loads consist of design platform loads, equipment live loads specified by the equipment manufacturers, and all other live loads transmitted by the internal structures.

A live load of 40,000 lb was considered acting on any one of the drywell equipment access doors at any given time.

Weight of the water used to fill the drywell from el 203 ft 4 in. to el 227 ft during refueling was considered a live load for the drywell design.

Weight of air during initial and final testing of the containment vessels was considered a live load.

All catwalks and platforms, with the exception of the personnel lock, were designed for a live load of 75 lb/ft^2.

A live load of 150 lb/ft^2 was considered for the personnel lock floor area.

C. Earthquake Loads (E, E^1)

Operating basis earthquake (OBE) loads E and design basis earthquake (DBE) loads E^1 obtained from the seismic analysis described in paragraph 3.8.2.4.6 were used in the containment design.

D. Test Pressure and Temperature Loads (P_t, T_t)

Upon completion of erection, the bare containment vessel and its penetrations and appurtenances were tested for 70-psig (125% of the design pressure) pressure followed by a leak rate testing at 56 psig. This is the initial containment pressure testing.

The containment vessel was also tested for leakage before the plant went into operation and a few other times during the life of the plant. This is considered as final testing in design.

The pressure P_t and the corresponding temperature at the time of the test T_t for initial and final pressure testing were considered in design.

E. Thermal Loads (T_o, T_a)

T_o = Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

T_a = Thermal loads under thermal conditions generated by the postulated break and including T_o.

F. Pipe Reaction Loads (R_o, R_a, R_e)

R_o = Pipe reactions during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

R_a = Pipe reactions under thermal conditions generated by the postulated break and including R_o.

R_e = Pipe reactions under thermal conditions during event causing external pressure.

G. External Pressure and Temperature Loads (P_e, T_e)

The following external pressure, P_e, and temperature, T_e, were considered in design:

1. Drywell and Vent Systems

Design external pressure	2 psig at 150°F
Operating external pressure	< 2 psig at 150°F

2. Suppression Chamber

Design external pressure	2 psig at 150°F
Operating external pressure	< 2 psig at 50° to 100°F

H. Pressure Loads Due to LOCA (P_a)

The design pressure and temperature of the containment are greater than the peak pressure and temperature that would result from a postulated complete blowdown of the reactor coolant. This might occur through the rupture of the RCS up to and including the hypothetical double-ended severance of the largest reactor coolant pipe. Pressure transients resulting from a LOCA (section 15.3) serve as the basis for a containment design pressure.

The design pressure, P_a, is not exceeded during any subsequent long-term pressure transients caused by the combined effects of heat sources. These effects are overcome by the combination of safety features and heat sinks.

The following pressure and temperature loads were used in the design:

1. Drywell and Vent Systems

Design internal pressure	56 psig at 340°F
Operating internal pressure	< 2 psig at 150°F

2. Suppression Chamber

Design internal pressure	56 psig at 340°F
Operating internal pressure	< 2 psig at 50° to 100°F

The design internal pressure is 90% of the maximum internal pressure.

I. Pipe Rupture Loads (Y_r, Y_j, Y_m)

Y_r = Equivalent static load on the structure generated by the reaction on the broken high-energy pipe during the postulated break, and including an appropriate dynamic load factor to account for the dynamic nature of the load.

Y_j = Jet impingement equivalent static load on a structure generated by the postulated break and including an appropriate dynamic load factor to account for the dynamic nature of the load.

The containment is designed for the following jet impingement loads resulting from pipe ruptures within the containment:

Location	Jet Force	Area of Influence
Drywell sphere	709,000 lb	3.94 ft^2
Drywell knuckle	472,000 lb	2.63 ft^2
Drywell cylinder up to el 203 ft 9 in.	472,000 lb	2.63 ft^2
Drywell head	32,600 lb	0.181 ft^2

The jet forces consist of steam and/or water at 340°F. Only one of the above jet forces is considered to act in the drywell at a given time.

Y_m = Missile impact equivalent static load on a structure generated by or during the postulated break, as from pipe whipping, and including an appropriate dynamic load factor to account for the dynamic nature of the load.

J. Containment Flooding Loads (F_L)

F_L = Loads generated by the post-LOCA flooding of the containment. In the event of a LOCA, the entire containment, including the suppression chamber, vent system, and the drywell, are flooded up to el 227 ft, and the resulting hydrostatic load, F_L, was considered in the containment design.

K. Hydrodynamic Loads for the Suppression Chamber

In performing large scale testing of an advanced design pressure suppression containment (Mark III), and during inplant testing of Mark I containments, suppression pool hydrodynamic loads not explicitly included in the original Mark I containment design basis were identified. These additional loads could result from dynamic effects of drywell air and steam being rapidly forced into the suppression pool during a postulated LOCA, and from suppression pool response to safety relief valve operation generally associated with plant transient operating conditions. Since these hydrodynamic loads were not explicitly considered in the original design of the Mark I containment, the NRC staff in early 1975 requested a detailed reevaluation of the containment system from each domestic utility with a Mark I containment. A two-phase program was established; it was described to the NRC in letters submitted during the week of May 5, 1975.

The phase I effort, called Short-Term Program (STP), provided a rapid confirmation of the adequacy of the containment to maintain its integrity under the most probable course of the postulated LOCA considering the latest available information on the important suppression pool dynamic loads. The STP was completed July 12, 1977, following the docketed submittal by Georgia Power Company (GPC) to the NRC of the HNP-2 plant-unique analysis report.(1) Review of this documentation led to issuance of the "Mark I Containment Short-Term Program Safety Evaluation Report," in December 1977.(2) This report concluded that licensed domestic boiling water reactor (BWR) Mark I facilities could continue to operate safely, without undue risk to health and safety of the public, during an interim period while the Long-Term Program (LTP) was conducted.

The phase II effort, the LTP, was initiated in June 1976. The LTP included detailed testing and analytical work to define more precisely the specific hydrodynamic loads appropriate for the anticipated life of the Mark I BWR facility. It also included detailed structural evaluation and modifications to restore the originally intended design-safety margins for the containment system.

The LTP was completed on September 16, 1983, following the docketed submittal by GPC to the NRC of the Plant Unique Analysis report.(21)

The hydrodynamic loads considered and a description of the pressure suppression chamber and drywell vent system modifications made during the STP and LTP are provided in supplement 3.8B.

3.8.2.3.2 Effects of the Induced Strains on the Shell

The primary membrane stresses were maintained within yield range for all loading combinations except when jet loadings were considered. As elastoplastic finite element analysis performed for jet impingement loading showed that the maximum computed total equivalent strain at the centerline of the jet load on the outer surface of the drywell shell backed up by concrete was

3.65%, which corresponded to a maximum computed equivalent stress of 45.1 ksi, which satisfies the fatigue requirements of figure I-9-1 of Section III of the ASME Code for 10 cycles.

3.8.2.3.3 Loading Combinations

The different loading combinations considered for the containment design were initial testing, final testing, normal operating, refueling, accident, and flooded. The loading combinations for the suppression pool hydrodynamic loads are included in supplement 3.8B.

A. Initial and Final Testings

The containment is subjected to the following loads associated with the initial and final testings for the containment:

$D + L + P_t + T_t + E$

$D + L + P_t + T_t + E^1$

B. Normal Operating

The containment vessel is under this condition for most of its lifetime. Applicable loads considered for this condition are:

$D + L + T_o + R_o + E$

$D + L + T_o + R_o + E^1$

$D + L + T_e + R_e + P_e + E$

$D + L + T_e + R_e + P_e + E^1$

C. Refueling

The plant is shut down for refueling operations about a month every year and the containment vessel is subjected to the following loads:

$D + L + E$

$D + L + E^1$

D. Accident

To maintain the integrity of the containment vessel, the following postulated LOCA loads were considered in the design:

$D + L + T_a + R_a + P_a + E$

$D + L + T_a + R_a + P_a + E^1$

$D + L + T_a + R_a + P_a + Y_r + Y_j + Y_m + E^1$

E. Flooded

The containment is flooded for accident recovery after a long period of time (months) following a LOCA. The loads corresponding to this condition are:

$D + L + E + F$

3.8.2.3.4 Loading Combinations on Localized Areas

A. Penetration Locations

Loads on penetrations due to thermal expansion, weight, earthquakes, and valve closure (when applicable) were obtained from corresponding piping system analysis and these penetration loads were applied to the shell at points of intersections and the combined effect of this load and containment internal design pressure were computed.

B. Drywell Embedment Zone

The drywell shell was analyzed at the point of embedment for the discontinuity stresses due to the pressure loads, differential thermal gradient loads, and shell dead and live loads, including seismic loads.

C. Drywell Knuckle Area

The drywell shell is analyzed in the region of the knuckle for the accident condition to determine the discontinuity stresses caused by pressure loads acting normal to the shell and by vertical loads resulting from dead, live, and seismic loads applied by the cylindrical shell.

D. Drywell Cone-to-Cylinder Intersections

Adequacy of the drywell cone-to-cylinder intersections is checked using ASME Code rules for design internal pressure acting at accident temperature coincident with the other loads specified in the loading combinations.

E. Drywell Top Head

The top head along with a portion of the cylinder above the flange is analyzed for the effects of the design internal pressure acting at accident temperature coincident with other loads specified in the loading combinations.

F. Drywell - Vent Line - Bellows Collar

The vent line along with its connection to the drywell and to the expansion bellows is analyzed for the effects of the design internal pressure, acting at accident temperature, in conjunction with the loads due to the bellows attachment resulting from the differential deflection of the vent line and suppression chamber due to thermal and pressure expansions and other loading specified in the loading combinations.

G. Drywell-to-Pedestal Intersection

The drywell shell in the region of the pedestal is analyzed for two sets of loading conditions. The first set of conditions is the construction condition with the operating and DBE loads along with the loads imposed by the weight of the shell and live loads contributed by scaffolding and other applicable temporary construction fixtures on the containment. The second set of loading conditions includes an overload test internal pressure of 70 psi in addition to all other loads considered above for the construction condition.

H. Drywell Flange

The drywell shell is analyzed in the region of the drywell flange for the accident condition to determine the discontinuity stresses. Pressure loads, thermal loads, bellows loads, and shell loads resulting from the preload on the flange bolts were considered for this analysis.

3.8.2.4 Design and Analysis Procedures

The steel containment vessel, which consists of a vertical freestanding cylindrical shell, elliptical top head, spherical bottom, and numerous penetrations and attachments, is considered to act as an independent structural component within the shield building.

The containment is analyzed for various loading combinations, taking into account the values of individual loads that generate the most significant stress condition for each component and member of the structure. The critical areas for analysis are as follows:

- Drywell.
- Pressure suppression chamber and vent system.
- Personnel and equipment access locks.

- Penetrations.
- Localized areas of discontinuities.

3.8.2.4.1 Drywell

The drywell shell is analyzed for stresses due to the loads and loading combinations given in paragraph 3.8.2.3. Primary membrane stresses are computed for each of the loading combinations and the resulting stresses are compared to ASME Code allowables. Membrane theory is used to compute primary membrane stresses due to internal pressure and vertical loads. Simple bending theory is used to determine the bending stresses due to horizontal shear and moment caused by lateral earthquake. Formulas from Section VIII of the ASME Code were used to design the top head, the cone, and the cylinder. The pressure stress resultants in the knuckle section are computed using the pressure area method outlined in L. P. Zick's paper.[(3)]

The sand pocket between el 113 ft 2 in. and el 114 ft 6 in. was considered to provide a transition zone at el 113 ft 2 in. and the drywell shell was assumed to dilate from a fixed to a free condition at this level. The drywell shell was considered as a freestanding shell fixed at the base and supported laterally by eight star truss connection lugs at ~ el 188 ft. No friction or bond resistance was considered to exist between the drywell and the concrete at the base.

3.8.2.4.2 Pressure Suppression Chamber and Vent System

The suppression chamber shell is analyzed for the effects of gaseous pressure loads, water pressure loads, and for the loads due to the seismic action on the weight of shell and contained water for the various design conditions given in paragraph 3.8.2.3.

Circumferential shell stresses are calculated based on an internal pressure that consists of internal gas pressure plus the hydrostatic head of water with seismic loads acting on the water using membrane theory.

Longitudinal stresses due to internal pressure consisting of internal gas pressure plus the hydrostatic head of water with seismic loads acting on the water are computed using membrane theory. Longitudinal stresses due to the weight of the shell and water acting together with seismic loads are computed by simple bending stress theories.

The suppression chamber shell is also checked for external pressure using applicable formulas in ASME Code, Section VIII.

Design and analysis procedures used in the evaluation of the pressure suppression chamber and vent system are summarized in supplement 3.8B.

3.8.2.4.3 Personnel and Equipment Access Locks

The personnel lock has two gasketed doors in series which are mechanically interlocked so that one door cannot be opened unless the other door is sealed. Each door is designed so that, with the other door open, it is capable of withstanding an internal pressure inside the containment vessel of 56 psi or an external pressure of 2 psi. In addition, the interior door is equipped with tiedown devices so that the space between the doors can be tested. The interior door and interior bulkhead are designed to withstand a jet load of 709 kips over a circular area of 3.94 ft^2. Analysis is accomplished by modeling the doors and bulkheads as beam structures.

The personnel lock barrel is designed as a cylindrical pressure vessel for internal and external pressures in accordance with the ASME Code rules. The shell insert plate is proportioned for area replacement and is checked for stresses due to weight of the lock and dynamic loading resulting from seismic activity. The lock floor is designed for a live load of 150 lb/ft^2.

The equipment hatch is arranged so that the drywell internal pressure and/or jet load tend to force the head onto the barrel, thus seating the gaskets. However, the external pressure tends to force the hatch open and so the closure bolts are preloaded sufficiently to maintain leaktightness. The bolts are preloaded to create adequate friction against the displacement due to the weight of the head and seismic loads acting on it.

The equipment hatch is provided with a floor at 130 ft to facilitate access into the drywell. The floor and its attachments to the barrel are designed for a 40-kip load evenly distributed on two wheels at 70-in. centers with vertical and horizontal seismic loads acting with it. Other parts of the floor are designed to support a live load of 150 lb/ft^2.

3.8.2.4.4 Penetrations

Two types of penetrations considered on the spherical and cylindrical sections of the containment are those that have no mechanical loads applied and those that have mechanical loads applied.

The analysis of penetrations with no external loads consists of proportioning the pipe neck for pressure according to ASME Code, Section VIII, and proportioning the insert reinforcing plate according to ASME Code, Section III, requirements. The insert reinforcing plates are proportioned using a computer program developed to check the adequacy of preselected reinforcing plate dimensions and weld sizes in accordance with the area replacement criteria of ASME Code, Sections III and VIII.

The following steps were followed in the analysis of penetration with external loads:

A. The pipe neck and the insert reinforcing plate were proportioned using the same methods as those applied for penetrations with no external loads.

B. Penetration loads on the suppression chamber shell obtained from the piping system analysis were resolved into radial and tangential components. These loads and the penetration loads on the drywell were used to check the stresses on the

individual penetration necks using a computer program. This computer program computes maximum stress intensity for both the inside and outside surfaces based on basic Mohr's circle equations. The principal stresses are printed out and the stresses resulting from bending, axial load, shear loads, and torsion, as well as the value of the loads which cause the maximum stressed condition, are obtained.

C. Initial pressure stresses are assumed in the insert reinforcing area in accordance with the requirements of ASME Code, Section III.

D. Stresses in the shell and insert plate are checked using methods of Welding Research Council Bulletin 107.[4] This analysis is done by a computer program which is designed to determine the stress intensities in a sphere or cylinder at 12 different points around an externally loaded circular attachment. The program superimposes those stresses resulting from external loads on the initial pressure stresses. Stresses are computed at three levels of plate thicknesses: outside, inside, and centerline. Points selected for checking the stresses are: edge of attachment, $1/2\sqrt{Rt}$ from the edge of attachment, and the edge of reinforcement where R is the inside radius of the vessel and t is the nominal thickness of the vessel. The program determines the normal stresses parallel to the vessel's longitudinal axis, the normal stresses in circumferential direction, and the shear stresses.

   Flanges and penetration caps were designed in accordance with the rules of ASME Code, Section VIII.

3.8.2.4.5 Localized Areas of Discontinuities

The localized areas of discontinuities, with the exception of cone-to-cylinder intersections, are analyzed for loadings associated with accident conditions using the program for shells of revolution developed by A. Kalnins of Yale University.[5] This program is based upon a method of analysis published in the "Journal of Applied Mechanics," Volume 31, September 1964 and is derived from the fact that a rotationally symmetric shell may be divided into a number of short segments in the meridional direction and that the stiffness properties of each of these segments can be determined in relation to eight fundamental variables. By enforcing equilibrium and compatibility between segments and applying boundary conditions, the values of the fundamental variables were determined for each segment. Values between each segment end were determined by integration.

Adequacy of the drywell cone-to-cylinder intersections was checked using ASME Code rules. The half-apex angle is 30 degrees and, as a result, paragraph UA-5 of ASME Code, Section VIII, was applied. The intersections were checked for the accident condition with an internal pressure of 56 psi and allowable stresses based on 340°F.

3.8.2.4.6 Seismic Analysis

The evaluation of the structural integrity of the steel containment vessel when excited by seismic motion is based on a dynamic analysis.

The steel containment vessel is designed to act as an independent structural component within the concrete bioshield building. The associated internals within the steel containment vessel are also designed to act as independent components. This independency and uncoupling of the major structural components of the containment internal system enables a very detailed dynamic mathematical model to be developed which provides for the realistic response of the containment system, and for which response spectra and/or time histories can be generated at the component interfaces, or at any point desired. These component response spectra and/or time histories were used to perform detailed dynamic analyses of the individual components.

In the dynamic analysis of the steel containment vessel component, a dynamic mathematical model is formulated which incorporates the general structural geometry and all significant boundary conditions present. The design of the numerous penetrations is such that any restraining forces on the steel containment vessel which could be developed can be considered as negligible. In the determination of the seismic response of the steel containment vessel a 2% damping value has been used for both OBE and DBE excitations.

The resulting equations of motion for the steel containment vessel were solved by the use of a large-capacity computer program. The solution algorithm used depends on the analytical method incorporated to evaluate the equations of motion for the system.

The results of the dynamic seismic analysis contain values for maximum translational accelerations, displacements, shears and moments, moments and rotations, as well as any response spectra and/or time histories desired at points throughout the steel containment vessel.

These resultant forces were then combined with the various loading conditions as described in paragraph 3.8.2.3 and in accordance with Subarticle NE-3131 of Section III of the ASME Code. These combined forces were used in the structural analysis of the various critical areas present within the steel containment vessel. By using a response spectra and/or time history, the cantilevered personnel locks, as well as other appurtenances, were dynamically analyzed.

For analytical purposes, each lock is assumed to be a beam cantilevered from the main body of the containment vessel. It is further assumed to vibrate in three independent directions as described in the following:

A. Case I: Lock vibration in the meridional plane of the vessel due to vertical earthquake. This condition imposes a moment on the shell in the meridional plane of the vessel.

B. Case II: Lock vibration in the circumferential plane of the vessel due to the combined effects of vessel translation and angular oscillation resulting from an applied horizontal earthquake acting perpendicular to the plane of the lock. This condition imposes a moment on the shell in the circumferential plane of the vessel.

C. Case III: Lock vibration radial to the vessel due to an applied horizontal earthquake acting in the plane of the lock. This condition imposes a radial load on the shell.

In order to determine the seismic accelerations acting on the lock, a 1 degree-of-freedom system is assumed. After its natural periods of vibration are calculated for the longitudinal, circumferential, and radial directions of the containment vessel, the lock accelerations are determined from the floor response spectrum curves which are developed by the seismic analysis of the containment itself using the time-history technique. Equivalent dynamic loads of vibration are obtained by multiplying the accelerations by the mass of the lock. By applying these accelerations at the center of gravity of the lock, forces, moments, and shears transmitted to the shell are determined.

The resulting stress intensities due to the addition of seismic loads to the various loading conditions for the steel containment vessel and its appurtenances were kept in accordance with the stress intensity limits as specified in Subarticle NE-3131 of Section III of the ASME Code.

3.8.2.4.6.1 **<u>Computer Program Utilized in the Seismic Dynamic Analysis</u>**. The seismic dynamic analysis was performed using a large-capacity computer. The program is capable of generating the required mass and stiffness matrices which are required to represent the distributed mass and stiffness of the actual structure.

The structure was modeled by the use of any or all types of the following finite elements:

- Three-dimensional beam element.
- Triangular and quadrilateral plate and thin shell elements.
- Triangular and quadrilateral axisymmetric elements.
- Boundary elements as required.

The program provides for the solution of the resulting equations of motion and yields the desired number of eigenvalues and eigenvectors as required. The program can then be optioned to provide the solution of a reduced, if desired, set of uncoupled equations for response to a specified form of excitation. The excitation used can be either symmetric or asymmetric.

The program provides as output the geometry and topology of the constructed dynamic mathematical model as well as all pertinent information required in its description. The resulting eigenvalues (frequencies of vibration), the associated eigenvectors (modes of vibration) which can be normalized, and participation factors are provided. In the performance of the response analysis, the output includes the modal and/or system generalized forces, displacements, velocities, accelerations, and forces. This resulting output can be requested at either the member level, the joint level, or both.

3.8.2.4.7 Computer Programs Used by Chicago Bridge and Iron Company (CB&I) for the Structural and Seismic Analysis of the Containment

3.8.2.4.7.1 CB&I Program 405 - Ring Analysis.[6]

A. Description

This is a program used for the analysis of a ring with a constant moment of inertia and modulus of elasticity. The loads are in the plane of the ring. The mathematics are based upon the Hardy-Cross Column Analysis for rings. The loads can be moments, tangential, or radial to the ring. The printouts are coefficients at incremental distances around the ring. The printout titles for the output are as follows:

X = angle and degrees as measured from reference axes.

V = a radial shear with force units acting in a radial direction through the ring.

T = an axial thrust in the ring with units of force.

M/R = a coefficient with units of force when multiplied by the radius to the centroid equals a moment.

EI/RR = a coefficient which when multiplied by the radius^2 is equal to the rotation of the ring at the point.

REI/RR = a coefficient when multiplied by the radius equals the radial deflection of the point.

CEI/RRR = a coefficient when multiplied by the radius^3 equals the tangential deflection of the point.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

The program is used to analyze a ring with constant moment of inertia and modulus of elasticity.

3.8.2.4.7.2 CB&I Program 601 - Stresses at Specific Points.

A. Description

This program is based on the mathematics of Program 405. In addition, the coefficients have been multiplied by the proper radius. Consequently, the thrust and moment only have to be divided by the area and section modulus respectively to find the stresses at specific points.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

This program is used in conjunction with the CB&I Program 405 to find the stresses at specific points.

3.8.2.4.7.3 CB&I Program 655 - Shear Transfer from the Ring to the Shell.[7]

A. Description

This program is supplementary to CB&I Programs 405 and 601 and is applied to transfer the shear in the ring into the shell between the rings. The influence of the loads on any ring is not evaluated beyond the adjacent rings.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

The program is used in conjunction with the CB&I Program 405 to transfer the shear in the ring into the shell between the rings.

3.8.2.4.7.4 CB&I Program 7-81 N - Shell Stress at Discontinuities.[5]

A. Description

This shells of revolution program based on the ASME paper, "Analysis of Shells of Revolution Subjected to Symmetrical and Nonsymmetrical Loads," by A. Kalnins,[5] is a standard computer program in the industry. The program computes the stresses and displacements in thin-walled, elastic shells of revolution when they are subjected to static edge loads, surface loads, or arbitrary temperature distribution over the surface of the shell. The geometry of the shell must be symmetrical; however, the shape of the median may be arbitrary. The shell wall

may consist of four layers of different orthotropic materials, and the thickness and elastic property of each layer may vary along the median.

The program numerically integrates the eight ordinary first-order differential equations of the thin shell theory derived by H. Reissner.

The CB&I version of the shells of revolution program incorporated modifications on the method of input and the format of output.

B. Validation

The results of the program were compared with those obtained by other shell programs, such as Seal and Cerl II, and were found to be in excellent agreement. Document traceability is available at CB&I.

C. Extent of Application

The program is used in the analyses of the equipment hatch and personnel lock for the containment structure and for computing the shell stresses at discontinuities with the exception of nozzles.

3.8.2.4.7.5 <u>CB&I Program 6-20 - Local Stresses in Shells</u>.[4]

A. Description

This program computes the local stresses in cylindrical and spherical shells due to a load or a combination of loads acting on a nozzle which penetrates the shell. The solution for local shell stresses is made using the dimensionless parameters (input) from the graphs in the Welding Research Council (WRC) Bulletin 107. When reinforcing is present, these parameters are found using the procedures of Biljaard as outlined in WRC 49 and 50.

Tests are performed in the cylinder and sphere subroutines to see if either an insert or pad plate or no reinforcing is present. Depending on the results of these tests, a particular set of denominators is computed for use in the stress calculations in the stress subroutine. When reinforcing is present, the program checks the stress at the edge of the reinforcing.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

The program is used to compute the local stresses in cylindrical and spherical shells due to a load or combination of loads acting on a nozzle which penetrates the shell.

3.8.2.4.7.6 <u>CB&I Program 860 - Rigid Attachment to Spherical Shell</u>.[(4)]

A. Description

This program computes shell stresses around a rigid attachment to a spherical shell due to any combination of loading - radial, shear, or moment. The program uses the nomenclature, the curves for coefficients, and the mathematics of the WRC Bulletin 107. Given the basic geometry of the attachment, the program computes the parameters as required from figures SR-2 and SR-3 and the shell stresses around the attachment.

If the width of reinforcing is < 1.65 times the square root of the spherical radius times either the thickness of the insert or an equivalent thickness for pads, the stresses are also checked at the edge of the reinforcing. All induced moments at the nozzle-to-shell junction and the inducted moment, Mx, at the edge of reinforcing are increased by 20% to satisfy WRC 49 and 50 by Biljaard. If the width of reinforcing is > 1.65 times the square root of the spherical radius times either the insert thickness or equivalent thickness, only the stresses at the nozzle-to-shell junction are computed. None of the induced moments is increased.

B. Validation

This program was verified and document traceability is available at CB&I.

C. Extent of Application

This program is used to compute shell stresses around a rigid attachment to a spherical shell due to any combination of loading - radial, shear, or moment.

3.8.2.4.7.7 <u>CB&I Program 1737</u>. The torus support system was originally analyzed by CB&I Program 1737. The description, validation, and extent of application are included in supplement 3.8B.

3.8.2.4.7.8 <u>CB&I Program 7-78 - Drywell Primary Membrane Stress Analysis</u>.[(8)]

A. Description

The program computes the primary membrane stresses and compares, to the ASME Code, allowable stresses for different loading combinations. In addition, the

program computes compressive stresses and compares to allowable buckling stresses.

The program uses the general primary membrane stress equations for axisymmetrically loaded shell of revolution derived in chapter 14 of "Theory of Plates and Shells," by Timoshenko.[8]

The program computes the allowable compressive stress resultants based on "Biaxial Compression - Equal Unit Forces" and "Biaxial Compression - Unequal Unit Forces" of the WRC Bulletin 69.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

The program is used to calculate the drywell primary membrane stresses for different loading combinations.

3.8.2.4.7.9 CB&I Program 772 -Nozzle Reinforcing.[9]

A. Description

This is a program for checking nozzle reinforcing. It is designed essentially for containment vessels and adheres to area replacement criteria specified by ASME Code, Sections III and VIII.[9] The program does no design work, merely checking the adequacy of preselected reinforcing plate dimensions and weld sizes.

B. Validation

The program by itself does not have the capability to analyze a structure. It merely checks the adequacy of reinforcing plate dimensions and weld sizes. Consequently, validation is not necessary.

C. Extent of Application

The program is used to check the adequacy of plates used for nozzle reinforcing.

3.8.2.4.7.10 CB&I Program 1027 - Stress Intensities.[4]

A. Description

This program determines the stress intensities in a sphere or cylinder at a maximum of 12 points around an externally loaded round or square attachment. Stresses resulting from external loads are superimposed on an initial pressure

stress situation. The program computes stresses at three levels of plate thicknesses: outside, inside, and centerline of plate; four points at the edge of attachment, at $1/2\sqrt{Rt}$ from the edge of attachment, and at the edge of reinforcement.

The program determines three components for each stress intensity:

σx = a normal stress parallel to the vessel's longitudinal axis.

$\sigma\phi$ = a normal stress in a circumferential direction.

T = a shear stress.

The program has an option whereby the penetration load is considered reversible or nonreversible in direction. Under the reversible option, only the data associated with the most severe loading situation is printed.

Most of the analyses and notation used in the program are taken directly from WRC Bulletin 107 of December 1968.[(4)] Use of the program required complete familiarity with this publication.

The program contains extrapolations of the curves for cylinders in WRC Bulletin 107 for γ up to 570.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

The program is used to analyze stress intensities in a sphere or cylinder around an externally loaded round or square attachment.

3.8.2.4.7.11 CB&I Program 1017 - Modal Analysis of Structures.[(10)(11)]

A. Description

Modal Analysis of Structures Using the Eigenvalue Technique

The purpose of the program is three-fold:

- To calculate the mass and stiffness matrices associated with the structural model.

- To determine the undamped natural periods of the model.

- To calculate the maximum modal responses of the structure, i.e., deflections, shears, and moments.

The stiffness and mass matrices may be required in order to perform a dynamic analysis of the structure. The maximum modal responses may be used to perform a spectral analysis.

The program has the following options:

- Vertical translation.
- Torsional modes.
- Soil-structure interaction.
- Liquid sloshing.
- Direct introduction of stiffness and mass matrices.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

The program is used to calculate the mass and stiffness matrices, undamped natural periods and maximum modal responses.

3.8.2.4.7.12 <u>CB&I Program 1044 - Seismic Analysis of Vessel Appendages</u>.[12]

A. Description

Appendages to a vessel may not significantly contribute structurally to the dynamic responses of a model of a vessel. However, appendages can affect the vessel locally by vibrating differently from the model of the vessel at the point of attachment.

The response spectrum method of analysis is not a strictly adequate way of obtaining the maximum appendage accelerations since it does not include the possible consequences of near resonance between the vessel model and the appendage model.

This paper describes the method used to evaluate the maximum elastic differential accelerations between an independently vibrating appendage model and an elastic-beam vessel model at the appendage elevation due to known excitations of the elastic beam model.

The method involves two distinct steps. Firstly, the necessary time-absolute acceleration records are computed at appendage elevations due to model excitations. Secondly, the maximum differential accelerations between each appendage model and the vessel model at the appendage elevation are obtained.

The time-absolute acceleration records at the appendage elevation are computed by use of a step-by-step matrix analysis procedure. The equations of motion for the vessel model are of the form:

$$[M]\{\ddot{u}\} + (AT/\pi)[K]\{\dot{u}\} + [K]\{u\} = [\diagdown M \diagdown]\{\ddot{u}_g\}$$

where:

$[M]$	=	mass matrix, order n x n, obtained from a modal analysis.
$[K]$	=	stiffness matrix, order n x n, obtained from a modal analysis.
A	=	portion of first mode critical damping for the model.
T	=	first mode period of the model.
$[\diagdown M \diagdown]$	=	a diagonal matrix, order n x n, with diagonal elements corresponding to elements of the mass matrix excited by translational accelerations.
$\{\ddot{u}\}$	=	n x 1 matrix of relative accelerations between the model base and the n degrees of freedom.
$\{\dot{u}\}$	=	n x 1 matrix of velocities corresponding to $\{\ddot{u}\}$.
$\{u\}$	=	n x 1 matrix of displacements corresponding to $\{\ddot{u}\}$.
$\{\ddot{u}_g\}$	=	n x 1 matrix of translation base acceleration.
n	=	degrees-of-freedom of vessel model.

By taking a small time increment (smaller than the smallest period obtained from the modal analysis) and letting accelerations vary linearly within the selected increment, the equations of motion can be integrated for the quantities $\{u\}$, $\{\dot{u}\}$, and $\{\ddot{u}\}$ over the selected time increments.(8) The values obtained are superimposed upon the values of these quantities existing at the beginning of the time increment. This process is repeated for the duration of the excitation. The time-absolute acceleration records for each translational degree-of-freedom are the sums of $\{\ddot{u}\}$ and $\{\ddot{u}_g\}$ taken throughout the history of the excitation.

The second step is similar to the first step. The equation of motion (n = 1) is written for the appendage as a single degree-of-freedom elastic model using the time-absolute acceleration record obtained in step 1 at the appendage elevation as the excitation. This equation is solved in the same manner used in step 1. The maximum absolute value of $\{\ddot{u}\}$ obtained is the quantity desired. It is the maximum differential acceleration between the appendage model and the vessel model due to a known excitation of the vessel model.

For any appendage, this two-step procedure should be executed three times. This is required to evaluate normal, tangential, and vertical appendage accelerations with respect to a vessel cross-section.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

The program is used to evaluate the maximum elastic differential accelerations between an independently vibrating appendage model and an elastic beam vessel model due to excitations of the elastic beam model.

3.8.2.4.7.13 CB&I Program 119 - Bolted Flange Design.[9]

A. Description

This program is used for the design of bolted flanges. The program checks the flange design based on Appendix II of ASME Code, Section VIII.[9] Bolt and flange stresses are computed for both the boltup and design conditions. If the bolt and gasket are not overstressed, the computer automatically calculates the required flange thickness or checks any supplied thickness. The minimum gasket width required to prevent crushing and the maximum pressure that the flange is capable of resisting under the design conditions are automatically calculated.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

This program is used for the design of bolted flanges in accordance with the requirements of Appendix II of Section VIII of the ASME Code.

3.8.2.4.7.14 CB&I Program 9-48 - Nozzle Analysis Program - All Loads Mechanical (NAPALM).[9]

A. Description

The basis for the program NAPALM is to analyze nozzles for mechanical loads and find the maximum stress intensity and location. The program analyzes at specified locations from the point of application of the mechanical loads. At each location the maximum stress intensity is calculated for both the inside and outside surfaces of the nozzle. The program includes an input option which results in the analysis to proceed for the sign and magnitude only, or it analyzes, with the magnitude of the loads constant, and varies the sign (plus or minus) for all combinations of loads. The program also uses an option for longitudinal pressure stress. With this option, the program considers this stress to exist and analyzes with it, or the program does not consider this stress and analyzes without it, or the program analyzes with and without this stress. This is to take into account the maximum stress condition, since due to the matching pipe configuration this stress may or may not exist.

The program also includes an option to input additional loads at the thermal sleeve junction to the nozzle. With this option at any location beyond the thermal sleeve junction toward the vessel, the additional loads are applied. These loads are added with their sign and magnitude to the mechanical loads applied elsewhere on the nozzle.

The program gives the largest value or maximum stress intensity for both the inside and outside surfaces and also its location angularly with respect to the axis of the nozzle. Also the principal stresses are printed out, and the stresses resulting from bending, axial load, shear loads, and torsion, as well as the value of the loads which caused this maximum stressed condition.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

This program is used to analyze nozzles for mechanical loads and to compute the maximum stress intensity. The program also identifies the location of the maximum stress intensity.

3.8.2.4.7.15 CB&I Program 10-06 - Rotation of Suppression Chamber Nozzle Loads.[4]

A. Description

This program rotates nozzle loads from a global coordinate system to a nozzle coordinate system per WRC Bulletin 107.[4] This program has been written for the torus-shaped suppression chambers.

The input loads may be applied at the shell or at the end of a pipe connected to the shell. The pipe may be inside or outside of the torus. However, the pipe must lie in the vertical plane containing the penetration point and a radius of the cylindrical segment. Output loads are applied at the shell.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

The program is used to rotate nozzle loads from a global coordinate system to a nozzle coordinate system for the torus-shaped suppression chambers.

3.8.2.4.7.16 **CB&I Program 1342 - Three-Dimensional Frame.**

A. Description

This is a three-dimensional-frame program to provide analysis capabilities for determining deflections, rotations, and member reactions on general space frames.

The program handles the following:

- Any support combination.
- Any member end condition.
- Distributed and concentrated member loads at any angle.
- Joint loads.
- Any number of loading conditions.
- Thermal stresses.
- Joint displacements.
- Shear deformations.
- Members that can carry only tension or compression.
- Rectangular or cylindrical coordinate input.
- Plotting option for geometry check.

The input coordinate system is similar to the "Stress" coordinate system and uses the global and local coordinate systems. The joint coordinates are input in the global system, and joint loads and restraints are input in this system.

Member properties and member loads are input in the local system. Conversion from the global to the local coordinate system is accomplished in the same manner as in the "Stress" program. The output consists of joint reactions and displacements which are in the global coordinate system and member end loads which are in the local coordinate system.

The units are not as flexible as the units used in the "Stress" program in that all loads must be in terms of kips and all dimensions must be in the terms of feet.

B. Validation

The program was verified and document traceability is available at CB&I.

C. Extent of Application

The program is used to provide analysis capabilities for determining deflections, rotations, and member reactions on general space frames.

3.8.2.5 Structural Acceptance Criteria

The fundamental acceptance criteria for the containment is the successful completion of the structural integrity bare-vessel test at 125% of design pressure.

The design and analysis methods, as well as the type of construction and construction materials, are chosen to allow assessment of the structure's capability throughout its service life. Additionally, surveillance testing provides further assurances of the structure's continuing ability to meet its design functions.

The actual and allowable stresses at critical sections of the containment are listed in table 3.8-2 for different loading combinations.

The low values of the resultant stresses identified for the first item of table 3.8-2, sheet 1, are obtained from primary principal membrane stresses and are due to the low operating internal pressure which is < 2 psi as identified in paragraph 3.8.2.3.1(H).
When shear stress components are zero, then:

$$\sigma_t = \sigma_1 = \sigma_\phi$$

$$\sigma_\ell = \sigma_2 = \sigma_x$$

$$\sigma_r = \sigma_3 = 0$$

Stress intensity, Sm, is the largest absolute value of:

$$S_{12} = |\sigma_1 - \sigma_2| = |\sigma_\phi - \sigma_x|$$

$$S_{23} = |\sigma_2 - \sigma_3| = \sigma_x$$

$$S_{31} = |\sigma_3 - \sigma_1| = \sigma_\phi$$

where:

$\sigma_t, \sigma_\ell, \sigma_r$	=	stress components in the tangential, longitudinal, and radial directions.
$\sigma_1, \sigma_2, \sigma_3$	=	principal stresses derived from $\sigma_t, \sigma_\ell,$ and σ_r.
σ_ϕ, σ_x	=	principal primary membrane stresses in the circumferential and meridional directions.
S_{ij}	=	stress difference in the i and j directions.
Sm	=	stress intensity.

The above method of computing stress intensity conforms to subsection NE-3215 of Section III of the ASME Boiler and Pressure Vessel Code. Table 3.8-2 was revised to show the stress intensities.

Figure 3.8-9 identifies the different levels at which stress computations were made and includes the thicknesses of the shell at those levels.

The calculated and allowable stresses at local areas of the containment, such as the personnel lock, are shown in table 3.8-3.

3.8.2.6 <u>Design Loading Combination Stress Limits</u>

The stress limits for different loading combinations are listed in table 3.8-4. A primary membrane stress limit of 0.9 Sy (specified minimum yield stress at appropriate temperature) has been used for DBE loading combinations to restrict the stresses within yield. For jet load impingement loadings on the shell backed up by bioshield concrete, an elastoplastic finite element analysis was performed to show that the computed strain is within the code allowables.

The compressive stress resultants were compared to the allowables obtained according to the paragraphs titled "Biaxial Compression - Equal Unit Forces" and "Biaxial Compression - Unequal Unit Forces" of the WRC Bulletin 69. The allowables used are found by assuming that the sphere reacts as a cylinder with a radius equal to the radius of the sphere. There are three cases of loading considered. The allowables for these three cases are:

- Uniaxial compressive stress resultant.

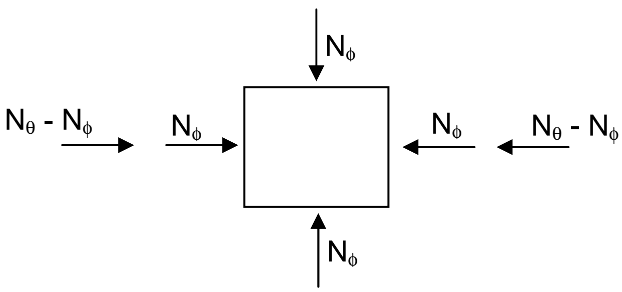
$$N_{ALL} = 1.8 \times 10^6 \frac{t^2}{R}$$

- Biaxial equal compressive stress resultants.

$$N_{ALL} = 0.9 \times 10^6 \frac{t^2}{R}$$

- Biaxial unequal compressive stress resultants.

  This case is treated as the summation of an uniaxial condition with the biaxial condition with equal stress resultants. (See sketch.)



$$\frac{N_\theta - N_\phi}{1.8 \times 10^6 \frac{t^2}{R}} + \frac{N_\phi}{0.9 \times 10^6 \frac{t^2}{R}} \leq 1$$

where:

N_{ALL} = allowable compressive stress resultant.

t = thickness of the shell.

R = radius of the equivalent cylinder.

N_θ = circumferential membrane stress resultant.

N_ϕ = meridional membrane stress resultant.

3.8.2.7 Materials, Quality Control, and Special Construction Techniques

The pressure parts and attachments to pressure parts of the containment vessel, penetrations, and appurtenances meet the requirements of Section NE-2000 of Section III of the ASME Code and were fabricated from the following materials:

A. Plate

- SA 36.

- SA 516, Grade 70.

- SA 240, Type 304.
- SA 537, Grade A.

B. Pipe

- SA 333, Grade 6.
- SA 312, Type 304.

C. Forgings

- SA 350, Grade LF2.
- SA 105, Grade II.
- SA 182, Type 304.
- SA 479, Type 304.

D. Bolting and Nuts

- SA 320, Grade L43.
- SA 193, Grade B7.
- SA 194, Grade 7.

E. Fittings

- A 234, Grade WPB.

The nonpressure parts of the containment were fabricated from the following materials:

- A 36.
- A 514, Grade F (T-1).
- A 53, Grade B.
- A 106, Grade B.
- A 283, Grade C.
- A 516, Grade 70.
- SA 105, Grade II.

- SB 443, Alloy 625.

The provisions of USNRC Generic Letter 89-09 were implemented for modification of the bellows assemblies for penetrations X-8 and X-9A. The material used to fabricate the replacement bellows assemblies conforms to the requirements of SB-443, Alloy 625.

The pressure parts and attachments to the pressure parts of the containment vessel, with the exception of the austenitic steel materials, meet the longitudinal Charpy V-notch impact test requirements of Section NE-2300, with minimum impact values not less than those specified in Appendix I of the ASME Code, Section III. The impact specimens were tested at 0°F.

The quality assurance provisions of Section NA-4000 of Section III of the ASME Code were followed in all phases of procurement, shop fabrication, and field installation of the containment.

The following surfaces were sandblasted in accordance with Specification SSPC-SP5 and were coated with the inorganic zinc primer Dimetcote 6 to a dry film thickness between 3.0-mils minimum and 5.0-mils maximum:

- Interior surface of the suppression pool.

- Interior and exterior surfaces of the vent lines, vent header, and downcomers within the suppression pool.

- Exterior surface of the vent-header supports.

- All other appurtenances and attachments within the suppression pool, with the exception of stainless steel surfaces.

The inorganic zinc surfaces below the waterline in the suppression pool were later coated, as required, with a DBA qualified, 100% solids epoxy installed by underwater application.

The surfaces listed below have been sandblasted in accordance with Specification SSPC-SP10 and were primed with the inorganic zinc Dimetcote 6 to a dry film thickness of 2.0-mils minimum and 4.0-mils maximum followed by a topcoat of the organic coating Ameron 90 to a dry film thickness of 3.0-mils minimum and 5.0-mils maximum.

- All exposed drywell interior surfaces above el 114 ft 6 in.

- Jet deflectors of the vent openings.

- Exterior surfaces of the drywell above the water seal support bracket at el 201 ft 4 in.

- Interior surfaces of the equipment hatches.

- All other appurtenances and attachments within the drywell, including both exposed surfaces of the reactor pressure vessel support pedestal shells.

The above protective coating operation has been carried out in full compliance with the quality assurance requirements for protective coatings applied to water-cooled nuclear power plants described in Regulatory Guide 1.54 (June 1973) except that American National Standards Institute (ANSI) N45.2-1971 was not used.

The design, furnishing and fabrication, delivery and unloading, erection, and code stamping were performed by CB&I using proven methods, tools, and equipment generally used by this type of industry.

3.8.2.8 Testing and Inservice Surveillance Requirements

This subsection describes the inspection and tests that are provided for the various systems or components of the primary containment as they apply during construction or plant operation.

3.8.2.8.1 Objectives

The objectives of these tests and inservice surveillance requirements are to ensure that:

- Leakage through the primary reactor containment and systems and components penetrating primary containment does not exceed allowable leakage rates specified in 10 CFR 50, Subsection 50.54, Appendix J, and the Technical Specifications.

- Periodic surveillance of primary reactor containment pentrations and isolation valves is performed so that proper maintenance and repairs can be made during the service life of the primary containment.

3.8.2.8.2 Leakage Testing To Verify Primary Containment Integrity

Fabrication procedures, nondestructive testing, and sample coupon tests were made in accordance with the ASME Code for Boilers and Pressure Vessels, Section III, Subsection NE, 1971 edition, including 1971 Summer Addenda. The integrity of the primary containment system was verified by a pneumatic test of the drywell and suppression chamber at 1.25 times their design pressure of 56 psig in accordance with Code requirements. An initial leakage test, performed in accordance with 10 CFR 50, Appendix J,[13] has also been successfully completed. These tests were completed upon erection of the primary containment.

The preoperational and periodic leakage tests of the primary containment and systems and components penetrating primary containment are performed in accordance with Appendix J to 10 CFR 50.[13] The integrated leak rate test (ILRT) is performed using the methods presented in BN-TOP-1 or ANSI/ANS-56.8-1994.[14] The testing methods, frequency, and acceptance criteria are specified in the Technical Specifications and are summarized briefly below. The terminology is consistent with reference 13.

3.8.2.8.2.1 Integrated Leak Rate Test (Type A Tests).

A. Objective

The objective is to confirm that the maximum allowable leak rate of 1.2 weight percent of the contained air per 24 h at peak calculated (test) pressure is not exceeded.

B. Test Methods

Both a reduced pressure test and a peak calculated pressure test were conducted prior to unit operation. ***TRM table T7.0-1*** provides a list of the type A tested items.

1. For the containment ILRT (type A) program, the test requires that data be collected at least hourly during type A testing. The time when containment conditions stabilize is monitored with the stipulation that the time period required for stabilization before the test is initiated is at least 4 h.

2. Systems which may communicate with the containment atmosphere under LOCA conditions are as follows:

    - Service air system.
    - H_2 and O_2 analyzer system.
    - Demineralized water system.
    - Nitrogen inerting system.
    - Purge and inerting system.
    - Fission products monitoring system.
    - Neutron monitoring system (NMS).
    - Main steam lines.
    - High-pressure coolant injection (HPCI) steam lines.
    - Reactor core isolation cooling (RCIC) steam lines.

    The above systems are drained and vented to the atmosphere in accordance with 10 CFR 50, Appendix J.

    In addition to the above systems, the drywell pneumatic system is depressurized and vented to ensure that no leakage of pressurized air into the containment occurs during the type A test. Also, the reactor pressure

vessel is vented allowing nuclear boiler system components to be subjected to containment atmosphere pressure as well as the water head created by maintaining the reactor water level at the normal operating level during the test.

3. The following is a tabulation of those systems in the containment which were not vented and drained during the type A test. The letters on the right side of the table refer to criteria paraphrased from 10 CFR 50, Appendix J, which are included after the table.

System	Criteria for Not Draining and Venting
• Nuclear boiler system (RPV and feedwater lines)	d
• Reactor recirculation system (RRS)	d
• CRD insert and withdraw lines	b
• Standby liquid control system (SLCS)	a
• HPCI system (torus suction)	d
• RCIC system (torus suction)	d
• Radwaste system	b, e
• Reactor water cleanup (RWC) system	a, f
• Core spray (CS) system	c
• RHR system	c
• Reactor building closed cooling water (RBCCW) system	b, g
• Chilled-water system	b, g
• Steam valve sealing system	a

Criteria:

a. The system is part of the reactor coolant boundary and does not open to the containment atmosphere normally or during a LOCA

and therefore is not drained. However, it is subjected to P_a as described in B.2 above.

b. The system is not part of the reactor coolant boundary and does not communicate with the containment atmosphere normally or during a LOCA.

c. The system is filled with water and is operating under post-LOCA conditions.

d. The system is required to maintain the plant in a safe condition during the test and therefore is not drained. However, it is subjected to P_a as described in B.2 above.

e. Radwaste system primary containment penetrations are discharge lines from submersible sump pumps. Each radwaste sump pump suction is sealed from the primary containment atmosphere by water in the sump during both normal and LOCA conditions.

   The containment isolation valves for each radwaste pump discharge line are normally closed and are provided with a primary containment isolation signal. Isolation valve leakage is monitored by air testing as part of the type C leakage test program. The valves are also subject to P_a through the sump during the type A test.

   Radwaste sump pump discharge piping is ASME III, Class 3, from the pump discharge to the containment penetration, where it is qualified ASME III, Class 2, to, and including, the outboard isolation valve. Piping from the pump discharges to, and including, the outboard isolation valves is also Seismic Category I.

f. The RWC piping within the primary containment is Seismic Category I, Quality Group A piping and is part of the reactor coolant boundary normally and post-LOCA. The RWC piping is connected to the nuclear boiler system through suction connections on the recirculation loop and the bottom head drain on the RPV. The lines are normally full of coolant and remain full post-LOCA, even if the postulated break occurs in the recirculation loop with the RWC connection, because of the interface with the RPV bottom head drain.

   RWC isolation valves are also tested with air as part of the type C leakage test program.

g. The RBCCW and chilled-water systems within the primary containment meet the criteria of GDC 57 for closed systems within the primary containment and meet BTP6.2-3 with the exception of

quality group classification of the piping. ***TRM table T7.0-1*** compares the design requirements of the Quality Group D RBCCW and chilled-water piping to the design requirements of comparable Quality Group B piping.

As closed systems, the RBCCW and chilled-water piping do not communicate with the primary containment atmosphere, normally or post-LOCA, and therefore are not vented or drained during the type A test.

Isolation valve leakage is monitored in the type C test program.

4. The supplemental test for the ILRT type A test was calibrated to leakage type test similar to the test described in ANSI 45.4 - 1972, Appendix C. The verification test was performed after the reduced pressure test and after the peak pressure test.

   A controlled leakage rate based on the ILRT result is imposed upon the containment by using the verification test portion of the integrated leak rate measurement system. The verification test portion of the measuring system allows a calibrated leak to be placed on the containment through the utilization of a bleedoff throttle valve. A flowmeter is installed downstream of the throttle valve to monitor the leak rate. The verification tests are run for a minimum of 5 h, during which the flowmeter data is taken hourly.

   During the verification test, the containment leakage measuring equipment measures a composite of the imposed leakage rate and the actual leakage rate. During the test, the leakage rate is calculated by subtracting the imposed leakage rate from the composite leak rate. The acceptance criteria for the test are as follows:

$$\left(L_i + L_{tm} + 0.25L_t\right) > L_{VM} > \left(L_i + L_{tm} - 0.25L_t\right)$$

   where:

   L_{tm} = containment leakage rate calculated during the ILRT at the same containment pressure at which the verification test is run.

   L_t = maximum allowable leakage rate for the ILRT at the same pressure as the verification test.

   L_{vm} = containment leakage rate calculated during the verification test.

   L_i = leakage rate imposed on the containment using the flowmeter.

5. The limiting conditions for operation and the surveillance requirements required for ILRT type A are included in the Technical Specifications.

C. Acceptance Criteria

To provide some margin against normal leakage deterioration which may occur during the period between leak rate tests, the allowable operational leak rate is derived by multiplying the maximum allowable test leak rate by 0.75. Specific acceptance criteria are provided in the Technical Specifications.

3.8.2.8.2.2 <u>Leak Tests of Penetrations and Isolation Valves (Type B and C Tests)</u>. Containment isolation valves (except for main steam isolation valves) and primary containment components which seal or penetrate the pressure-containing boundary are periodically tested at the peak calculated pressure. Test frequencies are established per 10 CFR 50, Appendix J, Option B as implemented by the Primary Containment Leakage Rate Testing Program.

The combined leakage rate of components subject to type B and C (except for main steam isolation valves) does not exceed 0.6 times the maximum allowable leakage rate. An additional restriction is placed on the personnel airlock which does not exceed a leakage rate of 0.05 times the maximum allowable.

The main steam isolation valves (MSIVs) are tested independently per Techncial Specification requirements.

All penetrations, seals, and isolation valves affected by these tests are listed in ***TRM table T7.0-1***. Specific testing and acceptance criteria are provided per the Primary Containment Leakage Rate Testing Program.

Additional information regarding the containment local leak rate (types B and C) test program is provided in paragraph 3.8.2.8.2.2.1.

3.8.2.8.2.2.1 Containment Local Leak Rate (Type B and C). Figures 3.8-16 through 3.8-23 provide diagrams of the typical type C test arrangement used. ***TRM table T7.0-1*** lists the specific reference figures which depict the orientation of each test boundary. The isolation valves listed in ***TRM table T7.0-1*** are tested with the test volume water filled, then pressurized with air or nitrogen, since the volume would remain water filled post-LOCA.

The personnel airlock barrel is tested at P_a per 10 CFR 50, Appendix J. The barrel test is performed by pressurizing between the inner and outer doors and verifies the overall pressure integrity of the barrel. A 10-psig pressure test is performed on the airlock door seals at the frequency specified by 10 CFR 50, Appendix J, to verify that the seal leakage rate is less than detectable.

3.8.2.8.2.3 <u>Drywell-to-Pressure Suppression Chamber Bypass Area Tests</u>. At the frequency specified by Technical Specifications, a leak rate test is performed to verify that significant leakage flow paths do not exist between the drywell and pressure suppression chamber. The existence of such leakage paths would result, in the event of a primary system rupture, in blowdown steam passing directly to the suppression chamber free-air space without

being condensed in the suppression pool. Since the design pressure of the containment is predicated on the experimentally verified assumption that all the blowdown steam is condensed in the suppression pool, the existence of bypass paths could possibly result in the containment design pressure being exceeded.

A. Objective

The objective of bypass area leak testing is to detect flow paths between the drywell and suppression chamber whose capacity is equal to or greater than the capacity of a 1-in.-diameter orifice.

The smallest pipe that is a part of the vent system is 1-in. pipe, whose failure could result in a drywell-to-suppression chamber leakage path. There are 12 of these 3/4-in., schedule 80 lines which serve as drain lines for the vent headers and vacuum breaker valves.

B. Test Method

To conduct the drywell-to-suppression chamber bypass area leak rate test, the drywell pressure is increased by ~ 1 psi with respect to the suppression chamber pressure and held constant. The 2-psig scram setpoint is not exceeded. The subsequent suppression chamber pressure transient (if any) is monitored with a manometer capable of detecting a small pressure increase. If the drywell pressure cannot be increased by 1 psi over the suppression chamber pressure, it would be because a significant leak path exists; in this case, the leakage source is identified and eliminated before primary system pressurization.

Drywell-Suppression Chamber Testing Program

The drywell-to-suppression chamber bypass area test is performed with all vacuum breakers between the suppression chamber and drywell lined up in their normal operating condition.

A pressure decay test is performed by increasing the drywell pressure by > 1 psig higher than the suppression chamber pressure. The pressure decay is monitored by using a manometer over a 10-min period.

Boundary Conditions for Drywell Testing Program

During the test period, there is no operation of the following equipment:

- RHR system in either the containment spray or pool cooling mode.
- RCIC system.
- HPCI system.
- Relief valves.

The objective of these restrictions is to prevent temperature variations in either the pool or suppression chamber airspace during the test. There are no energy dumps to the pool near the end of the refueling outage and a constant temperature situation is expected to exist in the suppression chamber at the time of the test.

C. Acceptance Criteria

With a differential pressure > 1 psi, the rate of change of the suppression chamber pressure must not exceed 0.25 in. of water per minute as measured over a 10-min period. In the event the rate of change exceeds this value, then the source of leakage will be identified and eliminated before power operation. Figure 3.8-24 shows the drywell and suppression chamber pressure transients assuming a 1-in. orifice leakage path to exist and assuming the drywell pressure was increased 1.25 psi in a 5-min period. Figure 3.8-25 shows the associated pressure differential between the drywell and suppression chamber. It can be seen that there is a 20-min period during which the differential would be greater than 1 psi; thus, there would be ample time to conduct a 10-min test.

3.8.2.8.3 Inspection and Testing Features

The following features of plant design were provided to allow testing and inspection in accordance with the above criteria and objectives.

3.8.2.8.3.1 **Penetrations and Seals**. Pipe penetrations, which must accommodate thermal movement, are provided with expansion bellows such as the penetration shown in figure 3.8-2. By use of the pressure test tap, a gas (nitrogen or other as required for leak detection) can be injected into the annulus, and by soap film, pressure decay, or other means, leakage can be detected and measured during shutdown without pressurizing the entire primary containment system. The test tap is plugged during normal operation to prevent leakage through the test tap in the event of a leak within the penetration.

Electrical penetrations are also provided with double seals and are also separately testable. The test taps and seals are located so that the tests of the electrical penetrations can be conducted without entering or pressurizing the drywell or suppression chamber.

All containment closures, which are fitted with resilient seals or gaskets, are separately testable. The covers on flanged closures, such as the equipment access hatch cover, the drywell head, and the access manholes, and personnel airlock doors are provided with double seals without pressurizing the entire containment system. Details of the containment airlock design which permits pressure testing are shown on figure 3.8-26.

3.8.2.8.3.2 Isolation Valves.

A. The test capabilities incorporated into the primary containment system to permit leak detection testing of containment isolation valves are separated into two categories.

The first category consists of those pipe lines which open into the containment and are not connected to the reactor vessel. In lines that contain two power-operated isolation valves in series, a test tap is provided between the valves which permits leakage monitoring of the first valve when the containment is pressurized. The test tap can also be used to pressurize between the valves to permit leakage testing of both valves simultaneously.

The second category consists of those pipe lines which are connected to the reactor vessel. In lines that contain two power-operated valves in series, a test tap is provided between the valves which permits leakage monitoring of the first valve when the reactor vessel is pressurized. The test tap can also be used to pressurize between the valves to permit leakage testing of both valves simultaneously when the reactor vessel is not pressurized. In lines that contain one inboard check valve and one outboard power-operated valve, a test tap is provided opposite the containment side of the outboard valve. Leakage through the inboard check valve can be monitored through the test tap by opening the outboard valve when the reactor is pressurized. Leakage through the outboard valve can be monitored by opening the inboard check valve when the reactor is pressurized.

B. A test connection is located between the two series check valves in each of the reactor feedwater lines. This test connection is used to leak test the outboard check valve with the inboard gate valve closed.

Another test connection is located on the reactor side of the inboard check valve between the inboard check valve and gate valve. This test connection is used to test the inboard check valve with the inboard gate valve closed.

C. A test connection is provided between the two valves in the reactor building to torus vacuum relief lines. With the inner air-operated valve held shut, leakage past the outer check valve is measured. Each of the two parallel lines would be tested individually. Thus, if the plant were in operation during the tests, the vacuum-breaking capability is still effective.

3.8.2.8.3.3 Drywell-to-Suppression Pool Vacuum Breaker Tests and Inspections. The drywell-to-suppression pool vacuum breakers are tested for operability monthly, using the redundant position indication installed on each valve. Each valve is cycled from both the main control room and the local panel. The indicating lights are monitored at each station as the valve cycles.

The vacuum breakers are visually inspected and leak tested by the method described in paragraph 3.8.2.8.2.3 at each refueling outage. The opening differential pressure for each valve is also checked at each refueling outage by measuring the force required to open the disc. This force would result from a 0.1-psid ΔP existing across the valve.

Operators are installed on the vacuum breakers to provide exercising capabilities. However, the valves self-actuate when the setpoint differential pressure exists across the disc and pop open in < 1 s. Therefore, an operational test for the determination of opening time is not feasible.

3.8.3 CONTAINMENT INTERNAL STRUCTURES

The containment internal structures are all Seismic Category I, consisting of:

- RPV pedestal.
- Reactor shield wall.
- Recirculation pump supports.
- Other structures.

3.8.3.1 <u>Description of Internal Structures</u>

A. RPV Pedestal

The pedestal consists of two concentric steel shells 18 ft 3 in. and 26 ft 3 in. in diameter with concrete fill in between the shells to provide mass and stability. The concrete strength is not considered in design. Stiffeners are provided at different locations to distribute the load uniformly over larger areas of the shell. The pedestal supports the RPV, reactor shield wall, intermediate platforms, CRD platform, pipe-whip restraints, pump restraints, pipe hangers, and snubbers. The bottom of the pedestal is anchored to the base slab by means of ninety-two 3-in.-diameter A-193 B7 anchor bolts which transfer the loads to the foundation. The reactor shield wall columns are directly welded to the top of the pedestal. Provisions are made at the top of the inner shell to inspect the reactor vessel bolting rings. The details of RPV pedestal are shown in drawing nos. H-25004 and H-25005.

B. Reactor Shield Wall

The reactor shield wall consists of 12 buildup steel columns with 3/8-in.-thick steel liner plate welded on both sides of the column flanges. A 1 3/4-in.-thick liner plate is provided on the outside flange of the core area for radiation shielding. Intermediate ring beams are provided at various levels to accommodate the restraints. The reactor shield wall is rigidly connected at the base to RPV pedestal

and laterally supported at el 188 ft 1/2 in. by a star truss. The star truss transfers seismic and other forces from the reactor vessel and the shield wall to the drywell shield concrete through the eight lugs in the drywell. The flat development of the reactor shield is shown in figure 3.8-27.

C. Recirculation Pump Supports

Recirculation pumps and motors are hung from platforms at el 127 ft 9 in. and el 148 ft 3 1/2 in. The snubbers for the pumps are attached to the RPV pedestal.

D. Other Structures

1. Inservice Inspection Platforms

Two major platforms at el 127 ft 9 in. and el 148 ft 3 1/2 in. are provided inside the drywell. The lower platform spans between the containment and the RPV pedestal and the upper platform spans between the containment and the reactor shield wall. Heavy steel I-beams and builtup box girders are used to carry pipe restraint and other loads. The beams are braced laterally to minimize torsion and to provide overall stability. Lubrite pads are provided at the drywell end of the beams for thermal movements. The other end of the beams are welded either to the pedestal at el 127 ft 9 in. or to the ring girder at el 148 ft 3 1/2 in. Typical connection details are shown in figure 3.8-28.

Other platforms are provided at various locations for inservice inspection access. The general arrangement of the platforms is shown in figure 3.8-28.

The inspection platforms provide access to inspect pipe welds, nozzle welds, and vessel welds in addition to providing working area for normal maintenance.

2. Inservice Inspection Doors

Inservice inspection doors provided in the reactor shield wall are used for inservice inspection of the nozzle welds at the outside face of the RPV. The doors are of heavy steel plates, up to 4 1/2-in.-thick, with 6 7/8 in. to 8-in. type 277M concrete fill, manufactured by Reactor Experiments, Inc. The door frames are tied to the reactor shield wall. The total thickness of doors varies from 13 3/8 in. to 15 in.

Typical arrangement of the door for the 28-in.-diameter recirculation line is shown on drawing no. H-29000.

3. Pipe Whip Restraints and Barrier Plates

Pipe whip restraints are provided inside the containment to protect it from pipe whip due to a high energy pipe break. A typical pipe whip restraint is a

steel bracket with wire ropes wrapped around the pipe and is shown on drawing no. H-29026.

Where pipe whip restraint installation is not possible due to limited space restrictions, barrier plates are provided to protect the containment integrity from pipe whip. The barrier plates in the cylindrical portion of the drywell are shown on drawing no. S-28345.

4. Drywell Concrete Floor at el 114 ft 6 in.

The reinforced concrete slab in the drywell at el 114 ft 6 in. provides a convenient level working area and supports the equipment.

5. Refueling Water Seal Assembly

The water seal assembly shown on drawing no. S-27793 is provided in the drywell cylinder at el 203 ft 4 l/2 in. to form a leaktight barrier for retaining and supporting the water above this level during refueling operation.

6. Miscellaneous Components

Miscellaneous components such as jet deflectors, monorails, spray headers and their supports are shown in figure 3.8-29.

3.8.3.2 Applicable Codes, Standards, and Specifications

The following regulations, codes, standards, regulatory guides, and specifications apply to the original design of the containment internal structures:

A. Regulations

- Title 10 Code of Federal Regulations Part 50, "Licensing of Production and Utilization Facilities."

- Title 29 Code of Federal Regulations Part 1910, "Occupational Safety and Health Standards."

B. Codes and Standard Specifications

- American Concrete Institute (ACI), "Building Code Requirements for Reinforced Concrete" (ACI 318-63).

- ACI, "Specifications for Structural Concrete for Buildings" (ACI 301-66).

- American Institute of Steel Construction (AISC), "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," 7th Edition.

- AWS, "Structural Welding Code" (AWS D1.1-72).

C. General Design Criteria, Regulatory Guides, Industry Standards, and Topical Reports

- 10 CFR 50, Appendix A - General Design Criteria for Nuclear Power Plants.

- GDCs 2, 3, 4, and 16.

- NRC Regulatory Guides.

  - Regulatory Guide 1.15, "Testing Reinforcing Bars for Category I Concrete Structures" (December 1972).

  - Regulatory Guide 1.29, "Seismic Design Classification" (August 1973).

  - Regulatory Guide 1.46 "Protection against Pipe Whip Inside Containment" (May 1973).

  - Regulatory Guide 1.55, "Concrete Placement in Category I Structures" (June 1973).

- Industry Standards.

  Nationally recognized industry standards, such as those published by the ASTM, are used whenever possible to describe material properties, testing procedures, fabrication methods, and construction methods.

- Bechtel Topical Reports.

  BN-TOP-2, "Design for Pipe Break Effects," Revision 2, May 1974.

For new modifications and analysis of modifications installed after the plant was put into operation, later editions of the following codes will be used:

- AISC - "Manual of Steel Construction."

- ACI - "Building Code Requirements for Reinforced Concrete" (ACI 318).

- ACI - "Specifications for Structural Concrete for Buildings" (ACI 301).

- American Welding Society (AWS) - "Structural Welding Code" (AWS D1.1).

For analysis or modification of original plant designs, a later edition of the codes listed above may be used; however, the applicable sections of the original plant design codes must be reviewed. Differences between the original design codes and a later edition of these codes should be documented. Wherever a code change that is applicable to the design has occurred,

a later edition of the code may be used if the change results in a more conservative design than the original design code, or the change results in an acceptable decrease in conservatism based upon a better knowledge or understanding of the condition by the code committee because of tests or experience. If the code change results in a less conservative design and this change is based upon a change in material quality or quality of installation, then the section from the original code edition will be used.

To account for changes in steel member properties and dimensions over the years, this information will be obtained from the AISC Code edition used for the original design.

3.8.3.2.1 Structural Specifications

Structural specifications are prepared to cover the areas related to design and construction of the plant structures. These specifications are prepared by Bechtel and Southern Company Services, Inc., specifically for these structures. The specifications emphasize important points of the industry standards for these structures and reduce options such as would otherwise be permitted by the industry standards. Unless specifically noted otherwise, these specifications do not deviate from the applicable industry standards and as such need not be included in the safety analysis report. These specifications cover the following areas:

- Furnishing and delivery of concrete.
- Purchasing, forming, placing, and curing of concrete.
- Furnishing, detailing, fabricating, delivery, and placing of reinforcing steel.
- Furnishing, delivery, and erection of structural steel.

3.8.3.3 <u>Loads and Loading Combinations</u>

The internal structures are designed for all credible conditions of loadings, including normal loads, seismic loads, and loads resulting from a LOCA.

Critical loading combinations are those caused by a postulated earthquake or by a pipe rupture within the containment. In addition to the loads listed below, the internal structures inside the torus were designed for hydrodynamic loads due to LOCA and safety relief valve discharge. The hydrodynamic loads and load combinations considered are summarized in supplement 3.8B.

3.8.3.3.1 Loads

The following loads are considered: dead loads, live loads, earthquake loads, pipe-rupture loads, thermal loads, pressure loads due to LOCA, hydrostatic loads, and impact loads.

A. Dead Loads (D)

Structural dead loads consist of the weight of platforms, all permanent equipment, major piping, and electrical ducts.

B. Live Loads (L)

Live loads consist of design floor loads, laydown loads, equipment live loads, and fuel handling equipment loads.

Live loads considered in design are:

1.	Floor at el 114 ft 6 in.	200 lb/ft^2
2.	Platforms and floors at el 127 ft 9 in. and el 148 ft 3 1/2 in.	150 lb/ft^2 plus 30-kip moving load
3.	Catwalk inside torus	75 lb/ft^2
4.	Inservice inspection platforms	100 lb/ft^2

Weight of water used to fill the drywell from el 203 ft 4 in. to el 227 ft during refueling was considered a live load for the refueling water seal assembly design.

C. Earthquake Loads (E, E^1)

The OBE loads, E, and DBE loads, E^1, derived from the seismic analysis in section 3.7 are used in design.

D. Pipe Rupture Loads (Y_r, Y_j, Y_m)

Y_r = Equivalent static load on the structure generated by the reaction on the broken high-energy pipe during the postulated break, including an appropriate dynamic load factor to account for the dynamic nature of the load.

Y_j = Jet impingement equivalent static load on a structure generated by the postulated break, including an appropriate dynamic load factor to account for the dynamic nature of the load.

Y_m = Missile-impact equivalent static load on a structure generated by or during the postulated break, as from pipe whipping, including an appropriate dynamic load factor to account for the dynamic nature of the load.

E. Thermal Loads (T_o, T_a)

T_o = Forces on structure or equipment due to thermal expansion of pipes or components under operating condition.

T_a = Forces on structure or equipment due to LOCA, including T_o.

F. Pressure Loads due to LOCA (P_a)

A LOCA results in an increased pressure of the surrounding area. This pressure load P_a does not include the jet forces resulting from rupture of pipes.

G. Pipe Reaction Loads

R_o = Pipe reactions during normal operating or shutdown conditions based on the most critical transient or steady-state condition.

R_a = Pipe reactions under thermal conditions generated by the postulated break and including R_o.

3.8.3.3.2 Loading Combinations

The three different loading combinations considered for the design of internal structures were: normal operation, refueling, and LOCA.

A. Normal Operation

The loading combinations for normal operating condition are:

$D + L + E + T_o + R_o$

$D + L + E^1 + T_o + R_o$

B. Refueling

During refueling operation, the drywell cylinder is filled with water up to el 228 ft 0 in. The water seal assembly is subjected to hydrostatic load during this period. The loading combinations for this condition are:

$D + L + E$

$D + L + E^1$

C. LOCA

The loading combinations used for the postulated LOCA are:

$D + L + E + Y_r + Y_j + Y_m + T_a + P_a + R_a$

$D + L + E^1 + Y_r + Y_j + Y_m + T_a + P_a + R_a$

3.8.3.4 Design and Analysis Procedures

The internal structures are designed to provide structural supporting elements for the nuclear steam supply system (NSSS) as well as required shielding. Basic supporting components are of structural steel. All design aspects are integrated with the design criteria of the NSSS supplier and include thermal and dynamic effects evident during earthquakes. The elastic working stress design method was used in Seismic Category I steel structures design.

Design of internal structures evolves around four basic systems:

- Recirculation.
- Main steam.
- Feedwater.
- Engineered safeguards.

3.8.3.4.1 RPV Pedestal

The RPV pedestal is designed as a freestanding structure fixed at the base. Basically, the design of the RPV pedestal is divided into four sections:

- General shell design.
- Shell stiffening.
- Pedestal top section.
- Penetrations.

The pedestal shells are analyzed for all combinations of loading described in paragraph 3.8.3.3.2.

In areas of major attachments, the shell is locally reinforced with a stiffener system to prevent local buckling of the shell plates and to distribute the loads over large areas of the pedestal.

The top section of the pedestal is designed to transmit the reactor vessel and the reactor shield wall column loads to the inner and outer shells.

The pedestal shell is reinforced in the areas of major penetrations. The analysis and design of the pedestal are based on conventional methods found in standard textbooks and handbooks used in the engineering profession.

3.8.3.4.2 Reactor Shield Wall

The reactor shield is designed without considering the concrete for structural strength. Concrete is used as filler material for shielding. The forces considered were: seismic forces, pipe loads, pipe restraints, platform loads, jet loads, and uniform internal pressure generated due to pipe break in the annulus formed by the reactor shield and RPV.

For seismic design, reactor shield was modeled as a lumped mass spring system coupled with the reactor building, drywell, RPV, and RPV pedestal. A space frame model consisting of columns and ring girders at various elevations is used for the stress program CE-309 to check the stresses in individual members for different combinations of loading that the shield is subjected to.

3.8.3.4.3 Recirculation Pump Supports

The design and analysis procedures are based on conventional methods found in standard textbooks and handbooks used in the engineering profession.

3.8.3.4.4 Other Structures

A. Inservice Inspection Platforms

Inservice inspection platforms are designed for applicable loads and loading combinations using conventional methods found in standard textbooks used in the engineering profession.

B. Inservice Inspection Doors

Inservice inspection doors are provided with door frames which transfer loads to the reactor shield wall. These doors are designed for jet forces due to a postulated complete circumferential break of the RPV nozzle combined with pressure differential acting on the inside door face. To prevent the doors from becoming missiles due to these forces, they are secured by bolting to the door frame. Door frames are secured to the reactor shield wall by welded connections. The jet force on the outside face of a door due to a pipe break in the vicinity is also considered in the design. The design and analysis procedures are based on conventional methods found in standard textbooks and handbooks used in the engineering profession.

C. Pipe-Whip Restraints and Barrier Plates

The postulated pipe break criteria and locations are identified in section 3.6. The pipe-whip restraints design includes the dynamic effects as described in Bechtel Topical Report BN-TOP-2, Rev 2. Barrier plates are provided to protect the containment from pipe whip. The ballistic research formula is used in the barrier plate design.

D. Drywell Concrete Floor at el 114 ft 6 in.

The drywell concrete floor at el 114 ft 6 in. is designed for applicable loads and loading combinations using conventional methods found in standard textbooks used in the engineering profession.

E. Refueling Water Seal Assembly

Refueling water seal assembly is designed for the hydrostatic load, bellows load, and seismic loads using conventional methods found in standard textbooks and handbooks used in the engineering profession.

F. Miscellaneous Components

Miscellaneous components such as jet deflectors, weld pads, spray headers and their supports, access ladders, handrails, and monorails are designed for applicable loads and load combinations using conventional methods found in standard textbooks and handbooks used in the engineering profession.

3.8.3.4.5 Computer Programs Used in the Analysis

A. CE 3O9 Structural Engineering Systems Solver (STRESS)(15)

1. Description

STRESS is a programming system for the solution of structural engineering problems. The system is capable of executing the linear, elastic, static analyses of two- and three-dimensional framed structures of the following types:

- Plane truss.
- Plane frame.
- Plane grid.
- Space truss.
- Space frame.

The programming system was originally developed at Massachusetts Institute of Technology in 1964(15) and is now in the public domain.

2. Validation

   The program has been verified by the ICES STRUDL II program. A sample problem of space-frame analysis was run using the CE 309 program and the commercially available versions (Version 1 and Version 2) of the ICES STRUDL II program. The results from these runs were found to be identical. Document traceability is available at Bechtel.

3. Extent of Application

   The program is used to obtain the member forces and displacements by stiffness method.

3.8.3.5 Structural Acceptance Criteria

The limiting values of stress and gross deformations are established by the following criteria:

- To maintain the structural integrity when subjected to the worst load combinations.

- To prevent structural deformations from displacing the equipment to the extent that the equipment suffers a loss of function.

The allowable stresses for different loading combinations described in paragraph 3.8.3.3.2 are shown in table 3.8-8.

A summary of actual and allowable stresses for different loading combinations of the inner and outer rings of the pedestal are shown in table 3.8-9. Table 3.8-10 shows the summary of stress levels at different elevations for reactor shield.

Structural deformations were found not to be a controlling criterion in the design of the internal structures.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

The basic materials used in the construction of the internal structures are found in table 3.8-11.

The internal structures are built of reinforced concrete and structural steel, using proven methods common to heavy industrial construction. All concrete work is done in accordance with ACI 318-63, "Building Code Requirement for Reinforced Concrete," and ACI 301-66, "Specifications for Structural Concrete for Buildings."

Mill test reports are obtained for all steel used with the exceptions of handrails, stairs, and ladders. Detailing, fabrication, and erection of the structural and miscellaneous steel are in accordance with the Manual of Steel Construction, 1969 edition. Welding is done in accordance with AWS D1.0-69, "Code for Welding in Building Construction."

No special techniques were employed in the construction of internal structures.

The effects of various amounts of radiation on the internal structures were considered in the design. Provisions were made to maintain a constant temperature in order to prevent any appreciable loss of structural strength.

3.8.3.7 Testing and Inservice Surveillance Requirements

The internal structures are not directly related to the functioning of the containment concept. Therefore, no testing or surveillance is required.

3.8.4 OTHER SEISMIC CATEGORY I STRUCTURES

Seismic Category I structures other than the containment and the internal structures are listed below:

- Reactor building.
- Diesel generator building.
- Control building.
- Intake structure.
- Lower portion of the RPV support pedestal.
- Main stack.
- Other outdoor Seismic Category I structures.

The relative locations of the intake structure, plant structures, main stack, cooling towers, and other buildings are shown on drawing no. E-10173.

3.8.4.1 Description of Structures

A. Reactor Building

The reactor building encloses the reactor, primary containment, auxiliary cooling systems, refueling and spent-fuel storage pools, and spent-fuel cask pit. The reactor building provides secondary containment for the reactor and primary containment for auxiliary systems. Primary containment for the reactor consists of the drywell and the pressure suppression chamber discussed in subsection 3.8.2. The reactor building is basically a reinforced concrete structure with structural steel framing, consisting of the following major structural components:

- Reinforced concrete foundation mat.
- Reinforced concrete floors supported by structural steel framing.
- Reinforced concrete or concrete block interior walls.
- Stainless-steel-lined reinforced concrete spent-fuel pool and spent-fuel cask pit, reactor well, steam dryer-separator storage pool, and fuel transfer canal.
- HPCI room integral with reactor building.
- Reinforced concrete exterior walls up to refueling floor level.
- Exterior walls above the refueling floor consisting of structural steel columns and prefabricated concrete panels.
- Reinforced concrete slab on metal roof deck system supported by steel framing.

Drawing nos. H-26096 and H-26098 through H-26105 show various plans and sections of the reactor building. The principal features of the new- and spent-fuel handling, storage, and shipment facilities are shown in drawing nos. H-26102 through H-26105.

Fuel-handling facilities are served by a 125-ton overhead crane capable of handling heavy loads, such as the spent-fuel cask concrete plugs, dryer, separator, and drywell and reactor vessel heads. A fuel-handling refueling platform runs on rails mounted on the operating floor.

Mechanical antiderailing devices mounted on the wheel assemblies of the overhead crane bridge and trolley prevent the crane from being dislodged from its rails due to horizontal motion during an earthquake. The vertical acceleration due to an earthquake is not large enough to overcome the crane's downward load due to gravity.

The spent-fuel pool and the spent-fuel cask pit walls and base slab are of 6-ft-thick reinforced concrete. The inside face of the walls and base slab are lined with 1/4-in.-thick stainless-steel liner plate to provide leaktightness. The prestressed concrete wall panels around the fuel-handling area of the operating floor protect the spent-fuel pool from the environment.

The diagonal corner rooms in the basement which house the RHR and CS system are designed for the hydrostatic load resulting from flooding due to torus leak. The diagonal rooms are separated from the torus by 2-ft-thick concrete walls for the entire height of the torus room. Each construction joint is provided with a water stop to prevent leakage of water. The maximum height of flooding of the torus room has been calculated assuming design basis accident (DBA) torus water

volume. All pipe penetrations below this level in the diagonal walls are sealed by stainless steel bellows. Entry into the diagonal rooms is from the floor above the torus room at el 130 ft msl; hence, flood protection barriers are not required to be broken for entry.

The HNP-2 reactor building is separated from the turbine building, radwaste building, control building, and the HNP-1 reactor building by a 3-in. gap. Refueling floors of both units are freely accessible from each other when both units are commissioned. Provisions are made to use the same overhead crane and refueling platform.

B. Diesel Generator Building

This reinforced concrete building, housing the diesel generators essential to safe plant shutdown upon loss-of-offsite power (LOSP), is a one-story box-type structure separated from all other buildings. Reinforced concrete interior walls are provided to physically separate the diesel generators from each other. Drawing no. H-12320 shows the general configuration of the building.

C. Control Building

The control building houses the control room and associated auxiliaries and is shared by both HNP-1 and HNP-2. The building is a reinforced-concrete structure with steel frame structure above el 164 ft, consisting of the following major structural components:

- Reinforced concrete foundation mat.
- Reinforced concrete floors with reinforced concrete beam and girder framing.
- Reinforced concrete or concrete block interior walls and reinforced concrete columns.
- Reinforced concrete (poured or prefabricated) exterior walls.
- Reinforced concrete slab on metal roof deck system supported by steel framing.

Drawing nos. H-12405, H-12406, H12627, H-12629, H-12631, H16249, H-22250, H-40429, and H-40430 show the general layout of the building. The control building is separated from the turbine and reactor buildings by a gap of 3 in.

D. Intake Structure

The intake structure constructed for HNP-1 is a reinforced concrete structure and is shared by HNP-2. Drawing no. H-12192 shows the intake structure and the equipment layout. The equipment provided is coarse trash racks with cleaners,

traveling screens, stop logs, service water, residual heat removal service water (RHRSW), and screen wash pumps. Table 3.8-12 lists the water velocity across the inlet screens at conditions of normal- and low-water river flow and for normal pumping rates and for pumping rates with all pumps running.

E. Lower Portion of RPV Support Pedestal

The lower portion of the RPV support pedestal is located outside the drywell and is a continuation of the concentric shells within the drywell. The lower portion of the pedestal is designed to transmit the loads developed for the pedestal and drywell to the foundation. Details of the lower portion of the pedestal are shown on drawing nos. H-25004 and H-25005.

F. Main Stack

The elevated gas release stack built at the site for HNP-1 is also used for HNP-2. This is a reinforced concrete structure 120-m high above ground level (el 119 ft 6 in.). The foundation is a reinforced concrete mat, octagonal in plan, supported by steel H piles. Drawing no. H-15650 shows the plans and elevations for the main stack.

G. Other Outdoor Seismic Category I Structures

The liquid nitrogen storage tank (chapter 9), the protective wall around the condensate storage tank (subsection 9.2.6), and the diesel generator fuel oil storage tanks (subsection 9.5.4) are designed to Seismic Category I requirements.

3.8.4.2 Applicable Codes, Standards, and Specifications

The following codes, standards, specifications, design criteria, NRC Regulatory Guides, and industry standard practices apply to the original design and construction of all Seismic Category I structures other than the containment and internal structures:

AISC	*Manual of Steel Construction*
ACI 318-63	*Building Code Requirements for Reinforced Concrete*
AWS D.1.0-69	*Welding in Building Construction*
NCIG-01 Rev. 2	*Nuclear Construction Issues Group (NCIG) Specifications for Visual Weld Acceptance Criteria for Structural Welding at Nuclear Plants*
AWS D.2.0-69	*Specifications for Welded Highway and Railway Structures*
ANSI N45.2.5	*Supplementary Quality Assurance Requirements*

	for Installation, Inspections, and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants
ASME	*Boiler and Pressure Vessel Code, Section III, "Nuclear Power Plant Components," 1971 Edition, including 1973 Winter Addenda*
American Association of State Highway Officials (AASHO)	*Standard Specifications for Highway Bridges*
SSBC	*Southern Standard Building Code, 1969 Edition*
CMAA	*Specifications for Electric Overhead Traveling Crane No. 70, 1970 Edition*
ICBO	*Uniform Building Code, 1970 Edition*
NRC Regulatory Guides	*Compliance is discussed in appendix A.*
Regulatory Guide 1.10	*Mechanical (Cadweld) Splices in Reinforcing Concrete Structures, (January 1973)*
Regulatory Guide 1.29	*Seismic Design Classification, (August 1973)*
Regulatory Guide 1.54	*Quality Assurance requirements for Protective Coatings Applied to Water-Cooled Nuclear Power Plants, (June 1973)*
Regulatory Guide 1.55	*Concrete Placement in Category I Structures, (June 1973)*
Regulatory Guide 1.59	*Design Basis Floods for Nuclear Power Plants, (August 1973)*
Regulatory Guide 1.64	*Quality Assurance Program Requirements for the Design of Nuclear Power Plants, (October 1973)*
Regulatory Guide 1.69	*Concrete Radiation Shields for Nuclear Power Plants, (December 1973)*
Regulatory Guide 1.76	*Design Basis Tornado for Nuclear Power Plants, (April 1974)*

General Design Criteria of 10 CFR 50

US Army Corps of Engineers Regulations with Respect to Dredging and Construction

American Society of Civil Engineers Paper 3269 for Wind Design Requirements [20]

American Iron and Steel Institute Specification for the Design of Light Gauge Cold-Formed Structural Members, 1968 Code of Federal Regulations, Title 29, Chapter XVII, Occupational Safety and Health Standards

For new modifications and analysis of modifications installed after the plant was put into operation, later editions of the following codes will be used:

- AISC - Manual of Steel Construction.
- ACI - Building Code Requirements for Reinforced Concrete (ACI 318).
- AWS - Welding in Building Construction (AWS D1.0).
- AWS - Specifications for Welded Highway and Railway Structures (AWS D2.0).
- AWS - Structural Welding Code (AWS D1.1).
- Southern Standard Building Code.
- Uniform Building Code.
- American Iron and Steel Institute Specification for the Design of Light-Gage Cold-Formed Steel Structural Members.

For analysis or modification of original plant designs, a later edition of the codes listed above may be used, however; the applicable sections of the original plant design codes must be reviewed. Differences between the original design codes and a later edition of these codes should be documented. Wherever a code change that is applicable to the design has occurred, a later edition of the code may be used if the change results in a more conservative design than the original design code, or the change results in an acceptable decrease in conservatism based upon a better knowledge or understanding of the condition by the code committee because of tests or experience. If the code change results in a less conservative design and this change is based upon a change in material quality or quality of installation, then the section from the original code edition will be used.

To account for changes in steel member properties and dimensions over the years, this information will be obtained from the AISC Code edition used for the original design.

3.8.4.3 Loads and Loading Combinations

All Seismic Category I structures are designed for all credible conditions of loadings, including normal loads, loads resulting from a pipe rupture where applicable, and loads due to adverse environmental conditions.

3.8.4.3.1 Loads

The following loads are considered in the design:

A. Dead Loads (D)

Structural dead loads consist of the weight of framing, roof, floors, walls, partitions, platforms, hangers, cable trays, pipes with fluid, and equipment dead loads as specified on the drawings supplied by the manufacturers of the equipment installed within the structure.

B. Live Loads (L)

Live loads consist of design floor loads, pool and tank liquid weights, and equipment live loads as specified on the drawings supplied by the manufacturers of the equipment installed within the structure. The live loads used in design are shown on table 3.8-13.

C. Earthquake Loads (E, E^1)

The OBE loads E and DBE loads E^1 derived from the seismic analysis in section 3.7 are used in design.

D. Pressure Loads Due to LOCA (P_a)

A LOCA results in an increased pressure of the surrounding area. This pressure load P_a does not include the jet forces resulting from rupture of pipes.

E. Thermal Loads (T_o, T_a)

T_o = Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

T_a = Thermal loads under thermal conditions generated by the postulated break, including T.

F. Wind and Tornado Loads (W) and (W_t)

The wind loadings and tornado loadings (W) and (W_t) are discussed in section 3.3.

All Seismic Category I structures listed in subsection 3.8.4 are designed to withstand the effects of the wind and tornado loadings and to provide protection against tornado missiles for all Seismic Category I systems and components within the structures.

The structures are analyzed for tornado loadings not coincident with the DBE.

G. Pipe Reaction Loads (R_o, R_a)

R_o = Pipe reactions during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

R_a = Pipe reactions under thermal conditions generated by the postulated break, including R_o.

H. Pipe Rupture Loads (Y_r, Y_j, Y_m)

Y_r = Equivalent static load on the structure generated by the reaction on the broken high-energy pipe during the postulated break, including an appropriate dynamic load factor to account for the dynamic nature of the load.

Y_j = Jet impingement equivalent static load on a structure generated by the postulated break, including an appropriate dynamic load factor to account for the dynamic nature of the load.

Y_m = Missile-impact equivalent static load on a structure generated by or during the postulated break, as from pipe whipping, including an appropriate dynamic load factor to account for the dynamic nature of the load.

I. Impact Loads (I)

Crane impact loads as per AISC are considered in the design of crane girders and their supports.

3.8.4.3.2 Loading Combinations

The following loading combinations are used for all Seismic Category I structures listed in subsection 3.8.4:

$D + L + T_o + R_o + E$

$D + L + I + E$

$D + L + T_o + R_o + W$

$D + L + T_o + R_o + E^1$

$D + L + T_o + R_o + W_t$

$D + L + T_a + 1.5P_a + R_a$[a]

$D + L + 1.25E + 1.25P_a + T_a + R_a + Y_r + Y_j + Y_m$[a]

$D + L + E^1 + P_a + T_a + R_a + Y_r + Y_j + Y_m$

3.8.4.3.2.1 Additional Load Combination Based on Document B.[16] The load combinations and acceptance criteria for Seismic Category I steel and concrete structures are in agreement with Document (B).[16] The load combinations and acceptance criteria used to check for conformance to Document (B) are found in table 3.8-14.

The load factors for the equations for Seismic Category I structures outside containment provided in paragraphs 3.8.4.3 and 3.8.4.5 were revised to agree with those in Document (B).

3.8.4.4 Design and Analysis Procedures

3.8.4.4.1 Biological Shield (Drywell Shield)

For the analysis of the biological shield which constitutes the interior wall of the reactor building, a three-dimensional finite element analysis is made. The CE 779 computer program described in the manual "SAP - A General Structural Analysis Program" by E. L. Wilson is used to perform this analysis. The model is made up of a combination of 5-ft-6-in.-thick shell elements and three-dimensional solid elements. The solid elements are used primarily to model the structure in the vicinity of the fuel pool and at lower elevations where the thickness of the shield is much > 6 ft. The triangular shell elements are used for transitions in the size of the grid and where required by geometry. The distribution of forces around the major penetrations for the equipment and personnel hatches is determined by deleting the shell elements in these regions. Furthermore, the effects of the large penetrations around the pipe chase are also considered by deleting the appropriate elements in this area. The openings in the shield for the fuel pool and dryer-separator pool plugs are also considered. The floor slabs at each elevation are assumed to offer only lateral restraint to the model. Rotational or vertical restraints are not considered at any slab level. The model is assumed to be completely fixed at el 114 ft 6 in. Necessary modifications are made to the lateral restraints in the vicinity of the pools to ensure that the loads from the pool are carried by the shield. The input for the analysis was prepared on the General Electric 635 computer. The output provides moments and forces in both the vertical and hoop directions for the shell elements and all six stresses at the center and on one face of the solid elements.

The axisymmetric elements of CE 316-4, which considers cracked section in concrete, were used to perform the thermal analysis. For the axisymmetric analysis, the model is assumed to

a. These loading combinations are used to evaluate the effects of high-energy pipe breaks.

be completely fixed at el 101 ft 10 in. A 70°F linear temperature gradient is assumed across each section. The results of the thermal analysis are used to determine the additional tensile reinforcement required for the shield due to temperature effects. Loadings used for different combinations in the three-dimensional finite element analysis were applied to this model as uniform ring load and the results were compared to those obtained from CE 779.

The reactor building seismic loads derived in section 3.7, attributable to the biological shield in proportion to the stiffness of the various members, were computed.

The critical values of moments and shears from the above analysis were used for reinforcing design and to check the adequacy of the thickness of the shield.

3.8.4.4.2 Other Seismic Category I Structures

The analysis procedures for other Seismic Category I structures are based on conventional methods. The elastic working stress design method was used in Seismic Category I steel structures.

The computer programs used in the analysis are described in paragraphs 3.8.4.4.3, 3.8.4.4.4, and 3.8.4.4.5.

3.8.4.4.3 Computer Programs Used for Biological Shield Analysis

3.8.4.4.3.1 CE 779 Structural Analysis Program (SAP).[17]

A. Description

The program performs the static and dynamic analyses of linear, elastic, and three-dimensional structures using the finite element method. The finite element library contains truss and beam elements, plane and solid elements, plate and shell elements, axisymmetric (torus) elements, and special boundary (spring) elements.

Element stresses and displacements are solved for either applied loads or temperature distributions. Concentrated loads, pressures, or gravity loads may be applied. Temperature distributions are assigned as an appropriate uniform temperature change in each element. Prestressing may be simulated by using artificial temperature changes on rod elements.

Dynamic response routines are available for solving arbitrary dynamic loads or seismic excitations using either modal superposition or direct integration. The program can also perform response spectrum and time-history analyses.

B. Validation

The solutions to test problems have been demonstrated to be essentially identical to the results obtained using the ASKA program, which was developed by Prof. A. J. Argyris (Institut fur Statik and Dynamik, Stuttgart) and to the Chan and Fermin program. The test problem solutions have also been compared to, and found to be in agreement with, the solutions of the programs from the ASME Library of Benchmark Computer Problems and Solutions. Document traceability is available at Bechtel.

C. Extent of Application

The program is used in the structural analysis of the containment shell at the region of the equipment hatch opening.

3.8.4.4.3.2 CE 316-4 Finite Element Stress Analysis.

A. Description

The program performs the static analyses of plane or axisymmetric structures using the finite element method in which a structure is idealized as an assemblage of finite elements. The finite elements are of either triangular or quadrilateral shape, connected at their corner (nodal) points. The applied loads may be concentrated, uniformly distributed, or inertial, or may be temperature distributions. At boundaries, displacements may be forced.

The program develops the force-displacement relationship (element stiffness matrix) for each individual element from its geometry and material properties. The element relationships are then assembled into an overall structure force-displacement relationship (structure stiffness matrix). Equilibrium equations are developed for each degree of freedom at each nodal point in terms of the structure force-displacement relationship, the unknown nodal point displacements by a modified Gaussian elimination scheme. Once the nodal point displacements are known, element stresses are calculated.

B. Assumptions

The stress and the strain are assumed to be constant within each element.

C. Validation

The program was verified by manual calculations. Document traceability is available at Bechtel.

D. Extent of Application

The program is used to compute stresses in the containment structure due to gravity, pressure, and thermal loads.

3.8.4.4.4 Computer Program Used for Other Seismic Category I Structures

3.8.4.4.4.1 CE-309 Structural Engineering Systems Solver.[14]

A. Description

STRESS is a programming system for the solution of structural engineering problems. The system is capable of executing the linear, elastic, static analyses of two- and three-dimensional framed structures of the following types:

- Plane truss.
- Plane frame.
- Plane grid.
- Space truss.
- Space frame.

The programming system was originally developed at the Massachusetts Institute of Technology in 1964 and is now in the public domain.

B. Validation

The program was verified by the ICES STRUDL II program. A sample problem of space frame analysis was run using the CE 309 program and the commercially available versions (Version 1 and Version 2) of the ICES STRUDL II program. The results from these runs were found to be identical. Document traceability is available at Bechtel.

C. Extent of Application

The program is used to obtain the flexibility matrices of the Seismic Category I structures. The flexibility matrices are used in the dynamic analyses of the structures.

3.8.4.4.5 Computer Programs Used for Seismic Analysis

3.8.4.4.5.1 SAP 1.9 Structural Analysis Program.[17]

A. Description

This program performs the static and dynamic analysis of linear elastic three-dimensional structures using the finite element method. Modeling can be done by a combination of the following:

- Three-dimensional truss and beam elements.
- Triangular membrane.
- Plate and shell elements.
- Three-dimensional isoperimetric hexahedron (brick) elements.
- Quadrilateral orthotropic shell elements.
- Sixteen-node thick shell elements.
- Special boundary elements.
- Three-dimensional curved beam elements.
- Triangular quadrilateral axisymmetric solid quadrilateral plane stress and plane strain elements.

The element stresses and displacements are solved due either to applied loads or temperature distributions. Concentrated loads, pressures, or gravity loads can also be applied. Available dynamic response routines are solved for arbitrary dynamic loads or seismic excitations using either modal superposition or direct integration. The program also does response spectrum analysis.

B. Validation

The solutions to test problems were demonstrated to be essentially identical to the results obtained using the ASKA program, which was developed by Prof. A. J. Argyris (Institut for Statik und Dynamik, Stuttgart) and to the Chan and Fermin program. The test problem solutions have also been compared to, and found to be in agreement with, the solutions of the programs from the ASME Library of Benchmark Computer Problems and Solutions. Document traceability is available at Bechtel.

C. Extent of Application

The program is used in the structural analysis of the containment shell at the region of the equipment hatch opening.

3.8.4.4.5.2 CE 917 Modal Dynamic Analysis.

A. Description

The program computes the reduced-stiffness matrix from the basic geometry input for plane-frame or truss models, or accepts the reduced-stiffness matrix for any structure as input. It calculates mode shapes, frequencies, participation factors, and modal damping values for a lumped-mass model. The special features of the program are:

1. It can accept either diagonal or full-mass matrices.

2. It generates output tape for input to CE 918, CE 920, and CE 931.

3. It can be used for horizontal or vertical earthquake with minimal input changes.

B. Validation

The solutions to the program were demonstrated to be substantially identical to the results obtained by manual calculations. Document traceability is available at Bechtel.

C. Extent of Application

The program is used to obtain the mode shapes and natural frequencies of Seismic Category I structures.

3.8.4.4.5.3 CE 918 Response Spectrum Analysis.

A. Description

This program is supplemental to the modal dynamic analysis program (CE 917). It computes the modal response of general plane-frame or truss models. Response spectrum technique is used, and output is expressed in terms of displacements, accelerations, support reactions, member forces and moments, and spring forces.

B. Validation

The solutions to the program were demonstrated to be substantially identical to the results obtained by manual calculations. Document traceability is available at Bechtel.

C. Extent of Application

The program is used to compute and plot the response spectra for the seismic analyses of Seismic Category I structures.

3.8.4.4.5.4 **CE 920 Time-History Analysis of Structures.**

A. Description

The program performs the earthquake response time-history analysis of lumped-mass models using mode superposition. Program input consists of frequencies, mode shapes, modal damping, and the base acceleration time history.

B. Validation

The solutions to the program were demonstrated to be substantially identical to the results obtained by manual calculations. Document traceability is available at Bechtel.

C. Extent of Application

The program is used to generate the time histories at Seismic Category I equipment locations in the structures.

3.8.4.4.5.5 **CE 921 Response Spectrum Calculations.**

A. Description

The program calculates response acceleration, velocity, and displacement spectra for a specified acceleration time history. It can produce printed plots of the calculated response spectra.

B. Validation

The solutions to the program were demonstrated to be substantially identical to the results obtained by manual calculations. Document traceability is available at Bechtel.

C. Extent of Application

The program is used to generate acceleration, velocity, and displacement spectra at Seismic Category I equipment locations and to print plots of these response spectra.

3.8.4.4.5.6 CE 931 Composite Damping for Soil-Structure Systems.

A. Description

This program calculates the composite modal damping, modal participation factors, and mode shapes for lumped soil-structure systems, in which the structures are represented by their fixed-base normal modes.

B. Validation

The solutions to the program were demonstrated to be substantially identical to the results obtained by manual calculations. Document traceability is available at Bechtel.

C. Extent of Application

The program is used to calculate the composite modal damping, modal participation factors, and mode shapes for lumped soil-structure systems.

3.8.4.5 Structural Acceptance Criteria

The limiting values of stress and gross deformations are established by the following criteria:

- To maintain the structural integrity when subjected to the worst loading combinations.
- To prevent structural deformations from displacing the equipment to the extent that it suffers a loss of function.

The allowable stresses for different loading combinations described in paragraph 3.8.4.3.2 are found in table 3.8-15.

Structural deformations were found not to be a controlling criterion in the design of Seismic Category I structures other than the containment and the internal structures.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

The Seismic Category I structures listed in subsection 3.8.4 are built of reinforced concrete and structural steel, using proven methods common to heavy industrial construction. No special construction techniques were employed in the construction of these structures.

The materials used in construction conform to all the referenced governing codes and standards that were in force on the date the contract for the material was signed (April 1, 1969) unless otherwise noted.

The basic materials used in the construction of the Seismic Category I structures are found in table 3.8-16.

Materials and their quality control requirements are described in the following paragraphs. After the construction phase of the unit was completed, several of these requirements had to be modified to allow for the use of smaller quantities of material, while maintaining the quality. The differences in the present quality control requirements and those used during construction are noted.

3.8.4.6.1 Reinforced Concrete

A. Concrete

All concrete work is done in accordance with ACI 318-63, "Building Code Requirements for Reinforced Concrete," and ACI 3O1-66, "Specifications for Structural Concrete for Buildings," except as otherwise stated herein or in the appropriate job specifications or design drawings.

The concrete is a dense, durable mixture of sound coarse aggregates, fine aggregates, cement, and water. In some areas, fly ash is substituted for portions of cement used in the concrete. Admixtures are added to improve the quality and workability of the plastic concrete during placement and to retard the set of concrete. The sizes of aggregates, water-reducing additives, and slumps are selected to maintain low limits on shrinkage and creep.

Concrete radiation shields were constructed in accordance with the requirements of Regulatory Guide 1.69 (December 1973).

B. Aggregates

Aggregates comply with ASTM C 33, "Specifications for Concrete Aggregates." Acceptability of the aggregates is based on the initial tests listed in table 3.8-17.

Certain user tests, as indicated in table 3.8-17, were performed during construction on the aggregates used in every 500 tons of concrete produced. Presently, the user tests on the aggregates are performed within 6 months prior to a job.

In addition, a daily inspection/control program is carried out during construction to ascertain the consistency in the potentially variable characteristics such as gradation and organic content.

C. Cement

Cement is either Type I, general use cement with no special properties, or Type II, low-alkali cement, in accordance with ASTM C 150-74, "Specification for Portland Cement," and is tested to comply with the requirements of ASTM C 114, "Chemical Analysis of Hydraulic Cement." Presently, Type II cement is only required when potentially reactive aggregates are used; however, during construction and Type II cement was used. The inspection and testing of cement, in addition to the initial tests performed by the cement manufacturer, are indicated in table 3.8-18.

During construction user tests were performed on the cement used in every 2800 tons of concrete produced. Presently, a certified mill test report is supplied stating compliance with ASTM C 150 for the cement used.

D. Fly Ash

Fly ash conforms to ASTM C 618-68T Class F, "Fly Ash and Raw or Calcined Natural Pozzolans for Use in Portland Cement Concrete," and is tested to comply with the requirements of ASTM C 311-68, "Sampling and Testing Fly Ash for Use as an Admixture in Portland Cement Concrete."

The producer is required initially to test and then submit data on each lot of fly ash furnished. User tests, as indicated in table 3.8-19, are performed for each 200-300 tons of concrete produced. In addition, periodic tests in accordance with ASTM C 109-64 are performed during construction to check the environmental effects of storage on fly ash.

Fly ash was not used in concrete used for walls, floors, and ceilings of background-sensitive areas, such as, instrument calibration stations, counting rooms, radiochemical laboratory, etc.

E. Water

During construction, water used in mixing concrete was free from injurious amounts of acid, alkali, organic matters, and other deleterious substances as determined by AASHO-T-26.

Presently, mixing water used for concrete is fresh, clean, and drinkable, except that undrinkable water may be used if it produces mortar cubes having 7- and 28-day strengths ≥ 90% of the strength of similar specimens made with water from a municipal supply, and will not cause a change in the setting time of Portland Cement of > 25%. The strength comparison shall be made on mortars (identical except for the mixing water) prepared and tested in accordance with "Method of Test for Compressive Strength Hydraulic Cement Mortars," ASTM C 109.

F. Admixtures

The selected water-reducing agent Pozzolith 80, manufactured by the Master Builders Company, possesses a shrinkage-reduction effect similar to the types prescribed by ASTM C 494, "Specifications for Chemical Admixtures for Concrete."

An air-entraining agent, MBVR, which conforms to ASTM C 260, manufactured by the Master Builders Company, is added to the concrete mix to increase workability.

Admixtures containing chlorides are not used.

G. Concrete Mix Design

Concrete mixes are designed in accordance with ACI 613-54, "Recommended Practice for Selecting Proportions for Concrete," using materials qualified and accepted for this work. Only concrete mixes meeting the design requirements specified for the structures are used.

Presently, concrete mixes are proportioned according to ACI 211.1, "Recommended Practice for Selecting Proportions for Normal and Heavyweight Concrete."

Trial mixes are tested in accordance with the applicable ASTM specifications as indicated below:

ASTM	Test
C 39	Compressive strength of molded-concrete cylinders
C 144	Slump of Portland Cement concrete
C 192	Making and curing concrete test specimens in the laboratory
C 231	Air content of freshly mixed concrete by the pressure method
C 173	Air content by the volumetric method

H. Concrete Testing

During construction, concrete is sampled and tested to ascertain conformance to the specifications. Concrete samples are taken from the mix in accordance with ASTM C 172, "Method of Sampling Fresh Concrete."

During construction, six cylinders, three sets of two cylinders each, were prepared from each sampling and cured in accordance with ASTM C 31, "Making and Curing Concrete Compressive and Flexural Strength Test Specimens in the Field." Presently, only two sets of cylinders are prepared.

The tests consist of the following:

- Determination of air content in accordance with ASTM C 231 or C 173.

- Slump test in accordance with ASTM C 143.
- Compressive strength test in accordance with ASTM C 39.
- Determination of temperature.

The frequency and extent of these tests are as follows:

- One complete test for each 50 yd^3 or less mixed at the batch plant.
- One complete test for each 50 yd^3 discharged from the trucks when ready mixed trucks were used.

In addition, all concrete discharged from the truck is visually examined by an experienced inspector during the course of discharge from the truck, and samples are obtained and tested whenever the concrete appears to have excessive slump.

The locations at which the sampled concrete is placed are marked.

I. Concrete Placement

All concrete for the base slab, drywell shield wall, and all other walls exceeding 2 1/2 ft in thickness has a placing temperature of not < 45°F nor more than 80°F. The concrete had a temperature of at least 55°F when placed for sections 0 to 12 in. and 50°F for sections 12 to 30 in.

If it is necessary to keep the temperature of the concrete from exceeding the above maximums, approved measures for reducing the temperature of the concrete are employed, such as:

- Cooling the mixing water.
- Cooling the aggregates by spraying with water.
- Shading the materials and facilities from direct rays of the sun.
- Insulating water-supply lines.
- Introducing flaked ice into the mix.
- Painting mixers, bins, and other appropriate storage and transporting facilities white.
- Working only at night.

In general, all procedures for hot weather concreting are in accordance with ACI 605-59.

During cold weather, concrete is not placed when the mean daily atmospheric temperature is below 40°F, or if it might be subject to freezing temperatures before final set has occurred. Whenever the outdoor temperature is below 40°F, the following procedures are implemented:

1. Prior to placing concrete, sufficient canvas and framework, or other type of housing are provided to maintain all concrete surfaces at least at their respective placement temperatures for not < 3 days. Concrete is protected from freezing for at least 7 days and kept wet during this period.

2. Salt or other chemicals for the prevention of freezing are not used and, when necessary, the concrete material was heated before mixing to maintain the required placement temperatures. Mixing water was not heated to more than 150°F and aggregates to more than 180°F.

3. No frozen materials were used in the concrete irrespective of whether or not the placement temperature criteria can be met.

4. Before concrete is placed, all ice, snow, and frost was completely removed from the surfaces which were in contact with the new concrete, and the temperature of these surfaces was raised within 10°F of the temperature of the concrete to be placed.

In general, all procedures for cold weather concreting are in accordance with ACI 306-66.

J. Bonding of Concrete Between Lifts

Horizontal construction joints are prepared for receiving the next lift by either wet sandblasting, by cutting with an air-water jet, or by bush hammering.

When wet sandblasting is employed, it is continued until all laitance, coatings, stains, and other foreign materials are removed. The surface of the concrete is washed thoroughly to remove all loose materials.

When air-water jet cutting is used, it is performed after initial set has taken place but before the concrete has taken its final set. The surface is cut with a high-pressure air-water jet to remove all laitance and to expose clean, sound aggregates, but not to undercut the edges of the larger particles of the aggregates. After cutting, the surface is washed and rinsed as long as there is any trace of cloudiness of the wash water. When it is necessary to remove accumulated laitance, coatings, stains, and other foreign materials, wet sandblasting is used before placing the next lift, to supplement air-water jet cutting.

The horizontal surface is wet immediately before the concrete is placed.

Surface-set retardant compounds are not used.

3.8.4.6.2 Reinforcing Steel

All reinforcing steel conforms to ASTM A 615-68, "Deformed Billet-Steel Bars for Concrete Reinforcement," Grade 60.

Mill test reports are obtained from the reinforcing steel supplier for each heat of steel to ensure that the physical and chemical properties of the steel are in compliance with the ASTM specifications. In addition, during construction user tests consisting of tension and bend tests, in accordance with ASTM A 615-68, were performed to supplement the standard mill tests. One tension test and one bend test were required for each 50 tons of each bar size from each heat of steel, with the exception that bend tests are not performed on No. 14 and No. 18 bars.

Bars No. 11 and smaller are generally lap spliced in accordance with ACI 318-63. Bars No. 14 and No. 18 are Cadweld spliced exclusively.

Splicing reinforcing bars by welding is not done.

Procedures for splicing reinforcing bars using the Cadweld process are defined in supplement 3.8C.

3.8.4.6.3 Structural and Miscellaneous Steel

All structural and miscellaneous steel conforms to the following ASTM specifications:

- Rolled shapes, bars, and plates — A 36-70a
- High-strength bolts — A 325-7l or A-490-71
- Stainless steel — A 240, Type 304

Mill test reports are obtained for all materials used with the exceptions of handrails, toe plates, kickplates, stairs, and ladders.

Detailing, fabrication, and erection of the structural and miscellaneous steel are in accordance with the Manual of Steel Construction, 1969 Edition.

Welding is done in accordance with AWS D 1.0-69, "Code for Welding in Building Construction."

NCIG-01 Rev. 2, "Visual Weld Acceptance Criteria For Structural Welding At Nuclear Plants," may be used in addition to AWS D1.0-69.

3.8.4.7 Testing and Inservice Surveillance Requirements

No structural preoperational testing of the Seismic Category I structures is planned. During the life of the plant, periodic inspections of the structures are made to employ visual inspection for apparent structural deterioration such as large cracks and excessive deflection of structural

members. All seam and plug welds in the spent-fuel pool liner plate were vacuum-box tested upon completion of the welding. Where vacuum-box testing was not possible, liquid penetrant testing was performed.

The spent-fuel pool has a system that provides for leakage to be detected at any time in the life of the plant. This system consists of troughs under the liner plate which lead to a collection system where leakage can be observed.

3.8.5 FOUNDATIONS AND CONCRETE SUPPORTS

The foundations for all Seismic Category I structures, other than Seismic Category I outdoor tank foundations, are supported by undisturbed Altamaha or Duplin formation with a static-bearing capacity in excess of 15,000 lb/ft^2. Seismic Category I outdoor tank foundations rest on fill compacted to 95% of the relative maximum dry density as determined by the Modified Proctor test. Each of the Seismic Category I structures is constructed on an individual mat foundation. A 3-in. gap is provided between the individual buildings to eliminate the possibility of interaction and impact between buildings during an earthquake.

3.8.5.1 Description of Foundations and Supports

A. Primary Containment

The drywell and suppression chamber are supported by the reactor building foundation mat. Drawing no. H-25000 shows the outline of the primary containment resting on the foundation slab. A description of the reactor building foundation mat is given in paragraph 3.8.5.1.

B. Reactor Building

The reactor building foundation is a 149-ft^2-reinforced concrete mat, 27-ft 2-in. thick at the middle drywell and reactor vessel support area and 12-ft 4-in. thick at other sections, bearing directly on the Duplin formation.

Drawing no. E-10173 shows the relative positions of the two reactor building foundations and other Seismic Category I structures' foundations. Suppression chamber seismic ties and support columns, RCIC turbine and pump, reactor vessel support pedestal, and the end walls are anchored to the base mat. Figure 3.8-30 shows the reactor building foundation mat general arrangement plan at el 87 ft and 101 ft 10 in.

Figure 3.8-31 shows typical details of end-wall anchorage to the base slab, reinforcing details for the transition zone between el 87 ft and 93 ft 2 in., general reinforcing for the base mat, and details of reinforcing directly under the reactor vessel pedestal.

C. Diesel Generator Building

The diesel generator building foundation, which is common to both HNP-1 and HNP-2, shown on drawing no. H-12320, is a reinforced concrete rectangular mat 196 ft times 103 ft 6 in., with the bottom of the mat at el 125 ft. The static foundation pressure below the structure is < 3 ksf. The relative position of the mat, with respect to the other structures, is shown on drawing no. E-10173.

D. Control Building

The control building foundation mat, which is common to both HNP-1 and HNP-2, shown on drawing nos. H-12405, H-12406, H-12627, H-12629, H-12631, H-16249, H-22250, H-40429, and H-40430 is a reinforced concrete rectangular mat 160 ft times 103 ft, with the bottom of the mat at el 105 ft. The foundation mat is separated from the HNP-1 and HNP-2 turbine building mats which are also found at the same elevation by a gap of 3 in. The average static foundation pressure below the structure is 6 ksf. The relative location of the mat with respect to the other structures is shown on drawing no. E-10173.

E. Intake Structure

The intake structure, which serves both HNP-1 and HNP-2, shown on drawing no. H-26099, is built on a reinforced concrete, rectangular mat 103 ft times 53 ft, with the bottom of the mat at el 52 ft. The average static foundation pressure is 5 ksf.

F. Lower RPV Pedestal

The lower pedestal is anchored to the reactor building foundation mat at el 101 ft 10 in. by ninety-two 3-in.-diameter anchor bolts as shown in figure 3.8-32.

G. Main Stack

The main stack foundation is an 11-ft-thick octagonal reinforced concrete slab bearing on H-bearing piles which transfer the loads to the Duplin formation.

- Plan dimensions - octagon with 36-ft inscribed radius.
- Yard, el 119 ft 6 in.
- Top of cap, el 108 ft 6 in.
- Bottom of cap, el 97 ft 6 in.
- Pile cutoff, el 98 ft 3 in.
- 164-14BP73 100-ton piles at 4- to 6-ft spacing in 5 rings with radii of 6 ft, 16 ft, 20 ft, 30 ft, and 34 ft, piles driven to el 20 ft.

- Loads on pile foundation of 114,000 kip-ft moment, 21,500 kips vertical load at pile cap.

- A shear of 800 kips is supported by the piles and pile cap.

Figure 3.8-33 shows the details of pile tip and vertical wall anchorage to the base slab. Drawing no. E-10173 shows the relative position of the main stack foundation to the other Seismic Category I structures.

H. Seismic Category I Outdoor Tank Foundations

There are two Seismic Category I outdoor tank foundations:

- Condensate storage tank foundation.

- Liquid nitrogen storage tank foundation.

The foundations of these tanks are 3-ft-thick reinforced concrete slabs bearing on fill compacted to 95% of the relative maximum dry density as determined by the Modified Proctor test.

The tank foundations are physically separated from each other and from other buildings as shown on drawing no. E-10173.

3.8.5.2 Applicable Codes, Standards, and Specifications

The following codes, standards, and specifications apply to the original design and construction of the foundations and concrete supports for all Seismic Category I structures.

ACI 318-63	*Building Code Requirements for Reinforced Concrete*
ACI 307	*Specifications for the Design and Construction of Reinforced Concrete Chimneys*
AISC	*Manual of Steel Construction*
AWS D.12.0	*Seventh Edition AWS D.12.0 Recommended Practice for Welding Reinforcing Steel, Metal Inserts and Connections in Reinforced Concrete Construction*
AWS D.1.0-69	*Welding in Building Construction*
NCIG-01 Rev. 2	*Nuclear Construction Issues Group (NCIG) Specifications for Visual Weld Acceptance Criteria For Structural Welding At Nuclear Plants*

SBCC	*Southern Standard Building Code, 1969 Edition*
ICBO	*Uniform Building Code, 1970 Edition*
CFR	*Title 29 Code of Federal Regulations, Chapter XVII, Occupational Safety and Health Standards*
CFR	*Title 10 Code of Federal Regulations Part 50, Licensing of Production and Utilization Facilities*
NRC Regulatory Guides	*Compliance is discussed in Appendix A.*
Regulatory Guide 1.10	*Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures (January 1973)*
Regulatory Guide 1.15	*Testing of Reinforcing Bars for Category I Concrete Structures (December 1972)*
Regulatory Guide 1.55	*Concrete Placement in Category I Structures (June 1973)*
Regulatory Guide 1.59	*Design Basis Floods for Nuclear Power Plants (August 1973)*
Regulatory Guide 1.64	*Quality Assurance Program Requirements for the Design of Nuclear Power (October 1973)*

Material specifications which were used to produce the concrete or the Seismic Category I foundations and concrete supports are given in paragraph 3.8.4.6.

For new modifications and analysis of modifications installed after the plant was put into operation, later editions of the following codes will be used:

- AISC - Manual of Steel Construction.
- ACI - Building Code Requirements for Reinforced Concrete (ACI 318).
- ACI - Specifications for the Design and Construction of Reinforced Concrete Chimneys (ACI 307).
- AWS - Welding in Building Construction (D1.0).
- AWS - Recommended Practice for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction (AWS D12.0).
- AWS - Structural Welding Code (AWS D1.1).

- Southern Standard Building Code.
- Uniform Building Code.

For analysis or modification of original plant designs, a later edition of the codes listed above may be used; however, the applicable sections of the original plant design codes must be reviewed. Differences between the original design codes and a later edition of these codes should be documented. Wherever a code change that is applicable to the design has occurred, a later edition of the code may be used if the change results in a more conservative design than the original design code, or the change results in an acceptable decrease in conservatism based upon a better knowledge or understanding of the condition because of tests or experience by the code committee. If the code change results in a less conservative design and this change is based upon a change in material quality or quality of installation, then the section from the original code edition will be used. To account for changes in steel member properties and dimensions over the years, this information will be obtained from the AISC Code edition used for the original design.

3.8.5.3 Loads and Loading Combinations

Foundation loads and loading combinations for all Seismic Category I structures are discussed in paragraphs 3.8.3.3 and 3.8.4.3.

3.8.5.4 Design and Analysis Procedures

3.8.5.4.1 Reactor Building Foundation

The analysis and design of the reactor building foundation mat was based on conventional one-way slab fixed at the periphery of the middle 27-ft 2-in.-thick section and simply supported at the diagonal and exterior walls subjected to uniform soil pressure. The soil material under the reactor building foundation mat has been assumed to be homogeneous, isotropic, elastic, and of uniform thickness.

3.8.5.4.2 Reactor Vessel Pedestal Foundation

The pedestal is assumed to be a short vertical cantilever beam fixed at the base. To attain fixity, the anchor bolts are prestressed for the normal operating OBE loads.

The structural response is assumed to be linear and elastic for all loading combinations.

It is assumed that friction or bond between the concrete and the lower drywell shell plates does not contribute to resisting the loads.

The embedded portion of the anchor bolt is coated with asphaltum and wrapped with tape and hence no bond is assumed to exist between the coated surface and the concrete.

Since the shear planes in the concrete overlap between the anchor bolts, the available shear area between the rows of bolts is neglected in design.

The inside and outside rings of the pedestal are full-butt welded to a circular base plate projecting sufficiently on either side to accommodate the stiffener attachments as shown in figure 3.8-32.

Sixty-four 3-in.-diameter A193 B7 anchor bolts are provided for the outer ring and 28 for the inner ring. The length of embedments for these anchor bolts are 10 ft 0 in. for the outer ring and 7 ft 0 in. for the inner ring. The lower ends of the anchor bolts are attached to 3-in.-thick embedded plates by a double bolting system.

The loads are transmitted to the foundation by bearing between the base plate and concrete and by uplift in the anchor bolts. To maintain fixity at the base of the pedestal, the anchor bolts are prestressed to normal OBE loads. The values of the torque and strain applied to obtain the prestressing required are indicated in figure 3.8-32.

For all combinations of loading, except construction, the lower pedestal is embedded in concrete and hence the horizontal shear force is transmitted directly by bearing on the concrete in the embedded zone.

For the construction condition, the anchor bolts transmit the horizontal shear force to the concrete as individual bolt loads not exceeding those permitted by the Uniform Building Code, 1970 Edition, table 26-1.

The loads for various loading combinations are distributed to the outer and inner rings of the pedestal according to the geometrical and structural properties of both rings. The base plate, stiffener plates, anchor bolts, embedded bearing plates, and connections are designed for these loads.

Figure 3.8-32 shows the number, size, and location of the anchor bolts for the outer and inner rings. The base plate detail, stiffener arrangement, embedded plate thickness and size, construction details, and requirements, along with the actual loads for different loading combinations, are also shown in figure 3.8-32.

3.8.5.4.3 Other Structures

The seismic techniques for analysis and design of the foundations for all other Seismic Category I structures are the conventional methods which involve simplifying assumptions such as are found in the theory of concrete structures. Stresses resulting from local moments, torques, concentrated reactions, and uniform loadings are computed by these methods. The soil under these buildings has been assumed to be homogeneous, isotropic, elastic, and of uniform thickness.

3.8.5.5 Structural Acceptance Criteria

The foundations of all Seismic Category I structures are designed to meet the same structural acceptance criteria as the structures themselves. These criteria are discussed in paragraphs 3.8.2.5, 3.8.3.5, and 3.8.4.5.

The allowable bearing pressure of 15,000 lb/ft^2 for all Seismic Category I structures recommended by the Law Engineering Testing Company was not exceeded for the most extreme loading combination.

The foundations of the individual structures are assumed to settle uniformly and independently and the estimated settlements for different buildings are discussed in supplement 2A.

All Seismic Category I structures rest on individual rigid mat foundations. In general, the sands which support the plant structures are dense and incompressible. The buildings are not structurally connected to each other and therefore settle independently. The estimated settlements of the individual plant structures are discussed in supplement 2A, section 2A.5.

The diesel generator building, intake structure, and main stack are physically separated by considerable distances and are on independent foundation mats as shown on drawing no. E-10173. The relative displacements between these buildings do not affect the safety objective.

The reactor building and the control building are physically separated by a 3-in. gap extending all the way through the foundation. Most of the total settlement occurs during construction. The maximum predicted post-construction settlement for the reactor building is in the range of 0.5 in., while it has been negligible for the control buildings, and hence their relative maximum displacement does not impair the integrity of these structures.

The horizontal forces were assumed to be resisted by sliding friction, and a minimum factor of safety against sliding for the most severe loading combination was well above 1.50.

The effects of overturning and floatation of all the structures were investigated, and a minimum factor of safety of 1.50 was maintained for the most critical loading combination.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

The foundations and equipment supports are built of reinforced concrete, using proven methods for heavy industrial construction. The description of the materials and the quality control procedures, as well as special construction techniques for foundations, are the same as those discussed in paragraphs 3.8.2.6, 3.8.3.6, and 3.8.4.6.

3.8.5.7 Testing and Inservice Surveillance Requirements

Testing and inservice surveillance are not required and are not planned for foundations of structures or supports. A discussion of the test program which serves as the basis for the soils investigation and foundation evaluation is found in chapter 2.

3.8.6 RESPONSES TO UNITED STATES NUCLEAR REGULATORY COMMISSION (USNRC) INSPECTION AND ENFORCEMENT (IE) BULLETINS (HNP-1 AND HNP-2)

This section provides a summary of the responses to the following two USNRC IE Bulletins:

- USNRC IE Bulletin 80-11, "Masonry Wall Design."
- USNRC IE Bulletin 79-02, "Pipe Support Base Plate Design Using Concrete Expansion Anchor Bolts."

3.8.6.1 Summary of Responses to USNRC IE Bulletin 80-11, "Masonry Wall Design"

3.8.6.1.1 Introduction

For HNP-1 and HNP-2, masonry wall design was reevaluated in accordance with the requirements of IE Bulletin 80-11, "Masonry Wall Design," May 8, 1980.

3.8.6.1.2 Reevaluation Approach

Concrete masonry walls were reevaluated by considering the relative potential for wall failure based on wall configuration, loading magnitudes, and span lengths. Detailed reevaluations were performed for the worst-case walls. A relatively large number of the lesser case walls were also reevaluated in detail to ensure the structural adequacy of each wall and to ensure that a large enough sample was selected to include all walls requiring a detailed reevaluation. The remainder of the lesser case walls in each priority were reevaluated by comparison with the worst-case walls. This ensured that the most critical walls were considered for prompt, detailed reevaluation.

Attachments to concrete masonry walls were identified during the plant walkdowns. The weight of each component attached to a wall was determined and proportioned to its supports on the wall. All pipes and conduits were assumed full for purposes of the analysis. Conservative weights were supplied for all pieces of equipment to ensure that future minor changes in equipment would not increase the load on the walls and to provide an additional safety factor for the analysis.

No major piping systems were found to be attached to any concrete masonry walls, and all systems that were attached were sufficiently rigid to ensure that the attachments would all experience the same acceleration as the wall. Therefore, the load due to each attachment multiplied by the acceleration of the wall was assumed to equal the inertial loads from that attachment.

The attachment inertial loads were combined directly with the wall inertial loads using the absolute sum method. In addition, moments obtained by multiplying the inertial load of each piece of equipment by the distance from the center of gravity of the reinforcing to the center of gravity of the equipment were also applied to the wall. Because most major loads on the walls came from individual pieces of equipment such as panelboards and pull boxes rather than from piping or conduit systems, the method used to account for equipment weights is conservative in the design of the wall.

Seismic analyses were performed using the floor response spectra for the floor location above the wall, and 3% and 5% dampings were used for the OBE and DBE, respectively, for both cracked and uncracked sections.

Consistent with the original design of the plant and with the FSAR, horizontal earthquake loads were applied in only one direction at a time.

For a horizontal earthquake acting perpendicular to a given wall, the wall was checked for all stresses due to the inertial load of the wall itself, inertial loads due to attached equipment, and static moments from attached equipment. These loads were all combined by the direct sum method. In addition, out-of-plane drift effects were included.

For a horizontal earthquake acting parallel to the wall, in-plane drift effects and equipment inertial loads were considered for the overall evaluation of the wall.

For a vertical earthquake, the inertial load moments due to attached equipment were applied to the wall. None of the walls are load bearing; therefore, the only other load considered was the inertial load of the wall itself due to a horizontal earthquake acting perpendicular to the wall.

For each of the conditions listed above, the wall was checked to ensure local load transfer from all attachments to the wall.

Analyses were performed using horizontal wall strips modeled as simply supported beams and/or finite element models, depending on the degree of complexity required to ensure the structural adequacy of the walls. Allowable wall stresses were based on the reevaluation criteria given in supplement 3.8C of this report. Supplement 3.8C also provides justification for the selected design criteria.

3.8.6.1.3 Function, Configuration, Type, and Strength of Materials, and Construction Practices for Masonry Block Walls

3.8.6.1.3.1 **Function of Walls**. Concrete masonry walls located in Class 1 buildings are all internal nonload bearing walls intended for use as partition walls, fire walls, and shield walls. None are intended or required to resist impact or pressurization loads such as tornados, missiles, pipe break, pipe whip, or jet impingement. A secondary function of these walls is to provide at least partial support for relatively light equipment and components such as small diameter piping, conduit, instrument lines, instrumentation, and electrical boxes.

3.8.6.1.3.2 **Wall Configurations**. Concrete masonry walls subject to reevaluation are single-wythe units constructed of normal-weight concrete blocks with nominal widths of 8 or 12 in. Horizontal joints are reinforced with extra-heavy single-wythe reinforcing trusses. Vertical reinforcing and cell fill are accomplished in two ways: some walls have every cell filled with concrete and No. 5 reinforcing bars spaced at 1 ft 4 in., while the rest of the walls have cell concrete and No. 6 reinforcing bars spaced at 2 ft 3 in. maximum. (Presently, the use of nonshrink grout is allowed in place of concrete for cell fill.) Vertical, reinforced concrete columns are strategically located along the walls to reduce the effective span length of the walls. All walls are recessed 1 in. into the supporting floor, with No. 4 reinforcing dowels projected into walls from the supporting floor at a spacing of 1 ft 4 in.

Masonry walls are anchored to structural concrete walls or columns with one or two dovetail stone anchors at each horizontal joint. In cases where dovetail anchor slots were not provided in the concrete, the masonry walls were anchored to the concrete with 3/16-in.-diameter wire ties attached to a structural shape which is in turn anchored to the concrete with 3/8-in.-diameter expansion bolts.

Each wall is set to within a minimum of 1/2 in. below the bottom of the concrete floor slab above, except for walls with suspended ceilings on both sides which extend one block course above the suspended ceiling. The gap between the masonry wall and the concrete slab is filled with insulating material.

The arrangement and location of concrete masonry walls in Class 1 buildings are shown on drawing nos. H-12320, H-12626 through H-12629, H-12631, H-12632, H-15851, H-15852, H-15854, H-16027, H-16029, H-16030, H-16249, H-22250, H-26098 through H-26105, H-40429, and H-40430. Figures 3.8-34 through 3.8-37 provide single-line wall sketches showing relative location and numbering scheme of the concrete masonry walls in the control building. Figures 3.8-38 through 3.8-45 show examples of wall surveillance sketches.

3.8.6.1.3.3 **Type and Strength of Materials**. A discussion is presented below for each of the primary materials used in the construction of concrete masonry walls at HNP.

Concrete Blocks

These are hollow concrete masonry units made from Portland cement and normal weight aggregates for hollow load bearing units conforming to ASTM C 90-70, Grade N, Type I. Exposed surfaces are specified to have a fine- to medium-coarse texture and uniform color throughout. All units are specified to be free of cracks, chips, or other imperfections that could impair the strength or permanence of the constructed walls.

Reinforcement

Horizontal wall reinforcement is extra-heavy (3/16-in. diameter) Durowall single-Wythe trusses manufactured from cold-drawn steel wire, conforming to ASTM A 82. The reinforcement is galvanized and side rods are deformed. Vertical wall reinforcing consists of deformed No. 5 and No. 6 bars, Grade 60, meeting the requirements of ASTM A 615-68. Dowels are No. 4 deformed bars.

Grout

Grout used to fill the masonry cells during construction was nonshrink grout having a minimum compressive strength of 4000 psi at 28 days. The mix proportions are specified as 1-part Portland Cement, 1-part concrete sand, sufficient grams per sack of cement of an aluminum powder required to cause initial and sustained expansion, and ~ 5 gal of water per sack of cement. The maximum aggregate size is No. 4 (1/2 in.), and the nominal slump is 0 in. Premixed, nonshrink grout may be used for concrete masonry unit fill material and must have a 4000-psi compressive strength in accordance with ASTM C 109. One set of grout cubes shall be cured and tested once per lot or at least once per month.

Mortar

Masonry mortar during construction was standard class mortar consisting of 1-part Portland Cement, 1/4-part hydrated lime, and 3-parts mortar sand with only sufficient quantities of water to produce the required workability. (Presently, the mortar type used shall be specified by the designer in accordance with ASTM C 270.)

Mortar Sand

Mortar sand is specified to conform to ASTM C 144, except that all sand is required to pass a No. 8 sieve and not < 97% is to be retained on No. 100 sieve.

Hydrated Lime

Hydrated lime for masonry mortar is specified to conform to ASTM C 207, Type S.

Aluminum Powder

Aluminum powder for nonshrink grout is specified to be commercial grade conforming to ASTM D 962, Type I, Class B.

<u>Dovetail Stone Anchors</u>

These are specified to be 3/16-in. times 10 1/4-in. times 24-in. long, formed from carbon steel and hot-dip galvanized, Hohmann, and Barnard, Inc. No. 304, or equal.

3.8.6.1.3.4 <u>Reinforcement Details</u>. Wall reinforcement details are discussed in paragraph 3.8.6.1.3.2. Also, typical block wall details, including reinforcing, are shown in figures 3.8.6-46 through 3.8.6-49.

3.8.6.1.3.5 <u>Construction Practices</u>. Detailed concrete specifications were prepared to govern all concrete work at HNP, including concrete masonry. Civil concrete inspectors were assigned to inspect the construction of concrete masonry walls to ensure that the walls were constructed in accordance with the drawings and specifications.

As a matter of general practice, during construction, approval was obtained from the architect/engineer for deviations from the requirements of the drawings and/or specifications.

Concrete blocks were filled on a course-by-course basis, and a wooden tamping rod was used ensure that all voids in the blocks were filled. Also, when the top course of block was laid, it was formed to the existing overhead floor and the blocks were poured full of grout. Detailed erection specifications for concrete masonry walls, reinforcing, and grout were prepared and properly implemented with appropriate inspections.

3.8.6.1.4 Discussion of Results and Conclusions

The concrete masonry walls in Class 1 buildings at the HNP were reevaluated in accordance with the requirements of IE Bulletin 80-11, "Masonry Wall Design."

The objective of the reevaluations was to verify that the masonry walls would perform their intended function under all postulated loads without endangering safety-related components or systems either attached to the walls or in proximity to the walls.

The status of the reevaluation for each wall is shown in table 3.8-20. Out of 166 walls surveyed, 5 were shown to be concrete walls and 53 were determined to have no safety-related equipment or systems attached to or in proximity to the walls. In these instances, no further analysis was required. In addition, six walls which had safety-related equipment in proximity were supported on both sides by a structural steel frame designed to protect the equipment during a seismic event; no further analysis was required for these walls. The remaining walls were analyzed for the as-built condition for all loads and load combinations utilizing a conservative approach based on a horizontal beam strip. A number of the worst-case walls were then modeled for STRUDL-DYNAL and had a finite element analysis run of the as-built condition, taking into account all loads and load combinations. A total of 10 walls was identified where calculated stresses exceeded allowable stresses. The results of the analysis showed that the effective horizontal span length of the wall should be reduced. Therefore, a point somewhere within the middle one-third of the span of the wall was chosen, considering

obstructions and installation problems, and one or more structural steel columns were erected to provide additional support for the wall. The column(s) were designed to withstand the applied loads and were used to brace the wall. The wall was then remodeled for STRUDL-DYNAL, including the steel column, and a finite element analysis was performed again for all postulated loads and load combinations. Resulting moments and shears for the masonry wall and support column were checked against allowables. In addition, bolts, plates, and welds for the support column were checked for worst-case loads. Wall number C130-14 A and B had an angle brace attached to the ceiling in addition to the structural steel column on each side of the wall to provide lateral shear support and reduce a local overstress condition to within allowables. Following the analysis using the modified models, each wall was determined to satisfy all stress allowables.

A total of eight walls has been modified by this method to bring calculated stresses to within design allowables under all postulated loading conditions. The remaining two walls where calculated stresses exceeded design allowables have been removed.

Two of the walls in the HNP-1 reactor building, located in the southeast corner at the personnel elevator on el 130 ft 0 in., have safety-related equipment in proximity and were included in the reevaluations.

In accordance with the criteria addressing interstory drift, the floor displacements resulting from the original building seismic analysis were used to determine the interstory wall strains. It was determined that the calculated masonry wall strains are less than the allowable strains presented in the reevaluation criteria. Therefore, no significant wall cracking is induced by interstory drift effects.

Local load transfer from attachments to the walls is accomplished by either bolting through the wall to a plate on the other side of the wall or by anchoring bolts directly into the wall. For single-Wythe walls, there are four postulated failure modes:

- Failure of the masonry mortar resulting in a single block pullout.
- Shear failure of the masonry around the plate.
- Shear cone failure around an individual bolt.
- Local crushing of the masonry under the bolt under the action of shear loads on the bolt.

Every piece of equipment attached to a concrete masonry wall was analyzed for each of these failure modes using conservative loads and spectral accelerations. Allowable loads at local attachments were based on code allowables for shear and bearing. Therefore, local stresses due to vertical and horizontal forces and moments were considered, and determined not to create an overstress condition when compared with code allowables.

Based on a review of the original seismic analysis of the masonry walls in Class 1 buildings at the HNP, the conservative reevaluations performed in accordance with the requirements of USNRC IE Bulletin 80-11, and the review of construction practices for masonry walls, it is

concluded that the concrete masonry walls at the plant perform their intended function during all postulated loading. For those walls showing stresses above design allowables during seismic loadings, fixes were implemented to reduce the stresses to within the design allowables.

3.8.6.1.5 Computer Program - BLOCK WALLS

Supplement 3.8C provides the description of the computer program BLOCK WALLS in sufficient detail to establish the applicability and validation of the program, with solutions checked against other solutions of classical problems.

3.8.6.2 Summary of Responses to IE Bulletin 79-02, "Pipe Support Base Plate Designs Using Concrete Expansion Anchor Bolts"

3.8.6.2.1 Introduction

For HNP-2, the pipe support base plate design was reevaluated in accordance with the requirements of IE Bulletin 79-02 (March 8, 1979), Revision 1 (June 21, 1979), Supplement No. 1 to Revision 1 (August 20, 1979), and Revision 2 (November 8, 1979).

3.8.6.2.2 Background

Concrete expansion anchors employed in nuclear power plant construction to provide a means to quickly and economically attach pipe support systems to the vertical and horizontal surfaces of concrete structures for years were installed with reliance on the skill of the craftspersons employed for such tasks, i.e., pipefitters and millwrights. However, instances have occurred where these anchors failed in service, raising questions regarding the degree of reliance to be placed on the safety factors assigned to the particular design.

Beginning in late February 1979, Georgia Power Company (GPC) notified Bechtel that several pipe supports provided for the reactor feedwater system piping housed within the turbine building had failed. The piping had fallen from its supported location to the turbine building floor, a distance of ~ 18 in. The cause of failure was determined to be the improper installation of the support system concrete expansion anchors. Due to this situation, GPC requested Bechtel to develop a program of expansion anchor testing to ensure public safety and plant reliability.

In March 1979, the USNRC issued IE Bulletin 79-02 which required nuclear plant owners to examine their construction and engineering records to verify pipe support base plate rigidity (an assumption used in expansion anchor loading calculations) and satisfactory expansion anchor selection and installation.

The USNRC agreed that a written review of the as-built conditions was sufficient as long as supporting documentation was available. However, when such documentation was not available, a suitable testing program would be required to prove the existence of the assumed safety factors in the design.

For the HNP, it was necessary to develop a suitable testing program to respond to IE Bulletin 79-02, as well as Revision 1, Supplement 1 to Revision 1, and Revision 2 of the Bulletin.

3.8.6.2.3 Discussion of Concrete Expansion Anchor Testing and Replacement Program

In order to ensure public safety, those systems essential to safe plant shutdown and/or accident mitigation were the subject of the anchor testing and inspection program. As an owner/operator of numerous central generating stations employing concrete expansion anchors, GPC agreed that plant reliability was not a consideration and, therefore, systems nonessential to safety were eliminated from the test program.

Discussions between Bergen-Paterson Pipesupport Corporation (the pipe-support designer and fabricator), Bechtel, and GPC revealed that little, if any, documentary evidence existed in support of the pipe-support base plate rigidity assumption. A review of construction as-built records and an inspection of the physical plant revealed that expansion anchor substitutions had been made, in many cases without supporting documentation. In addition, GPC construction quality control had not employed an anchor installation inspection program that was sufficiently documented to verify correct anchor installation techniques.

In light of the above, a base plate design verification program was initiated, employing Bergen-Paterson in the development of base plate loads, and Bechtel in the determination of base plate rigid/flexible conditions. An anchor testing program was also developed and implemented to verify proper expansion anchor installation.

A description of the above actions follows in the form of responses to specific attributes of IE Bulletin 79-02.

3.8.6.2.3.1 **Base Plate Flexibility and Design Criteria**. As part of the original design, Bechtel provided Seismic Category I piping analyses and forwarded supported system design loads to Bergen-Paterson. Bergen-Paterson supplied design and fabrication of the pipe-support systems and shipped the support assemblies to the plant site for installation by the piping contractor, M. W. Kellogg.

Because flexibility of the base plate was not specifically taken into account in determining the concrete anchor bolt loads during the original design phase, GPC initiated a program to take base plate flexibility into account and reassess the concrete anchor bolt load. Bergen-Paterson was employed to develop design loads (through dimensional forces and moments) at the centroid of each attachment to the pipe-support base plates. Bechtel utilized this data to determine the adequacy of the as-found base plate anchor systems. This determination was accomplished through analyses based on an empirical-analytic technique (developed by

Bechtel) which takes into account design parameters such as flexibility of the base plates and concrete anchors stiffness (based on actual load-displacement curves furnished by the anchor bolt manufacturer). This method was verified with appropriate finite element solutions.

A computer program for the empirical-analytic technique was implemented for determining the bolt loads for routine applications. The program requires plate dimensions, number of bolts, bolt size, bolt spacing, bolt stiffness, the applied forces, and the allowable bolt shear and tension loads as inputs. The allowable loads for given bolt are determined based on the concrete edge distance, bolt spacing, embedment length, shear cone overlapping, manufacturer's ultimate capacity, and a design safety factor. Supplement 3.8D provides criteria for determining expansion anchor bolt loads in pipe support base plates.

The program computes the bolt forces and calculates shear-tension interaction value based on the allowable loads. The following interaction equation is considered adequate:

$$\left[\frac{\text{Design Tension}}{\text{Allowable Tension}}\right]^2 + \left[\frac{\text{Design Shear}}{\text{Allowable Shear}}\right]^2 \leq 1$$

An interaction value > 1 indicates bolt inadequacy (safety factor less than required).

For special cases where the design of the support does not lend itself to this method, standard engineering analytical techniques with conservative assumptions were employed.

If any bolt on a base plate failed in the analysis, one or more of the following actions were taken.

A. The base plate was reanalyzed assuming the failed bolts(s) carry zero load.

B. The base plate was reanalyzed assuming bolt replacement.

C. When feasible, additional expansion anchors were added, and the base plate was reanalyzed.

D. Larger and/or thicker plates were substituted and reanalyzed.

E. The existing base plates were stiffened to redistribute the loads to other concrete anchors and reanalyzed.

F. Additional braces were added to the support to distribute the loading and were reanalyzed.

G. In those instances where repair/corrective actions resulted in relocation of a piping support, Bechtel analyzed the effect of such modification on the piping system.

H. Corrective action based on existing field conditions was proposed.

3.8.6.2.3.2 **Safety Factors for Expansion Anchors**. A minimum safety factor of 4.0 between the bolt design load and the bolt ultimate capacity was verified to exist for wedge-type anchors, and a minimum safety factor of 5.0 was maintained for the self-drilling shell-type anchors. It should be noted that in light of the experienced failure rate for shell-type anchors, support devices were modified as necessary to eliminate reliance on shell-type anchors employed in tensile load configurations.

However, for factored loadings (which include accident/extreme environmental loads), a safety factor of 3.0 could have been used commensurate with the provisions of Section B.7.2 of the "Proposed Addition to Code Requirements for Nuclear Safety Related Concrete Structures," (ACI 349-76), August 1978. Also based on the HNP program of 100% verification of acceptable anchor bolts, it would have been justifiable to reduce the safety factor to 2.0.

For snubbers and anchors, DBE (safe shutdown earthquake) loads were included directly for determining design bolt loads. For rigid hangers and restraints, OBE loads were used in determining bolt design loads in the actual calculation of shear-tension interactions. Since the interaction values typically have an additional margin which can accommodate increased loading, and since seismic loads on rigid supports comprise only a part of the entire design load, the bulletin factors of safety are, in general, satisfied for DBE (safe shutdown earthquake) loadings.

3.8.6.2.3.3 **Cyclic Loads**. In the original design of the piping system, Bechtel considered deadweight, thermal seismic, and dynamic operating loads, where applicable, in the generation of the static equivalent pipe support design loads.

The safety factors used for concrete expansion anchors were not increased for those portions of the support load which are cyclic in nature. The use of the same safety factor for cyclic and static loads is based on the FFTF tests.(18) The test results indicate:

A. The expansion anchors successfully withstood 2 million cycles of long-term fatigue loading at a maximum intensity of 0.20 of the static ultimate capacity. When the maximum load intensity was steadily increased beyond the aforementioned value and cycled for 2000 times at each load step, the observed failure load was about the same as the static ultimate capacity.

B. The dynamic load capacity of the expansion anchors, under simulated seismic loading, was about the same as their corresponding static ultimate capacities.

C. Preload is not a requirement for the anchor bolts to function in a dynamic loading environment.

3.8.6.2.3.4 **Expansion Anchor Testing Program**. Due to insufficient documentation of the existing installations, an expansion anchor testing program was developed and implemented.

The expansion anchor testing program was conducted in accordance with written procedures HNP-2-11004, "Surveillance Procedure for Identifying Anchors Used For Hangers in Safety Systems," and HNP-2-11005, "Inspection and Testing Procedure for Concrete Expansion Anchors."

The following system piping was included in the test program:

A. All large bore (> 2 1/2-in.-nominal diameter) piping systems required to function and/or support the function of systems to mitigate the consequences of the DBA discussed in chapter 15.

B. Computer analyzed piping system ≤ 1/2-in.-nominal diameter, in safety-related systems.

C. If < 2 1/2-in.-diameter pipe was supported using and engineered field procedure, i.e., the "cookbook method," the support systems were not included in this program, unless that portion of pipe was originally analyzed with the main piping system. In that case, < 2 1/2-in.-diameter pipe supports were inspected from the main pipe to the first anchor on the smaller line.

The specific systems or portions of systems for HNP-2 which had 100% expansion anchor testing or replacement are as follows:

- Primary steam drainage (computer analyzed portion).
- SLCS (pump suction and discharge piping up to containment penetration, Seismic Category I portion).
- Process radiation monitoring (PRM) system (containment penetration to first anchor after second isolation valve) HNP-2 only.
- HPCI system (containment isolation portion).
- RCIC system (containment isolation portion).
- H_2 and O_2 analyzer system (containment isolation portion).
- Drywell pneumatic system (containment isolation portion).
- Diesel oil system (oil piping from day tank to diesel, starting air and cylinder jacket cooling water).
- N_2 inerting system (containment isolation portion).

Since 100% testing of wedge-type expansion anchors and replacement of self-drilling type anchors with wedge type was performed on the above listed systems, it is felt that the supports employing expansion anchors subject to higher

concern with regard to system operability have been covered by the program. (NOTE: Small pipe inside the containment relies on welded supports for operability.)

Other supports outside the containment supported by "cookbook" methods have conservatisms inherent to this method of pipe supporting and since no major items which would affect system operability were identified during the testing or replacement of those small pipe supports which were covered by this program, plant safety is not considered to be in jeopardy.

D. All the anchors not required to take tension loading through their support systems were not included in this program.

E. Containment penetrations smaller than 2 1/2-in. piping installed with motor- or air-operated isolation valves (a heavy concentration of weight) and supported using standard cookbook methods; these support systems were included in this program up to the first anchor beyond the second isolation valve.

Piping and instrument diagrams, isometric drawings for the large bore and small bore piping, and, if necessary, physical piping drawings, were yellow-lined to identify the piping and systems which were to be subjected to the anchor test program. The program included the following systems:

2B21	Nuclear boiler system
2C11	Control rod drive system (Seismic Category I portion only)
2C41	SLCS (complete pump suction and discharge piping up to containment penetration Seismic Category I portion)
2D11	PRM system (from containment penetration to first anchor beyond second isolation valves)
2E11	RHR system
2E11	RHRSW system (including intake structure)
2E21	CS system
2E41	HPCI system
2E51	RCIC system
2G11	Radwaste system (from containment penetration to first anchor beyond second isolation valves)

2G31	RWC (containment isolation portion and connection to feedwater F039)
2G41	Fuel pool cooling system (RHR connection only)
2G51	Torus drainage and purification system (Seismic Category I portion only)
2N11	Main steam (MSIVs to first anchor beyond turbine stop valves and branches 2 1/2 in. and larger to first isolation valve)
2N21	Feedwater system (portion bounded by hangers and/or supports 2N21-RFW-H10, H8, H9, H6, H11, H12, H13, and R5)
2P11	Condensate supply system (Seismic Category I portion only)
2P21	Demineralized water supply system (containment isolation portion only)
2P33	H_2 and O_2 analyzer system (containment isolation portion only)
2P41	Service water system (reactor building, diesel building, and intake structure)
2P42	RBCCW system (containment isolation portion only)
2P51	Service air system (containment isolation portion only)
2P52	Instrument air system (containment isolation portion only)
2P64	Chilled water system (containment isolation portion only)
2P70	Drywell pneumatic system (containment isolation portion only)
2R43	Diesel oil system (oil piping from day tank to diesel, starting air and cylinder jacket cooling water)
2T46	Standby gas treatment system
2T48	Drywell-to-torus ΔP (containment isolation portion only) Containment purge and inerting system N_2 inerting system (containment isolation portion only)
	All systems within the primary containment which employ concrete expansion anchors for pipe support attachments

The initial step in this program was to identify the supports which employed anchor bolts and to verify the type of anchors used in these supports. This was accomplished by walking down the systems and noting the attachment to the building as being welded or employing wedge- or shell-type anchors. After identification of supports and anchor type, testing was started.

Wedge anchors were subjected to the following tests:

- Ultrasonic verification of bolt length.
- Determination of anchor bolt diameter using a Go-No-Go gauge.
- Verification of bolt preload using a calibrated torque wrench. Verification of proper installation of bolts was made using torque values based on manufacturer's data.

Shell-type anchors were tested by first removing the bolt from the shell, verifying its diameter, and verifying that a wedge could be seen inside the shell. The shell length was verified by measuring from the shoulder of the shell to the wedge and adding the standard wedge dimension to it. This would approximate the length of the shell for comparison to vendor data. Thread engagement was checked by reinserting the bolt without rotation until the bolt was seated against the starting thread of the shell. The distance from the underside of the bolt head to the surface of the plate was measured and recorded as thread engagement.

Prior to applying a torque wrench for load verification, the anchor shell was checked to be certain that it was not against the plate. If it was determined to not be against the plate, the bolt was reinserted and torqued to the specified value. The bolt was then removed to determine if any movement of the shell occurred.

In cases where the shell appeared to be seated against the plate prior to the reinsertion of the bolt, an attempt was made to shim the plate away from the wall. The bolt was then inserted and torqued.

Shell-type anchors subjected to the torque test were considered acceptable if no visible slippage was detected. If slippage did occur, an attempt was made to reseat and retest the shell.

Data collected for anchors was recorded and forwarded to design engineers for evaluation.

Failure rates experienced were such that the testing program included the total anchor population in lieu of a statistical sample. Ultimately in the interest of economics and safety, a management decision was made to simply remove and replace the self-drilling anchors (2500 anchors) with the more easily installed and tested wedge-style stud anchors. The new wedge-type anchors were installed per IE Bulletin 79-02. The existing wedge-type anchors were evaluated for design requirements per IE Bulletin 79-02, and corrective action was recommended as required.

3.8.6.2.3.5 **Expansion Anchor Bolts in Concrete Block Walls**. A walkdown inspection of HNP-1 and HNP-2 was performed to determine the extent that expansion anchor bolts were used in concrete block walls to attach piping supports within the scope of IE Bulletin 79-02.

No supports were identified for the safety-related systems which were inspected.

3.8.6.2.3.6 __Structural Shapes Attached Directly to Walls__. The scope of the testing and replacement programs for HNP-2 included all supports relying on expansion anchor bolts for support of the piping covered in the program, whether utilizing base plates or structural shapes attached directly to walls. It should be noted that structural shapes were generally not attached directly to the building walls. Only a few cases were identified during the program and these were given the same consideration as the other supports.

3.8.6.2.3.7 __Inaccessible Anchor Bolt Testing__. This paragraph is not applicable to HNP-2.

3.8.6.2.3.8 __Inspection Documentation__. Inspection documentation for the HNP-2 testing and replacement program is available at the site.

3.8.7 SEISMIC EVALUATION OF RADWASTE FACILITIES BUILDINGS (HNP-1 AND HNP-2)

The following radwaste facilities were analyzed and evaluated in accordance with the Branch Technical Position[19] ETSB-11-1, Rev 1, Section V, and the USNRC Regulatory Guide 1.143 and found to be acceptable:

- Radwaste building (HNP-1).
- Radwaste addition building (HNP-1).
- Radwaste building (HNP-2).
- Waste gas treatment building (common to both units).
- Off-gas recombiner building (HNP-1).

3.8.7.1 __Seismic Model__

In order to seismically analyze each of the radwaste system structures, a simplified model fixed at the base was used. The mass points were located at elevated slab locations, and the weight of each mass consists of the weight of the slab plus half the weight of walls below and above the slab. Twenty-five percent of the live load on the slab was also included. The model did not consider structure-soil interaction, structure-structure interaction, or lateral torsional coupling.

3.8.7.2 Modal Analysis

A modal analysis was performed on each of the radwaste system buildings to establish the natural frequencies and associated mode of vibrations using Bechtel computer program CE 917.

3.8.7.3 Response Spectrum Technique

The response spectrum technique was used to determine the response of the structures to the earthquake. Using the design spectra presented in the USNRC Regulatory Guide 1.60 and a scale factor of 0.08, the spectral acceleration associated with each of the natural frequencies was determined. A building damping value of 4% was used based on the USNRC Regulatory Guide 1.61. The structural response, including the deflection, acceleration, shear and moment, for each mode were then evaluated. Because there were no closely spaced modes, the total response of the structure was taken as the square root of the sum of the squares of the individual modal responses. The computation of this portion of the analysis was accomplished by Bechtel computer program CE 918. Only a horizontal ground motion was considered.

3.8.7.4 Time-History Analysis

In order to determine appropriate seismic loads for the design of equipment and piping systems attached to the structure, a time-history analysis was performed.

Input motion was defined at the foundation of each building by using a Bechtel standard synthetic time-history scaled to 0.08g (OBE). A building damping value of 4% was used. Both the input ground motion and the damping value are consistent with the USNRC Regulatory Guides 1.60 and 1.61. The response spectrum curves were computed for 0.5, 1, 2, 3, 4, and 5% damping at each mass point for all the buildings in the radwaste systems. The floor spectrum curves cover a frequency range for 0.1 Hz to 33 Hz. Two sets of curves, one for the east-west direction, and one for the north-south direction, were plotted for each elevation above the base. The computation and plotting of the response spectra was accomplished by use of Bechtel computer programs CE 920 and CE 921.

3.8.7.5 Computer Programs

Description, validation, and extent of application for computer programs CE 917, CE 918, CE 920, and CE 921 are included in paragraph 3.8.4.4.3.

3.8.7.6 Structural Evaluation

Using results from the seismic analysis of the radwaste facilities, structural evaluations were made, considering seismic forces. Structural evaluation included:

- Overturning.

- Combined flexural and axial stress at base of building.

- Shearing stress at base of building combined with torsional shear stresses.

- Local bending of interior and exterior walls.

3.8.7.7 Seismic Evaluation of Charcoal Adsorbers (HNP-1 and HNP-2)

Using applicable floor response spectra of the waste gas treatment building, seismic evaluations were made for the charcoal adsorber vessels. Seismic response spectra curves were developed from 0.08g (OBE) lateral fixed-base time-history analysis, using Bechtel synthetic horizontal time history as the input ground motion.

Seismic evaluation of the charcoal adsorber vessels has shown that the design is adequate to withstand seismic accelerations, using the static analysis methods.

<u>DOCUMENTS INCORPORATED BY REFERENCE INTO THE FSAR</u>

Technical Requirements Manual Table T7.0-1, Primary Containment Penetrations.

REFERENCES

1. "Structural Evaluation of Pressure Suppression System for E. I. Hatch Nuclear Plant-Unit 2, Mark I Containment," Docket No. 50-366, July 1977, Bechtel.

2. "Mark I Containment Short-Term Program Safety Evaluation Report," <u>NUREG-0408</u>, Nuclear Regulatory Commission, December 1977.

3. Zick, L. P., "Circumferential Stresses in Pressure Vessel Shells of Revolution," <u>Journal of Engineering for Industry</u>, May 1963.

4. Wichman, K. R., Hopper, A. G., and Mershon, J. L., "Local Stresses in Spherical and Cylindrical Shells Due to External Loadings," Welding Research Council Bulletin 107, August 1965, revised December 1968.

5. Kalnins, A., "Analysis of Shells of Revolution Subjected to Symmetrical and Nonsymmetrical Loads," presented at the Summer Conference of the Applied Mechanics Division, Boulder, Colorado, June 9-11, 1964, of the American Society of Mechanical Engineers.

6. Grinter, L. E., <u>Theory of Modern Steel Structures</u>, McMillan Company, New York, 1949.

7. NAST-TN 129.

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9. ASME Boiler and Pressure Vessel Code, Sections III and VIII.

10. Blume, Newmark, and Corning, <u>Design of Multistory Reinforced Concrete Buildings for Earthquake Motions</u>, Portland Cement Association, 1961.

11. Hurty and Rubinstein, Dynamics of Structures, Prentice Hall, 1964.

12. Wilson, E. and Clough, R. W., <u>Symposium on the Use of Computers in Civil Engineering</u>, Paper 45, Lisbon, October 1-5, 1962.

13. 10 CFR 50, Appendix J, "Reactor Containment Leakage Testing for Water Cooled Power Reactors," February 14, 1973.

14. BN-TOP-1, "Testing Criteria for Integrated Leakage Rate Testing of Primary Containment Structures for Nuclear Power Plants," Revision 1, November 1, 1972, Bechtel.

15. Fenjes, S. J., Logchar, R. D., and Mauch, S. P., Stress Reference Manual, M.I.T. Press, Cambridge, Mass., 1964.

16. Document (B), Structural Engineering Branch Directorate of Licensing, Structural Design Criteria for Evaluating the Effects of High-Energy Pipe Breaks on Category I Structures Outside the Containment, June 1973.

17. "A Refined Quadrilateral Element for Analysis of Plate Bending," Proceedings of the (second) Conference of Matrix Methods in Structural Mechanics, Wright-Patterson AFB, Ohio, 1968.

18. Drilled-In Expansion Bolts Under Static and Alternating Loads, Report No. BR-5853-C-4, Bechtel, January 1975.

19. "Design Guidelines for Radioactive Waste Management System Installation in Light-Water Cooled Nuclear Power Reactor Plants," Branch Technical Position ETSB No. 11-1, Revision 1.

20. American Society of Civil Engineers Paper 3269, "Wind Forces on Structures," Transactions Vol. 126, Part II, 1961.

21. "Plant Unique Analysis Report for E. I. Hatch Nuclear Plant Unit 2 Mark I Containment Long-Term Program, "Revision 1, Docket No. 50-366, Bechtel, September 1983.

22. "ASME Section III Component Replacements (Generic Letter 89-09)," USNRC, May 8, 1989.

TABLE 3.8-2 (SHEET 1 OF 11)

SUMMARY OF CONTAINMENT STRESSES

Location: Drywell Cylinder

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Primary Principal Membrane Stresses Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	380	548	548	17,500
	DBE	598	548	598	32,670
Operating (exterior pressure)	OBE	-1481	-548	1481	17,500
	DBE	-1698	-548	1698	32,670
Refueling	OBE	-2222	0	2222	17,500
	DBE	-2549	0	2549	34,200
Accident (interior pressure)[(a)]	OBE	7671	15,335	15,335	17,500
	DBE	7878	15,335	15,335	29,930
Accident (exterior pressure)[(a)]	OBE	-1378	-548	1378	17,500
	DBE	-1585	-548	1585	32,670
Flooded	OBE	468	14,479	14,479	24,200

TABLE 3.8-2 (SHEET 2 OF 11)

Location: Section 1 - Upper Knuckle Section el 168.12 ft

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Primary Principal Membrane Stresses Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	115	385	385	17,500
	DBE	180	487	487	32,670
Operating (exterior pressure)	OBE	-447	-899	899	17,500
	DBE	-513	-1000	1000	32,670
Refueling	OBE	-672	-1035	1035	17,500
	DBE	-771	-1188	1188	34,200
Accident (interior pressure)[(a)]	OBE	2319	9429	9429	17,500
	DBE	2382	9525	9525	29,930
Accident (exterior pressure)[(a)]	OBE	-416	-851	851	17,500
	DBE	-479	-947	947	32,670
Flooded	OBE	141	4595	4595	34,200
Flooded - filling	OBE	-482	-778	778	34,200
Construction stage	OBE	-356	-550	550	17,500
	DBE	-443	-685	685	34,200
Construction stage - wind		-185	-285	285	34,200
Final testing (interior pressure)	OBE	2319	9429	9429	17,500
	DBE	2382	9525	9525	32,670
Final testing (exterior pressure)	OBE	-416	-851	851	17,500
	DBE	-479	-947	947	32,670

TABLE 3.8-2 (SHEET 3 OF 11)

Location: Section 2 - Middle Knuckle Section el 164.91 ft

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Primary Principal Membrane Stresses Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	154	798	798	17,500
	DBE	250	1164	1164	32,670
Operating (exterior pressure)	OBE	-529	-2242	2242	18,500
	DBE	-625	-2608	2608	32,670
Refueling	OBE	-711	-2966	2966	17,500
	DBE	-905	-3475	3475	34,200
Accident (interior pressure)[a]	OBE	2903	16,946	16,946	17,500
	DBE	2995	17,298	17,298	29,930
Accident (exterior pressure)[a]	OBE	-494	-2108	2108	17,500
	DBE	-586	-2460	2460	32,670
Flooded	OBE	657	10,671	10,671	34,200
Flooded - filling	OBE	-788	-3034	3034	34,200
Construction stage	OBE	-416	-1602	1602	17,500
	DBE	-518	-1996	1996	34,200
Construction stage - wind		-213	-819	819	34,200
Final testing (interior pressure)	OBE	2903	16,946	16,946	17,500
	DBE	2995	17,298	17,298	32,670
Final testing (exterior pressure)	OBE	-494	-2108	2108	17,500
	DBE	-586	-2460	2460	32,670

TABLE 3.8-2 (SHEET 4 OF 11)

Location: Section 3 - Lower Knuckle Section el 163.50 ft

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Primary Principal Membrane Stresses Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	218	684	684	17,500
	DBE	320	909	909	32,670
Operating (exterior pressure)	OBE	-707	-1763	1763	17,500
	DBE	-809	-1988	1988	32,670
Refueling	OBE	-993	-2193	2193	17,500
	DBE	-1142	-2521	2521	34,200
Accident (interior pressure)[(a)]	OBE	4093	14,690	14,690	17,500
	DBE	4190	14,905	14,905	29,930
Accident (exterior pressure)[(a)]	OBE	-663	-1667	1667	17,500
	DBE	-761	-1882	1882	32,670
Flooded	OBE	807	9095	9095	34,200
Flooded - filling	OBE	502	1126	1126	34,200
Construction stage	OBE	-513	-1133	1133	17,500
	DBE	-641	-1413	1413	34,200
Construction stage - wind		-265	-584	584	34,200
Final testing (interior pressure)	OBE	4093	14,690	14,690	17,500
	DBE	4190	14,905	14,905	32,670
Final testing (exterior pressure)	OBE	-663	-1667	1667	17,500
	DBE	-761	-1882	1882	32,670

TABLE 3.8-2 (SHEET 5 OF 11)

Location: Section 4 - Sphere - Knuckle Transition el 162.67 ft

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	-1027	1720	2747	17,500
	DBE	-1254	1948	3202	42,345
Operating (exterior pressure)	OBE	-1720	1027	2747	17,500
	DBE	-1948	1254	3202	42,345
Refueling	OBE	-2313	2313	4626	17,500
	DBE	-2643	2643	5286	45,000
Accident (interior pressure)(a)	OBE	8429	10,984	10,984	17,200
	DBE	8112	21,202	11,202	35,370
Accident (exterior pressure)(a)	OBE	-1624	931	2555	17,500
	DBE	-1842	1148	2990	42,345
Flooded	OBE	-1820	11,526	13,346	45,000
Flooded - filling	OBE	-2311	2474	4785	45,000
Construction stage	OBE	-1196	1196	2392	17,500
	DBE	-1467	1467	2934	45,000
Construction stage - wind		-657	657	1314	45,000
Final testing (interior pressure)	OBE	8429	10,984	10,984	17,500
	DBE	8112	11,202	11,202	42,345
Final testing (exterior pressure)	OBE	-1624	931	2555	17,500
	DBE	-1842	1148	2990	42,345

TABLE 3.8-2 (SHEET 6 OF 11)

Location: Section 5 - Upper Drywell Beam Seats el 148.29 ft

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Primary Principal Membrane Stresses Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	-951	1644	2595	17,500
	DBE	-1131	1824	2955	42,345
Operating (exterior pressure)	OBE	-1644	951	2595	17,500
	DBE	-1824	1131	2955	42,345
Refueling	OBE	-1759	1759	3518	17,500
	DBE	-1988	1988	3976	45,000
Accident (interior pressure)[(a)]	OBE	8455	10,958	10,958	17,200
	DBE	8281	11,132	11,132	35,370
Accident (exterior pressure)[(a)]	OBE	-1598	905	2503	17,500
	DBE	-1772	1079	2851	42,345
Flooded	OBE	-1484	13,270	14,754	45,000
Flooded - filling	OBE	-1725	3975	5700	45,000
Construction stage	OBE	-749	749	1498	17,500
	DBE	-897	897	1794	45,000
Construction stage - wind		-464	464	928	45,000
Final testing (interior pressure)	OBE	8455	10,958	10,985	17,500
	DBE	8281	11,132	11,132	42,345
Final testing (exterior pressure)	OBE	-1598	905	2503	17,500
	DBE	-1772	1079	2851	42,345

TABLE 3.8-2 (SHEET 7 OF 11)

Location: Section 6 - Drywell Bottom Head Intersection el 128.34 ft

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Primary Principal Membrane Stresses Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	-2061	2755	4816	17,500
	DBE	-2390	3084	5474	42,345
Operating (exterior pressure)	OBE	-2755	2061	4816	17,500
	DBE	-3084	2390	5474	42,345
Refueling	OBE	-2873	2873	5746	17,500
	DBE	-3251	3251	6502	45,000
Accident (interior pressure)[(a)]	OBE	7345	12,068	12,068	17,200
	DBE	7021	12,392	12,392	35,370
Accident (exterior pressure)[(a)]	OBE	-2708	2015	4723	17,500
	DBE	-3032	2339	5371	42,345
Flooded	OBE	-3231	18,137	21,368	45,000
Flooded - filling	OBE	-3474	8796	12,270	45,000
Construction stage	OBE	-1316	1316	2632	17,500
	DBE	-1559	1559	3118	45,000
Construction stage - wind		-852	852	1704	45,000
Final testing (interior pressure)	OBE	7344	12,069	12,069	17,500
	DBE	7020	12,393	12,393	42,345
Final testing (exterior pressure)	OBE	-2709	2016	4725	17,500
	DBE	-3033	2340	5373	42,345

TABLE 3.8-2 (SHEET 8 OF 11)

Location: Section 7 - Lower Drywell Beam Seats el 127.75 ft

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Primary Principal Membrane Stresses Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	-1579	2099	3678	16,200
	DBE	-1831	2351	4182	38,970
Operating (exterior pressure)	OBE	-2099	1579	3678	26,200
	DBE	-2351	1831	4182	38,970
Refueling	OBE	-2191	2191	4382	16,200
	DBE	-2481	2481	4962	41,400
Accident (interior pressure)[a]	OBE	5476	9084	9084	15,900
	DBE	5228	9332	9332	32,600
Accident (exterior pressure)[a]	OBE	-2064	1544	3608	16,200
	DBE	-2312	1792	4104	38,970
Flooded	OBE	-2491	13,671	16,162	41,400
Flooded - filling	OBE	-2675	6731	9406	41,400
Construction stage	OBE	-1041	1041	2082	16,200
	DBE	-1230	1230	2460	41,400
Construction stage - wind		-679	679	1358	41,400
Final testing (interior pressure)	OBE	5475	9085	9085	16,200
	DBE	5227	9333	9333	38,970
Final testing (exterior pressure)	OBE	-2065	1545	3610	16,200
	DBE	-2313	1793	4106	38,970

TABLE 3.8-2 (SHEET 9 OF 11)

Location: Section 8 - Drywell Ventline Intersection el 118.92 ft

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	-3188	3708	6896	16,200
	DBE	-3706	4226	7932	38,970
Operating (exterior pressure)	OBE	-3708	3188	6896	16,200
	DBE	-4226	3706	7932	38,970
Refueling	OBE	-3939	3939	7878	16,200
	DBE	-4509	4509	9018	41,400
Accident (interior pressure)[a]	OBE	3881	10,679	10,679	15,900
	DBE	3369	11,191	11,191	32,600
Accident (exterior pressure)[a]	OBE	-3659	3139	6798	16,200
	DBE	-4171	3651	7822	38,970
Flooded	OBE	-5327	17,547	22,874	41,400
Flooded - filling	OBE	-5585	10,637	16,222	41,400
Construction stage	OBE	-2091	2091	4182	16,200
	DBE	-2454	2454	4908	41,400
Construction stage - wind		-1398	1398	2796	41,400
Final testing (interior pressure)	OBE	3876	10,684	10,684	16,200
	DBE	3363	11,197	11,197	38,970
Final testing (exterior pressure)	OBE	-3664	3144	6808	16,200
	DBE	-4177	3657	7834	38,970

TABLE 3.8-2 (SHEET 10 OF 11)

Location: Section 9 - Bottom Head Embedment el 113.17 ft

Loading Condition		Primary Principal Membrane Stresses Meridional (psi σ_x)	Primary Principal Membrane Stresses Circumferential (psi σ_Φ)	Stress Intensity (psi S_m)	Allowable Stress (psi)
Operating (interior pressure)	OBE	-5923	6443	12,366	16,200
	DBE	-6953	7473	14,426	38,970
Operating (exterior pressure)	OBE	-6443	5923	12,366	16,200
	DBE	-7473	6953	14,426	38,970
Refueling	OBE	-6973	6973	13,946	16,200
	DBE	-8089	8089	16,178	41,400
Accident (interior pressure)[a]	OBE	1177	13,383	13,383	15,900
	DBE	155	14,405	14,405	32,600
Accident (exterior pressure)[a]	OBE	-6363	5843	12,206	16,200
	DBE	-7385	6865	14,250	38,970
Flooded	OBE	-11,362	24,102	35,464	41,400
Flooded - filling	OBE	-11,775	17,475	29,250	41,400
Construction stage	OBE	-4014	4014	8028	16,200
	DBE	-4837	4837	9674	41,400
Construction stage - wind		-2489	2489	4978	41,400
Final testing (interior pressure)	OBE	1157	13,403	13,403	16,200
	DBE	133	14,427	14,427	38,970
Final testing (interior pressure)	OBE	6383	5863	12,246	16,200
	DBE	-7407	6887	14,294	38,970

TABLE 3.8-2 (SHEET 11 OF 11)

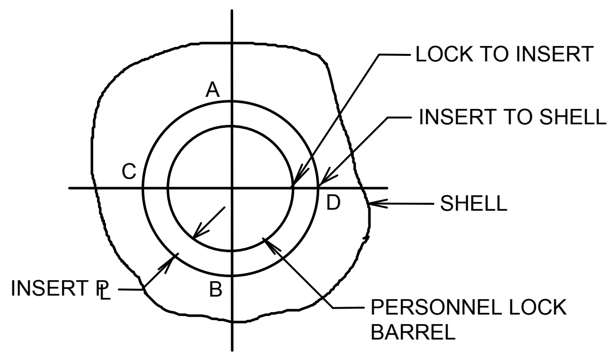
Location: Suppression Chamber Shell

Loading Condition	Shell Stresses – Top Half – Circumferential Stress (psi)	Shell Stresses – Top Half – Longitudinal Stress (psi)	Shell Stresses – Bottom Half – Circumferential Stress (psi)	Shell Stresses – Bottom Half – Longitudinal Stress (psi)	Allowable Stress (psi)
Operating OBE	624	3906	2347	4623	17,500
Operating DBE	624	5174	2526	5929	34,200
Accident OBE[a]	17,474	12,095	17,444	11,944	17,500
Accident DBE[a]	17,474	12,823	17,633	12,736	29,970
Flooded OBE	28,718	19,055	28,453	18,724	34,200

a. Does not include jet impingement loadings.

TABLE 3.8-3 (SHEET 1 OF 6)

SUMMARY OF STRESSES AT PERSONNEL LOCK AREA



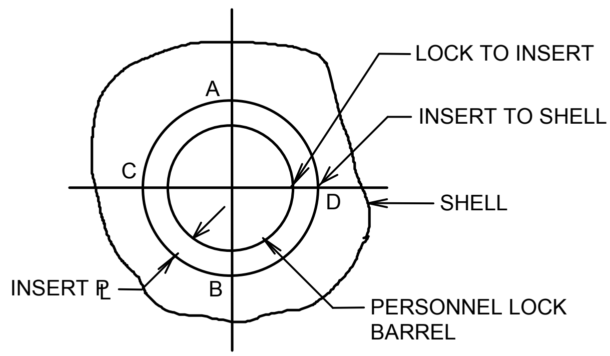
Membrane Stresses at Lock to Insert - OBE Vessel Empty

Type of Stress	A	B	C	D
Radial (psi)	17,559	17,517	17,380	17,696
Tangential (psi)	17,518	17,506	17,469	17,555
Maximum surface (psi)	17,559	17,517	17,469	17,696
Allowable surface (psi)	26,250	26,250	26,250	26,250

Membrane Stresses at Insert to Shell - OBE Vessel Empty

Type of Stress	A	B	C	D
Radial (psi)	17,615	17,545	17,330	18,150
Tangential (psi)	17,540	17,520	17,465	17,595
Maximum surface (psi)	17,615	17,545	17,465	18,150
Allowable surface (psi)	19,250	19,250	19,250	19,250

TABLE 3.8-3 (SHEET 2 OF 6)



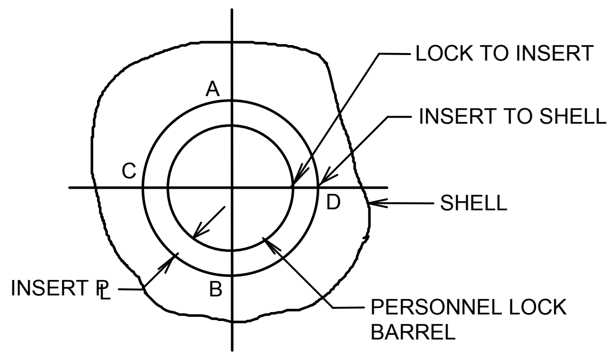
Membrane Stresses at Lock to Insert - DBE Vessel Empty

Type of Stress	A	B	C	D
Radial (psi)	17,601	17,531	17,422	17,710
Tangential (psi)	17,532	17,512	17,483	17,561
Maximum surface (psi)	17,601	17,531	17,483	17,710
Allowable surface (psi)	26,250	26,250	26,250	26,250

Membrane Stresses at Insert to Shell - DBE Vessel Empty

Type of Stress	A	B	C	D
Radial (psi)	17,690	17,580	17,405	17,865
Tangential (psi)	17,565	17,535	17,360	17,610
Maximum surface (psi)	17,690	17,580	17,405	17,865
Allowable surface (psi)	19,250	19,250	19,250	19,250

TABLE 3.8-3 (SHEET 3 OF 6)



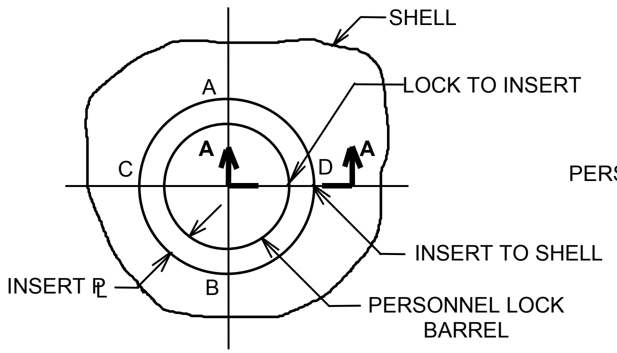
Membrane Stresses at Lock to Insert - OBE Vessel Flooded

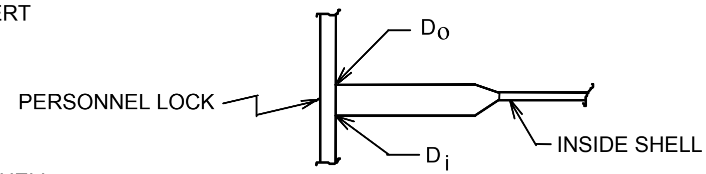
Type of Stress	A	B	C	D
Radial (psi)	17,574	17,524	17,396	17,702
Tangential (psi)	17,519	17,509	17,470	17,558
Maximum Surface (psi)	17,574	17,524	17,470	17,702
Allowable surface (psi)	26,250	26,250	26,250	26,250

Membrane Stresses at Insert to Shell - OBE Vessel Flooded

Type of Stress	A	B	C	D
Radial (psi)	17,640	17,560	17,360	17,840
Tangential (psi)	17,545	17,525	17,470	17,850
Maximum surface (psi)	17,640	17,560	17,470	17,850
Allowable surface (psi)	19,250	19,250	19,250	19,250

TABLE 3.8-3 (SHEET 4 OF 6)





SECTION A-A

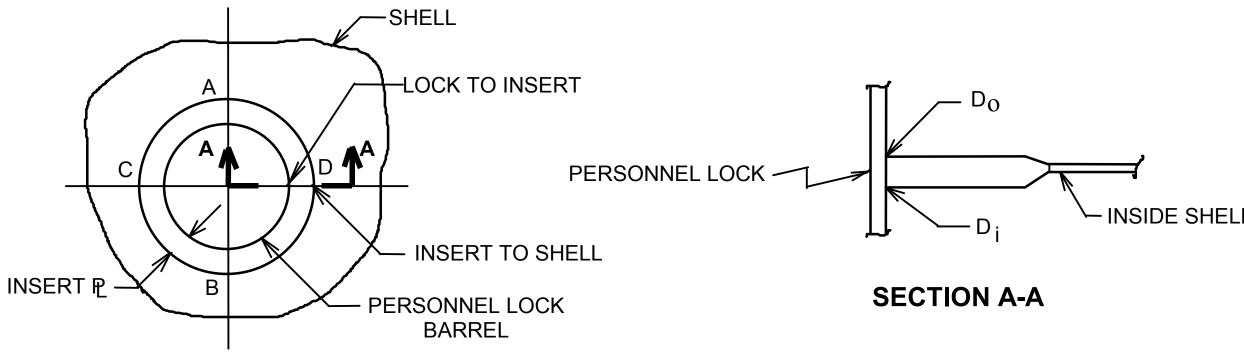
Surface Stresses at Lock to Insert - OBE Vessel Empty

Type of Stress	A_0	A_i	B_0	B_i	C_0	C_i	D_0	D_i
Radial (psi)	26,481	26,383	26,293	26,259	25,695	26,791	27,079	25,831
Tangential (psi)	26,318	26,282	26,264	26,252	26,091	26,411	26,491	26,123
Maximum surface (psi)	26,481	26,363	26,293	26,259	26,091	26,791	27,079	26,123
Allowable surface (psi)	52,500	52,500	52,500	52,500	52,500	52,500	52,500	52,500

Surface Stresses at Insert to Shell - OBE Vessel Empty

Type of Stress	A_0	A_i	B_0	B_i	C_0	C_i	D_0	D_i
Radial (psi)	17,905	17,675	17,655	17,675	16,875	18,135	18,685	17,105
Tangential (psi)	17,630	17,550	17,560	17,520	17,343	17,687	17,847	17,413
Maximum surface (psi)	17,905	17,675	17,655	17,675	17,343	18,135	18,685	17,413
Allowable surface (psi)	52,500	52,500	52,500	52,500	52,500	52,500	52,500	52,500

TABLE 3.8-3 (SHEET 5 OF 6)



SECTION A-A

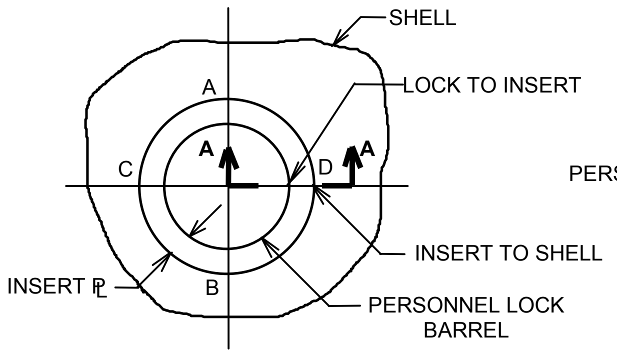
Surface Stresses at Lock to Insert - DBE Vessel Empty

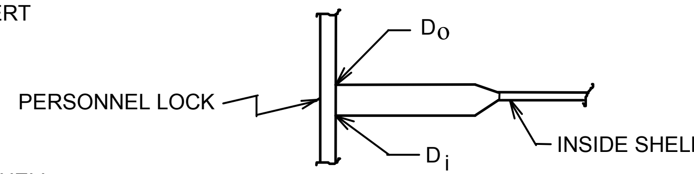
Type of Stress	A_0	A_i	B_0	B_i	C_0	C_i	D_0	D_i
Radial (psi)	26,642	26,440	26,332	26,270	25,856	26,868	27,118	25,842
Tangential (psi)	26,367	26,303	26,277	26,253	26,140	26,432	26,504	26,124
Maximum surface (psi)	26,642	26,440	26,332	26,270	26,140	26,868	27,118	26,124
Allowable surface (psi)	52,500	52,500	52,500	52,500	52,500	52,500	52,500	52,500

Surface Stresses at Insert to Shell - DBE Vessel Empty

Type of Stress	A_0	A_i	B_0	B_i	C_0	C_i	D_0	D_i
Radial (psi)	18,185	17,850	17,785	17,625	17,155	18,265	18,815	17,165
Tangential (psi)	17,710	17,580	17,600	17,530	17,423	17,717	17,887	17,393
Maximum surface (psi)	18,185	17,805	17,785	17,625	17,423	18,265	18,815	17,393
Allowable surface (psi)	52,500	52,500	52,500	52,500	52,500	52,500	52,500	52,500

TABLE 3.8-3 (SHEET 6 OF 6)





SECTION A-A

Surface Stresses at Lock to Insert - OBE Vessel Flooded

Type of Stress	A_0	A_i	B_0	B_i	C_0	C_i	D_0	D_i
Radial (psi)	26,535	26,387	26,185	26,267	25,753	26,539	27,097	25,837
Tangential (psi)	26,335	26,289	26,271	26,253	26,109	26,419	26,497	26,123
Maximum surface (psi)	26,535	26,387	26,271	26,267	26,109	26,539	27,097	26,123
Allowable surface (psi)	52,500	52,500	52,500	52,500	52,500	52,500	52,500	52,500

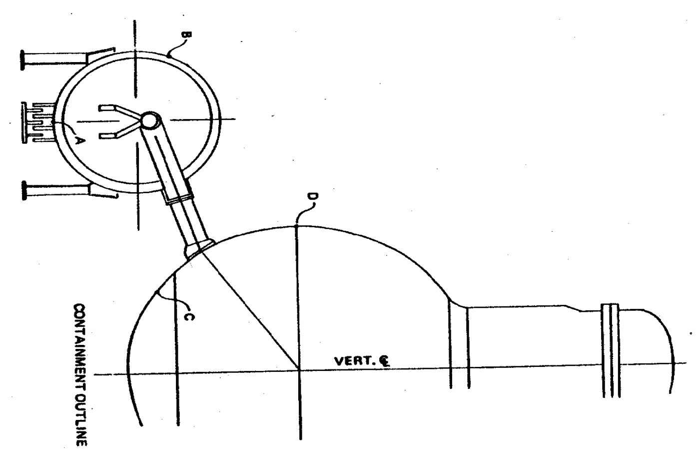
Surface Stresses at Insert to Shell - OBE Vessel Flooded

Type of Stress	A_0	A_i	B_0	B_i	C_0	C_i	D_0	D_i
Radial (psi)	18,000	17,720	17,710	17,590	16,980	18,120	18,730	17,190
Tangential (psi)	17,654	17,565	17,575	17,525	17,369	17,703	17,861	17,439
Maximum surface (psi)	18,000	17,720	17,710	17,590	17,369	18,120	18,730	17,439
Allowable surface (psi)	52,500	52,500	52,500	52,500	52,500	52,500	52,500	52,500

TABLE 3.8-4

SUMMARY OF STRESS LIMITS

Critical Section	Applicable Code	Type of Stress	Operating Condition OBE	Operating Condition DBE	Accident, Initial, and Final Testing Conditions[a] OBE	Accident, Initial, and Final Testing Conditions[a] DBE	Flooded Condition OBE
A	ASME III-MC	Membrane	Sm	0.9 Sy	Sm	0.9 Sy	0.9 Sy
B	ASME III-MC	Membrane	Sm	0.9 Sy	Sm	0.9 Sy	0.9 Sy
C	ASME III-MC	Membrane	Sm	0.9 Sy	Sm	0.9 Sy	0.9 Sy
D	ASME III-MC	Membrane	Sm	0.9 Sy	Sm	0.9 Sy	0.9 Sy



a. Does not include jet impingement loadings.

TABLE 3.8-8

ALLOWABLE STRESSES FOR DIFFERENT LOADING COMBINATIONS

Loading Combinations	Allowable Stresses
$D + L + E + T_o + R_o$ $D + L + E$ $D + L + E + Y_r + T_a + P_a + R_a$ (for pedestal only) $D + L + E + Y_j + T_a + P_a + R_a$ (for pedestal only)	Normal code allowable stresses (AISC for structural steel, ACI for reinforced concrete). The customary increase in allowable stresses, when earthquake loads are considered, is not permitted.
$D + L + E^1$ $D + L + E + Y_r + Y_j + Y_m + T_a + P_a + R_a$	Stresses do not exceed: 150% of AISC allowables for structural steel, 90% of yield stress for reinforcing bars, 85% of ultimate stress for concrete.
$D + L + E^1 + T_o + R_o$ $D + L + E^1 + Y_r + Y_j + Y_m + T_a + P_a + R_a$	No functional failure is permitted. Usually stresses are not allowed to exceed the yield point of the material for steel and the ultimate strength for concrete. If these limits are exceeded, energy absorption is determined and compared to the energy input from the earthquake. The design is such that energy absorption capacity exceeds energy input.

TABLE 3.8-9 (SHEET 1 OF 2)

SUMMARY OF STRESSES ON PEDESTAL RINGS

Location: Intersection of the Inner Ring and the Drywell el 107 ft 2 5/32 in.

	Outer Ring				Inner Ring			
Loading Case	Axial Stress (ksi)	Moment Stress (ksi)	Total Stress (ksi)	Allowable Stress (ksi)	Axial Stress (ksi)	Moment Stress (ksi)	Total Stress (ksi)	Allowable Stress (ksi)
Normal operating OBE	6.01	8.85	14.86	22.80	6.01	6.23	12.24	22.80
Normal operating DBE	6.64	16.67	23.31	34.20	6.64	11.73	18.37	34.20
Refueling OBE	6.18	8.85	15.03	22.80	6.18	6.23	12.41	22.80
Refueling DBE	6.81	16.67	23.48	34.20	6.81	11.73	18.54	34.20
Accident OBE, vert jet	6.44	10.21	16.65	22.80	6.44	7.18	13.62	22.80
Accident OBE, horiz jet	6.01	11.33	17.34	22.80	6.01	7.97	13.98	22.80
Accident DBE, vert jet	7.07	18.03	25.10	34.20	7.07	12.68	19.75	34.20
Accident DBE, horiz jet	6.64	19.15	25.79	34.20	6.64	13.47	20.11	34.20

Location: el 116 ft 3 in.

Normal operating OBE	6.01	7.27	13.28	22.80	6.01	5.11	11.12	22.80
Normal operating DBE	6.64	13.82	20.46	34.20	6.64	9.72	16.36	34.20
Refueliing OBE	6.18	7.27	13.45	22.80	6.18	5.11	11.29	22.80
Refueling DBE	6.81	13.82	20.63	34.20	6.81	9.72	16.53	34.20
Accident OBE, vert jet	6.44	8.63	15.07	22.80	6.44	6.07	12.51	22.80
Accident OBE, horiz jet	6.01	9.01	15.02	22.80	6.01	6.34	12.35	22.80
Accident DBE, vert jet	7.07	15.18	22.25	34.20	7.07	10.67	17.74	34.20
Accident DBE, horiz jet	6.64	15.57	22.21	34.20	6.64	10.95	17.59	34.20

TABLE 3.8-9 (SHEET 2 OF 2)

Location: el 125 ft 11 in.

Loading Case	Outer Ring Axial Stress (ksi)	Outer Ring Moment Stress (ksi)	Outer Ring Total Stress (ksi)	Outer Ring Allowable Stress (ksi)	Inner Ring Axial Stress (ksi)	Inner Ring Moment Stress (ksi)	Inner Ring Total Stress (ksi)	Inner Ring Allowable Stress (ksi)
Normal operating OBE	4.93	5.63	10.56	22.80	4.93	3.96	8.89	22.80
Normal operating DBE	5.46	10.84	16.30	34.20	5.46	7.62	13.08	34.20
Refueling OBE	5.10	5.63	10.73	22.80	5.10	3.96	9.06	22.80
Refueling DBE	5.63	10.84	16.47	34.20	5.63	7.62	13.25	34.20
Accident OBE, vert jet	5.19	6.95	12.14	22.80	5.19	4.88	10.07	22.80
Accident OBE, horiz jet	4.93	6.57	11.50	22.80	4.93	4.62	9.55	22.80
Accident DBE, vert jet	5.72	12.15	17.87	34.20	5.72	8.55	14.27	34.20
Accident DBE, horiz jet	5.46	11.78	17.24	34.20	5.46	8.29	13.75	34.20

Location: el 132 ft 2 in.

Loading Case	Outer Ring Axial Stress (ksi)	Outer Ring Moment Stress (ksi)	Outer Ring Total Stress (ksi)	Outer Ring Allowable Stress (ksi)	Inner Ring Axial Stress (ksi)	Inner Ring Moment Stress (ksi)	Inner Ring Total Stress (ksi)	Inner Ring Allowable Stress (ksi)
Normal operating OBE	4.30	4.83	9.13	22.80	4.30	3.40	7.70	22.80
Normal Operating DBE	4.83	9.42	14.25	34.20	4.83	6.62	11.45	34.20
Refueling OBE	4.30	4.83	9.13	22.80	4.30	3.40	7.70	22.80
Refueling OBE	4.83	9.41	14.24	34.20	4.83	6.62	11.45	34.20
Accident OBE, vert jet	4.56	5.95	10.51	22.80	4.56	4.18	8.74	22.80
Accident OBE, horiz jet	4.30	5.26	9.56	22.80	4.30	3.70	8.00	22.80
Accident DBE, vert jet	5.08	12.84	17.92	34.20	5.08	9.03	14.11	34.20
Accident DBE, horiz jet	4.83	9.85	14.68	34.20	4.83	6.92	11.75	34.20

TABLE 3.8-10

SUMMARY OF STRESS LIMITS ON REACTOR SHIELD

Location (el)	Actual Factor as Per AISC Section 1.6 for Combined Stresses Using Critical Load Combination	Allowable Factor as Per AISC Section 1.6 for Combined Stresses
148 ft 3 1/2 in.	0.58	1.0
151 ft 9 in.	0.59	1.0
156 ft 1 9/16 in.	0.78	1.0
162 ft 1 in.	0.65	1.0
168 ft 6 in.	0.73	1.0
178 ft 11 in.	0.73	1.0
188 ft 1/2 in.	0.69	1.0

TABLE 3.8-11

BASIC MATERIALS FOR CONSTRUCTION OF INTERNAL STRUCTURES

1. Concrete		f′c = 4000 psi
2. Reinforcing steel		
Deformed bars	ASTM A 615 Grade 60	fy = 60,000 psi
3. Structural and miscellaneous steel		
Rolled shapes, bars, and plates	ASTM A 36	fy = 36,000 psi
	ASTM A 441	fy = 42,000 to 50,000 psi
	ASTM A 516 Grade 70	fy = 38,000 psi
High-strength bolts	ASTM A 325 ASTM A 193-B7	
Nuts for the pedestal	ASTM A 194 Grade 7	
Washers for the pedestal	ASTM A 516 Grade 70	

TABLE 3.8-12

WATER VELOCITY AT INTAKE STRUCTURE

	el 71.5 ft Normal Water (ft/s)	el 61.7 ft Low Water (ft/s)
Velocity at inlet		
Normal pumping requirement	0.40	1.08
All pumps operating	0.78	2.12
Velocity through screens		
Normal pumping requirements	0.91	2.82
All pumps operating	1.79	5.55

TABLE 3.8-13 (SHEET 1 OF 2)

LIVE LOADS ON STRUCTURES

	Beams and Slabs	Girders and Columns
General		
Roof (minimum)	20 lb/ft^2	20 lb/ft^2
Offices	50 lb/ft^2	40 lb/ft^2
Stairways and walkways	100 lb/ft^2	80 lb/ft^2
Assembly rooms	100 lb/ft^2	80 lb/ft^2
Concentrated loads[a]	4000 lb	4000 lb
Reactor Building (excluding drywell and torus area)		
Floor at el 130 ft		
General	600 lb/ft^2	600 lb/ft^2
In corners	250 lb/ft^2	200 lb/ft^2
Near equipment hatches	1000 lb/ft^2	1000 lb/ft^2
Near railroad airlock	Cooper E72 locomotive wheel loads	
Floor at el 158 ft, el 185 ft, and el 205 ft	200 lb/ft^2	200 lb/ft^2
Floor at el 228 ft		
General	1000 lb/ft^2	800 lb/ft^2
Cask areas	250,000 lb (6-ft diameter)	250,000 lb (6-ft diameter)
New-fuel storage area	600 lb/ft^2	600 lb/ft^2
Spent-fuel pool and dryer separator storage pool	Water plus equipment stored	Water plus equipment stored
Torus Area		
Floor el 87 ft	150 lb/ft^2 or torus water load	
Intake Structure		
Valve pit slab	200 lb/ft^2	200 lb/ft^2
Pump room slab	200 lb/ft^2	200 lb/ft^2
Grating floor	100 lb/ft^2	100 lb/ft^2
Base slab	75 lb/ft^2	

TABLE 3.8-13 (SHEET 2 OF 2)

	Beams and Slabs	Girders and Columns
Control Building		
Base slab	250 lb/ft^2	
All floors	350 lb/ft^2	350 lb/ft^2
Laydown area	1000 lb/ft^2	1000 lb/ft^2
Diesel Generator Building		
Base slab	200 lb/ft^3	

Crane Loads

Crane and elevator loads are considered live loads. A 25% impact increase to live load is used for traveling-crane support girders and columns. A 100% impact increase to live load is used for elevator supports.

a. For design of floor elements only. Applied at the point of maximum moment or shear. It is not cumulative and not carried to columns.

TABLE 3.8-14

LOADING COMBINATION AND ACCEPTANCE CRITERIA

Loading Combination	Acceptance Criteria
$D + L + T_a + 1.5\ P_a + R_a$	Stresses do not exceed 150% of AISC allowables for structural steel, 90% of yield stress for reinforcing bars, 85% of ultimate stress for concrete.
$D + L + 1.25\ E + 1.25\ P_a + T_a + R_a + Y_r + Y_j + Y_m$	No functional failure is permitted. Usually stresses are not allowed to exceed the yield point of the material for steel and the ultimate strength for concrete. If these limits are exceeded, energy adsorption is determined and compared to the energy input from the earthquake. The design is such that energy adsorption capacity exceeds energy input.

where:

D, L, E, P_a, T_a, R_a, Y_r, Y_j, and Y_m t are defined in paragraph 3.8.3.3.

TABLE 3.8-15

ALLOWABLE STRESSES FOR DIFFERENT LOADING COMBINATIONS

Loading Combination	Acceptance Stresses
$D + L + T_o + R_o + E$ $D + L + I + E$ $D + L + C + E$	Normal allowable code stresses (AISC for structural steel, ACI for reinforced concrete). The customary increase in allowable stresses, when earthquake loads are considered, is not permitted.
$D + L + T_o + R_o + W$	Normal code allowable stresses.
$D + L + T_a + 1.5P_a + R_a$[a]	Stresses do not exceed: 150% of AISC allowables for structural steel, 90% of yield for reinforcing bars, 85% of ultimate stress for concrete.
$D + L + T_o + R_o + E^1$ $D + L + T_o + R_o + W_t$ $D + L + 1.25E + 1.25P_a + T_a + R_a + Y_r + Y_j + Y_m$[a] $D + L + E^1 + P_a + T_a + R_a + Y_r + Y_j + Y_m$	Stresses are limited to the minimum yield point as a general case. However, in a few cases when missile loads are included, stresses may exceed yield point. In such cases, an analysis, using the limit-design approach, is made to determine the energy absorption capacity which should be such that it exceeds the energy input. The resulting distortion is limited to ensure no loss of function and an adequate factor of safety against collapse.

a. These loading combinations and structural acceptance criteria were used to evaluate the effects of high-energy pipe breaks.

TABLE 3.8-16

BASIC CONSTRUCTION MATERIALS FOR SEISMIC CATEGORY I STRUCTURES

1. Concrete	f′c = 4000 psi (all structures) and	5000 psi (certain areas of refueling floor only)
2. Reinforcing steel		
Deformed bars	ASTM A 615, Grade 60	fy = 60,000 psi
3. Structural and miscellaneous steel		
Rolled shapes, bars, and plates	ASTM A 36	fy = 36,000 psi
High-strength bolts	ASTM A 325 or A 490	
Stainless steel	ASTM A 240, Type 304	

TABLE 3.8-17

AGGREGATE TESTS

ASTM No.	Title	Results to Be Achieved	Initial Test	User's Test	Daily Test
C-33	Gradation	To conform with specification	X		X
C-40	Organic impurities	To conform with specification	X		
C-87	Mortar-making properties	To conform with specification	X		
C-88	Soundness	To conform with specification	X		
C-117	Material finer than No. 200 sieve	Design mix calculations	X	X	
C-127	Specific gravity and absorption (coarse aggregates)	Design mix calculations	X	X	
C-128	Specific gravity and absorption (fine aggregates)	Design mix calculations	X	X	
C-131	Los Angeles abrasion	To conform with specification	X		
C-136	Sieve analysis	To conform with specification	X		
C-142	Clay lumps	To conform with specification	X		
C-227	Potential reactivity (mortar bar)	To conform with specification	X		
C-289	Potential reactivity (chemical)	To conform with specification	X		
C-295	Petrographic	To conform with specification	X		

TABLE 3.8-18

CEMENT TESTS

ASTM No.	Type of Test	Initial Test	User's Test	Daily Test
C-109	Compressive strength	X	X	
C-114	Chemical analysis	X	X	
C-115	Fineness-turbidimeter	X	X	
C-151	Autoclave expansion (soundness)	X	X	
C-185	Air content	X	X	
C-266	Time of setting Gilmore	X	X	

TABLE 3.8-19

FLY ASH TESTS

ASTM No.	Type of Test	Initial Test	User's Test
C-114	Chemical analysis loss on ignition	X	X
C-185	Air entrainment of mortar	X	X
C-188	Specific gravity	X	X
C-204	Fineness	X	X
C-311	Sampling and tests	X	X

TABLE 3.8-20 (SHEET 1 OF 13)

MASONRY WALL REEVALUATION RESULTS

Wall No.[(a)]	Safety-Related Equipment[(b)]	Wall Analysis[(c)]
C112-1A -1B	Yes	MR
C112-2A -2B	Yes	SA
C112-3A -3B	No	Not required
C112-4A -4B	Yes	SA
C112-5A -5B	Yes	SA
C112-6A -6C	Yes	SA
C112-6B -6D	No	Not required
C112-7A -7C	No	Not required
C112-7B -7D	No	Not required
C112-8A -8C	No	Not required
C112-8B -8D	No	Not required
C112-9A -9C	No	Not required
C112-9B -9D	No	Not required

TABLE 3.8-20 (SHEET 2 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C112-10A -10C	Yes	SA
C112-10B -10D	Yes	SA
C112-11A -11B	Yes	SA
C112-12A -12B	Yes	SA
C112-13A -13B	No	Not required
C112-14A -14B	Yes	SA
C112-15A -15B	Yes	SA
C112-16A -16B	Yes	SA
C112-17A -17B	Yes	SA
C112-18A -18B	Yes	SA
C112-19A -19B	Yes	SA
C112-20A -20B	Yes	SA
C112-21A -21D	Yes	SA
C112-21B -21E	Yes	SA

TABLE 3.8-20 (SHEET 3 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C112-21C -21F	Yes	SA
C112-22A -22B	Yes	SA
C112-23A -23B	Yes	SA
C112-24A -24B	Yes	SA
C112-25A -25B	Yes	SA
C112-25C -25D	Yes	SA
C112-26A -26B	Yes	SA
C112-27A -27B	Yes	SA
C112-28A -28B	Yes	SA
C112-29A -29B	Yes	SA
C112-30A -30B	Concrete walls	Not required
C112-31A -31B	Concrete walls	Not required
C112-32A -32B	Concrete walls	Not required
C112-33A -33B	Concrete walls	Not required

TABLE 3.8-20 (SHEET 4 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C112-34A -34B	Yes	SA
C130-1A -1B	Yes	MR
C130-2A -2B	Yes	MR
C130-3A -3B	Yes	MR
C130-4A -4B	No	Not required
C130-5A -5B	Yes	SA
C130-6A -6B	Yes	SA
C130-7A -7B	Yes	MR
C130-8A -8B	Yes	SA
C130-9A -9B	Yes	SA
C130-10A -10B	Yes	SA
C130-11A -11B	Yes	SA
C130-12A -12B	Yes	SA
C130-13A -13B	Yes	SA

TABLE 3.8-20 (SHEET 5 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C130-14A -14B	Yes	MR
C130-15A -15B	Yes	SA
C130-16A -16B	Yes	SA
C130-17A -17B	Yes	SA
C130-17C -17D	Yes	SA
C130-18A -18B	No	Not required
C130-18C	No	Not required
C130-19A -19B	No	Not required
C130-20A -20B	Yes	SA
C130-20C -20D	Yes	SA
C130-21A -21B	Yes	SA
C130-21C -21D	Yes	SA
C130-22A -22B	Yes	SA
C130-23A -23B	No	Not required

TABLE 3.8-20 (SHEET 6 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C130-24A -24B	No	Not required
C130-25A -25B	No	Not required
C130-25C -25D	No	Not required
C130-26A -26B	No	Not required
C130-27A -27B	No	Not required
C130-28A -28B	No	Not required
C130-29A -29B	No	Not required
C130-30A -30B	No	Not required
C130-31A -31B	No	Not required
C130-32A -32B	No	Not required
C130-33A -33B	No	Not required
C130-34A -34B	No	Not required
C130-35A -35B	Yes	SA
C130-36A -36B	No	Not required

TABLE 3.8-20 (SHEET 7 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C130-37A -37B	No	Not required
C130-38A -38B	No	Not required
C130-39A -39B	Yes	MR
C130-39C -39D	Yes	MR
C130-40A -40B	Yes	SA
C130-41A -41B	Yes	MR
C130-42A -42B	No	Not required
C130-43A -43B	No	Not required
C130-44A -44B	Yes	SA
C130-44C -44D	Yes	SA
C130-45A -45B	Yes	SA
C130-46A -46B	Yes	SA
C130-47A -47B	Yes	SA
C130-47C -47D	Yes	SA

TABLE 3.8-20 (SHEET 8 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C130-47E -47F	Yes	SA
C130-48A -48B	Yes	SA
C130-48C -48D	Yes	SA
C130-49A -49B	No	Not required
C130-49C -49D	No	Not required
C130-49E -49F	No	Not required
C130-50A -50B	Yes	SA
C130-51A -51B	Yes	SA
C130-52A -52B	Yes	SA
C130-53A -53B	Yes	SA
C130-54A -54B	Yes	SA
C130-55A -55B	Yes	SA
C130-56A -56B	Yes	SA
C130-57A -57B	Yes	SA

TABLE 3.8-20 (SHEET 9 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C130-58A -58B	Yes	SA
C130-59A -59B	Yes	SA
C130-60A -60B	Yes	SA
C130-61A -61B	Yes	SA
C130-62A -62B	Yes	SA
C130-63A -63B	Yes	SA
C130-64A -64B	Yes	SA
C130-65A -65B	Yes	SA
C130-38C -38D	No	Not required
C147-1A -1B	Yes	SA
C147-1C -1D	Yes	SA
C147-2A -2B	Yes	SA
C147-3A -3B	Yes	SA
C147-4A -4B	Yes	SA

TABLE 3.8-20 (SHEET 10 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C147-5A -5B	No	Not required
C147-6A -6B	Yes	SA
C147-7A -7B	No	Not required
C147-8A -8B	No	Not required
C147-9A -9B	Yes	SA
C147-10A -10B	Yes	SA
C147-10C -10D	Yes	SA
C164-1A -1B	Yes	SA
C164-2A -2B	Yes	SA
C164-3A -3B	Yes	SA
C164-4A -4B	Yes	MR
C164-5A -5B	Yes	SA
C164-6A -6B	No	Not required
C164-7A -7B	Yes	SA

TABLE 3.8-20 (SHEET 11 OF 13)

Wall No.[a]	Safety-Related Equipment[b]	Wall Analysis[c]
C164-8A -8B	No	Not required
C164-9A -9B	Yes	SA
C164-10A -10B	No	Not required
C164-11A -11B	Yes	SA
C164-12A -12B	Yes	SA
C164-13A -13B	Yes	SA
C180-1A -1B	No	Not required
C180-2A -2B	No	Not required
C180-3A -3B	No	Not required
C180-4A -4B	No	Not required
R130-1A -1B	Yes[a]	Not required
R130-2A -2B	Yes[a]	Not required
R130-3A -3B	Yes	SA
R130-4A -4B	Yes	SA

TABLE 3.8-20 (SHEET 12 OF 13)

Wall No.[(a)]	Safety-Related Equipment[(b)]	Wall Analysis[(c)]
R158-1A -1B	Yes[(a)]	Not required
R158-2A -2B	Yes[(a)]	Not required
R158-3A -3B	Yes[(a)]	Not required
R158-4A -4B	Yes[(a)]	Not required
R158-5A -5B	No	Not required
R158-6A -6B	No	Not required
R185-1A -1B	No	Not required
R185-2A -2B	No	Not required
R185-3A -3B	No	Not required
R185-4A -4B	Concrete wall	Not required
R203-1A -1B	No	Not required
R203-2A -2B	No	Not required
R203-3A -3B	No	Not required
S108-2A -2B	Yes	SA

TABLE 3.8-20 (SHEET 13 OF 13)

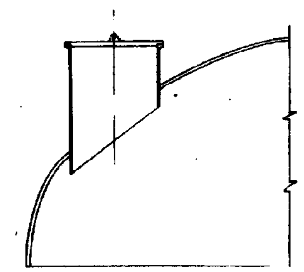
a. Wall numbering system: Example: C112-20A.

C is used to denote control building; R denotes reactor building; and S denotes main stack.

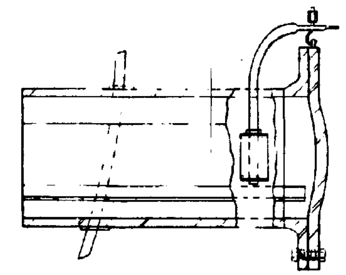
112 indicates the floor elevation supporting the wall. 20 indicates the consecutive wall number, and A indicates the particular side of the wall in question.

b. Yes - Safety-related equipment is attached to or in proximity to wall. No - No safety-related equipment is attached to or in proximity to wall.

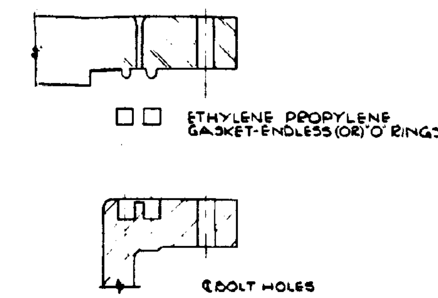
c. SA - Stresses are within design allowables. MR - Modification required due to one or more stresses above design allowables. Necessary modifications were completed.



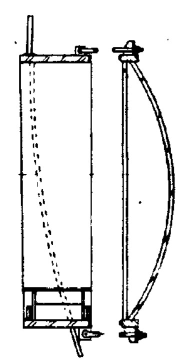
DRYWELL HEAD PERSONNEL ACCESS HATCH



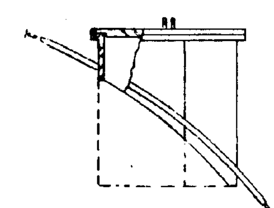
CRD REMOVAL HATCH



TYPICAL TESTABLE DOUBLE SEAL



TYP 10'-0" DIA EQUIPMENT DOOR ASS'Y



TYPICAL SUPPRESSION CHAMBER MANHOLE ENTRANCE

PERSONNEL AND EQUIPMENT ACCESS LOCKS

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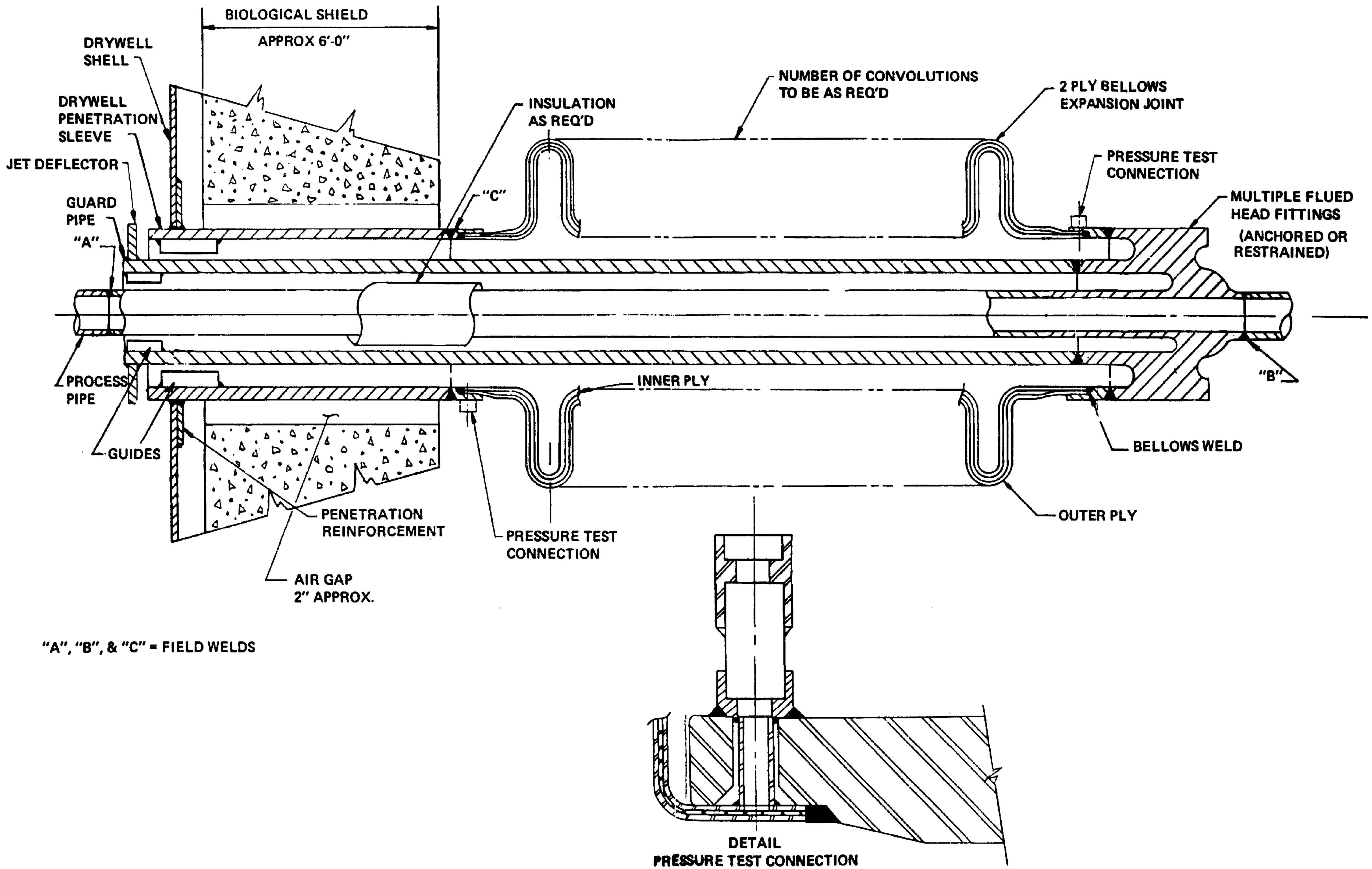
UNIT 2

PERSONNEL AND EQUIPMENT ACCESS LOCKS FOR DRYWELL AND SUPPRESSION CHAMBER

FIGURE 3.8-1



"A", "B", & "C" = FIELD WELDS

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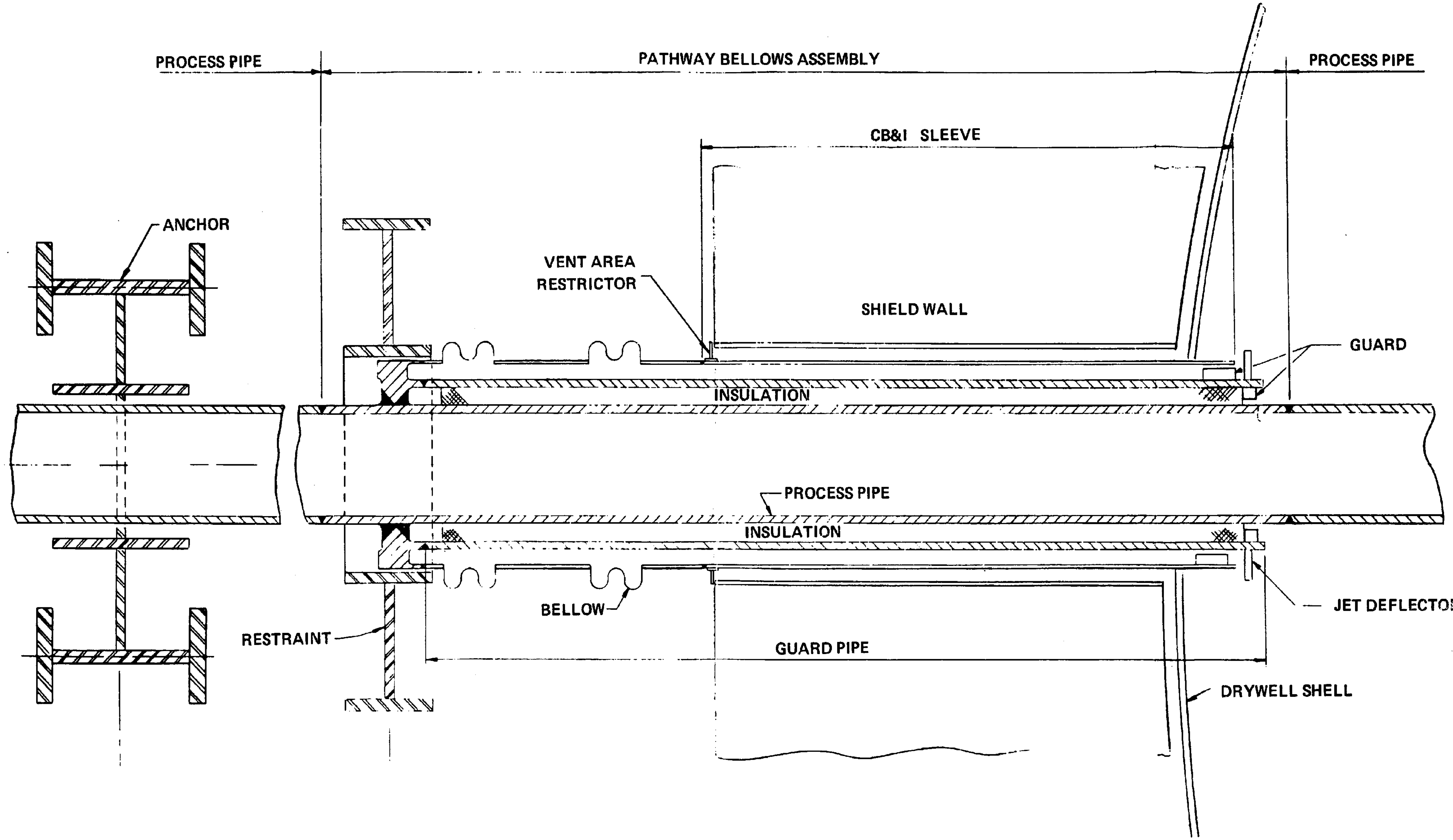
PIPE PENETRATIONS – TYPE 1
ACCOMMODATE THERMAL MOVEMENTS

FIGURE 3.8-2 (SHEET 1 OF 4)



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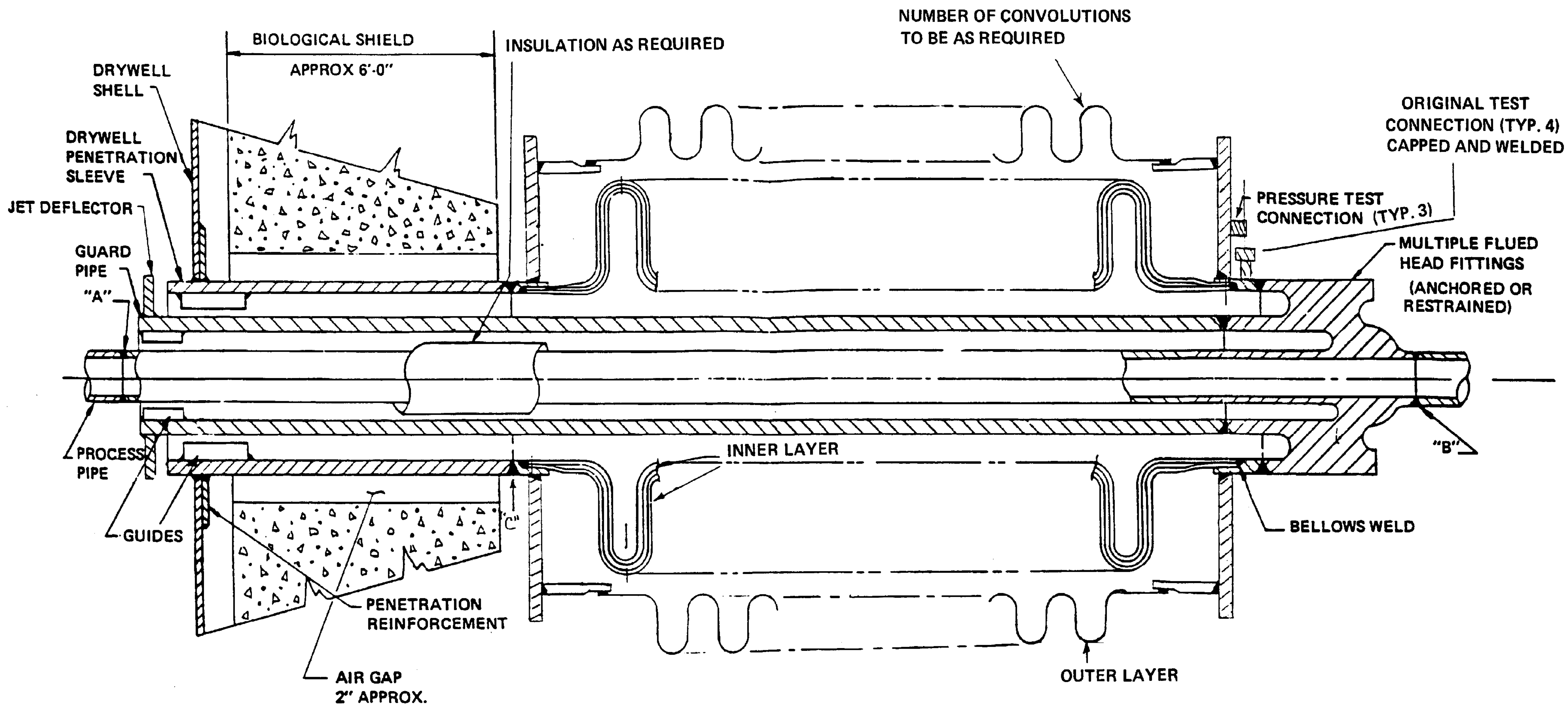
PIPE PENETRATIONS – TYPE 1A
(MAIN STEAM LINE ONLY)
ACCOMMODATE THERMAL MOVEMENTS

FIGURE 3.8-2 (SHEET 2 OF 4)



"A", "B", & "C" = FIELD WELDS

CLAMSHELL - DESIGN BELLOWS ASSEMBLIES FOR PENETRATION X-8.

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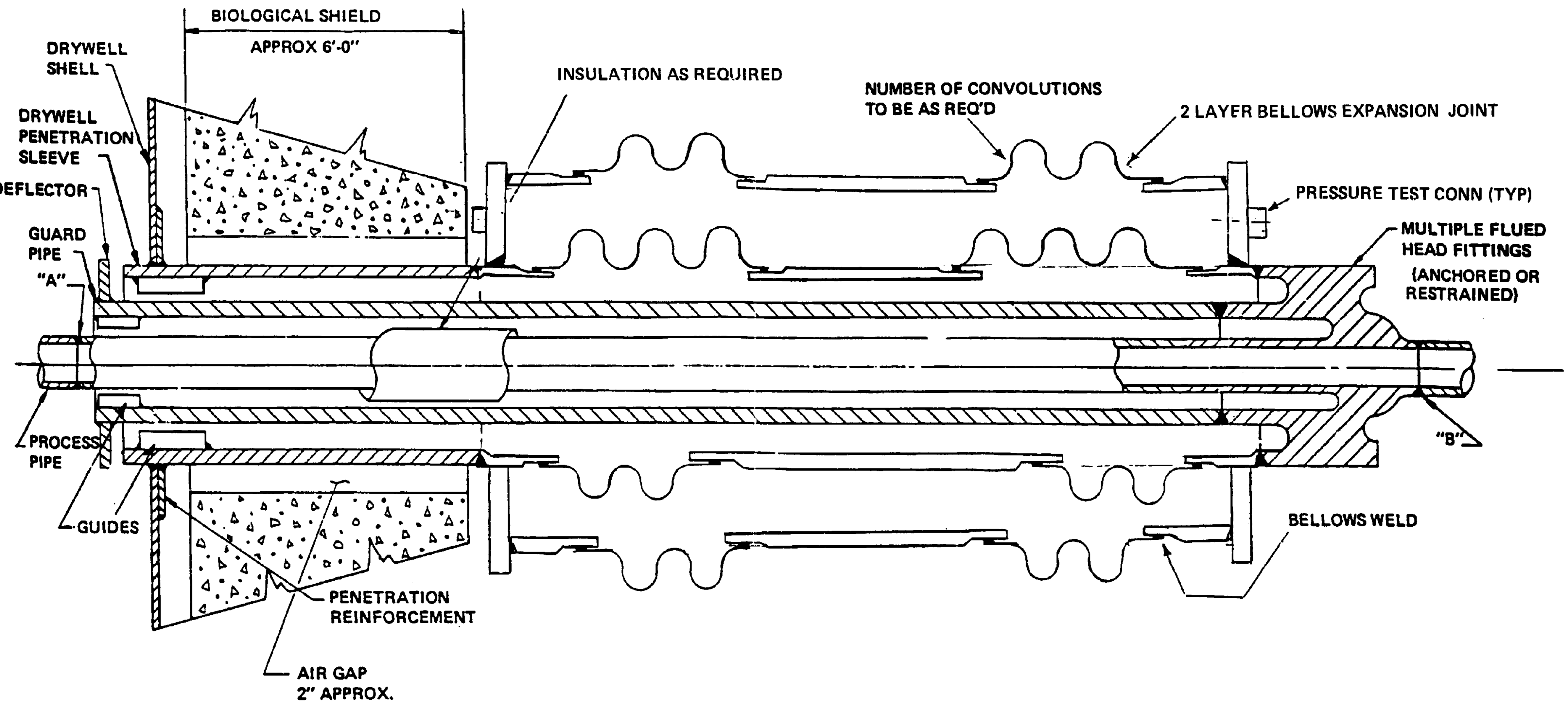
PIPE PENETRATIONS – TYPE 1B
ACCOMMODATE THERMAL MOVEMENTS

FIGURE 3.8-2 (SHEET 3 OF 4)



"A", "B", & "C" = FIELD WELDS

CLAMSHELL BELLOWS ASSEMBLIES DETAILS FOR PENETRATION X - 9A

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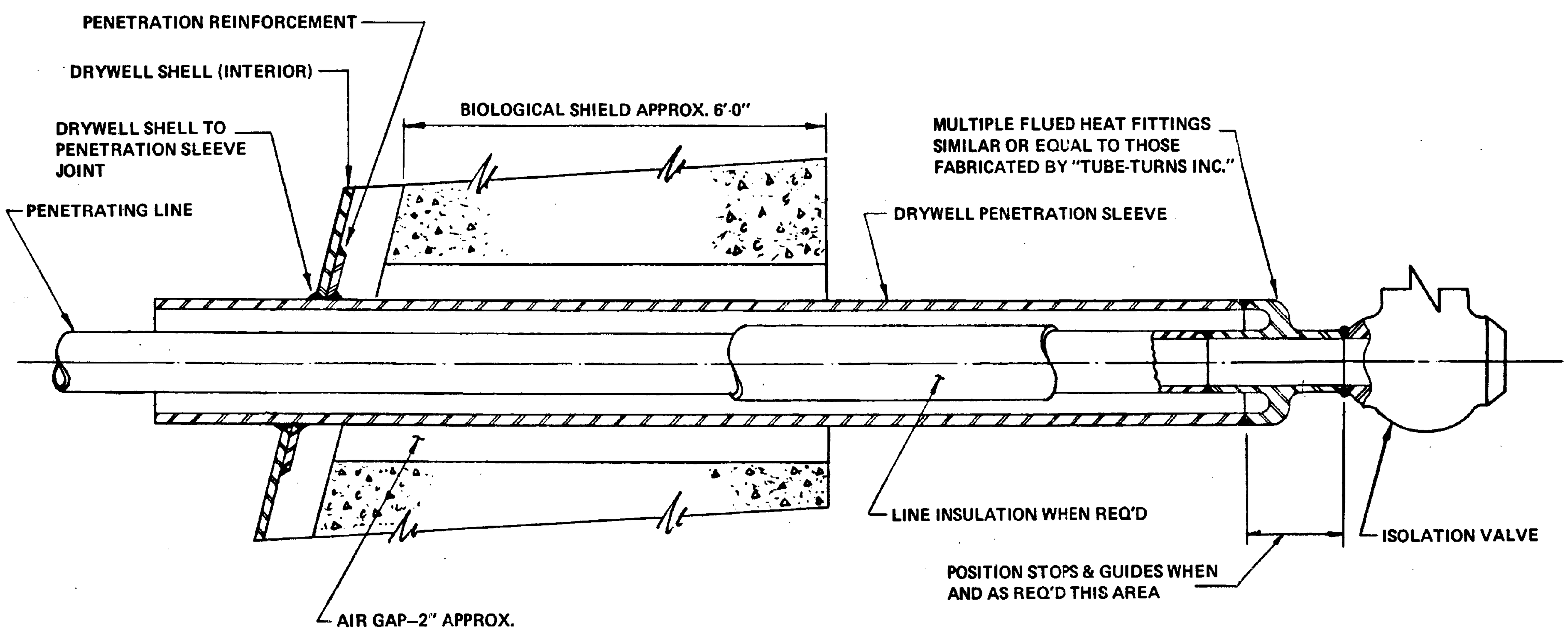
SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	PIPE PENETRATIONS – TYPE 1C ACCOMMODATE THERMAL MOVEMENTS FIGURE 3.8-2 (SHEET 4 OF 4)



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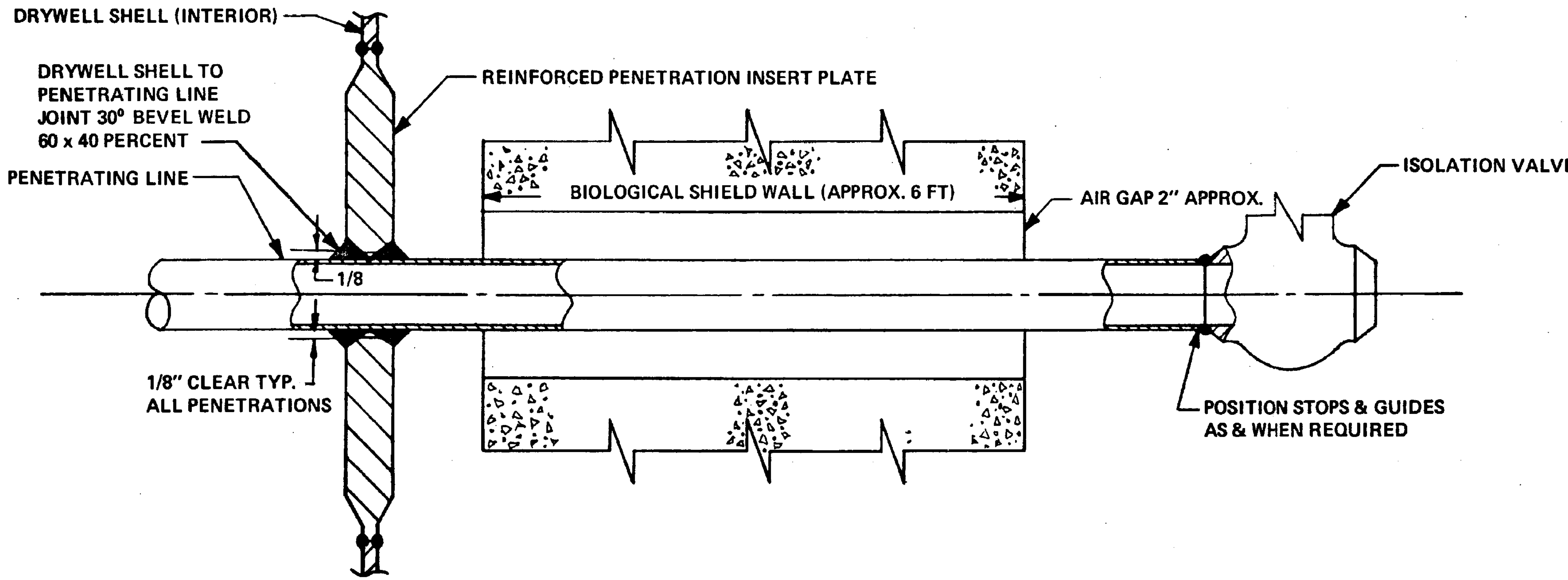
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

PIPE PENETRATIONS – TYPE 2.1 – THERMAL MOVEMENT RELATIVELY SMALL (SMALL BORE PIPING ONLY)

FIGURE 3.8-3



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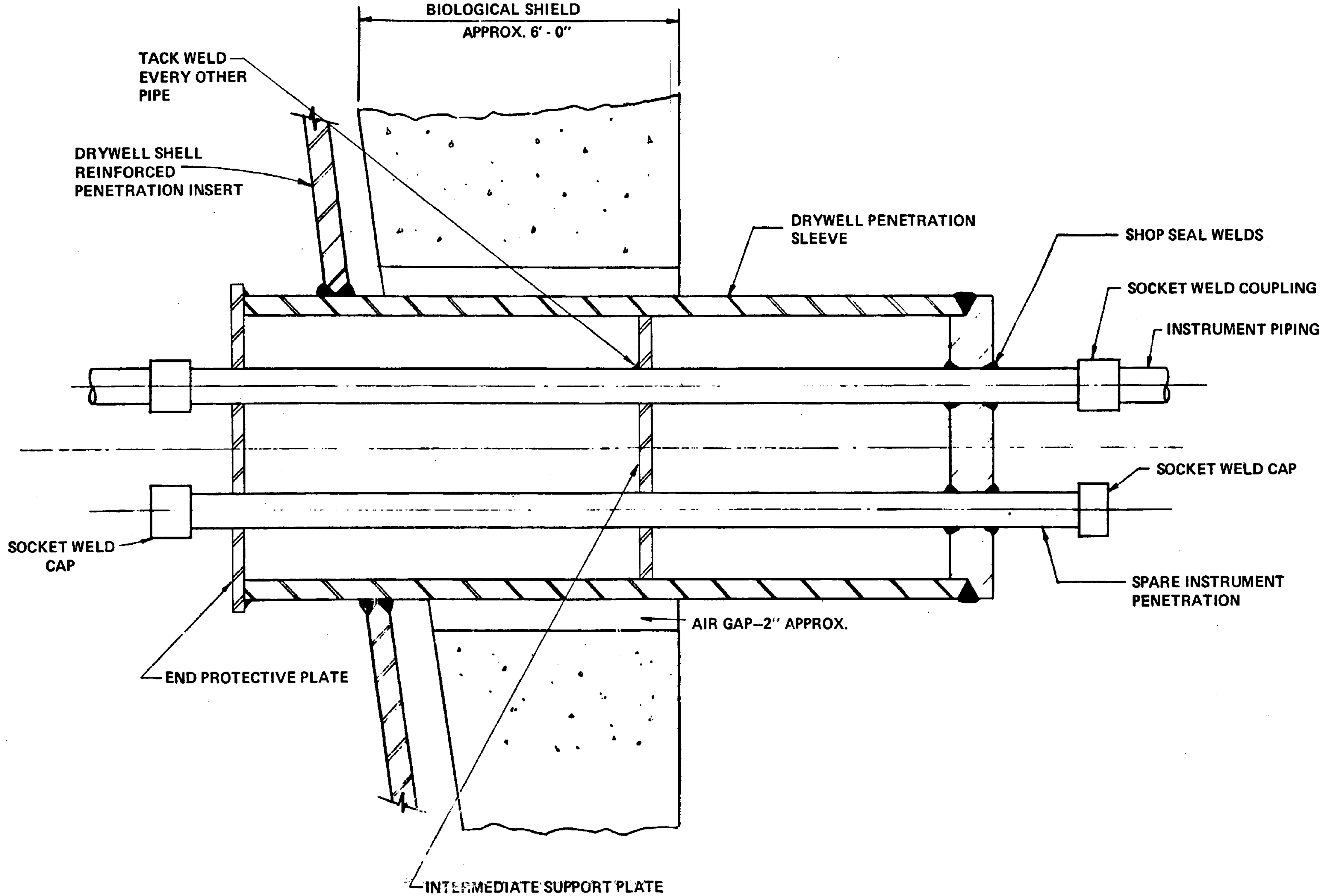
PIPE PENETRATIONS – TYPE 2.2 – THERMAL MOVEMENTS RELATIVELY SMALL

FIGURE 3.8-4



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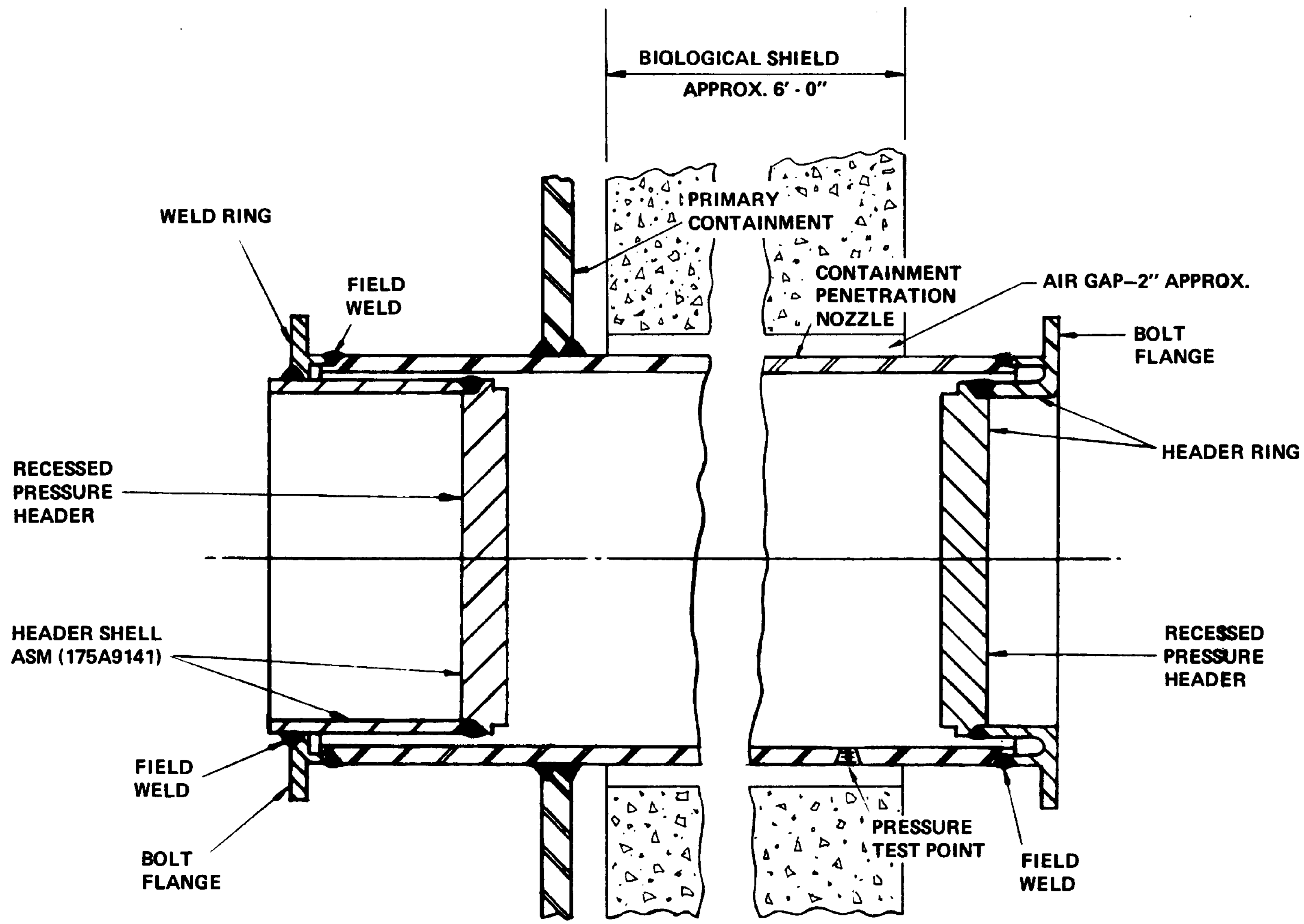
TYPICAL INSTRUMENT PENETRATION

FIGURE 3.8-5



ELECTRICAL PENETRATION ASSEMBLY STRUCTURAL COMPONENTS

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EF DWG SX-25401 REV A

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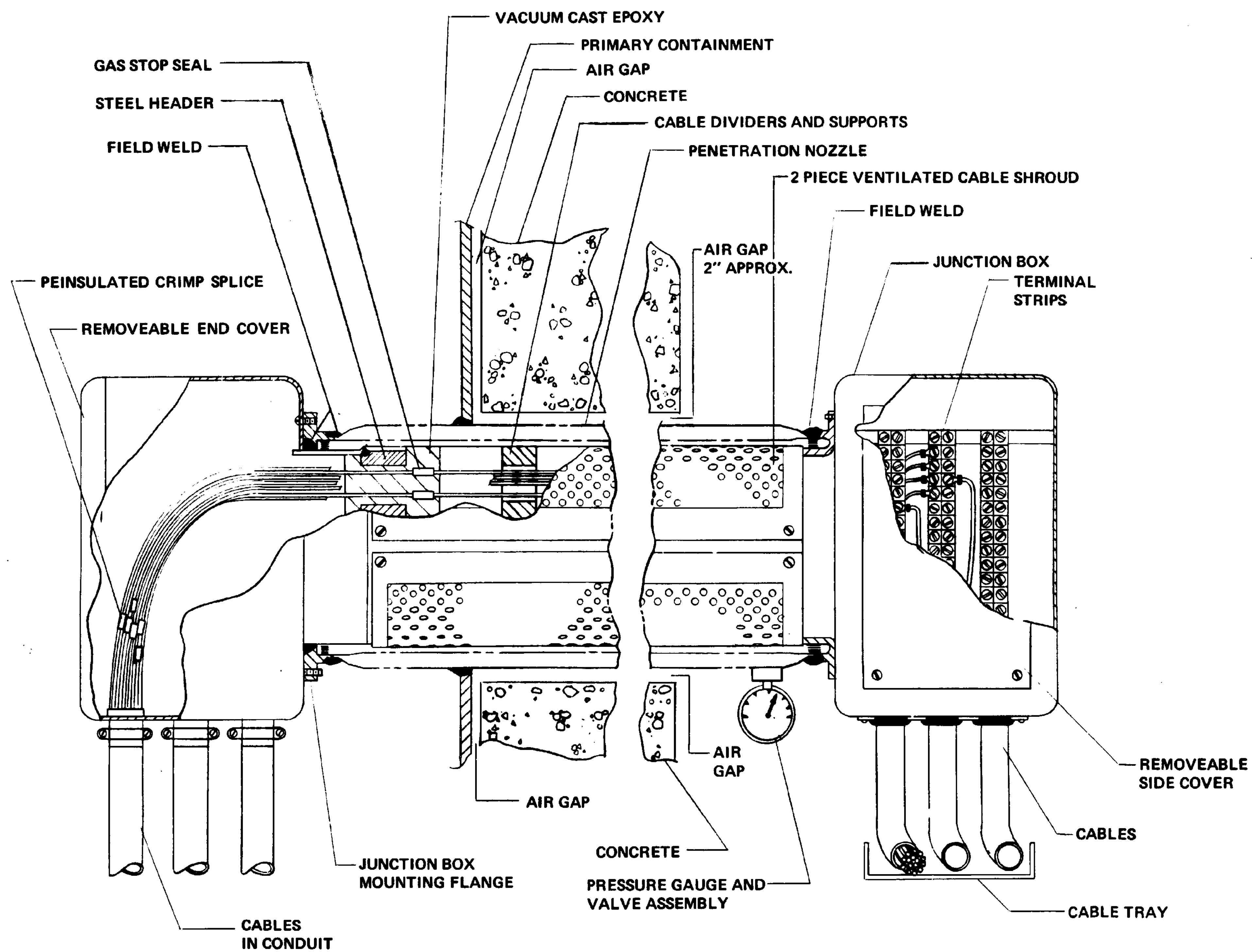
TYPICAL ELECTRICAL PENETRATION STRUCTURAL COMPONENTS

FIGURE 3.8-6



INSTALLED ELECTRICAL PENETRATION ASSEMBLY

ACAD 2030807

REV 19 7/01

EF DWG SX-25401 REV A

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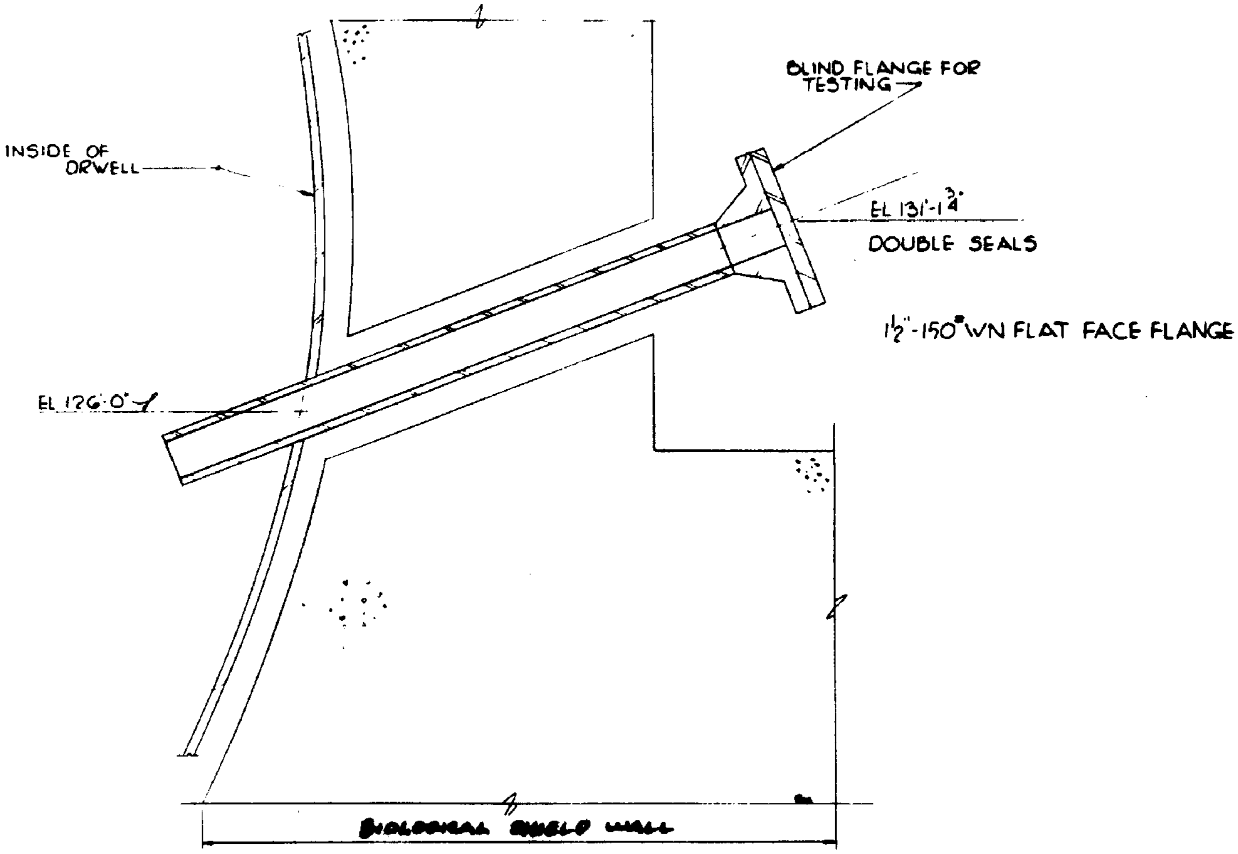
UNIT 2

TYPICAL ELECTRICAL PENETRATION ASSEMBLY DETAIL

FIGURE 3.8-7



TYPICAL TRAVERSING IN-CORE PROBE PENETRATION

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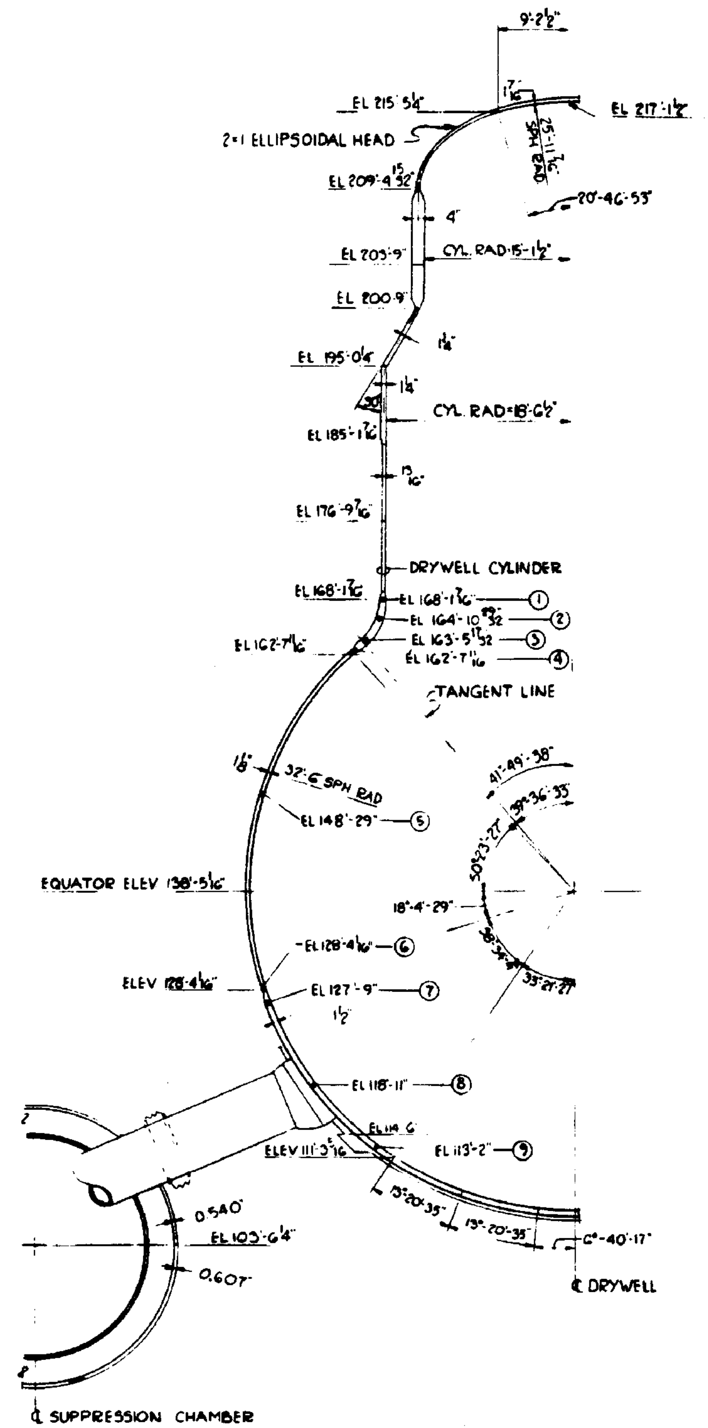
TRAVERSING INCORE PROBE PENETRATION

FIGURE 3.8-8



DRYWELL BASIC GEOMETRY

SHELL THICKNESS & STRESS LOCATIONS

SECTIONAL ELEVATION

ACAD 2030809

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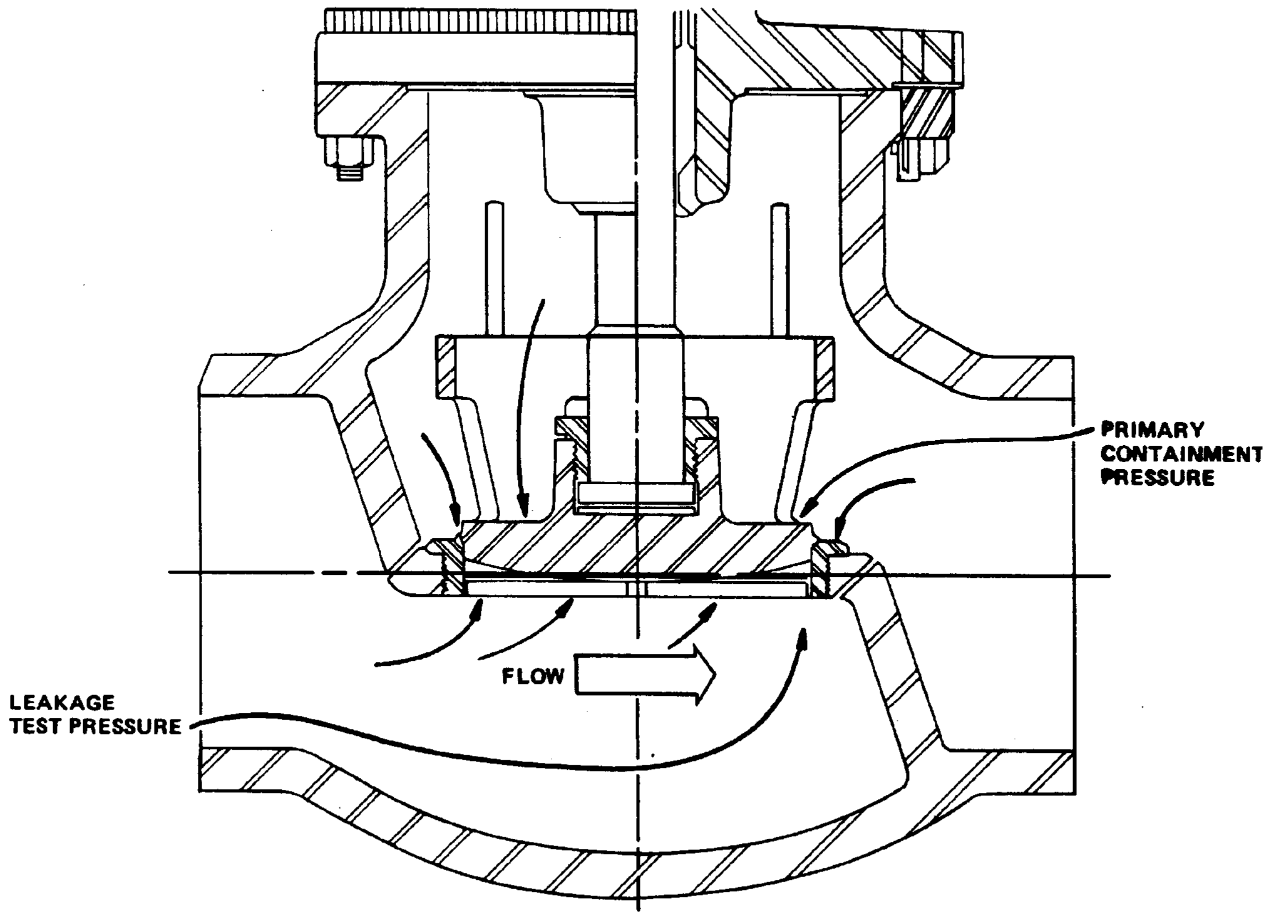
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

CONTAINMENT GEOMETRY
SHELL THICKNESS AND STRESS
LOCATIONS

FIGURE 3.8-9

APPLICABLE TO VALVES: 2E21-F015A, B



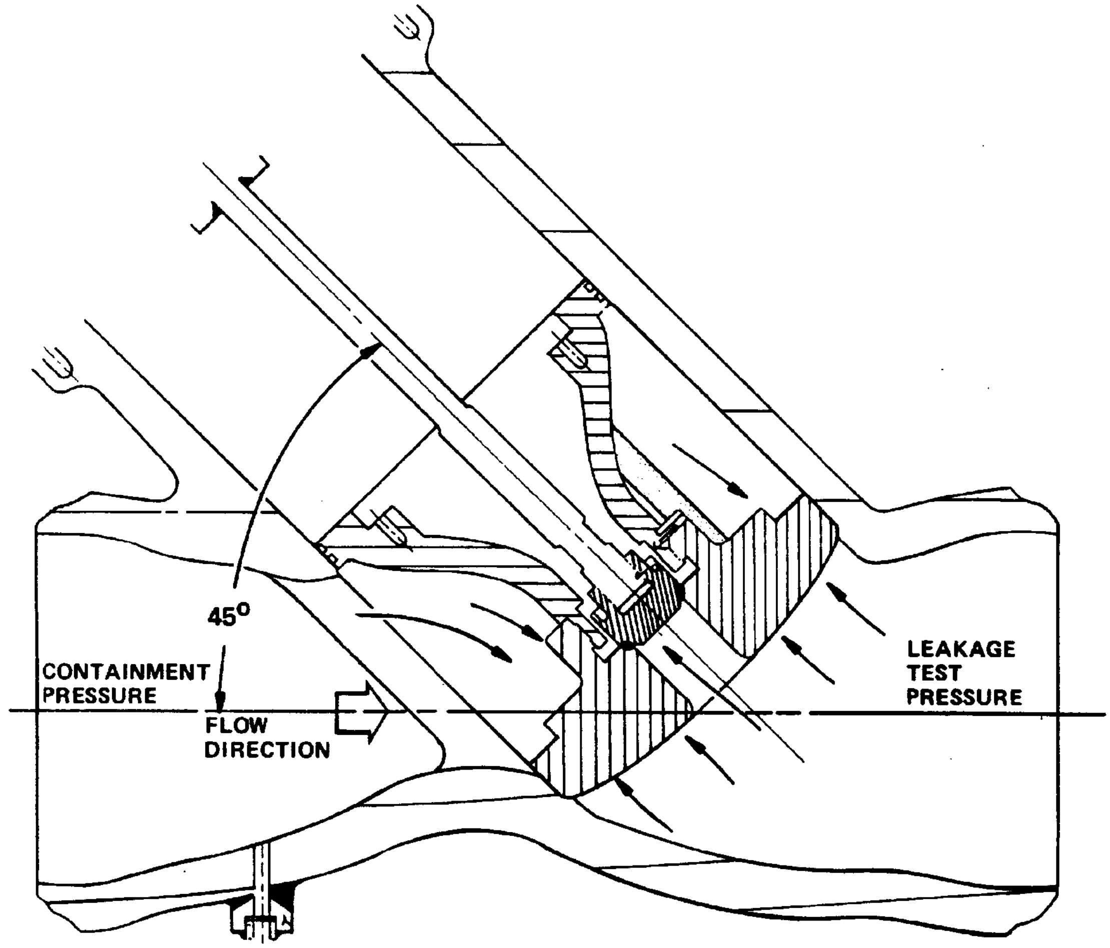
FORCES CAUSED BY THE APPLICATION OF LEAKAGE TEST PRESSURE UNDER THE VALVE DISC ACT AGAINST THE SEATING FORCE CREATED BY THE STEM ACTING ON THE DISC. FORCES DUE TO CONTAINMENT PRESSURE ACT ON TOP OF THE DISC AND ARE ADDITIVE TO THE SEATING FORCES OF THE STEM AGAINST THE DISC AND TEND TO SEAT THE VALVE MORE TIGHTLY.

ACAD 203810

REV 19 7/01

SOUTHERN COMPANY *Energy to Serve Your World®* SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	GLOBE VALVE
	FIGURE 3.8-10

APPLICABLE TO VALVES: 2B21-F022A, B, C, D



FORCES DUE TO THE LEAKAGE TEST PRESSURE ACT AGAINST THE SEATING FORCES OF THE VALVE. AS THE VALVE IS DESIGNED TO USE UPSTREAM PRESSURE TO PROVIDE A TIGHT SEAT, PRESSURE FORCES FROM THE CONTAINMENT DIRECTION WILL TEND TO SEAT THE VALVE.

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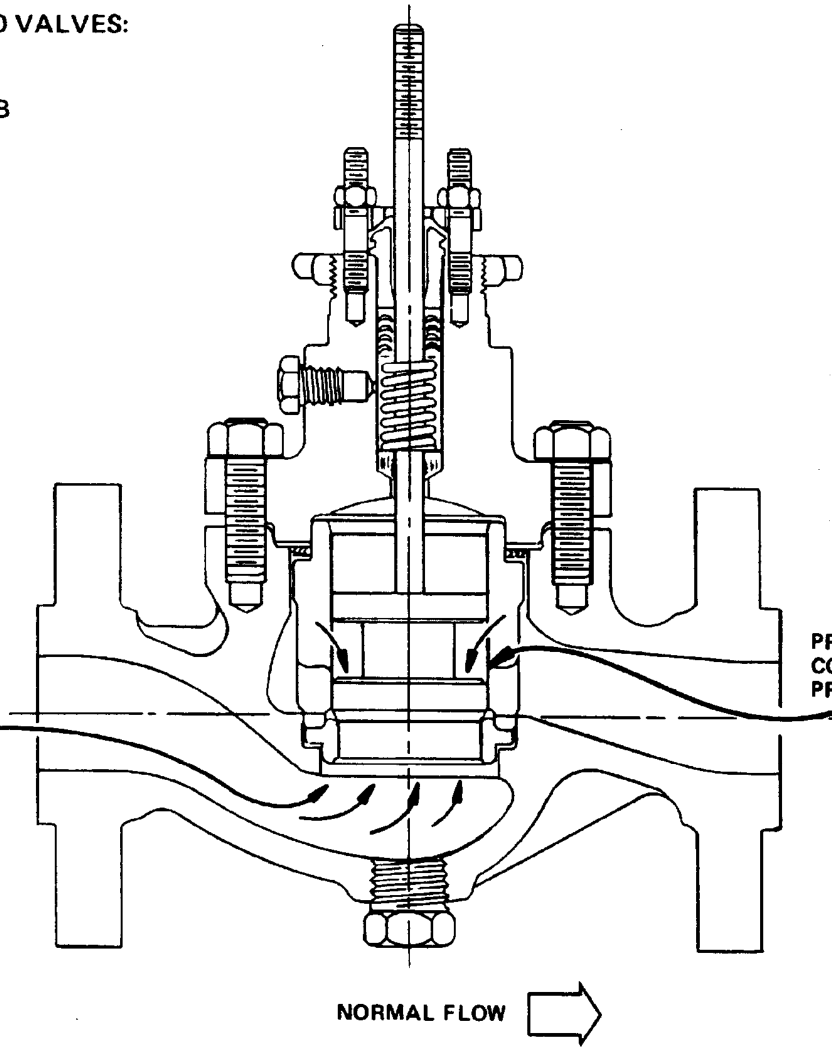
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

MAIN STEAM ISOLATION
GLOBE VALVE

FIGURE 3.8-11

APPLICABLE TO VALVES:
2T48-F114
2T48-F116
2T48-F118A, B
2T48-F322
2T48-F327



THE SUBJECT VALVES ARE OF THE UNBALANCED FLOW TO OPEN DESIGN; THEREFORE, WITH AN OBSERVED PRESSURE DROP IN THE REVERSE FLOW DIRECTION, AN ADDITIONAL SEATING LOAD WILL BE EXPERIENCED DUE TO THE HIGHER PRESSURE AT THE OUTLET OF THE VALVE BEING REGISTERED ON TOP OF THE VALVE PLUG, THUS SUPPLYING A FORCE IN THE DOWNWARD DIRECTION. THEREFORE, PRIMARY CONTAINMENT PRESSURE WILL TEND TO SEAT THE VALVE MORE TIGHTLY, WHEREAS TEST PRESSURE APPLIED ON THE SIDE OPPOSITE CONTAINMENT ACTS AGAINST THE SEATING FORCES.

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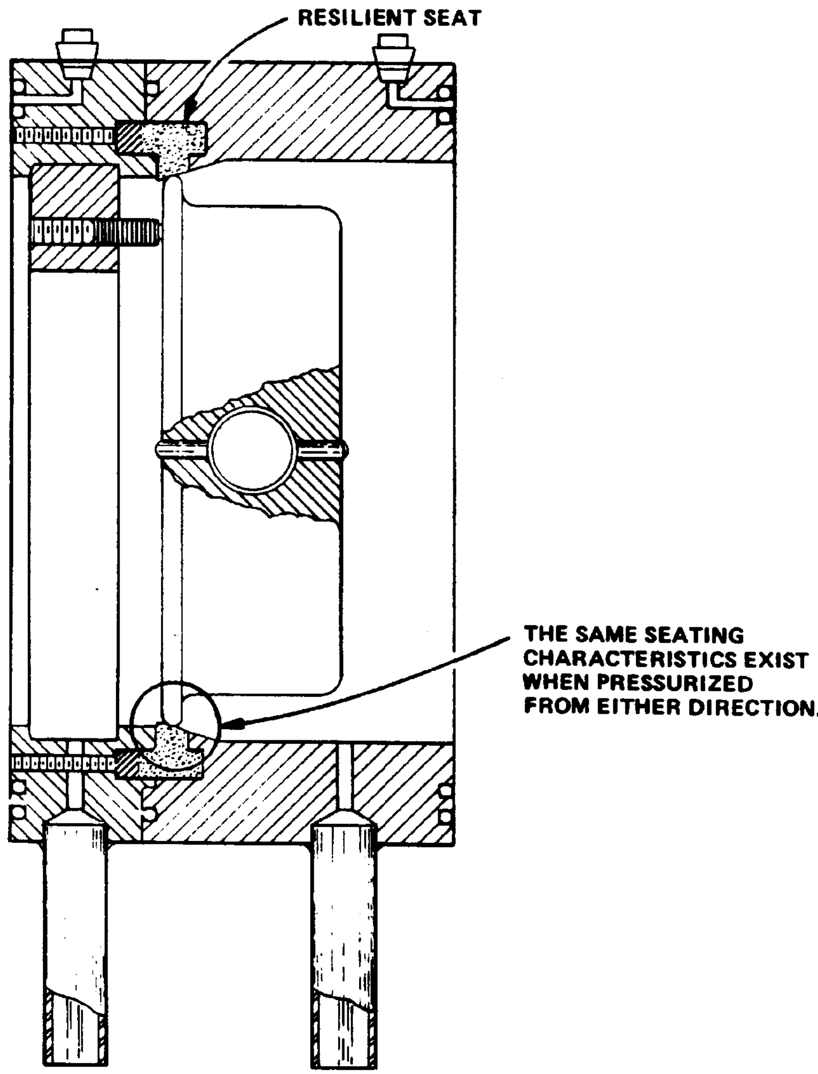
UNIT 2

CONTROL VALVE

FIGURE 3.8-12

APPLICABLE TO VALVES:
2T48-F307
2T48-F309
2T48-F310
2T48-F311
2T48-F318
2T48-F319



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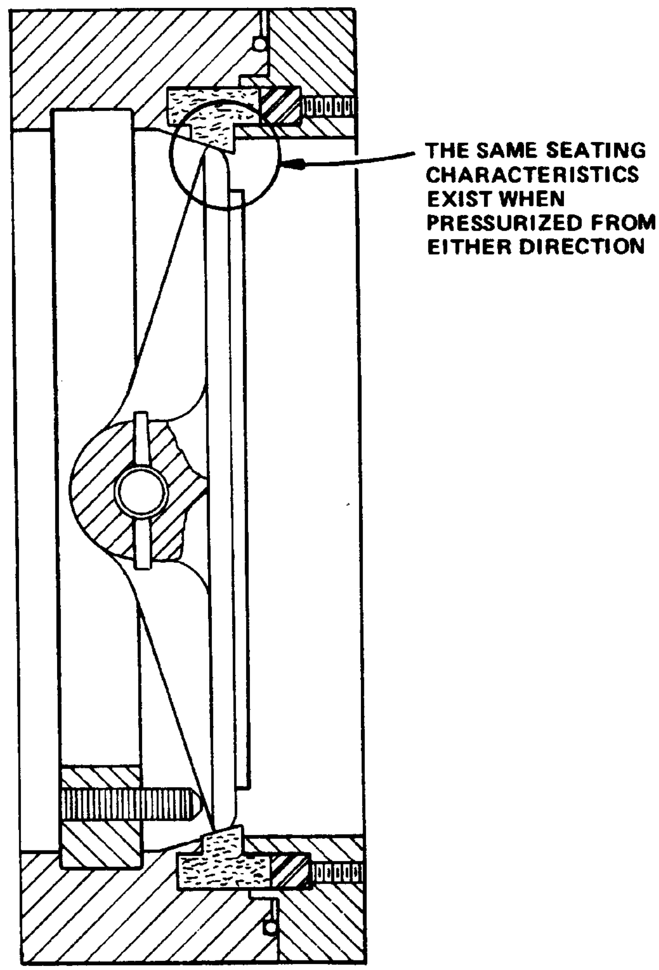
UNIT 2

BUTTERFLY VALVE

FIGURE 3.8-13

APPLICABLE TO VALVES: 2E11-F065, A, B, C, D



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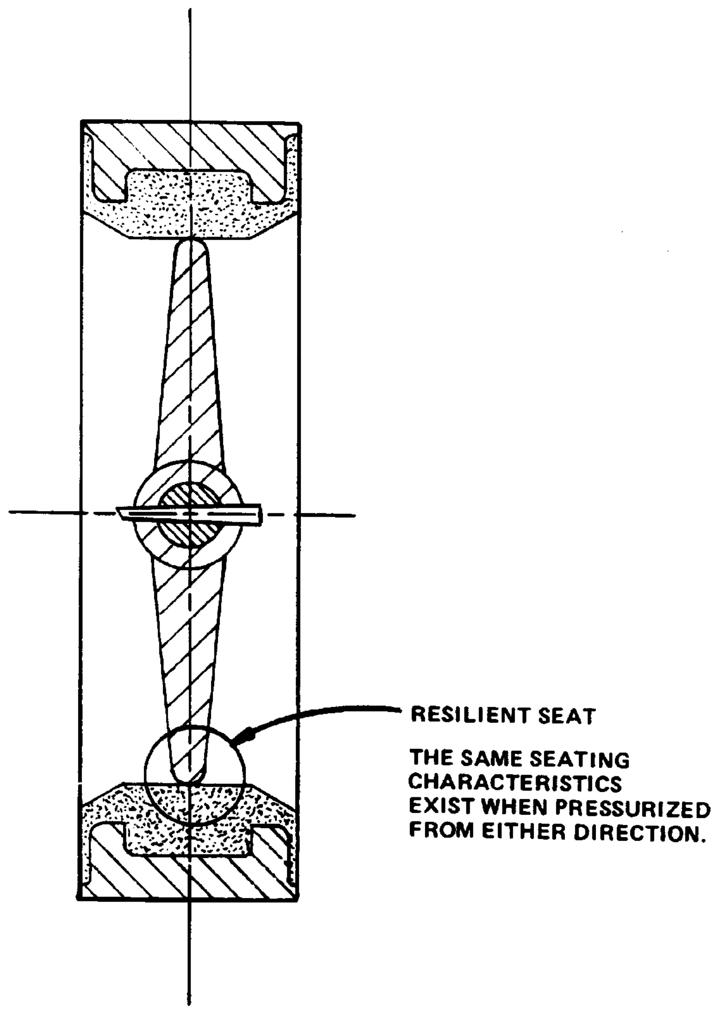
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

BUTTERFLY VALVE

FIGURE 3.8-14

APPLICABLE TO VALVES: 2E41-F051
2E51-F003
2E21-F019A, B



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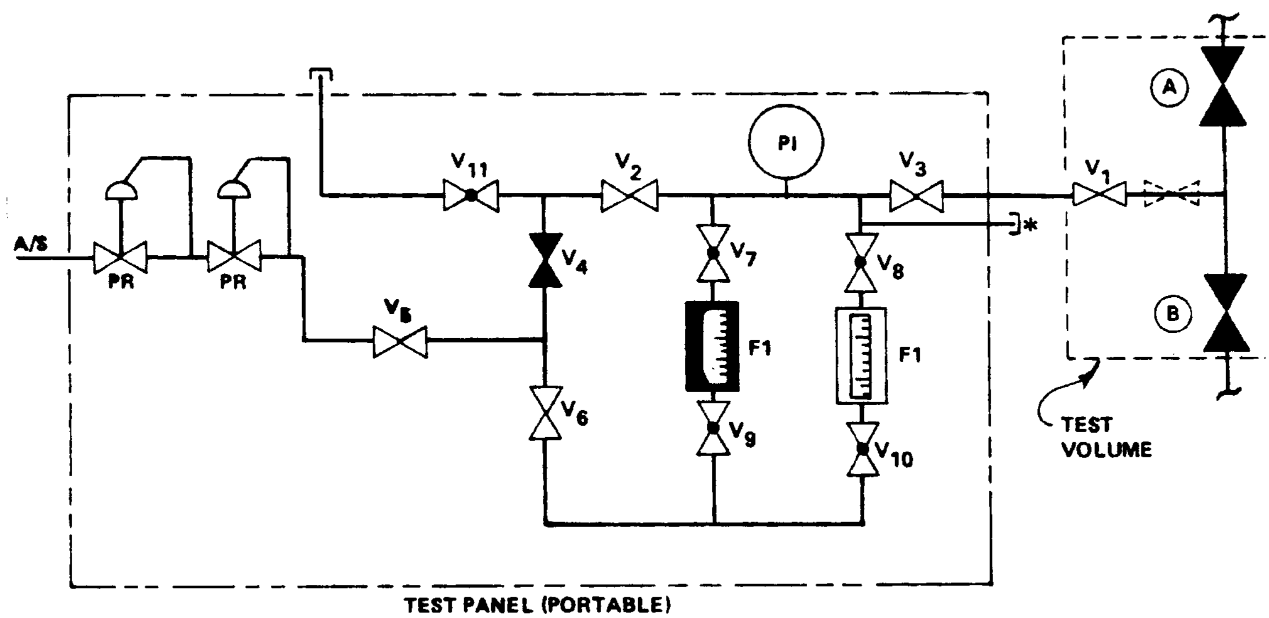
UNIT 2

BUTTERFLY VALVE

FIGURE 3.8-15



A/S – AIR OR NITROGEN SUPPLY
FI – DUAL SCALE FLOWMETER
PI – PRESSURE GAGE ABSOLUTE

ACAD 2030816

REV 19 7/01

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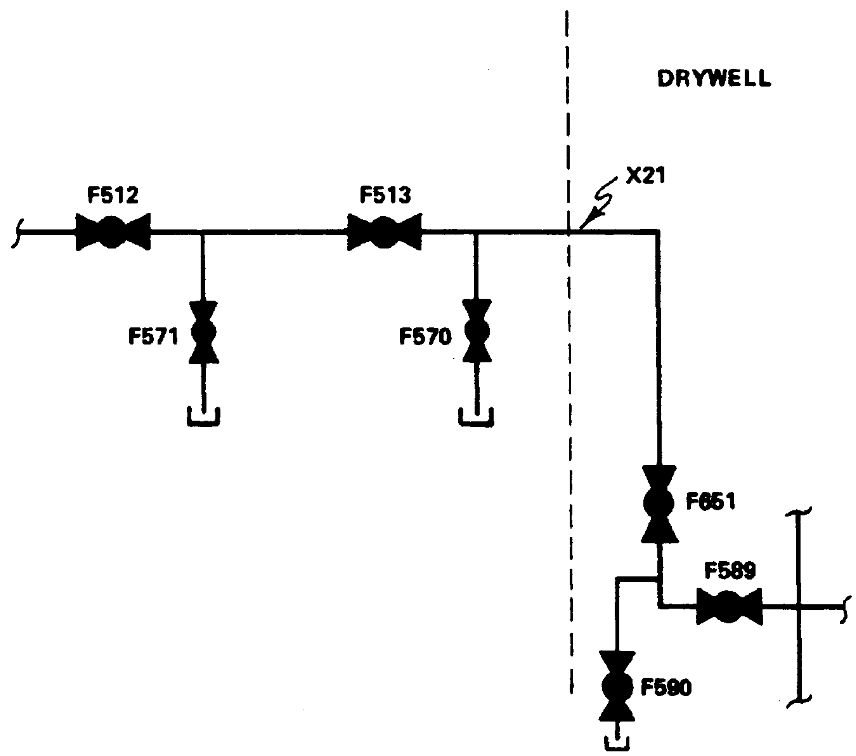
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TYPICAL TYPE C TEST ARRANGEMENT

FIGURE 3.8-16



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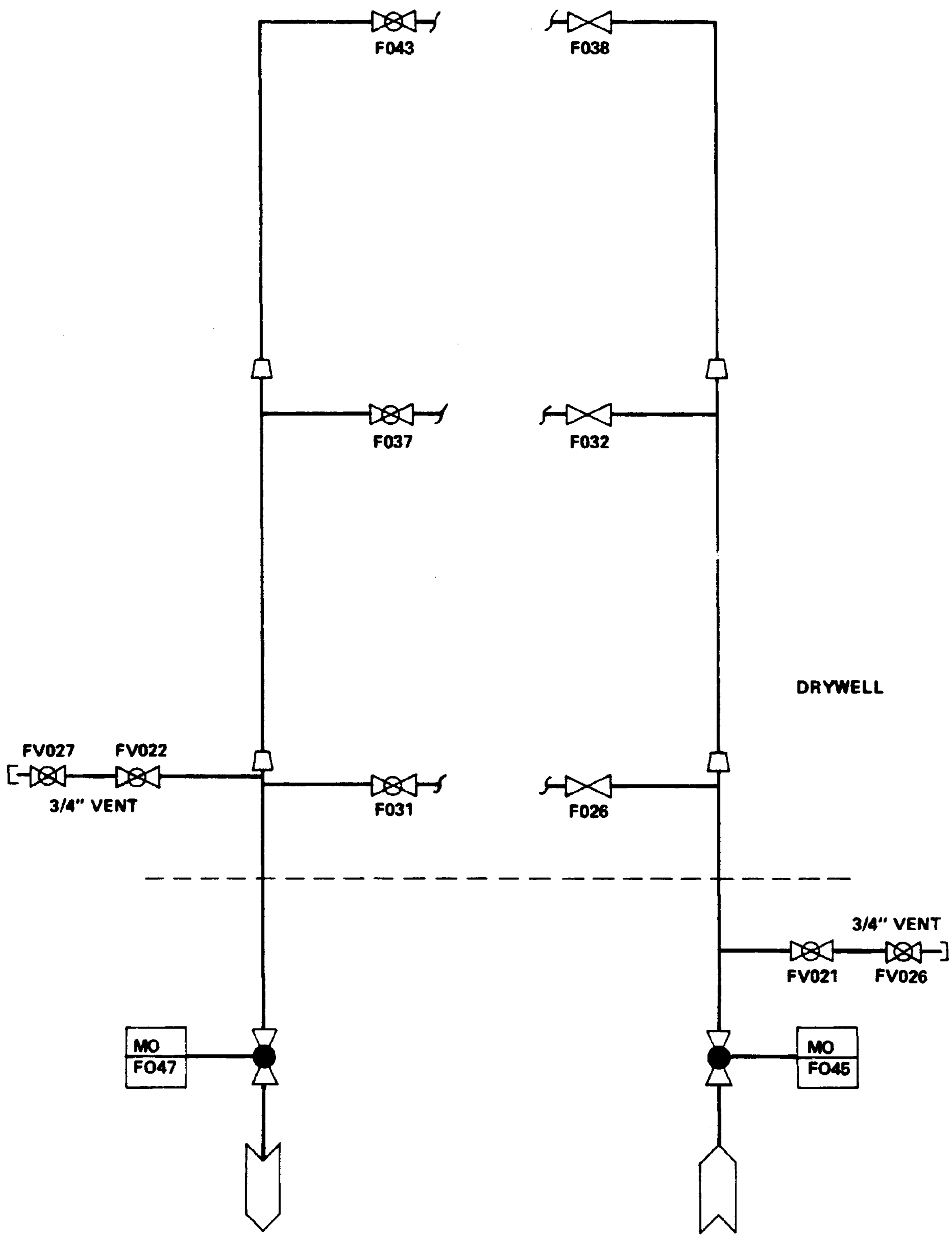
UNIT 2

2P51 SERVICE AIR SYSTEM

FIGURE 3.8-17



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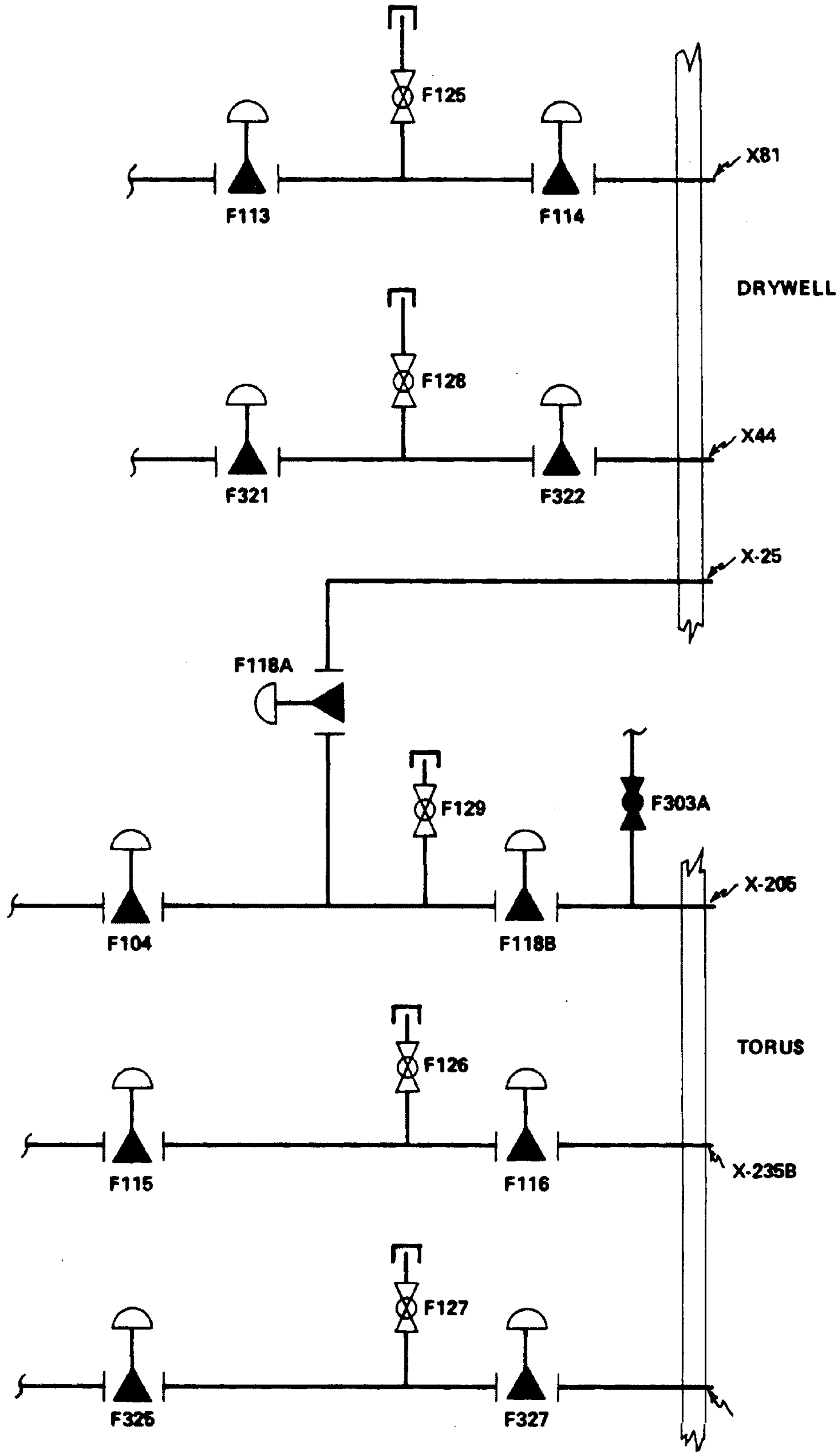
2P64 CHILLED WATER SYSTEM

FIGURE 3.8-18



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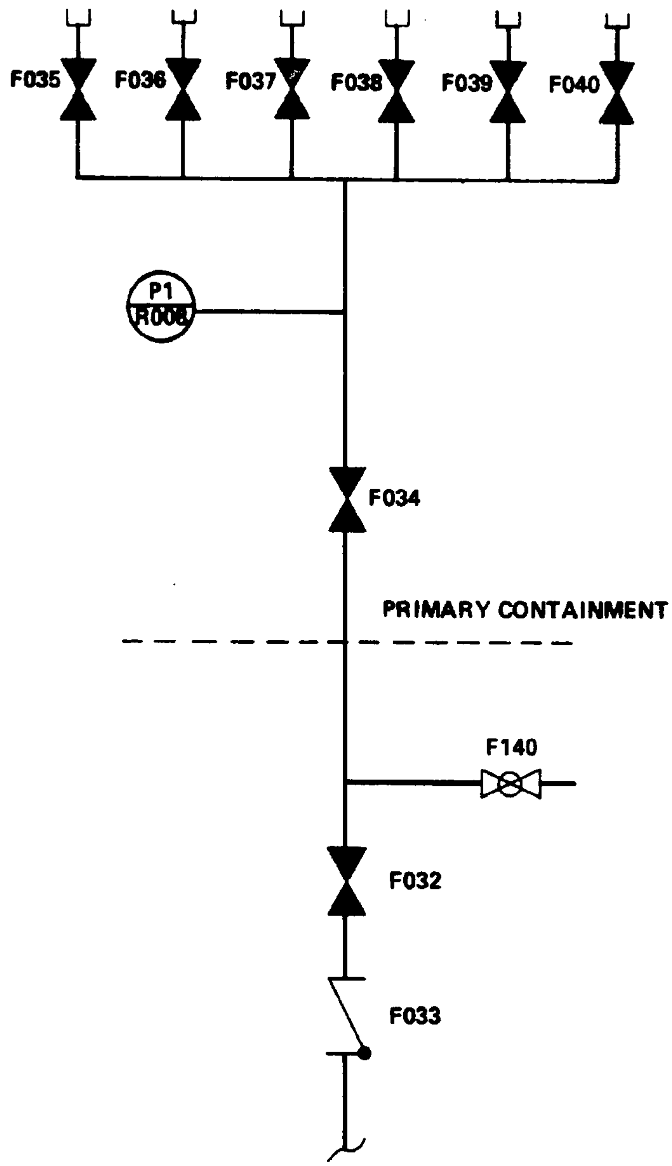
2T48 NITROGEN INERTING SYSTEM

FIGURE 3.8-19



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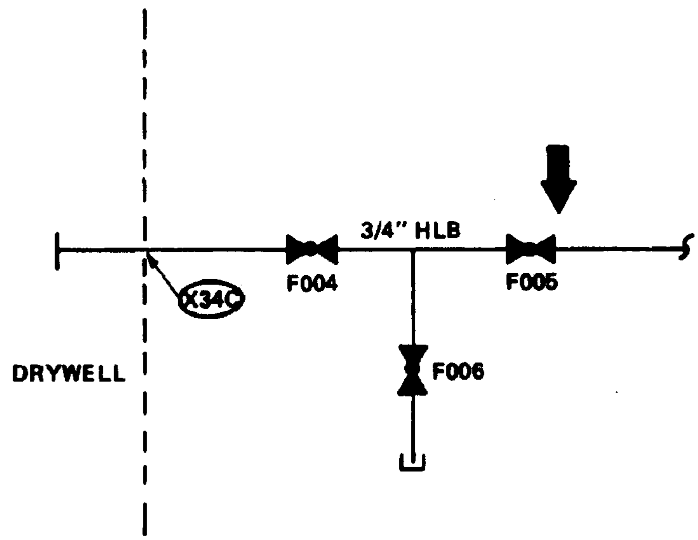
UNIT 2

2P21 DEMINERALIZED WATER SYSTEM

FIGURE 3.8-20



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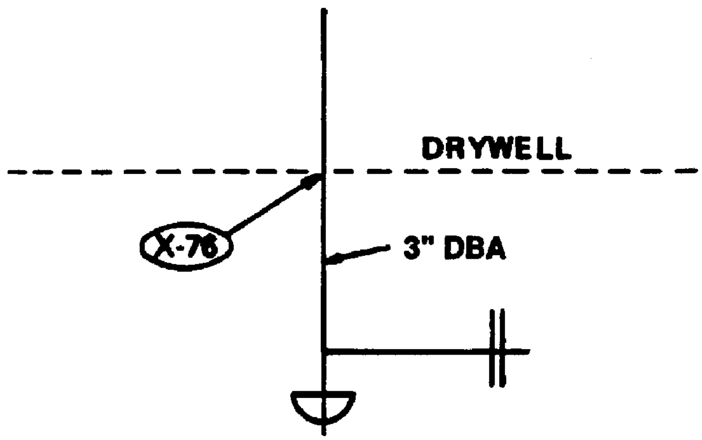
UNIT 2

ILRT CONNECTION

FIGURE 3.8-21



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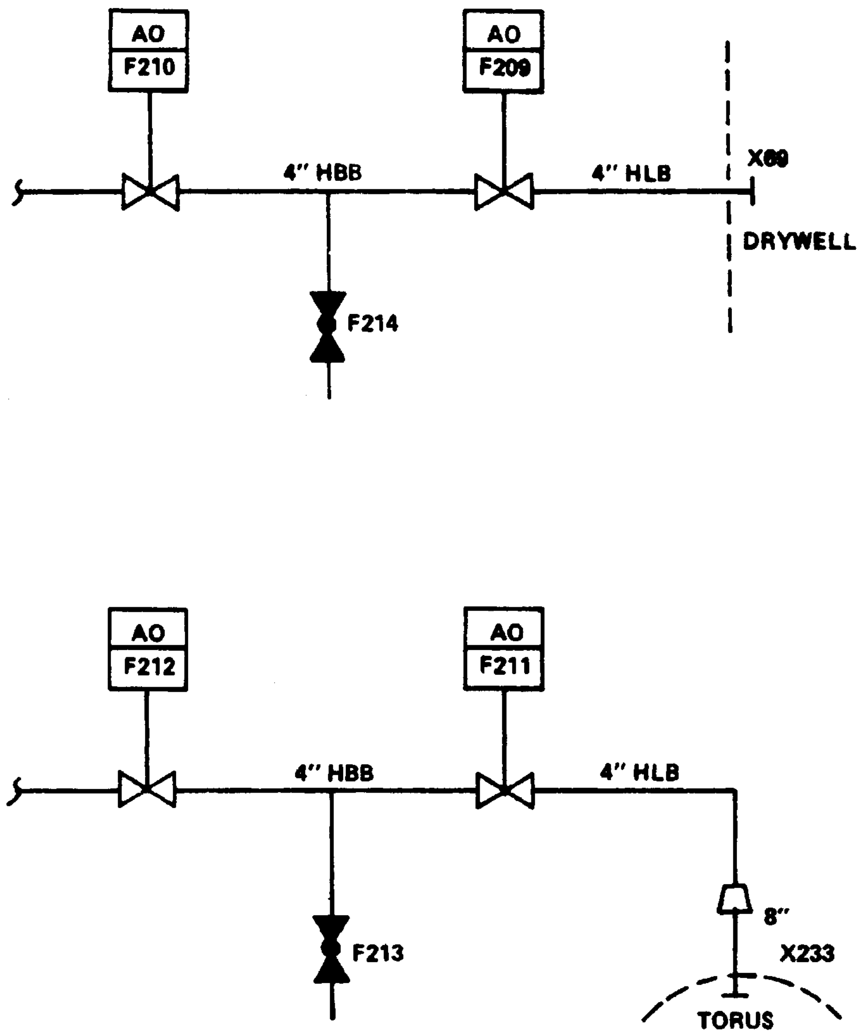
UNIT 2

FIRE PROTECTION

FIGURE 3.8-22



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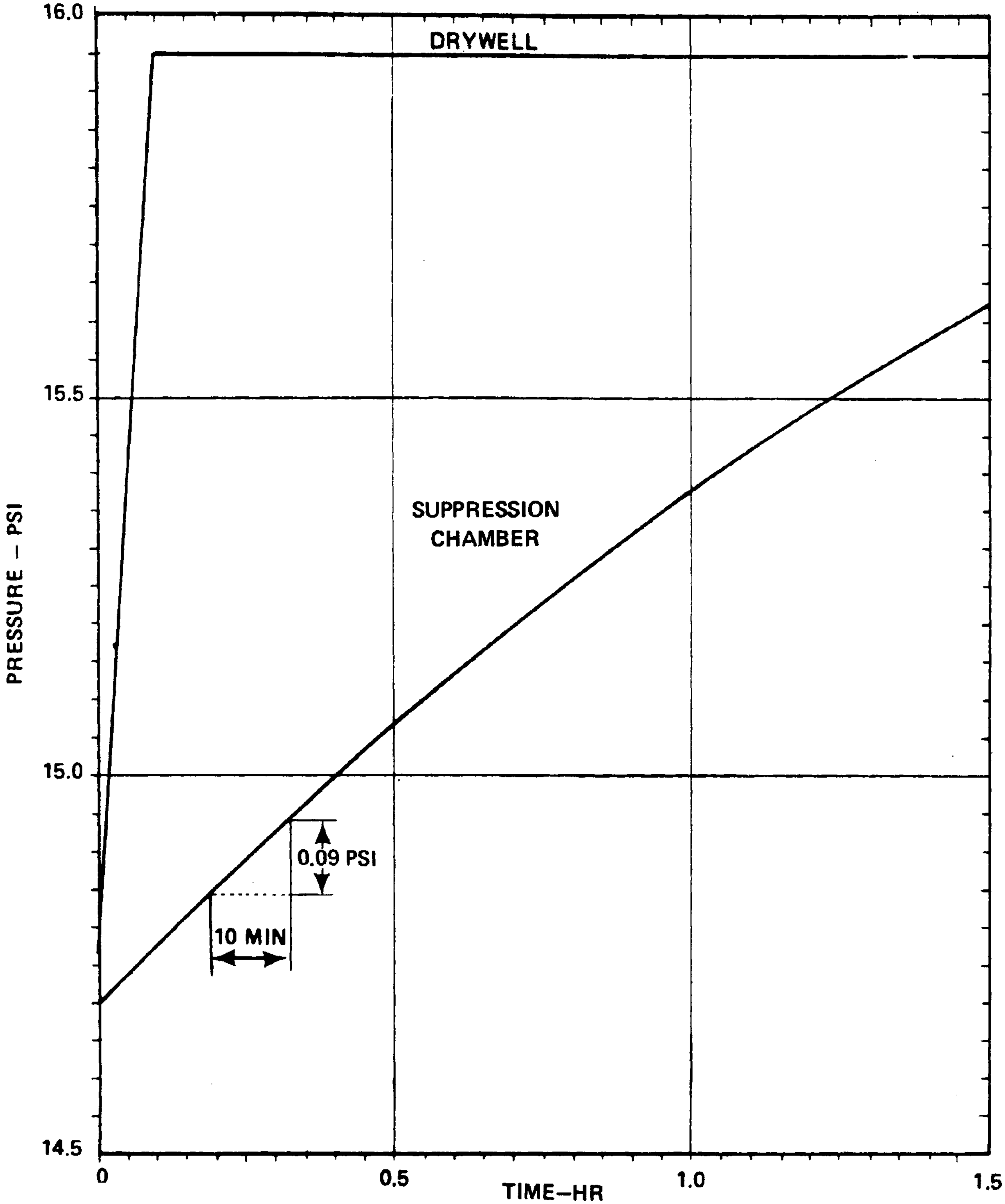
UNIT 2

TORUS TO DRYWELL
ΔP

FIGURE 3.8-23



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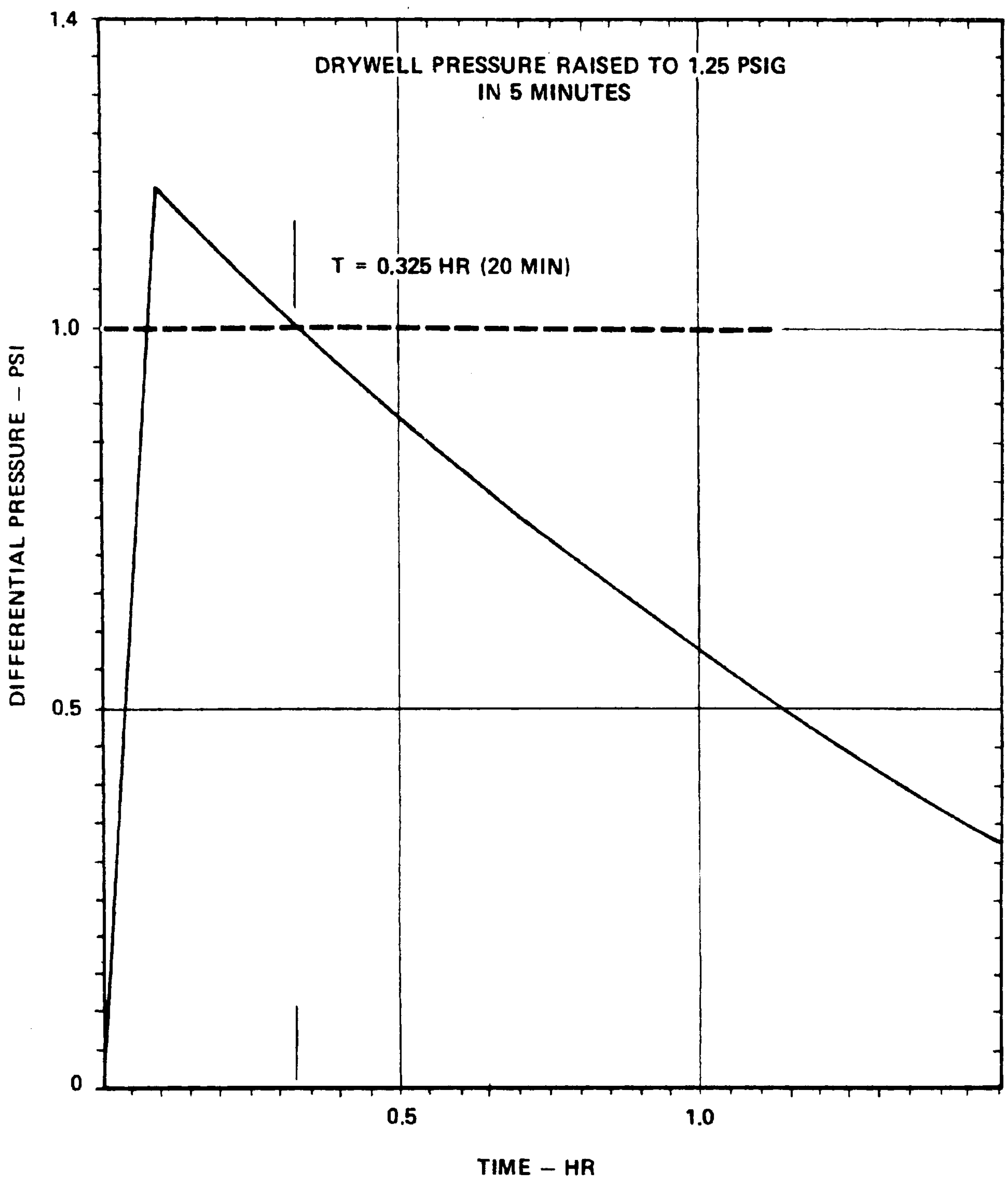
DRYWELL/SUPPRESSION CHAMBER LEAK TEST CONTAINMENT PRESSURE RESPONSE WITH LEAK EQUIVALENT TO 1-in. DIAMETER ORIFICE

FIGURE 3.8-24



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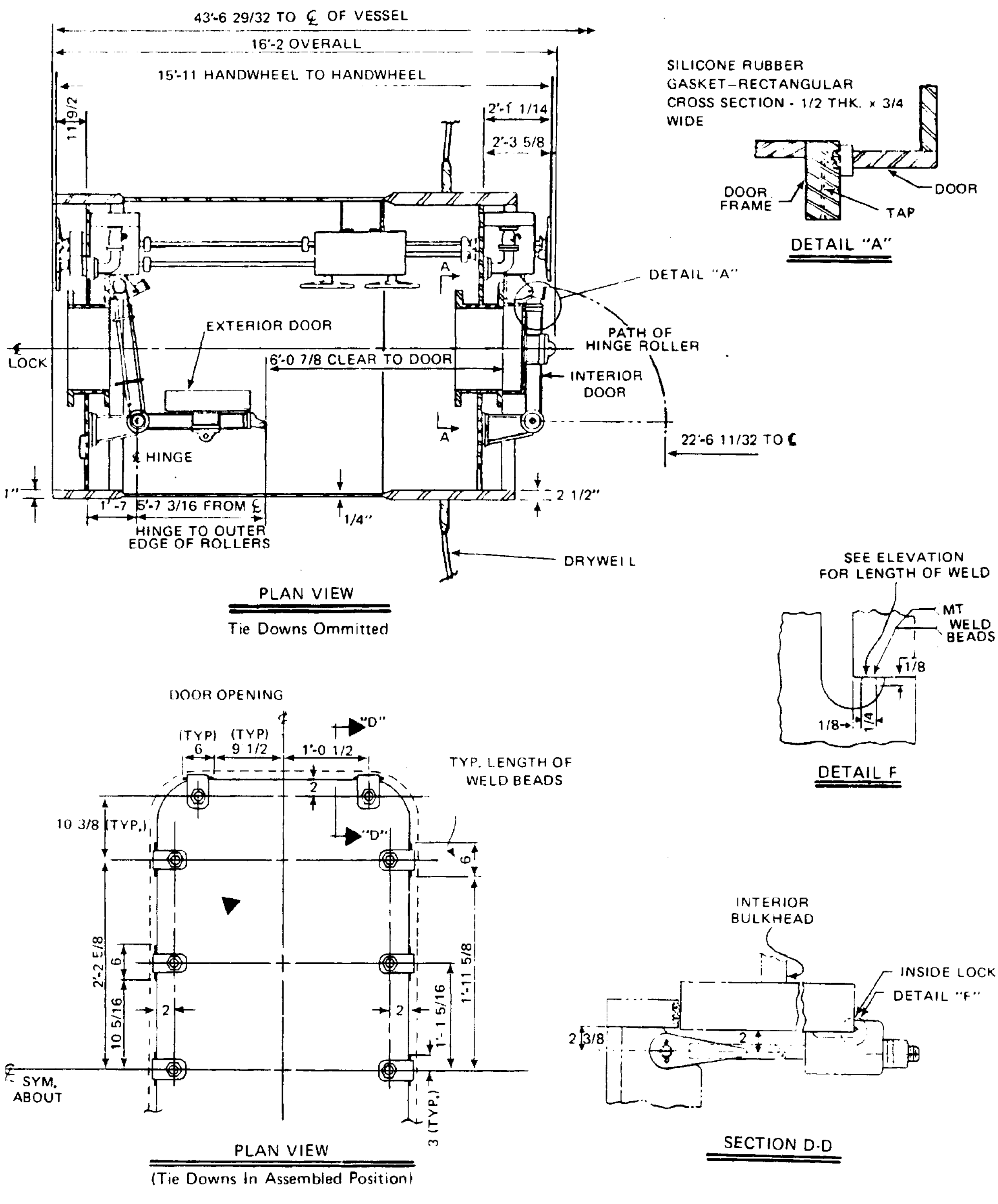
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

DRYWELL SUPPRESSION CHAMBER LEAK TEST CONTAINMENT DIFFERENTIAL RESPONSE WITH LEAKAGE EQUIVALENT TO 1-in. DIAMETER ORIFICE

FIGURE 3.8-25



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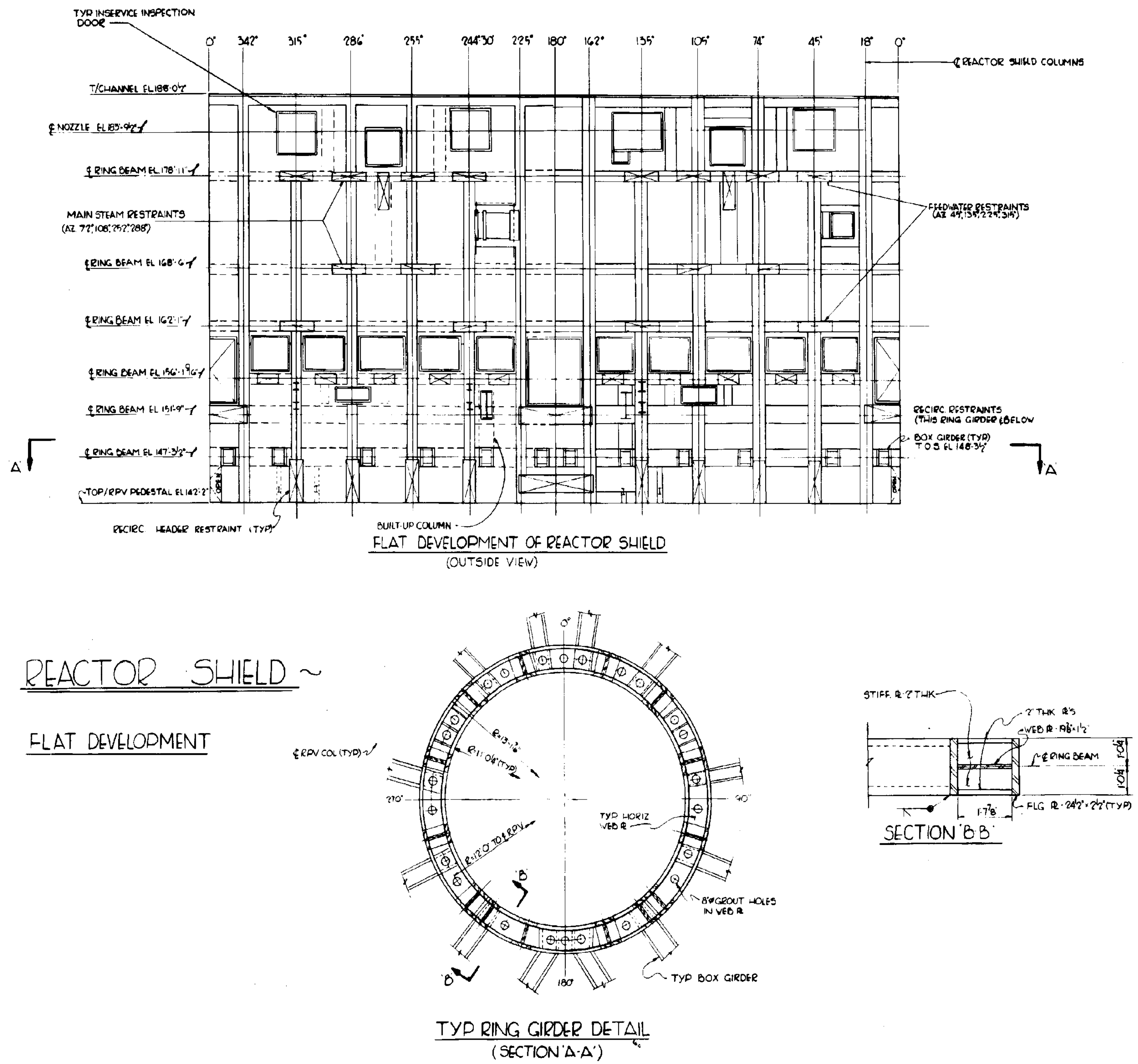
UNIT 2

DETAILS OF CONTAINMENT AIRLOCK

FIGURE 3.8-26



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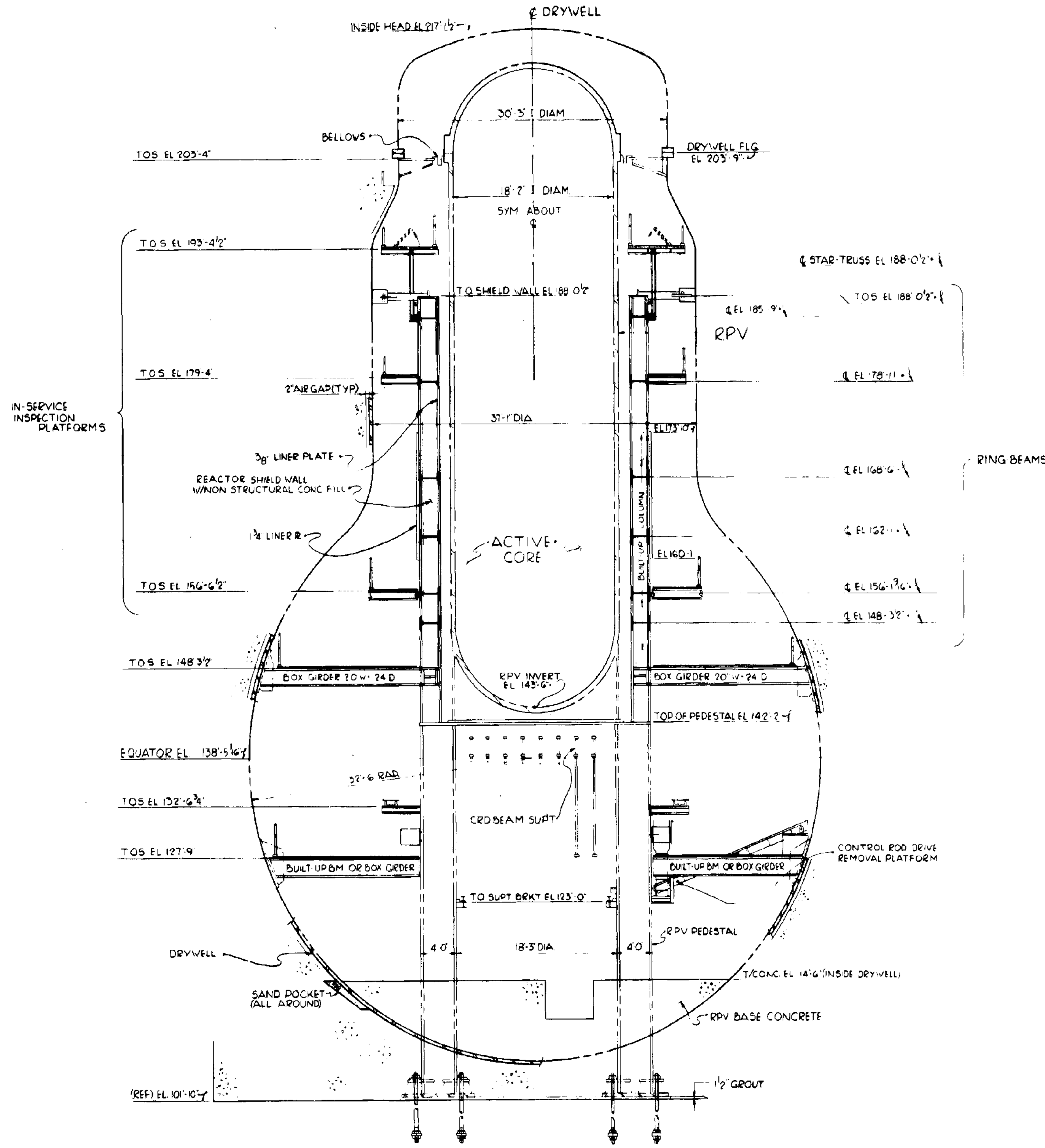
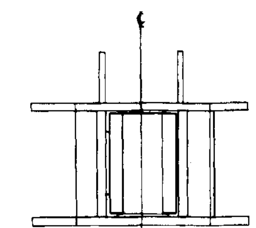
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

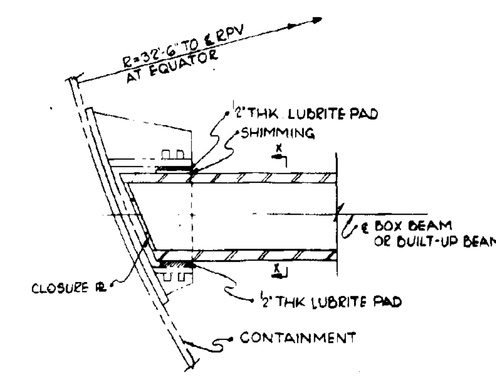
FLAT DEVELOPMENT OF REACTOR SHIELD

FIGURE 3.8-27





VIEW X
(TAKEN FROM BEAM SEAT DETAIL)



TYP BEAM SEAT DETAIL

ACAD 2030828

DRYWELL INTERNAL STRUCTURES

ACAD 2030828

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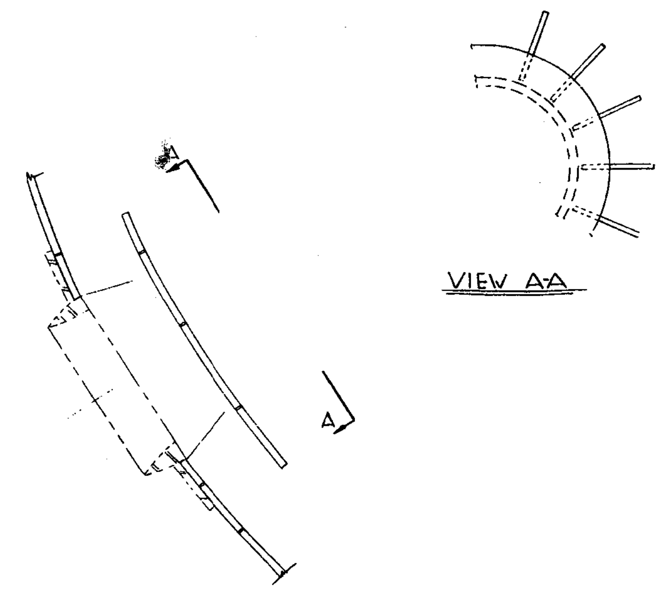
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

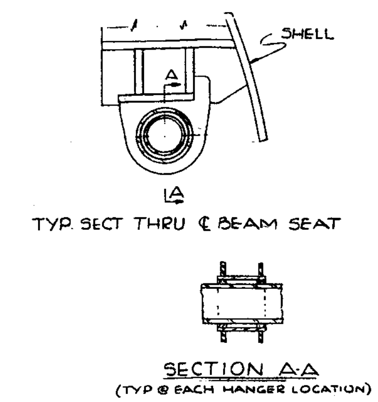
DRYWELL INTERNAL STRUCTURES

FIGURE 3.8-28



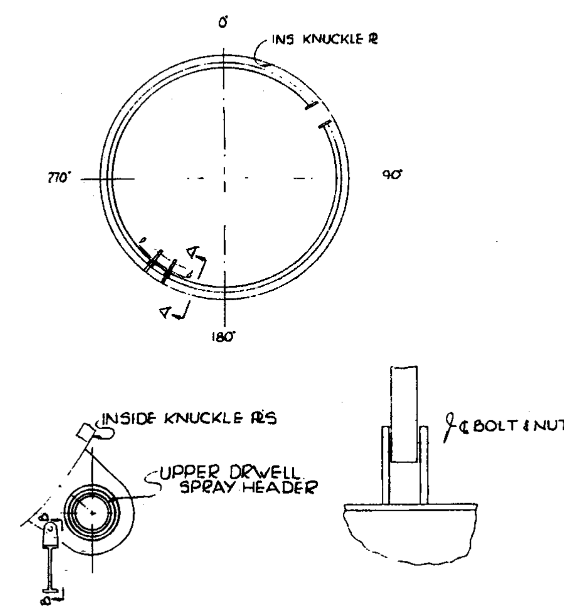
VIEW A-A

DRYWELL JET DEFLECTION



LOWER SPRAY HEADER SUPPORT

MISCELLANEOUS INTERNAL ~ STRUCTURAL COMPONENTS



SECTION A-A

VIEW BB

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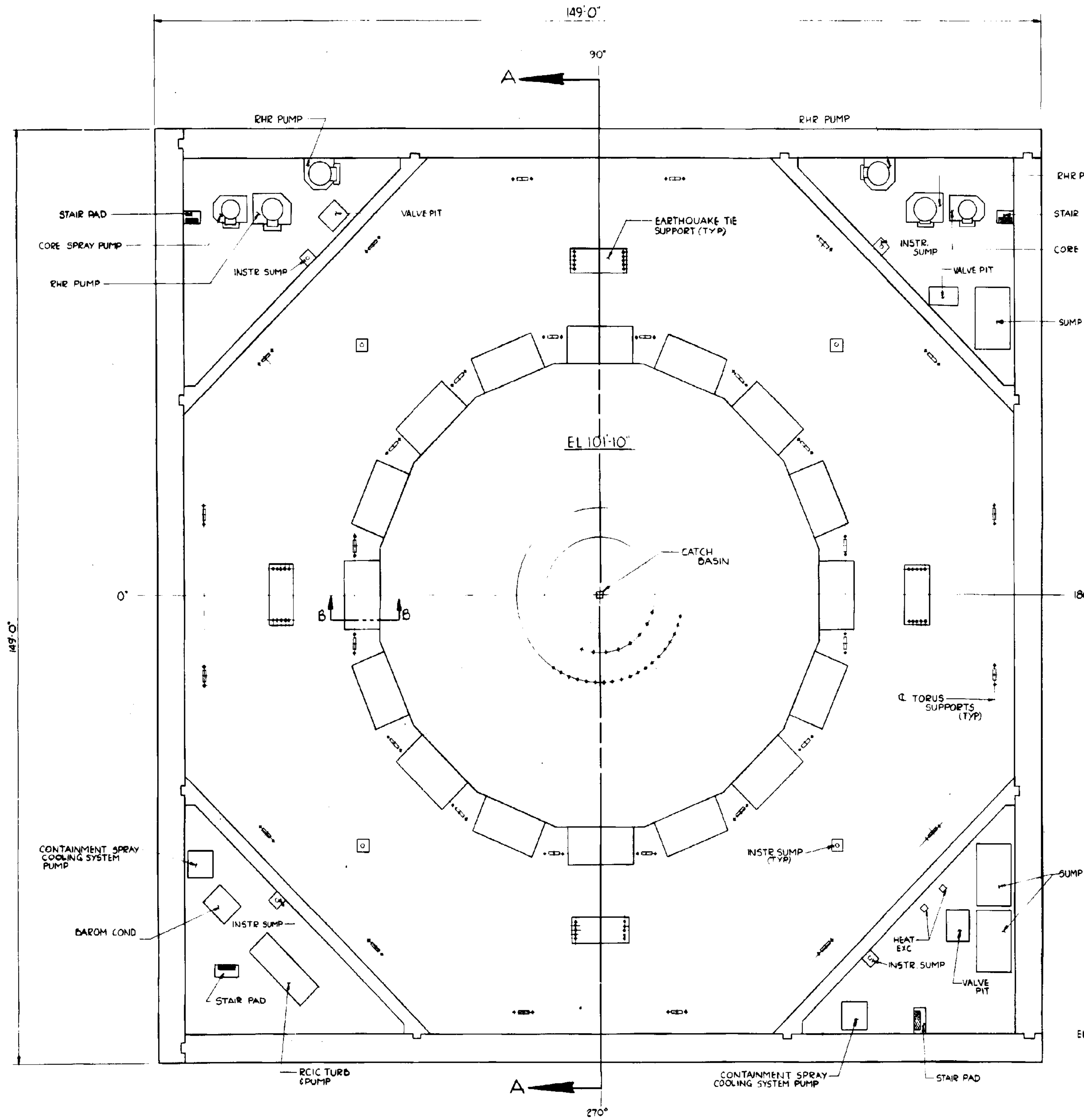
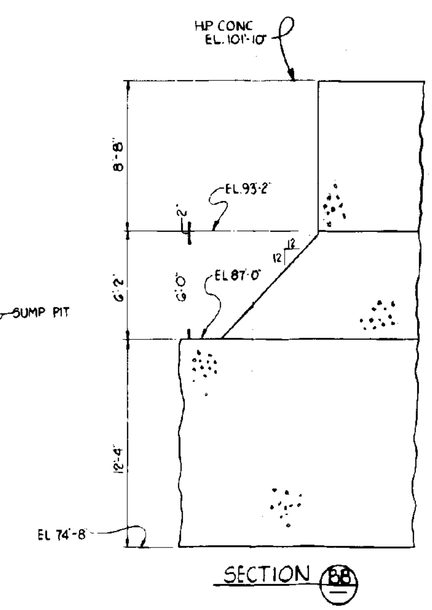
UNIT 2

MISCELLANEOUS INTERNAL STRUCTURAL COMPONENTS

FIGURE 3.8-29



REACTOR BUILDING FOUNDATION PLAN



SECTION B-B

ACAD 2030830

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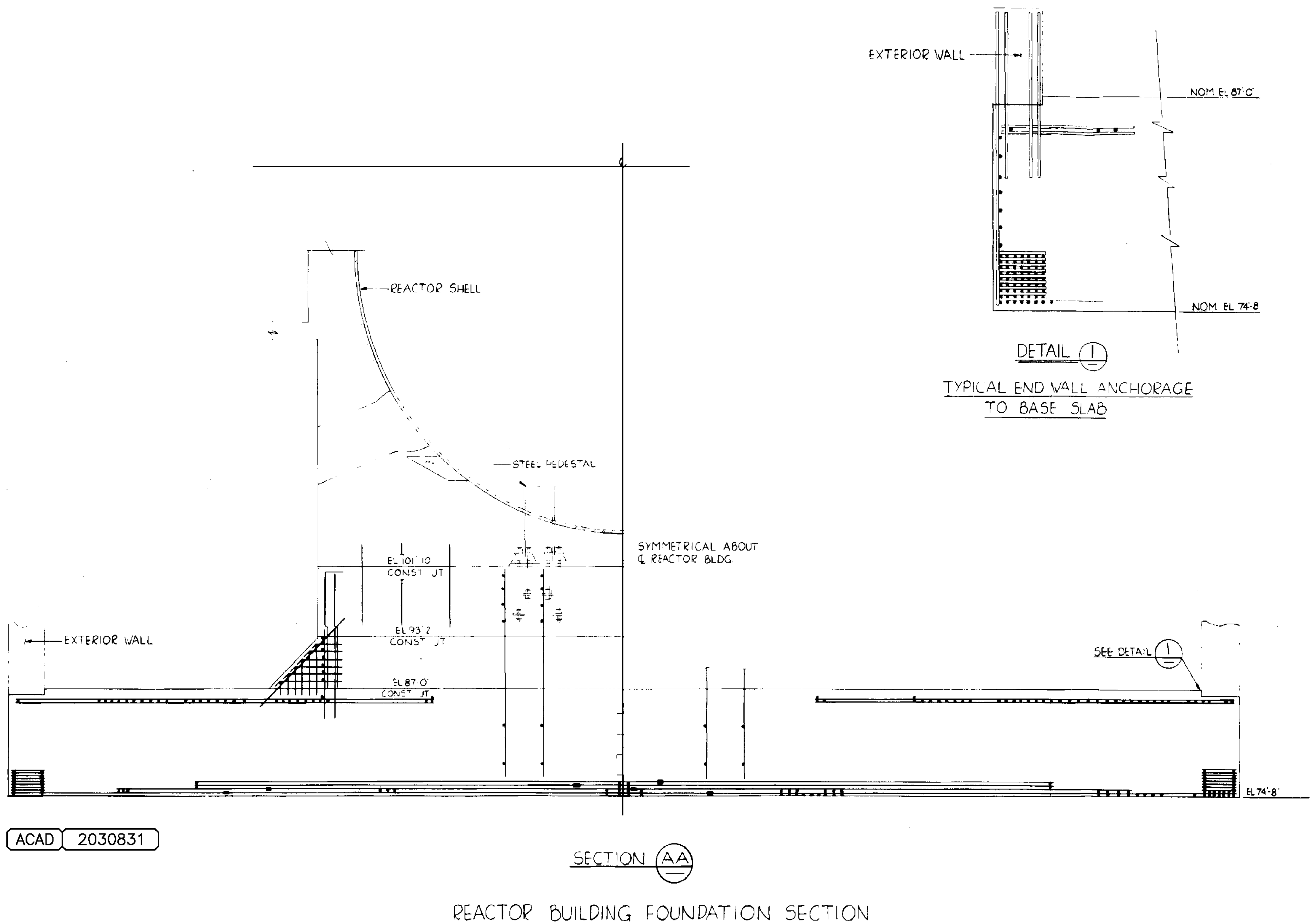
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

REACTOR BUILDING FOUNDATION PLAN AND SECTION

FIGURE 3.8-30



ACAD 2030831

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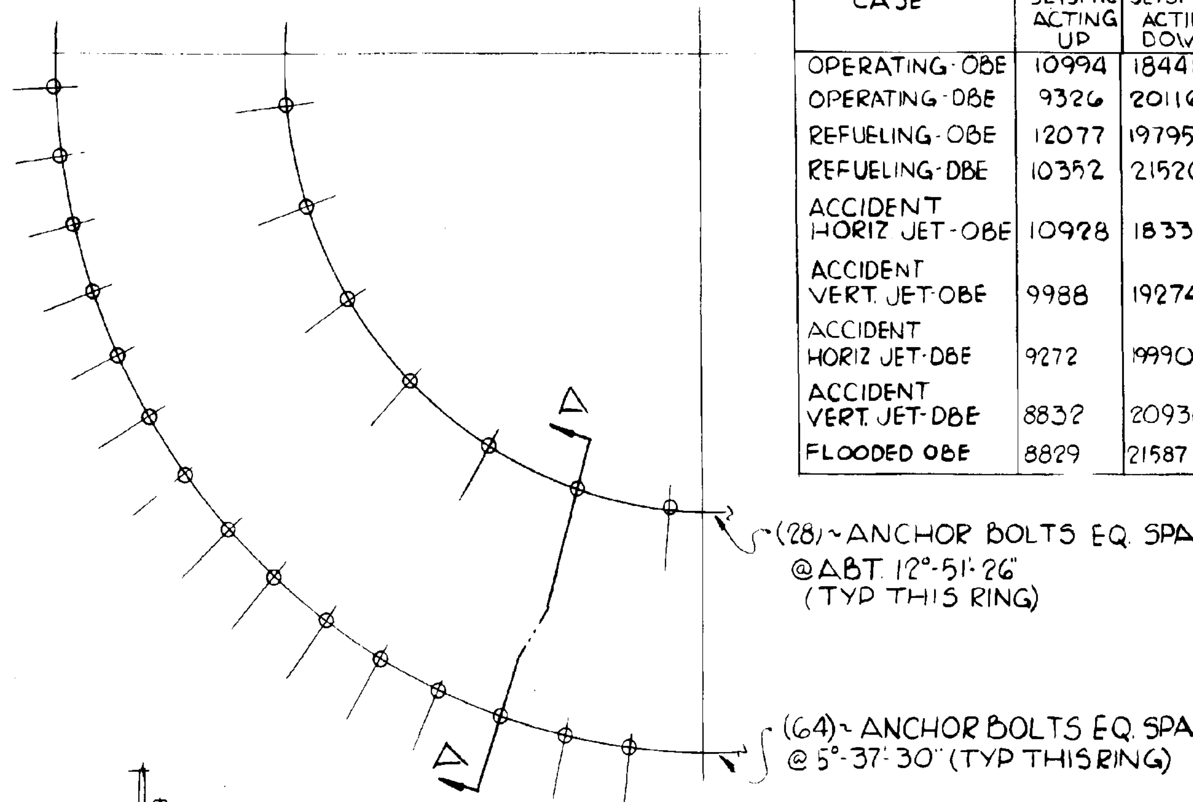
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

SECTION OF REACTOR BUILDING FOUNDATION

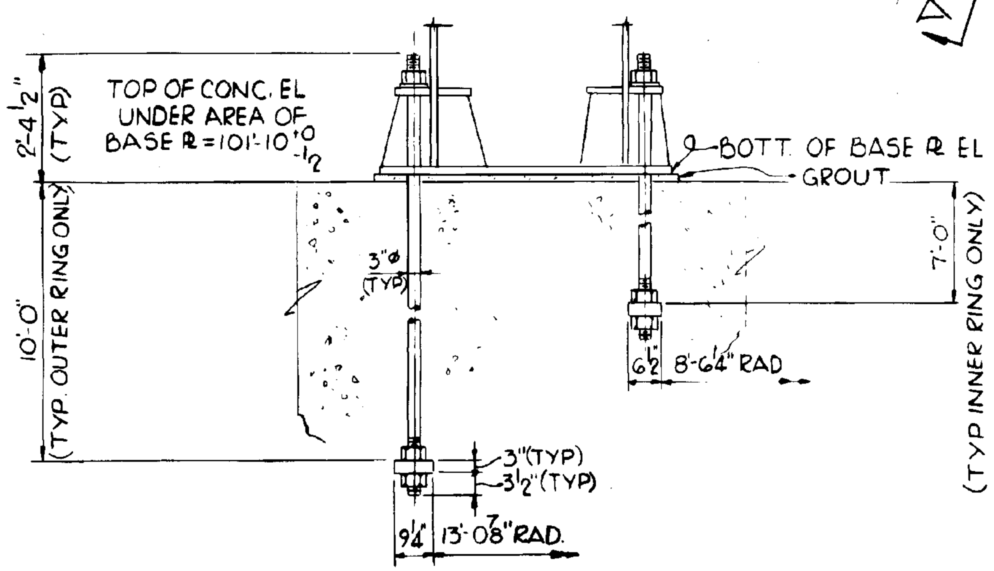
FIGURE 3.8-31

SUMMARY OF LOADS AT BASE OF PEDESTAL EL 101'-11½"

LOADING CASE	VERTICAL LOAD: WITH SEISMIC ACTING UP	VERTICAL LOAD: WITH SEISMIC ACTING DOWN	OVER-TURNING MOMENT (FT-K)	HORIZ. SHEAR (K)	MAX BOLT LOAD (K): OUTS RING	MAX BOLT LOAD (K): INS. RING	MAX. CONC BEARING (PSI): OUTS RING	MAX. CONC BEARING (PSI): INS RING
OPERATING-OBE	10994	18448	104371	1949	79	56	861	881
OPERATING-DBE	9326	20116	136245	2615	150	147	1034	1043
REFUELING-OBE	12077	19795	104371	1949	69	40	892	917
REFUELING-DBE	10352	21520	136245	2615	140	132	1066	1081
ACCIDENT HORIZ JET-OBE	10928	18334	137628	2889	137	126	1000	1000
ACCIDENT VERT. JET-OBE	9988	19274	104371	1949	88	71	1880	903
ACCIDENT HORIZ JET-DBE	9272	19990	168962	3555	206	216	1171	1160
ACCIDENT VERT. JET-DBE	8832	20930	136245	2615	159	162	1053	1065
FLOODED OBE	8829	21587	135278	3026	152	152	1064	1079



PLAN VIEW



SECTION A-A

PRE-TENSIONING VALUES		
	STRAIN	TORQUE
OUTS. BOLTS	0.0747 IN	2789 FT LB
INS. BOLTS	0.0412 IN.	2053 FT. LB.

PEDESTAL ANCHOR BOLTS

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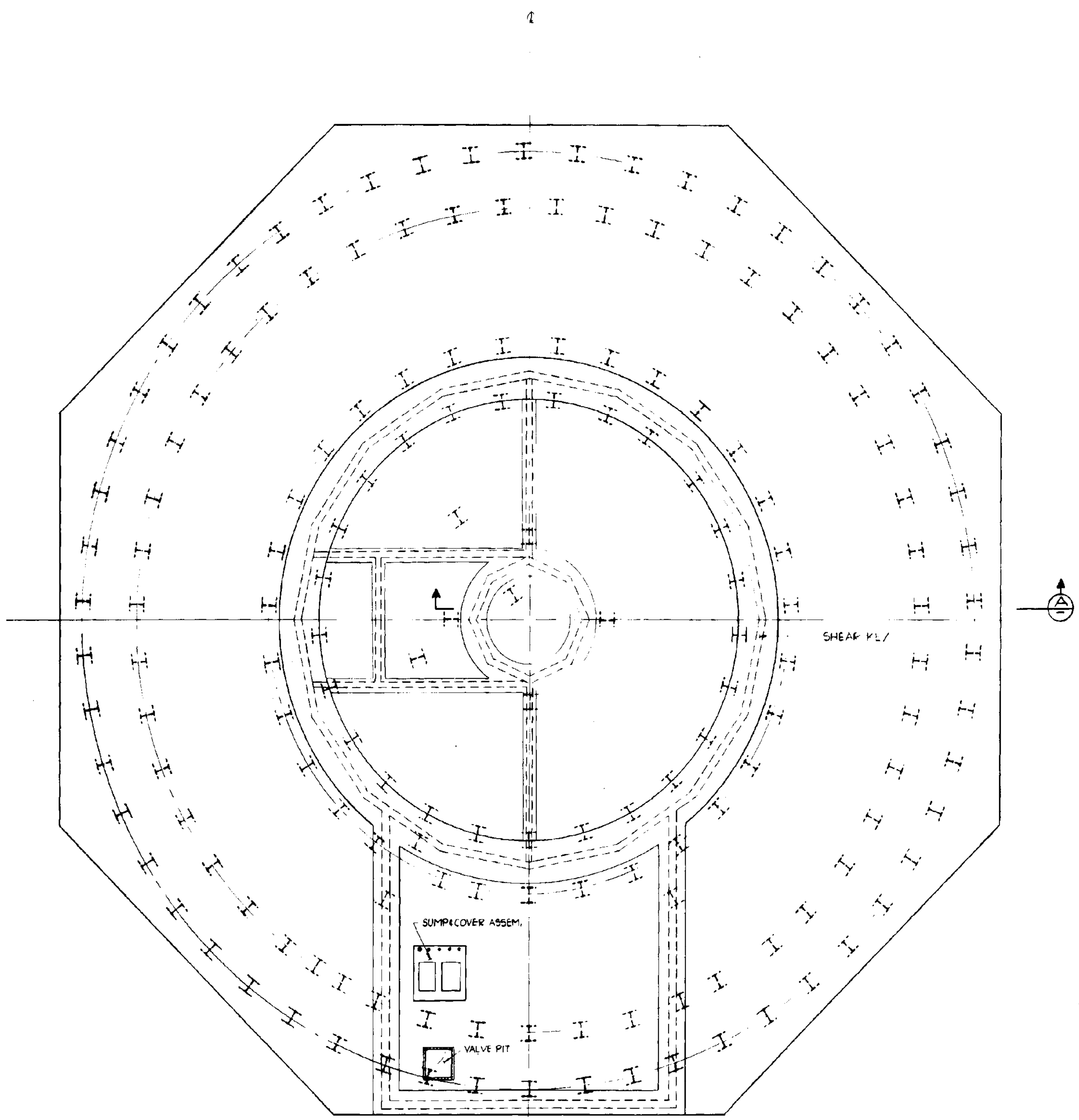
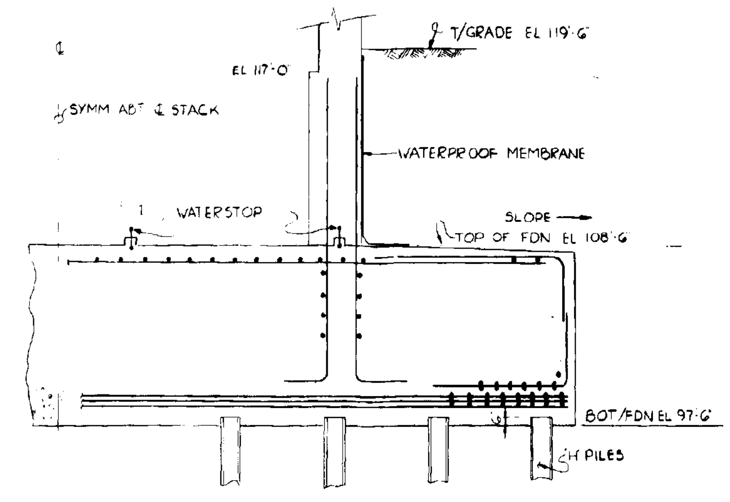
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

RPV PEDESTAL FOUNDATION
ANCHOR BOLTS

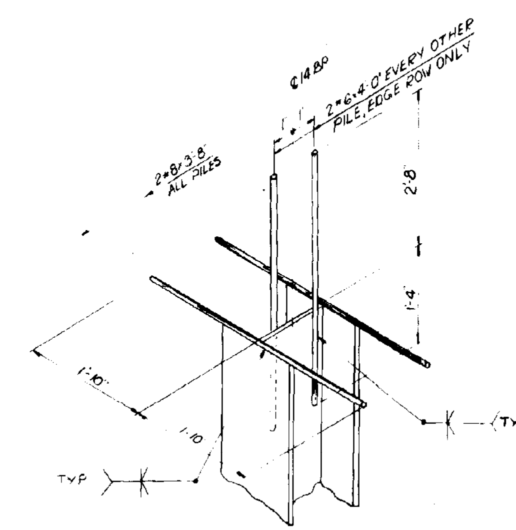
FIGURE 3.8-32



MAIN STACK FOUNDATION PLAN



SECTION (A)



PILE ANCHORAGE DETAIL

ACAD 2030833

REV 19 7/01

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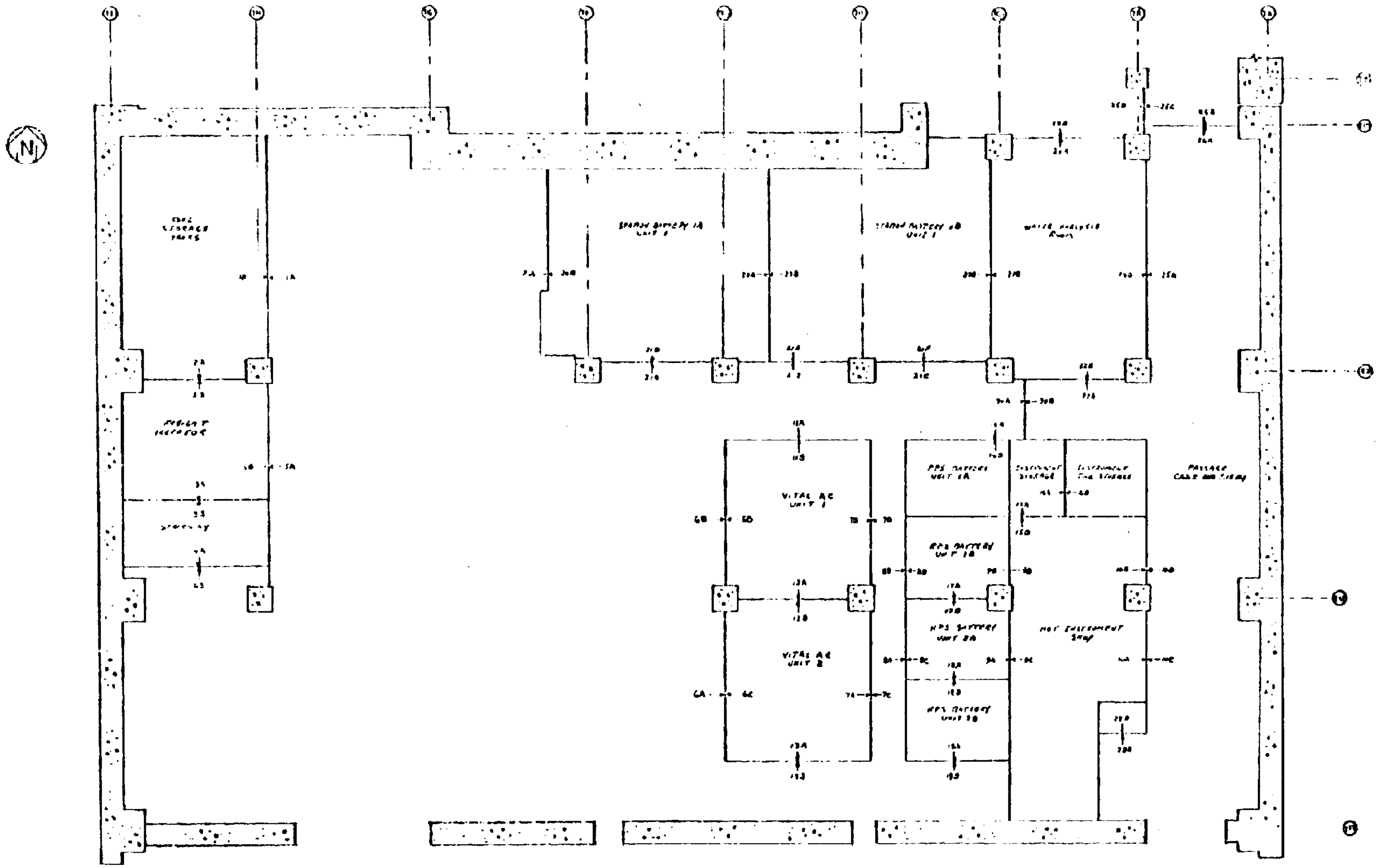
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

MAIN STACK FOUNDATION
PLAN AND SECTIONS

FIGURE 3.8-33



ACAD 2030834

REV 19 7/01

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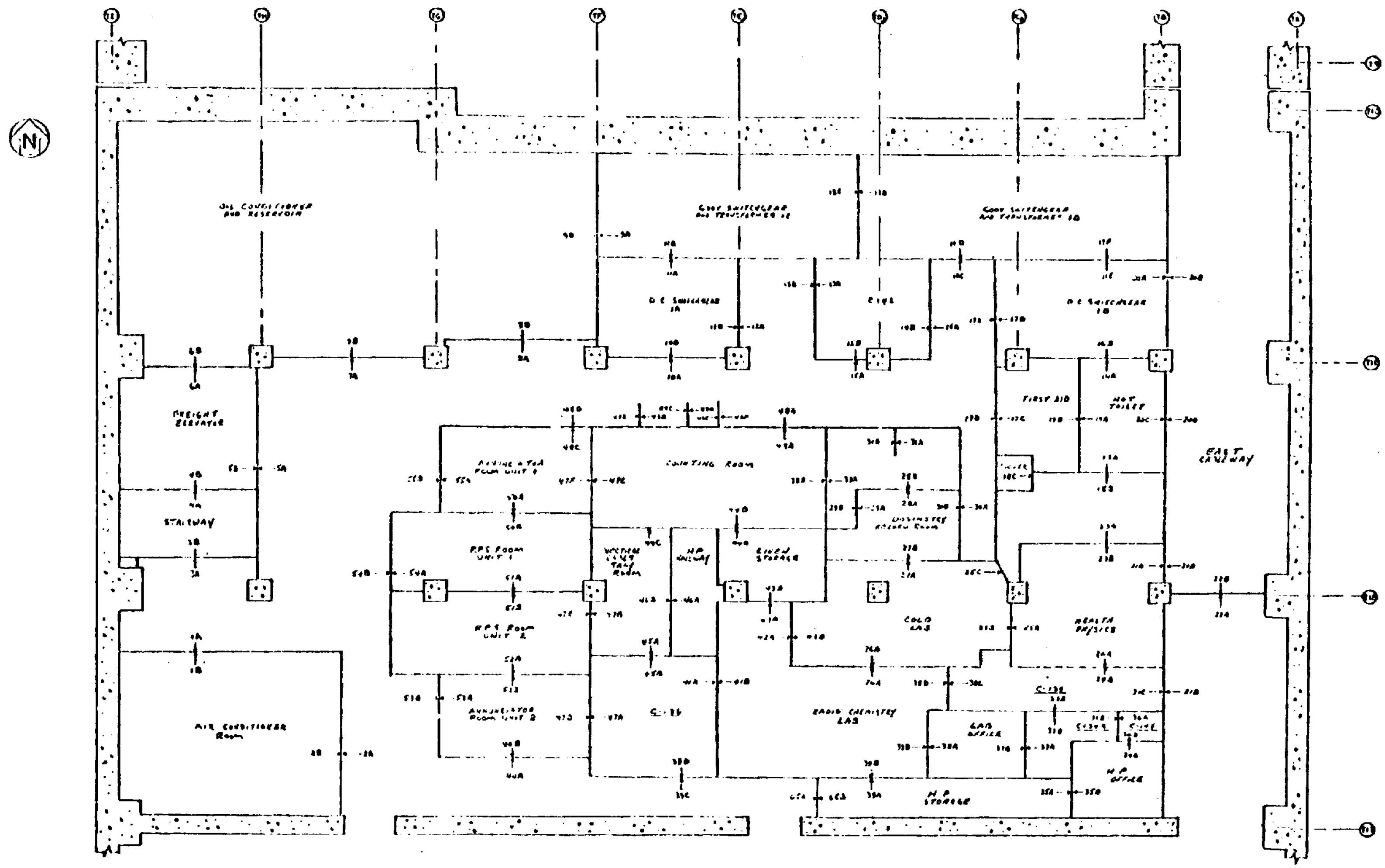
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

MASONRY WALL SINGLE-LINE
SKETCHES – CONTROL BUILDING

FIGURE 3.8-34



ACAD 2030835

REV 19 7/01

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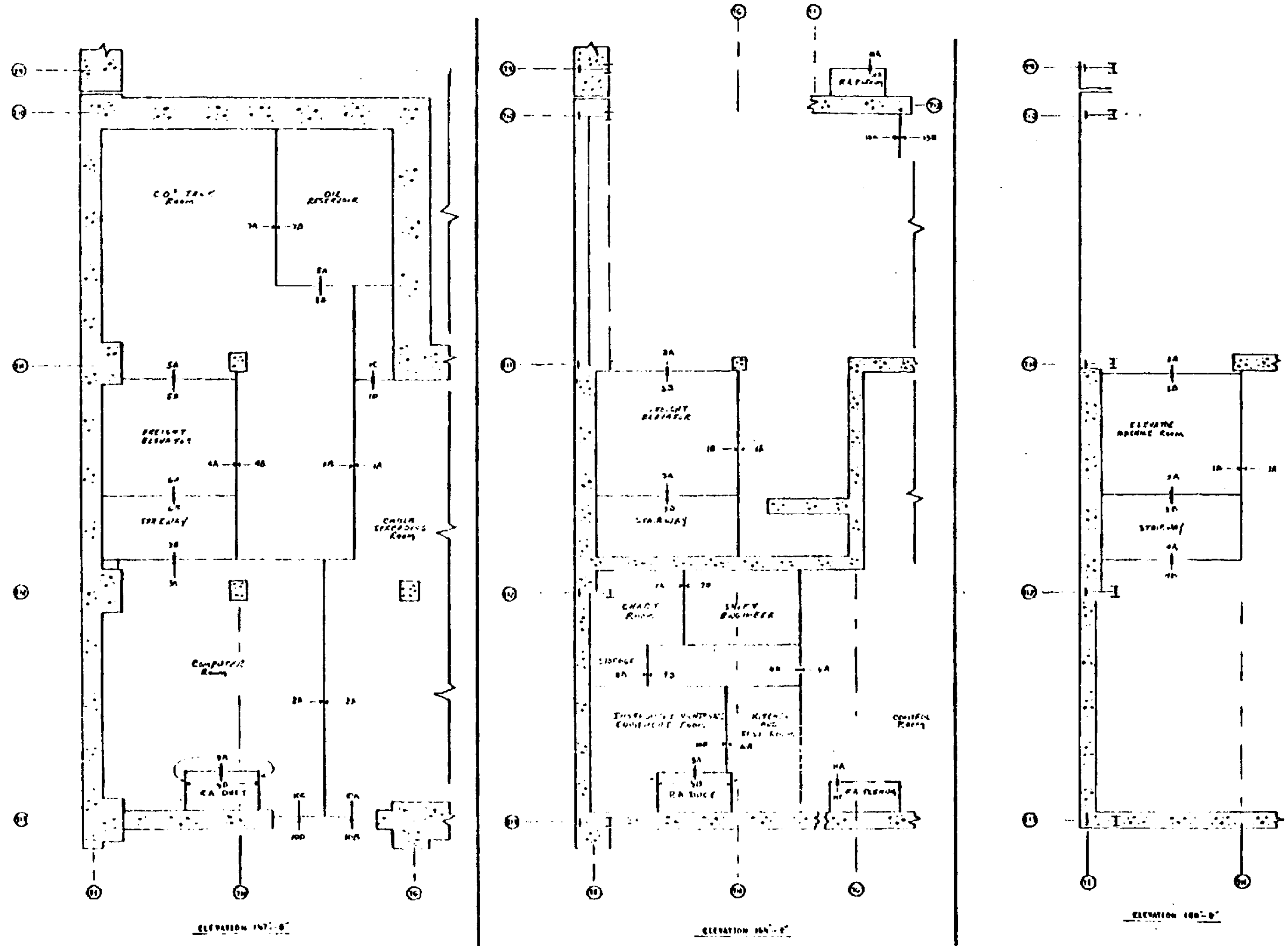
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

MASONRY WALL SINGLE-LINE SKETCHES – CONTROL BUILDING

FIGURE 3.8-35



ACAD 2030836

REV 19 7/01

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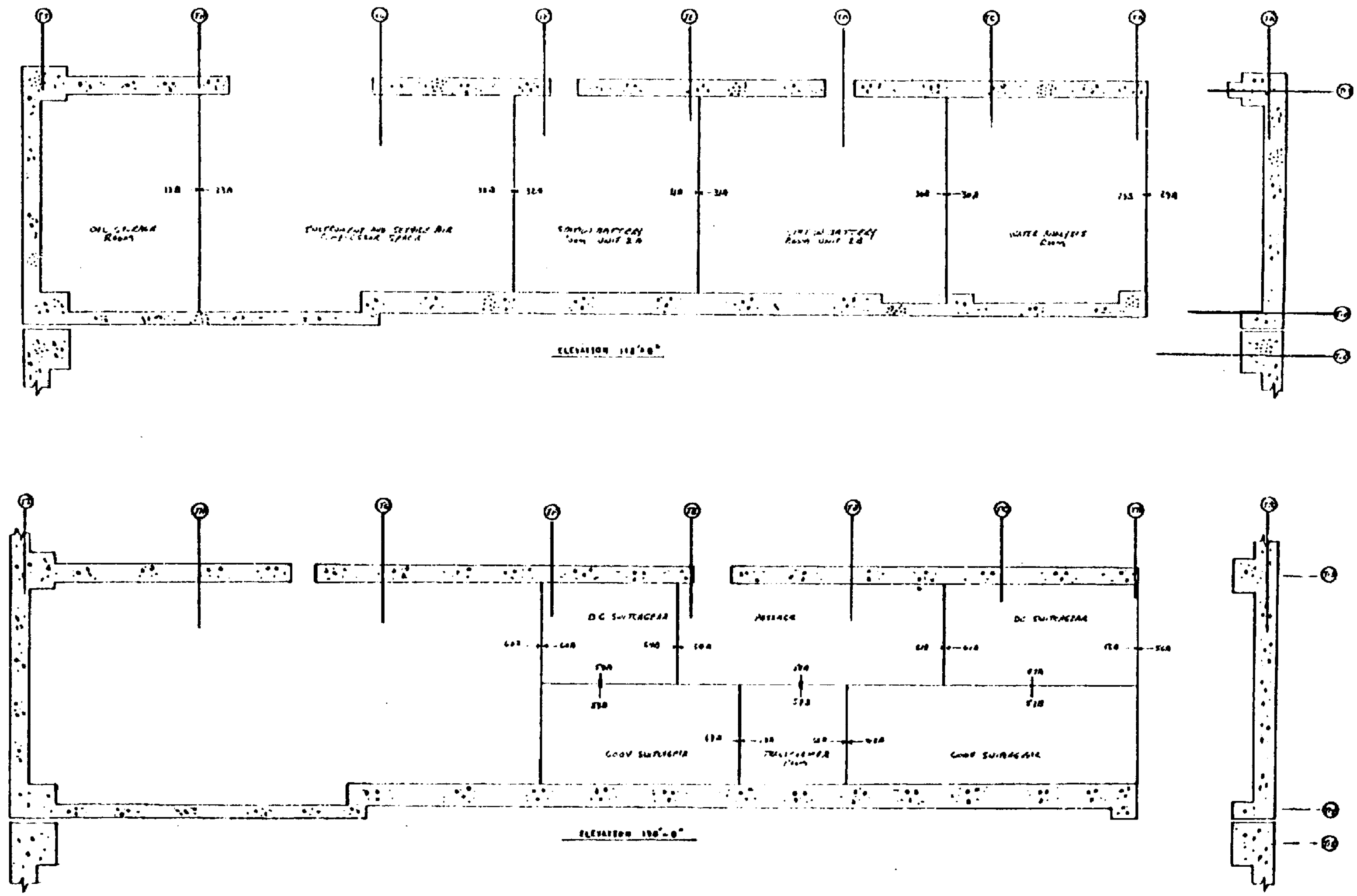
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

MASONRY WALL SINGLE-LINE
SKETCHES – CONTROL BUILDING

FIGURE 3.8-36



ACAD 2030837

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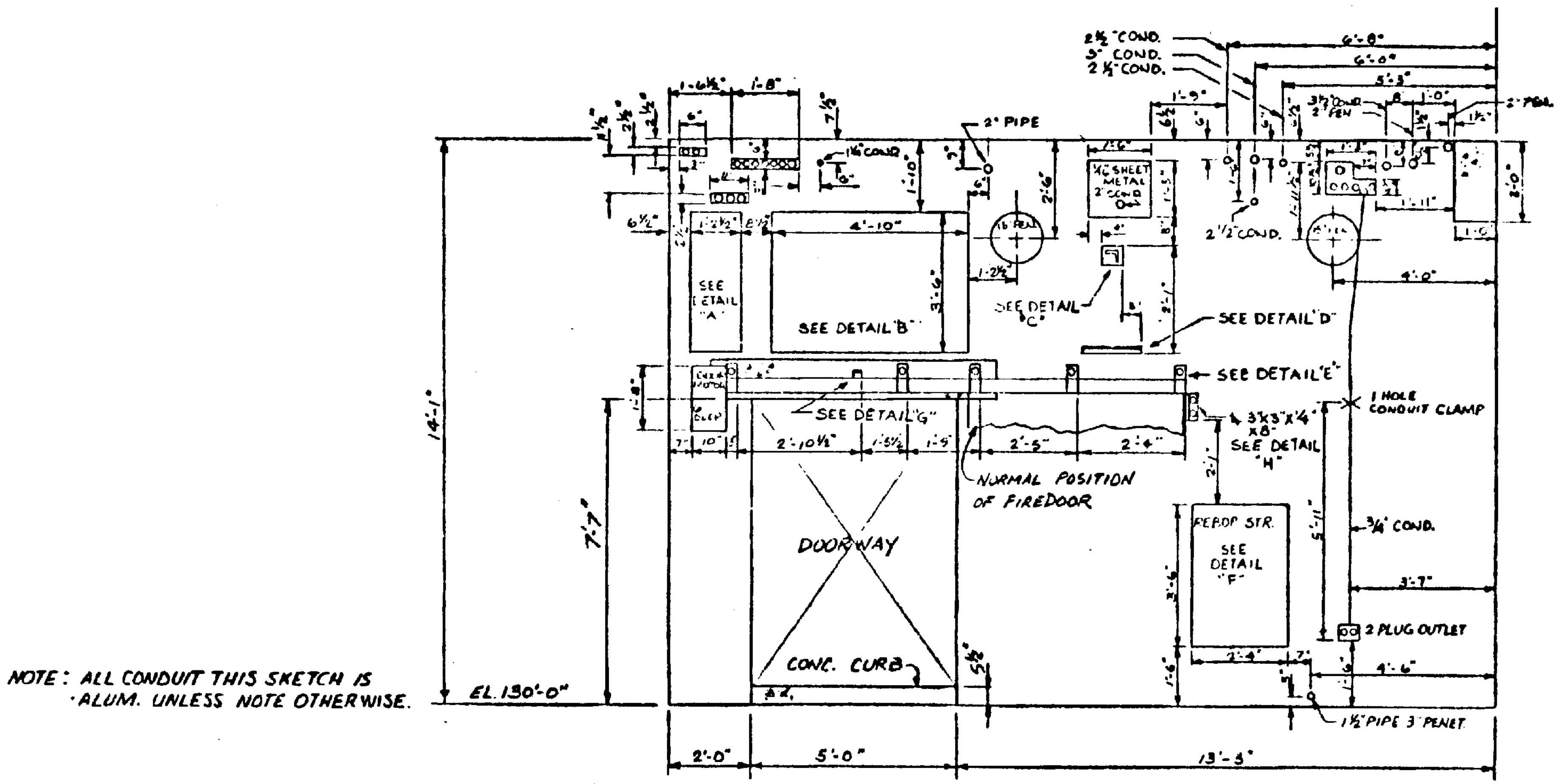
SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

MASONRY WALL SINGLE-LINE SKETCHES – CONTROL BUILDING

FIGURE 3.8-37



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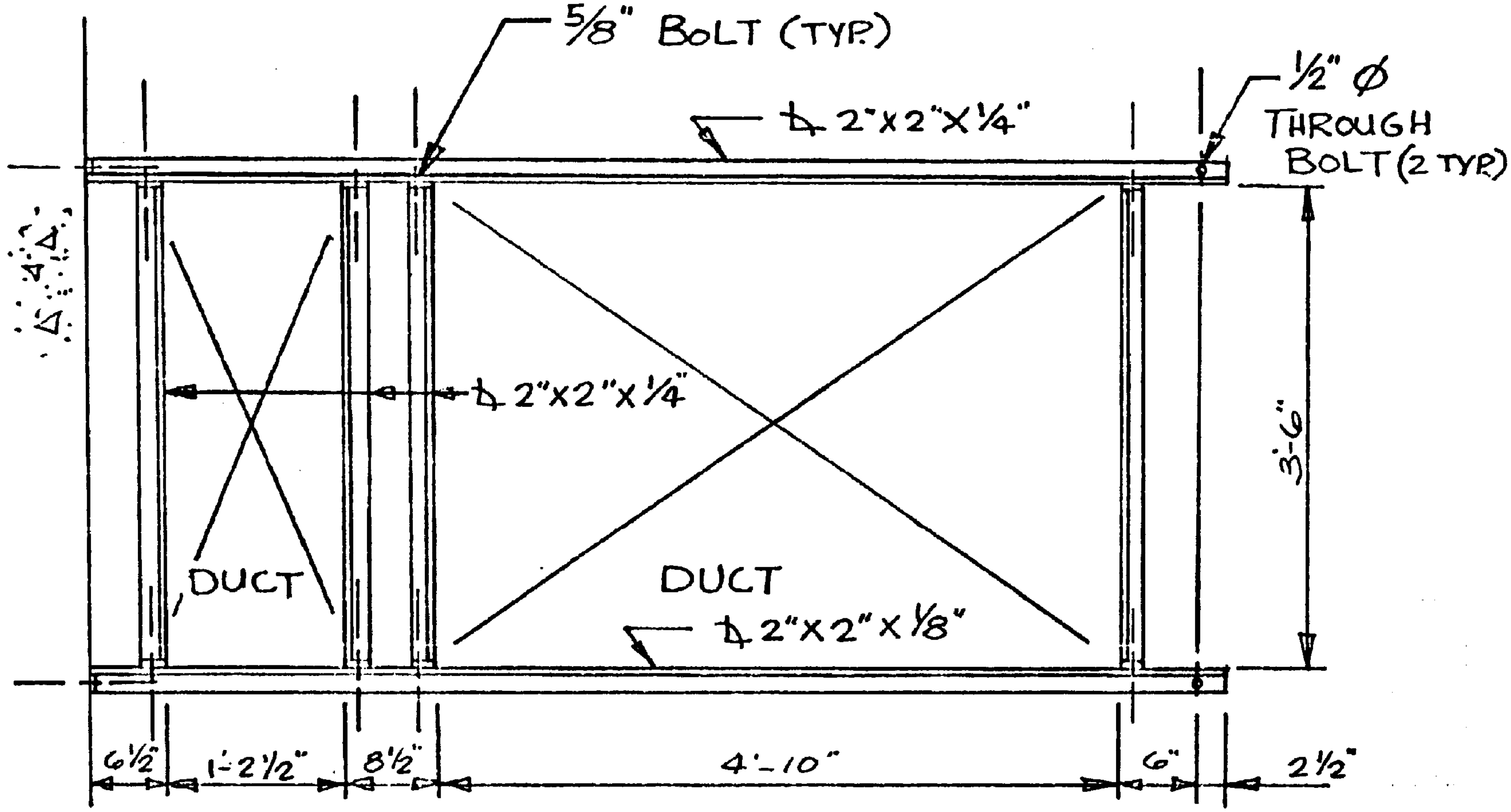
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

EXAMPLE OF WALL SURVEILLANCE SKETCH

FIGURE 3.8-38



NOTE: ALL ∠ MADE OF SHEET METAL

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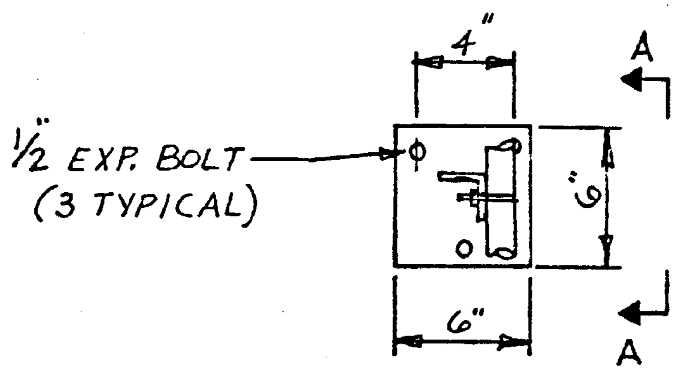
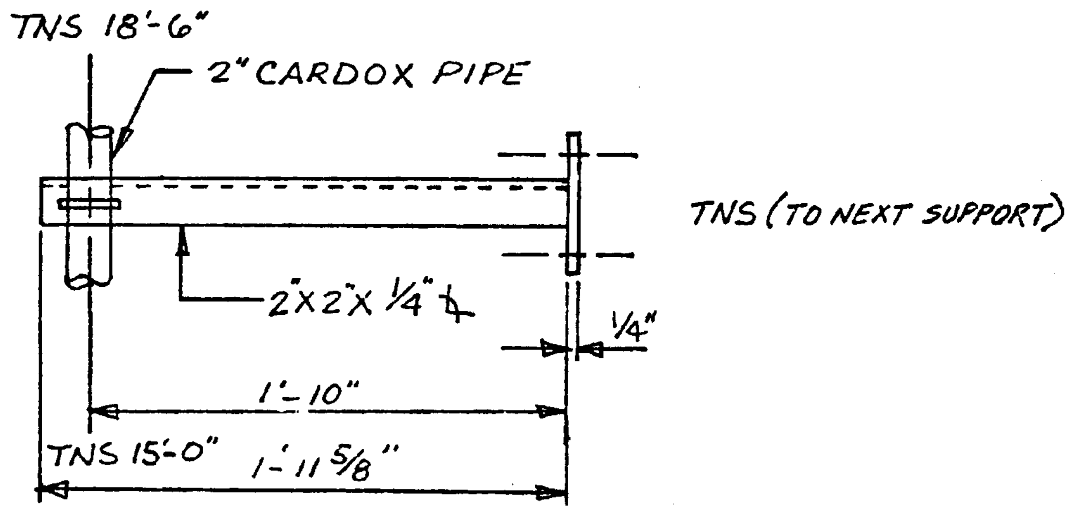
UNIT 2

MASONRY WALL NO. C-130-7A

FIGURE 3.8-39



DETAIL "C"



SECTION
A-A

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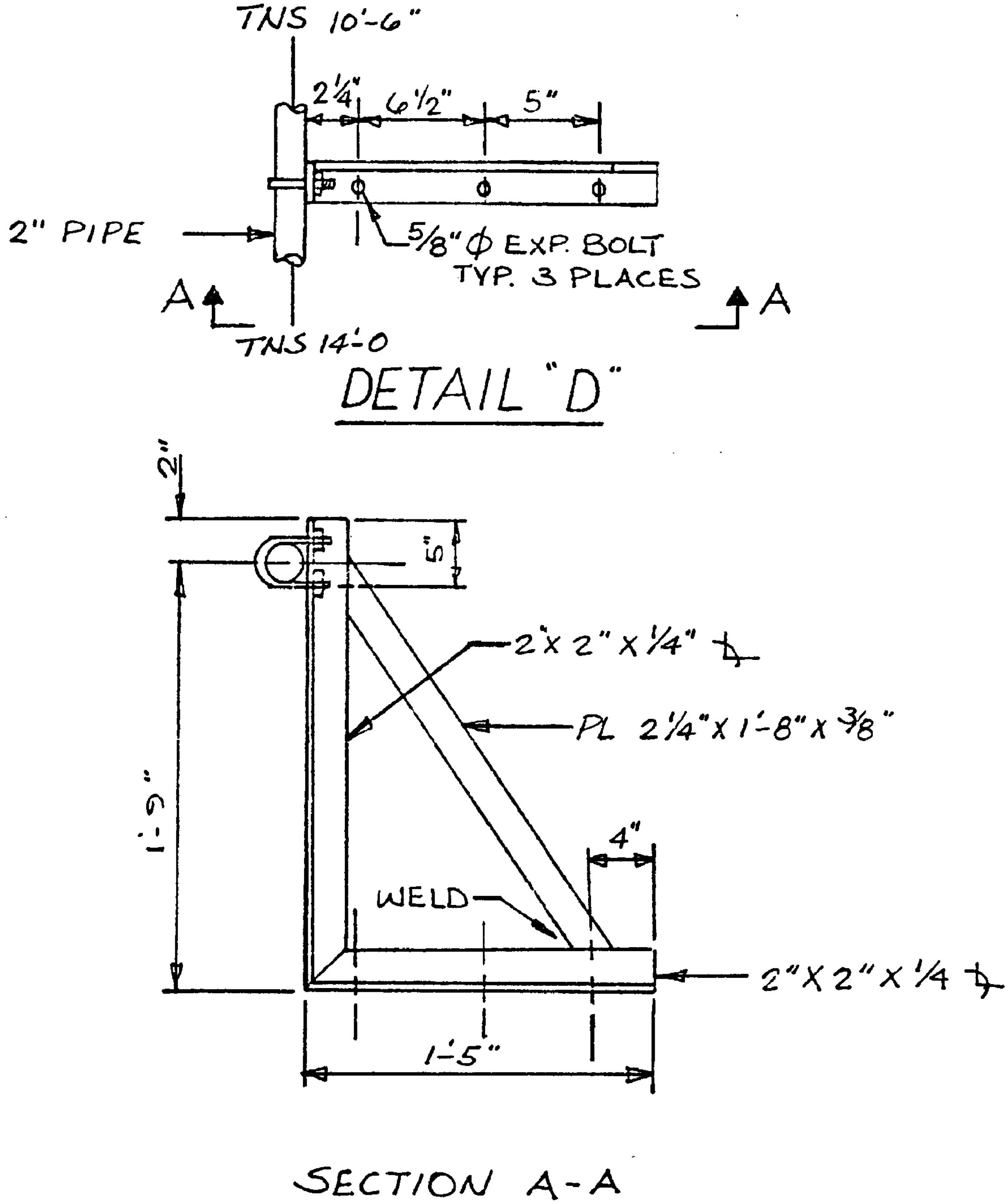
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

DETAILS FOR MASONRY WALL
NO. C-130-7A

FIGURE 3.8-40



ACAD 2030841

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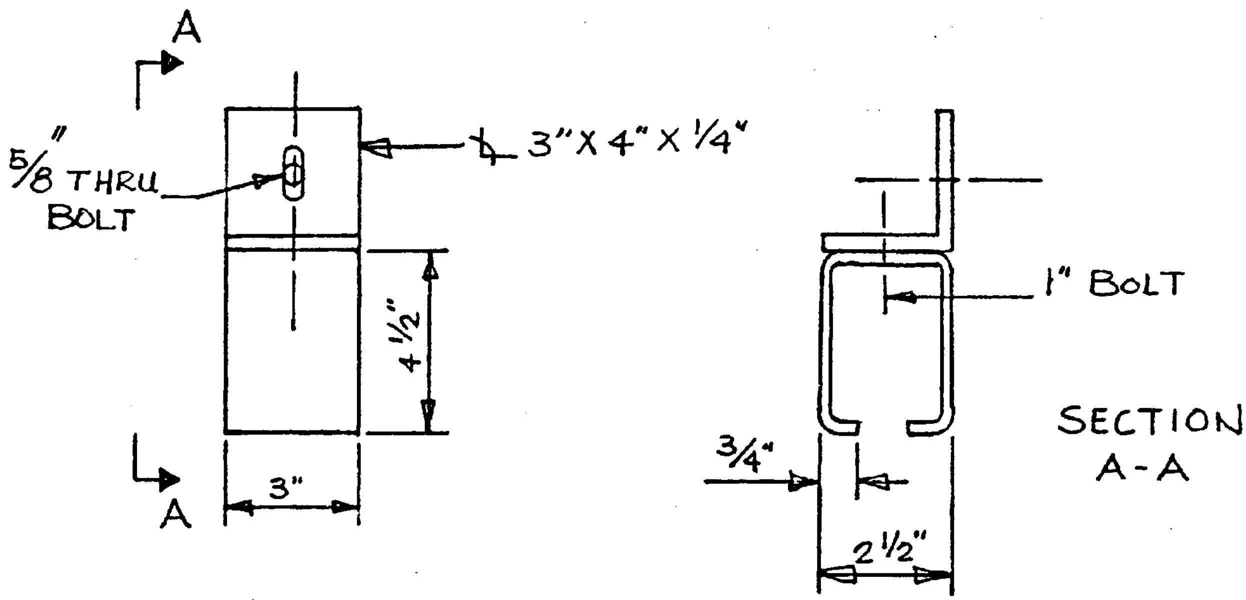
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

DETAILS FOR MASONRY WALL
NO. C-130-7A

FIGURE 3.8-41



DETAIL "E"

STANDARD FIREDOOR HANGER

ACAD 2030842

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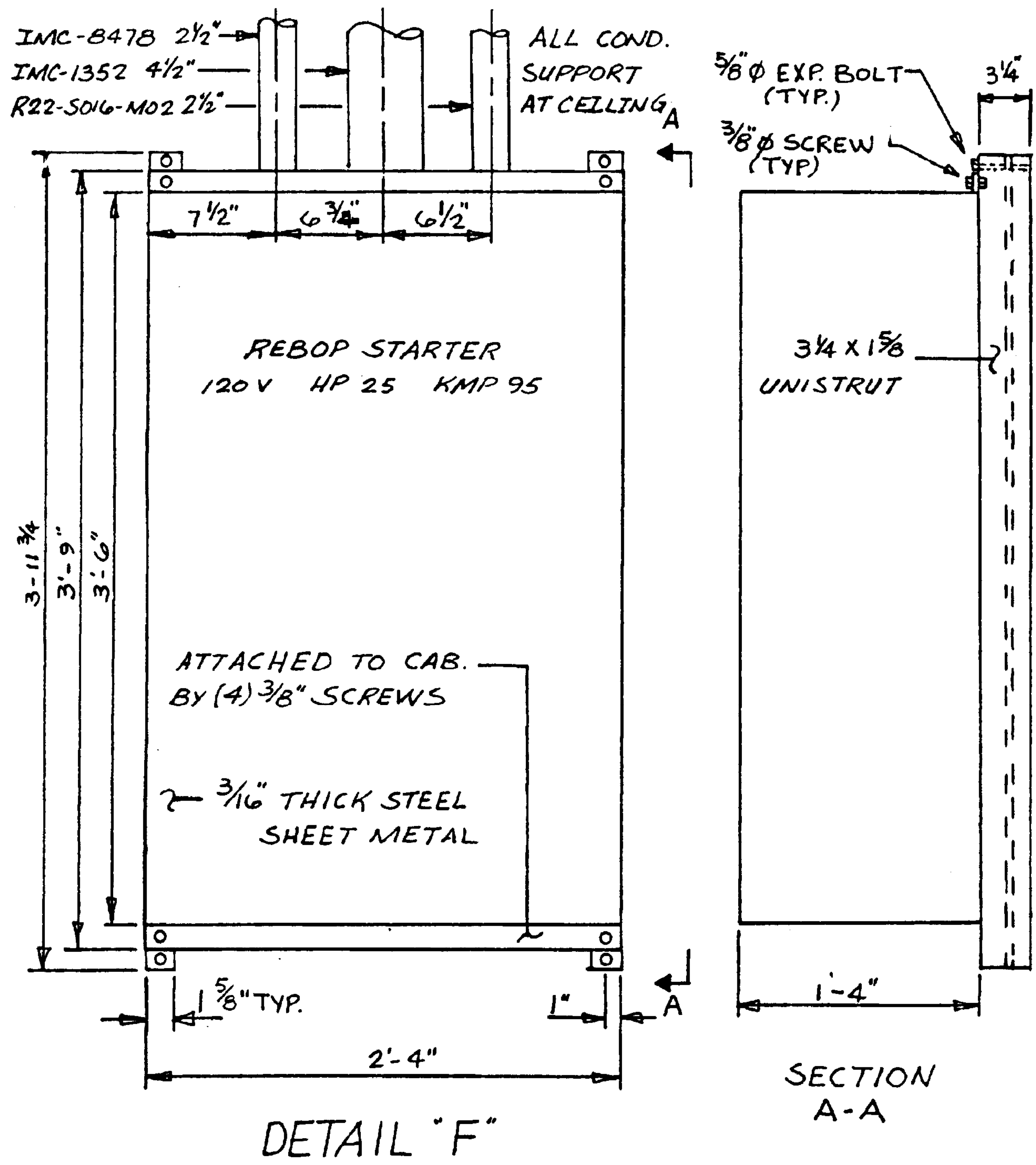
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

DETAILS FOR MASONRY WALL
WALL C-130-7A

FIGURE 3.8-42



ACAD 2030843

REV 19 7/01

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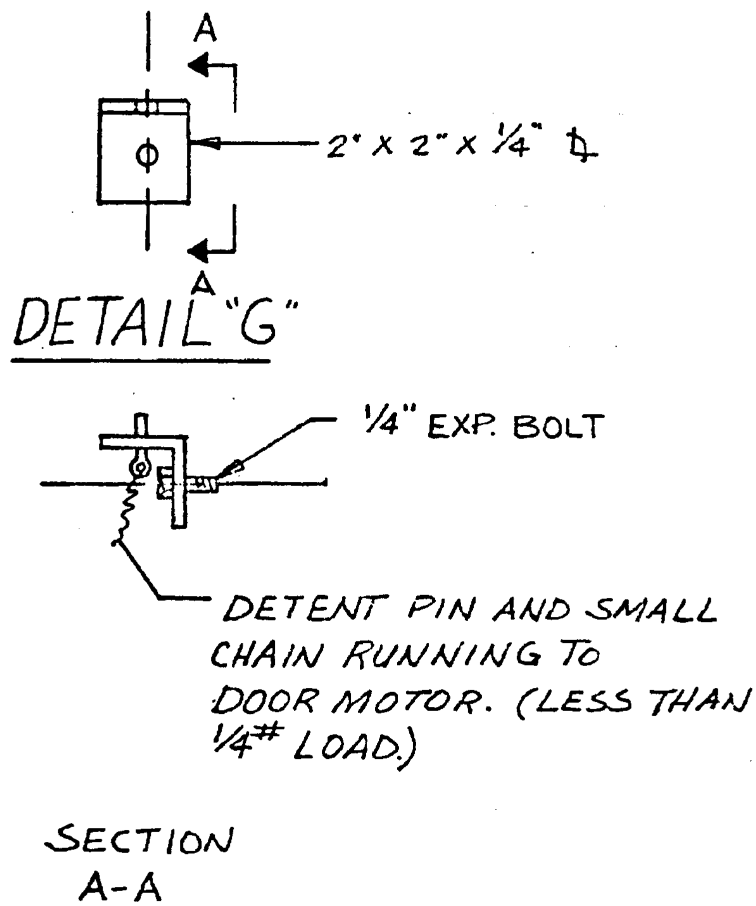
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

DETAILS FOR MASONRY WALL
WALL C-130-7A

FIGURE 3.8-43



ACAD 2030844

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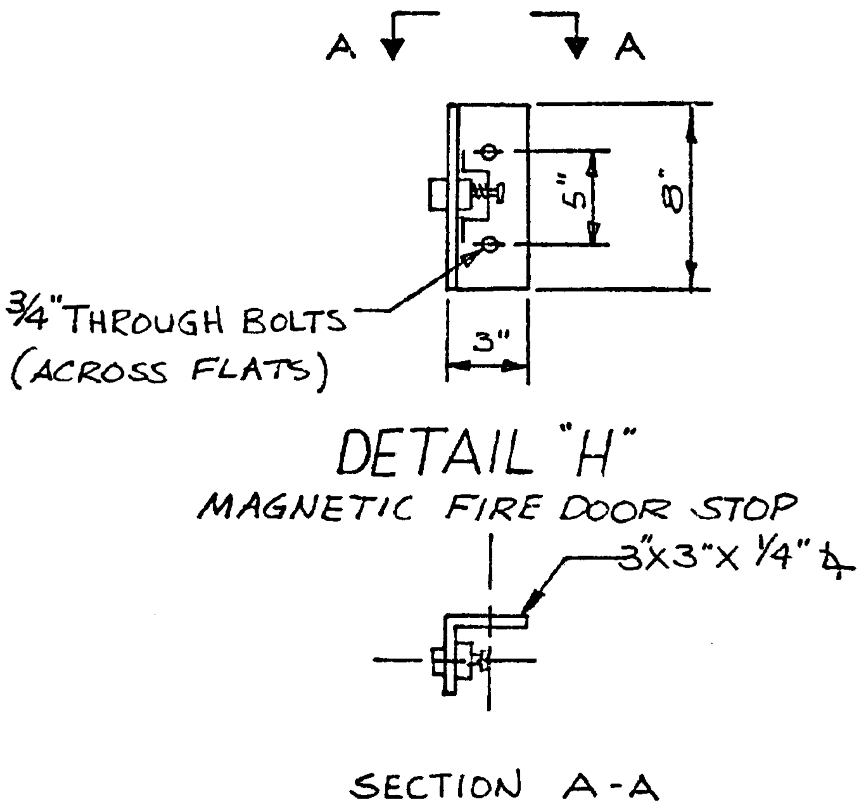
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

DETAILS FOR MASONRY WALL
WALL C-130-7A

FIGURE 3.8-44



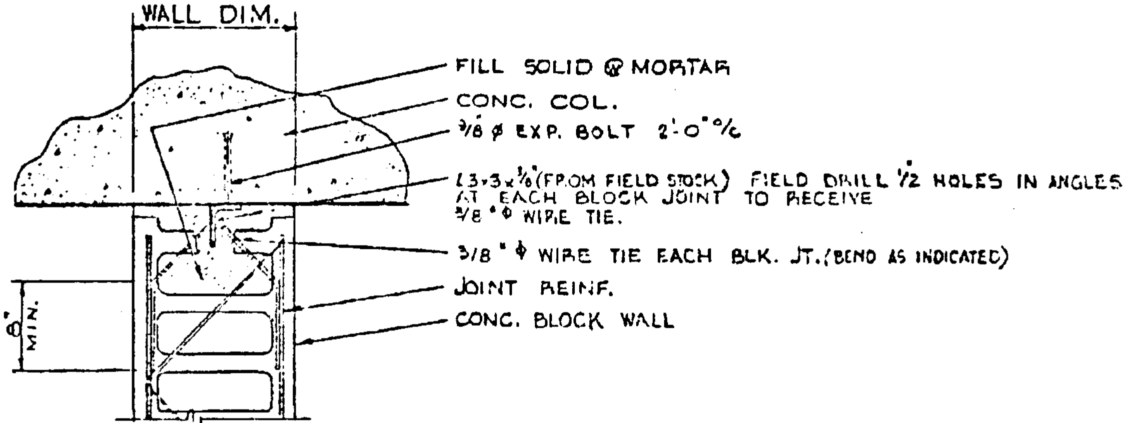
ACAD 2030845

REV 19 7/01

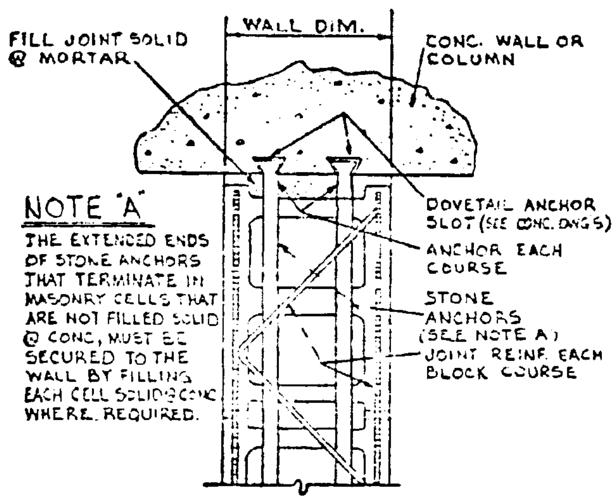
SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

DETAILS FOR MASONRY WALL
WALL C-130-7A

FIGURE 3.8-45



TYP. PLAN @ HORZ. BLOCK JOINT
WHERE MASONRY BUTTS CONC. COL'S OR WALLS
@ NO DOVETAIL SLOTS C-C
SC. 1 1/2" = 1'-0"



TYP. PLAN @ HORZ. BLK. JOINT WHERE
MASONRY WALLS BUTT CONCRETE -
DETAIL "B" (DOUBLE ANCHOR SLOT)

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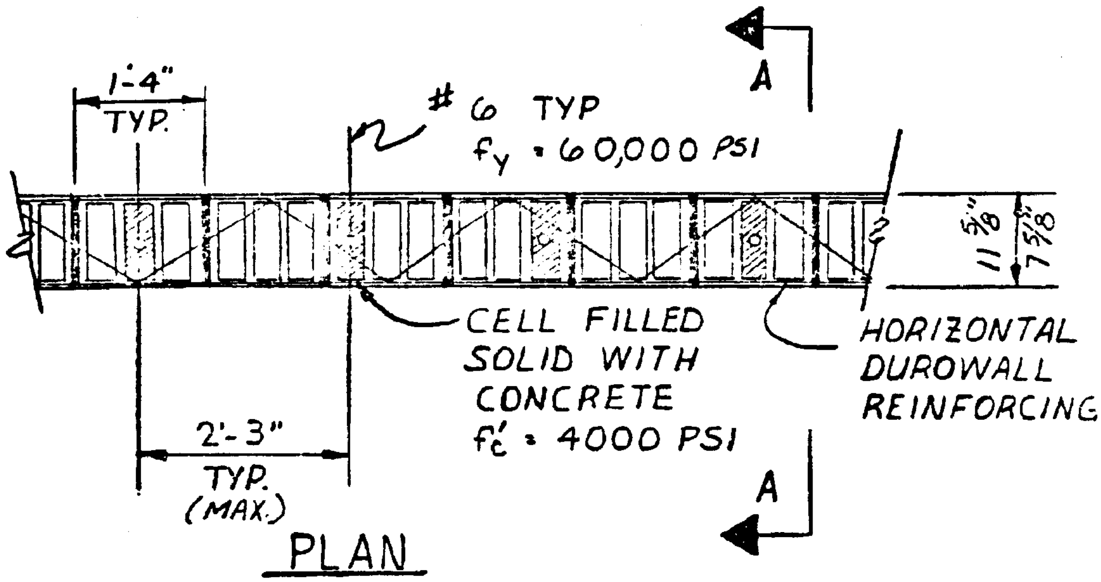
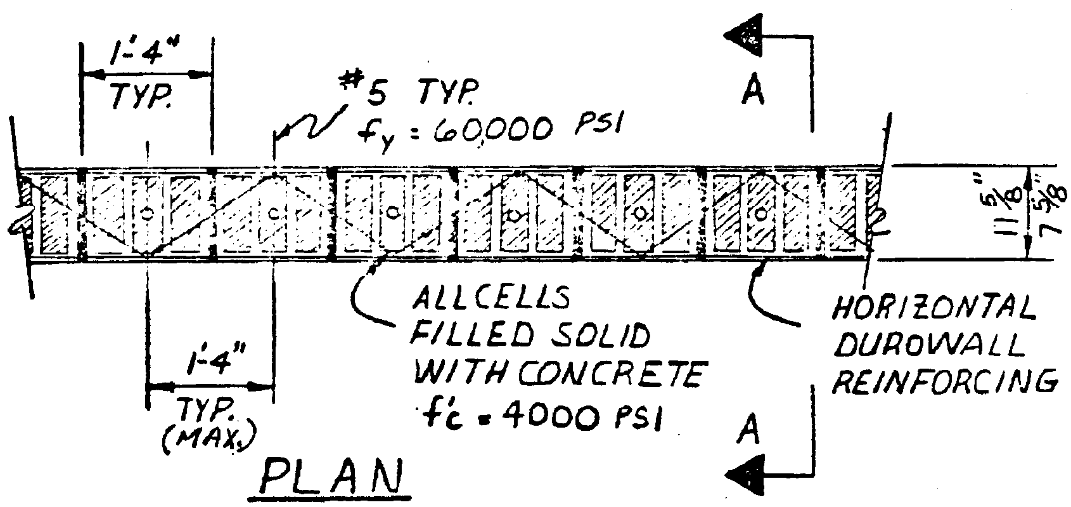
UNIT 2

TYPICAL MASONRY WALL DETAILS

FIGURE 3.8-46

2 TYPES OF CONCRETE BLOCK WALLS





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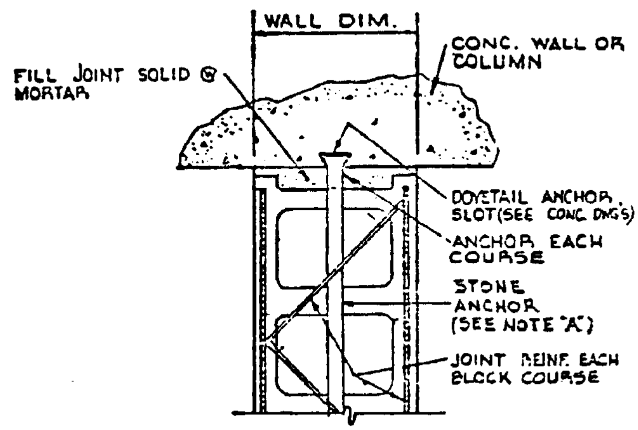
UNIT 2

TYPICAL MASONRY WALL DETAILS

FIGURE 3.8-47



TYP. PLAN @ HORZ. BLK. JOINT WHERE MASONRY WALLS BUTT CONCRETE - DETAIL "A" (SINGLE ANCHOR SLOT)

SC. 1 1/2" = 1'-0"

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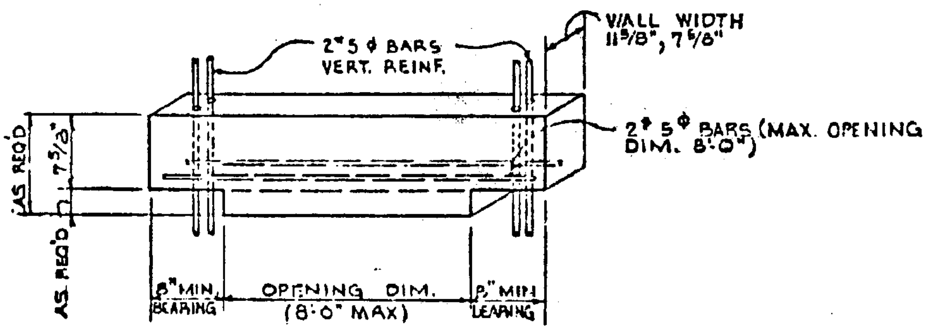
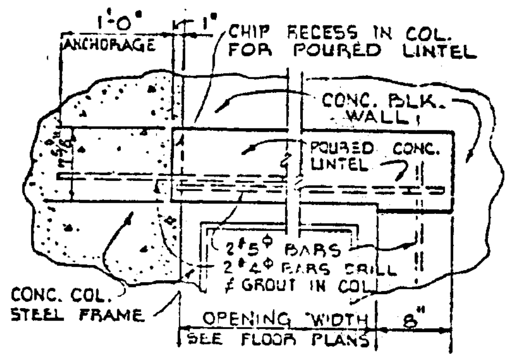
TYPICAL MASONRY WALL DETAILS

FIGURE 3.8-48



TYP. PRECAST CONC. LINTEL DETAIL A-A



TYP. ELEV. @ LINTEL WHERE OPENING BUTTS CONC. COLUMN—C-C

SC. 1"=1-0"

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UNIT 2

TYPICAL MASONRY WALL DETAILS

FIGURE 3.8-49

SUPPLEMENT 3.8A

MECHANICAL SPLICING OF REINFORCING BARS USING THE CADWELD PROCESS

3.8A.1 SCOPE

Mechanical splicing of deformed reinforcing bars for full-tensile loading is accomplished with Cadweld connectors, and the procedure used is in accordance with Regulatory Guide 1.10, "Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures," January 1973, except as noted in the Qualification of Operators and Joint Acceptance Standards. The average tensile strength of the Cadweld joints is greater than the minimum tensile strength for the particular grade of reinforcing steel as specified in the appropriate American Society of Testing Materials (ASTM) standard. The minimum tensile strength of the splices exceeds 125% of the minimum yield strength for each grade of reinforcing steel as specified in the appropriate ASTM standard.

3.8A.2 PROCESS

All splices are made by the Cadweld process (Erico Products, Inc.) using clamping devices, sleeves, charges, etc., as specified by the Cadweld instruction sheets for T series connections. The C series and C-16 series materials are not permitted.

3.8A.3 QUALIFICATIONS OF OPERATORS

Prior to the production splicing of reinforcing bars, each operator or crew, including the foreman or supervisor for that crew, prepares and tests a joint, in place of two joints required by Regulatory Guide 1.10, for each of the positions used in production work. These splices are made and tested in strict accordance with the specification, using the same ASTM grade and size of bar spliced in the production work. To qualify, the completed splices must meet the Joint Acceptance Standards for workmanship, visual quality, and minimum tensile strength. A list containing the names of qualified operators and their qualification test results is maintained at the jobsite. The qualified crew was not required to requalify in accordance with Regulatory Guide 1.10.

3.8A.4 PROCEDURE

All joints are made in strict accordance with the manufacturer's instruction as presented in Erico Products, Inc. Bulletin RB10M-670, "1970 Cadweld Rebar Splicing," plus the following additional requirements:

A. A manufacturer's representative, experienced in Cadweld splicing of reinforcing bars, is present at the jobsite at the outset of the work to demonstrate the equipment and techniques used for making quality splices. He is also present for

at least the first 50 production splices to observe and verify that the equipment is being used correctly and that quality splices are being obtained.

B. The splice sleeves, cartridges, asbestos wicking, ceramic inserts, and graphite parts are stored in a clean, dry area with adequate protection to prevent absorption of moisture.

C. Each splice sleeve is visually examined immediately prior to use to ensure the absence of rust and other foreign material on the inner surface.

D. The graphite molds are preheated with an oxyacetylene torch to 300°F minimum to drive off moisture immediately prior to use.

E. Bar ends to be spliced are in good condition with full-size undamaged deformations. The bar ends are power brushed to remove all loose mill scale, rust, concrete, and other foreign material. Prior to power brushing, all water, grease, and paint are removed by heating the bar ends with an oxyacetylene or propane torch.

F. A permanent line, marked 12 in. back from the end of each bar, serves as a reference point to confirm that the bar ends are properly centered in the splice sleeve.

G. Immediately before the splice sleeve is placed into final position, the previously cleaned bar ends are preheated with an oxyacetylene or propane torch to ensure complete absence of moisture.

H. Special attention is given to maintaining the alignment of sleeve and guide tube to ensure a proper fill.

I. When the temperature is below freezing or the relative humidity is above 65%, the splice sleeve is externally preheated with an oxyacetylene or propane torch after all materials and equipment are in position.

J. The reinforcing bar deformations which become engaged in the Cadweld splice are not ground, flame-cut, or altered in any way except for the longitudinal ribs which are ground to a diameter not less than the other bar deformations.

K. An adequate escape route is provided for gases generated during the casting of horizontal splices. For splices in bars smaller than No. 11, this is done by inserting a hairpin piece of soft twisted wire at the top of the splice between the rebar and the sleeve.

L. The packing material at the ends of the horizontal splices and at the top of the vertical splices is not hard packed. The material is firmly in place but loose enough to allow the escape of gases.

3.8A.5 ONSITE USER TESTS

The onsite user test program for reinforcing steel splices is described below:

A. Every operator is required to pass a qualification test.

B. All splices are visually inspected. As indicated in section 3.8A.7 of this supplement, unsatisfactory splices are replaced.

C. For each crew, after qualification, tests are made for each position as follows:

Sister Splice Program

The following tensile program is used:

- One out of the first lot of 10 production splices for each position, bar size, and grade of bar.
- One production splice and 3 "sister splices" from the next 90 splices for each position, bar size, and grade of bar.
- Three splices out of the next and subsequent lots of 100 splices for each position, bar size, and grade of bar; one-fourth of these splices from production splices and three-fourths from "sister splices."

A "sister splice" is defined as a 3-ft-long test bar spliced in sequence with, and in an otherwise identical manner as, the production splices.

3.8A.6 JOINT ACCEPTANCE STANDARDS

The following criteria are used for judging the acceptability of Cadweld joints:

A. Sound nonporous filler metal must be visible at both ends of the splice sleeve and at the tap hole in the center of the splice sleeve. Filler metal, which is usually recessed 1/4 in. from the end of the sleeve due to the packing material, is not considered as poor fill.

B. Splices which contain slag or porous metal in the riser, tap hole, or at the ends of the sleeves (general porosity) are rejected. A single shrinkage bubble present below the riser is not detrimental and is distinguished from general porosity as described above.

C. The Cadweld splices, both horizontal and vertical, may contain voids at either or both ends of the Cadweld splice sleeve. At the end of the Cadweld splice sleeves, the acceptable size void for a No. 18 splice does not exceed 3 in.^{2} per end of splice sleeve. The area of the void is assumed to be the circumferential length as

measured at the inside face of the sleeve multiplied by the maximum depth of wire probe minus 3/16 in.

D. The average tensile strength of the Cadweld joints is greater than the minimum tensile strength for the particular grade of reinforcing steel as specified in the appropriate ASTM standard. The minimum strength of the Cadweld joints must be greater than 125% of the specified minimum yield strength for the particular bar. If any of the tested specimens failed, two additional random splices of the same lot were tested, and if both passed the test, the lot was accepted. If one or both failed, the entire lot was rejected. This criterion is more conservative than the one described in Regulatory Guide 1.10.

3.8A.7 REPAIRS

Joints which do not meet the quality acceptance standards of section 3.8A.6 are rejected and completely removed. The bars are then rejointed with a new splice.

SUPPLEMENT 3.8B

PLANT-UNIQUE ANALYSIS OF MARK I CONTAINMENT SYSTEM

3.8B.1 INTRODUCTION

The Hatch Nuclear Plant Unit 2 (HNP-2) containment system is one of the first-generation General Electric (GE) boiling water reactor (BWR) nuclear steam supply systems (NSSSs) housed in a containment structure designated as the Mark I containment system. The original design of the Mark I containment system considered postulated accident loads previously associated with containment design, which included pressure and temperature loads associated with a loss-of-coolant accident (LOCA), seismic loads, dead loads, jet-impingement loads, hydrostatic loads due to water in the suppression chamber, overload pressure test loads, and construction loads. However, since the establishment of the original design criteria, additional loading conditions have been identified that arise in the functioning of the pressure-suppression concept utilized in the Mark I containment system. These additional loads result from the dynamic effects of drywell air and steam being rapidly forced into the suppression pool (torus) during a postulated LOCA and from suppression pool response to safety relief valve (SRV) operation generally associated with plant transient operating conditions. Because these hydrodynamic loads were not considered in the original design of the Mark I containment system, the Nuclear Regulatory Commission (NRC) determined that a detailed reevaluation of the Mark I containment system was required.

A two-phase program was identified in the NRC in May 1975. The first-phase effort, called the Short-Term Program (STP), provided a rapid assessment of the adequacy of the containment to maintain its integrity under the most probable course of the postulated LOCA. Thus, the first phase demonstrated the acceptability of continued operation during the performance of the second phase, called the Long-Term Program (LTP). In the LTP, detailed testing and analytical work was performed to define the specific design loads against which the containment was assessed to establish conformance to established acceptance criteria.

The STP was completed in July 1977 following the docketed submittal by Georgia Power Company (GPC) to the NRC of the HNP-2 plant unique analysis report.[1] Reevaluation of the Mark I containment system (LTP) was completed in September 1983, following the docketed submittal by Georgia Power Company to the NRC of the HNP-2 Plant Unique Analysis Report (PUAR).[2]

3.8B.2 PLANT UNIQUE ANALYSIS REPORT

The Mark I containment system reevaluation results are detailed in the PUAR submitted to the NRC in September 1983 and revised in December 1989. (See Reference 2). Many of the analyses presented in Reference 2 assumed a 95°F initial suppression pool temperature. Reference 3 documents the acceptability of the Reference 2 analyses at higher initial pool temperature (≤110°F). The PUAR demonstrates that the configuration of the plant, including structural modifications and load mitigation devices, meets the NRC requirements for the Mark

I LTP as documented in the Mark I containment Long-Term Program Safety Evaluation Report, NUREG-0661.[4]

In summary the report provides the following:

- A review of the event sequences involving the Mark I containment system related phenomena for the postulated LOCA and safety relief valve (SRV) actuation conditions.

- A description of the major structural components of the HNP-2 containment system that were evaluated. The description includes both before and after status structural modifications.

- A review of the design criteria used, which includes both the design specification covering the fabrication and erection of the modifications and the structural acceptance criteria applying to the design analysis.

- A discussion of the system changes/additions made to the containment system to mitigate loads.

- A description of the loads and load combinations as applied in the HNP-2 analysis.

- A review of the computer programs used in the analysis.

- A summary of the analytical methods and models employed in evaluating each of the structural components.

- A summary of the analytical results for each structural component and a comparison with allowables, based on the structural acceptance criteria that demonstrate that the upgraded design-safety margins have been achieved.

3.8B.3 DESCRIPTION OF LTP MODIFICATIONS

The components significantly affected by the postulated LOCA and SRV actuation events are the suppression chamber, vent system, torus internal structures, SRV piping and supports, and the torus-attached piping and supports. Detailed analysis of the components determined that structural modifications and system changes were required to establish the NRC design safety margins specified for the Mark I LTP. Table 3.8B-1 presents a summary of the LTP modifications to the HNP-2 containment system. The modifications are in addition to the STP changes summarized in Appendix A of the PUAR.[2]

3.8B.4 EXPANDED OPERATING DOMAIN OPERATION

A containment loads analysis was performed to demonstrate that ample margins for containment integrity remain for plant operation in the expanded operating domain (EOD) at the maximum core inlet

subcooling condition which was 100% of original rated power (2436 MWt) and 87% flow with reduced feedwater temperature. This analysis[5] which evaluated the containment pressure and temperature response and the containment hydrodynamic loads for a postulated design basis LOCA, was based on the methodology developed for the Mark I Long-Term Containment Program and documented in the Mark I Containment Program Load Definition Report.[6] The results of this analysis showed that the peak containment pressure in the EOD with reduced feedwater temperature was 46.7 psig which was higher than the value reported in NEDO-24569[7] of 43.0 psig, but below the design value of 56 psig and within the design margins shown in the PUAR. The containment hydrodynamic loads with EOD conditions were also within the design margins shown in the PUAR. The results of analysis and evaluations for the effect of power uprates up to a licensed 100% RTP of 2804 MWt on containment loads including operation in the EOD are discussed in subsection 3.8B.5.

3.8B.5 <u>POWER UPRATE OPERATION</u>

A containment loads analysis was performed for extended power uprate conditions to assure adequate margins existed for operation at 2763 MWt. The results, summarized in reference 9, were acceptable. The containment system performance analysis included short- and long-term pressure and temperature responses, LOCA containment dynamic loads, and SRV containment dynamic loads. The analyses included the EOD for a core power of 2763 MWt and final feedwater temperature operation. The effect on containment loads was subsequently evaluated in reference 10 to support operation at 2804 MWt and in references 11 and 12 for an increase in reactor operating pressure to 1060 psia. The evaluations indicate that containment loads increase slightly due to the operating pressure increase but remain acceptable. The evaluation for the pressure increase showed that the peak containment pressure in the EOD for a core power of 2804 MWt with reduced feedwater temperature (see section 6.2) is 50.1 psig, which is below the 56 psig design limit.

3.8B.6 <u>OPERATION DURING PERIOD OF EXTENDED OPERATION</u>

An analysis of the cumulative fatigue usage factor (CFUF) for the torus shell was performed to account for the period of extended operation. (See subsection 18.1.1 for a definition of the term "period of extended operation.") This analysis demonstrated the need to track actual thermal and dynamic loading events to ensure the torus shell maintains an actual CFUF $\leq$ 1.0 through the period of extended operation. The most limiting event for the torus is the steam blowdown resulting from the lifting of one or more main steam safety relief valves. The component cyclic or transient limit program (subsection 18.2.12) performs tracking of operational events. The CFUF analysis is a time-limited aging analysis and is described in section 18.5

REFERENCES

1. "Structural Evaluation of Pressure Suppression System for E. I. Hatch Nuclear Plant-Unit 2, Mark I Containment," Bechtel Power Corporation, Docket No. 50-366, July 1977.

2. "Plant Unique Analysis Report for E. I. Hatch Nuclear Plant-Unit 2, Mark I Containment Long-Term Program," Bechtel Power Corporation, Revision 2, Docket No. 50-366, December 1989.

3. "Elimination of the Suppression Pool Temperature Limit for Plant Hatch Units 1 and 2," EAS-19-0388, General Electric Company, March 1988.

4. "Safety Evaluation Report Mark I Containment Long-Term Program," NUREG-0661, U. S. Nuclear Regulatory Commission, July 1980.

5. "Limiting Reload Licensing Events for E. I. Hatch Nuclear Plant Unit 1 and Unit 2," EAS-65-1088, General Electric Company, October 1988.

6. "Mark I Containment Program Load Definition Report," Revision 2, NEDO-21888, General Electric Company, November 1981.

7. "Mark I Containment Program Plant Unique Load Definition: Unit 2," NEDO-24569, General Electric Company, September 1981.

8. "Power Uprate Safety Analysis Report for Edwin I. Hatch Plant Units 1 and 2," NEDC-32405P, General Electric Company, December 1994.

9. "Extended Power Uprate Safety Analysis Report for Edwin I. Hatch Plant Units 1 and 2," NEDC-32749P, General Electric Company, July 1997.

10. “Safety Analysis Report for Edwin I. Hatch Units 1 and 2 Thermal Power Optimization,” NEDC-33085P, GE Nuclear Energy, December 2002.

11. “10-PSI Dome Pressure Increase Project Report for Edwin I. Hatch Units 1 and 2,” GE-NE-0000-0003-0634-01, Revision 1, GE Nuclear Energy, July 2003.

12. RER 03-254, Reactor Operating Pressure Increase From 1050 psia to 1060 psia, Engineering Evaluation.

TABLE 3.8B-1 (SHEET 1 OF 2)

LTP MODIFICATION SUMMARY

Component Category	Modification Description
Torus	Addition of shell T-stiffeners
	Addition of stiffeners to existing saddle supports
Vent system	Addition of vent header deflectors under vent header in the non-vent bays
	Modification of existing downcomer ties
	Addition of stiffener plates to downcomer-vent header intersection
	Addition of stiffener plates to vent header intersection at vacuum breaker locations
	Addition of stiffener plates to vent lines at the SRV line penetration locations
	Addition of pipe braces to existing vacuum breaker drain lines
Internal structures	Modification of catwalk inside torus
	Modification of monorail inside torus
SRV piping	Addition of T-quencher discharge devices inside torus
	Addition of vacuum breakers to low-low set (LLS) SRV discharge lines (SRVDLs)
SRV piping supports	Addition of T-quencher supports Support beams Beam supports Gusset plate reinforcing
	SRV line M intermediate support
	Addition/modification of SRVDL supports inside drywell

TABLE 3.8B-1 (SHEET 2 OF 2)

Component Category	Modification Description
Torus attached piping and support inside torus	Addition of elbows to RHR test lines
	Modification to return line restraints
	Removal of spare piping - X227A
Suppression pool temperature monitoring	Addition of thermowells and half couplings
SRV logic change	Main steam isolation valve isolation logic level change
	SRV LLS logic

SUPPLEMENT 3.8C

DESIGN CRITERIA FOR REEVALUATION OF CONCRETE MASONRY WALLS

3.8C.1 GENERAL

3.8C.1.1 PURPOSE

Provided in supplement 3.8C are the design requirements and criteria used to reevaluate the structural adequacy of concrete block walls as required by the Nuclear Regulatory Commission (NRC) I&E Bulletin 80-11, Masonry Wall Design (May 8, 1980).

3.8C.1.2 SCOPE

The reevaluation determined whether the concrete masonry walls would perform their intended function under all postulated loads and load combinations. Concrete masonry walls not supporting safety systems but whose collapse could result in the loss of required function of safety-related equipment or systems were evaluated in the same manner as walls that support safety systems. Verification of wall adequacy included support condition, global response of wall, and local transfer of load. Evaluation of anchor bolts and embedments was not considered to be within the scope of I&E Bulletin 80-11.

3.8C.2 GOVERNING CODE

The American Concrete Institute (ACI) Building Code Requirements for Concrete Masonry Structures (ACI 531-79) was used as the basis for structural reevaluation. Supplemental allowables for cases not directly covered in the governing code are specified in this supplement.

3.8C.3 LOADS AND LOAD COMBINATIONS

Loads and load combinations are as specified in the Final Safety Analysis Report (FSAR) for concrete design. Load factors consistent with FSAR requirements were applied to all loads in all postulated load combinations.

Masonry walls were designed to withstand dead loads, live loads and both operating basis earthquake (OBE) and design basis earthquake (DBE) loads. The walls are not subjected to other loads such as wind, tornadoes, missiles, pipe whips, jet impingement, or differential pressure. All walls are nonload bearing and are not included in the overall building shear wall system.

The walls are relied upon to act only as interior partition walls or to provide shielding. During normal operation or seismic events, the walls are not subjected to extreme thermal loads. The

walls were originally designed to provide a specified fire rating; therefore, it is assumed that the walls will not fail under thermal loads.

3.8C.4 DESIGN ALLOWABLES

For walls subjected to normal unfactored dead loads, live loads, and OBE loads, the design allowables are as follows:

A. Masonry

Allowable values for tension, compression, shear, bond, and bearing stresses are those given in ACI 531-79.

B. Core Concrete Or Cell Grout

Values used for stress in core concrete were the same as allowables for concrete given in ACI 318. The allowable tension stress is given as 2.5 $\sqrt{f'_c}$. This value is stated as having a safety factor of 3.

C. Reinforcing Steel

The allowable values for tension and compression stresses are those given in Section 10.2, ACI 531-79.

D. Seismic Loads

Consistent with FSAR guidelines, the 33% increase in allowable stresses for masonry and reinforcing steel due to OBE loads is not permitted.

E. Collar Joints

Multiple Wythe walls are assumed to act independently under earthquake loads.

Design allowables for load combinations that include factored loads and/or DBE loads are as follows:

A. Masonry

The allowable masonry stresses are 1.67 times the values given in A and B above.

B. Reinforcing Steel

The allowable steel stresses are 0.9 fy, provided that lap splice lengths and embedment can develop a stress level equal to this value. To determine splice and anchorage requirements, allowable bond stresses may be increased by 1.67 in calculations.

The damping values used for uncracked sections are as follows:

A. For OBE analysis, a 3% value of critical damping is used.

B. For DBE analysis, a 5% value of critical damping is used.

The damping values used for cracked sections are as follows:

A. For OBE analysis, a 3% value of critical damping is used.

B. For DBE analysis, a 5% value of critical damping is used.

The extreme tensile fiber stress for use in determining the lower-bound uncracked moment capacity is $6\sqrt{f'_c}$ for core concrete or cell grout and 2.4 times the code allowable flexural tensile stress for masonry.

3.8C.5 ANALYSIS AND DESIGN

3.8C.5.1 GENERAL

A. The concrete masonry structures were analyzed according to working stress principles using factored loads.

B. The walls were designed, in general, to span horizontally in accordance with the original design. Walls were also designed to span vertically or for two-way action on a case-by-case basis.

C. Consistent with FSAR criteria, concrete columns, pilasters, and walls framing into the wall under consideration are taken as rigid supports under seismic loading.

D. Section properties are based on actual masonry unit dimensions rather than on nominal sizes.

3.8C.5.2 STRUCTURAL RESPONSE OF MASONRY WALLS

A. Equivalent Moment of Inertia

To determine the out-of-plane frequencies of masonry walls, the uncracked behavior and capacities of the walls (step 1) and, if applicable, the cracked behavior and capacities of the wall (step 2) were considered.

Step 1 - Uncracked Condition

The equivalent moment of inertia of an uncracked wall is obtained from a transformed section consisting of the block, mortar, cell grout, and core concrete. Alternatively, the cell grout and core concrete (neglecting block and mortar on the tension side) may be used.

Step 2 - Cracked Condition

If the applied moment (M_a) due to all loads in a load combination exceeds the uncracked-moment capacity, the wall is considered to be cracked. In this event, the equivalent moments of inertia are computed as follows:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 (I_t) + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

$$M_{cr} = f_r \left(\frac{I_t}{y}\right)$$

where:

I_e = equivalent moment of inertia.

M_{cr} = uncracked-moment capacity.

M_a = applied maximum moment on the wall.

I_t = moment of inertia of the transformed section.

I_{cr} = moment of inertia of the cracked section.

f_r = modulus of rupture (2.4 times the allowable flexural tensile stress for masonry).

y = distance of neutral plane from tension face.

If the use of I_e results in an applied moment M_a which is less than M_{cr}, then the wall is verified for M_{cr}.

B. Modes of Vibration

A parametric study concluded that, under all boundary conditions and aspect ratios tested, the first mode of vibration accounted for over 99% of the total moment and displacement of the walls. The effect of the first 3 modes was considered in the analysis, and was assumed to contribute ~ 100% of the total moment and displacement.

C. Frequency Variations

Uncertainties in structural frequencies of the masonry walls because of variations in structural properties and mass were taken into account. Significant variables include mass, boundary conditions, modulus of elasticity, extent of cracking, vertical load, in-plane and out-of-plane loads, and two-way action. Because of these uncertainties, the effect of variations was considered. It is considered conservative to use the lower-bound frequency if it is on the higher-frequency side of the response spectrum peak; however, if the lower-bound frequency is on the lower side of the peak, the peak acceleration is used if a more detailed analysis was not performed.

D. Selection of Appropriate Response Spectra

The seismic acceleration at each frequency is the greater number from the response spectra of the floor above or the floor below the wall for all walls except simple cantilevers. The response spectra for the lower floor was used for cantilever walls.

Since portions of the control building are shared by both HNP-1 and HNP-2, the response spectra for HNP-2 were used for initial analyses since the accelerations are greater at each frequency in the HNP-2 spectra. However, if more detailed analyses were required, the response spectra used in the original design were used.

3.8C.5.3 STRUCTURAL STRENGTH OF MASONRY WALLS

A. Boundary Conditions

The walls were designed as either one-way or two-way spans with either free, simply supported, or fixed edges. A simple support was assumed when the joint was capable of shear transfer under all loading conditions. Fixed support conditions were assumed when the joint was capable of flexural tensile stress transfer to the support under all loading conditions.

B. Application of Concentrated Out-of-Plane Loads

<u>One-Way Bending</u>

Out-of-plane loads are applied as point loads on a beam strip equal in width to two times the wall thickness. Local moments are determined by using beam theory and taking into account both in-plane and out-of-plane loads.

<u>Two-Way Bending</u>

On a case-by-case basis, two-way action may be considered. For these cases, conservative analysis techniques are employed with all out-of-plane loads and openings being considered in the seismic analysis.

C. Interstory Drift Effects

Interstory drift effects are derived from the original dynamic analysis. Adequacy of the walls with regard to in-plane drift effects are based on a strain criteria that are based on experimental and analytical results for walls cyclically loaded to failure. The strain criteria apply for all confined nonstructural walls. A nonstructural wall is defined as follows:

1. A nonstructural wall does not carry a significant part of the story shear or moment.

2. A nonstructural wall does not significantly modify the behavior of adjacent structural elements.

   In other words, the behavior of the structure must be substantially the same whether such walls are present or not.

A conservative value to use for acceptable levels of strain for confined masonry is 0.001 in./in. For a wall 17 ft in height, the allowable story drift is 0.204 in.

D. Stress Calculations

All stress calculations are performed by conventional methods prescribed by the Working Stress Design Method.

E. Analytical Techniques

In general, classical design techniques were used in the evaluation. Refined methods utilizing computer analyses or dynamic analyses were also used on a case-by-case basis.

3.8C.6 JUSTIFICATION OF SELECTED ITEMS IN DESIGN CRITERIA

3.8C.6.1 PURPOSE

The purpose of this subsection is to justify and elaborate on certain selected items contained within the criteria. It is not an attempt to present all the background information that was analyzed in an effort to arrive at a consistent, structurally sound criteria for the reevaluation of existing concrete masonry walls, but is instead a summary of items upon which certain portions of the criteria are based.

3.8C.6.2 DAMPING

Accelerations at the boundaries of the wall were computed without taking the walls into account. Since the walls are nonstructural, nonload-bearing elements, their presence has no effect on the overall response of the structure. Therefore, the evaluation method for the walls is similar to that used for equipment qualification, and damping values realistic for concrete block walls in general were used. Test data and industry concensus indicates that 3% and 5% damping (for operating basis earthquake (OBE) and design basis earthquake (DBE), respectively) are reasonable for the uncracked case and that 4% and 7% are reasonable for the cracked case. Although the higher damping values are reasonable and conservative for the cracked case, 3% and 5% damping were used for both the cracked and uncracked condition since these spectra were already available. The difficulties and uncertainties involved in obtaining response spectra for higher damping values outweigh the benefits obtained by using the resulting lower spectral accelerations in the analysis.

3.8C.6.3 IN-PLANE ACCEPTANCE CRITERIA

In-plane effects may be imposed on masonry walls by the relative displacement between floors during seismic events. However, nonstructural walls are not considered to carry a part of the associated story shear; and any portion which they might carry, determined by their stiffness, is extremely difficult to define. In addition, since the experimental evidence demonstrates that the apparent in-plane strength of masonry walls depends heavily upon the in-plane stress boundary conditions, load or stress on the walls is not a reasonable basis for acceptance criteria.

However, examination of the test data indicates that the gross shear strain of a wall is a reliable indicator for predicting the onset of significant cracking. A significant crack is considered to be a crack in the central portion of the wall extending at least 10% of the wall's width or height. Cracking along the interface between a block wall and adjacent concrete members does not limit the integrity of the wall unless it affects boundary conditions for out-of-plane analysis, and was not considered when examining available test data.

Because of the absence of test data that examined the behavior of masonry walls subjected to simultaneous in-plane and out-of-plane behavior, no general acceptance criteria for the coupled condition were established. Instead, the criteria for in-plane effects are conservative so that a reasonable margin remains for out-of-plane loading. However, boundary condition

requirements, as specified in the acceptance criteria, must be met before that boundary condition may be assumed for out-of-plane loading, even with the existence of a crack along the interface of the wall and adjacent concrete members.

Test data indicate that for confined masonry (supported on the bottom and two sides) significant cracking is initiated at strains in excess of ~ 0.001 in./in. Reinforcing and grout have no effect on this value. The data also show that this strain level is not sensitive to the magnitude of the initially applied vertical load. Since all masonry walls at Plant Hatch meet the criteria for confined walls, the question of shear strain allowables for unconfined walls was not addressed.

For width to height ratios of 0.5 to 2.5, an excellent correlation has been found between a nonlinear analysis of an equivalent diagonal strut and available test data in predicting the onset of significant cracking. Dividing the predicted strain level (using the equivalent strut method) by two, gives a value for allowable strain of ~ 0.001 in./in. Available test data indicate that this value provides a sufficient margin for out-of-plane loading.

3.8C.6.4 OUT-OF-PLANE DRIFT EFFECT

The internal wall moments due to deflections caused by out-of-plane interstory drift were determined for each wall height, location, and orientation and compared to the ultimate allowable moment for each wall. In all cases, calculated moments were found to be less than allowables.

3.8C.6.5 ALLOWABLE STRESSES

Most available test data were analyzed to obtain reasonable allowable stresses for masonry, grout, and reinforcing for different loadings and types of stress. For areas where no test data were available to back up chosen values, conservative engineering judgment was used.

Using test data results utilizing materials, height-to-thickness ratios, and loadings appropriate for the walls under consideration, values presented in the criteria for axial compression have a safety factor of 3 for 93% of all walls tested. A 1.67 increase for DBE still leaves a safety factor of 1.8 for 93% of all walls tested. Assuming that masonry can develop 85% of its specified compressive strength at any section,[a] the value given for flexural compression has a safety factor of 2.6 at the worst-case extreme fiber of the unit.

Values for bearing stress allowables were based on test data used in determining values in the American Concrete Institute code. These values are less than allowed by the ATC-3-06 provision.

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a. This is an assumption practiced for many years.

Comparison of results from recent extensive tests evaluating the shear strength of concrete block walls, with values for shear stress allowables presented in the acceptance criteria, yields the following safety factors:

Unfactored loads	2.0 to 3.0
Factored loads	1.2 to 1.76

Ductility associated with walls subjected to stress levels in the range of factored loads near maximum permissible levels provides an added safety factor.

Allowable reinforcing stresses in reinforced concrete masonry walls are the same as for reinforced concrete and have the same safety factors.

All multiple-Wythe walls are designed as single-Wythe walls. That is, the allowable tension and shear stresses for collar joints are assumed to be zero.

3.8C.7 COMPUTER REEVALUATION OF REINFORCED CONCRETE MASONRY WALLS

3.8C.7.1 INTRODUCTION

The fortran computer code BLOCK WALLS was developed to analyze block walls for axial load and flexural effects due to external and/or seismic loading. The block wall was analyzed as a simplified three-degree-of-freedom beam model. The modal analysis technique was used in conjunction with the response spectrum method to obtain the seismic response of the wall model. An iterative method was used to determine the actual stress and section properties (effective moment of inertia) of a wall section. Convergence criteria were established to verify that the assumed section condition results in the same inertial loading for two successive iterations.

The working stress method for concrete analysis was used for stress calculations. Finally, the calculated stresses were checked against the established allowables.

3.8C.7.1.1 Determination of Section State (Cracked Versus Uncracked) - Iteration Procedure

A. For the first iteration, the wall was assumed to be uncracked.

B. As a result of A above and based on the calculated inertial forces, the section was checked for cracking.

C. If cracked conditions existed, an effective moment of inertia was determined using the following American Concrete Institute (ACI) formula:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 (I_t) + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

$$M_{cr} = f_r \left(\frac{I_t}{y}\right)$$

where:

M_{cr} = uncracked-moment capacity.

M_a = applied maximum moment on the wall.

I_t = moment of inertia of the transformed section.

I_{cr} = moment of inertia of the cracked section.

f_r = modulus of rupture.

y = distance of neutral plane from tension face.

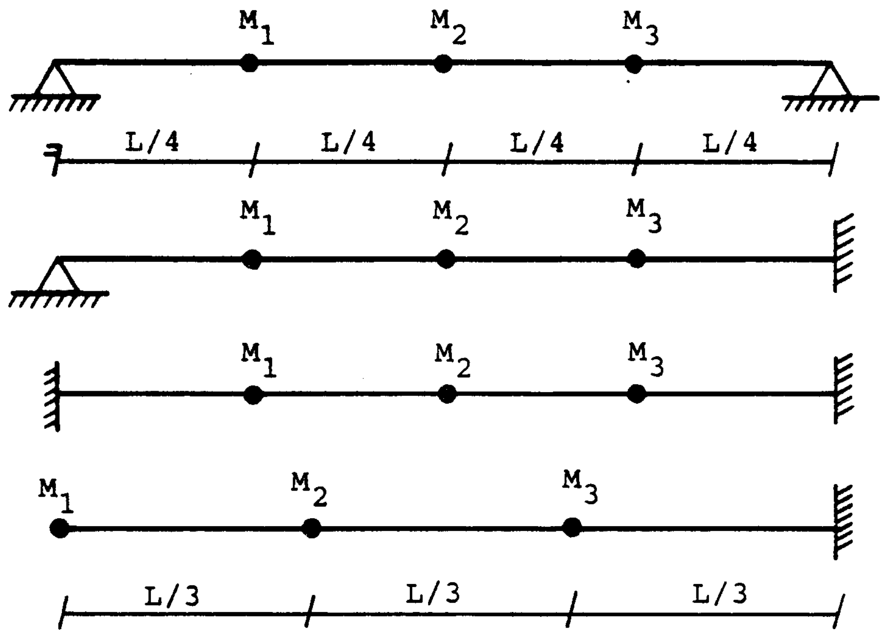
D. A new iteration was initiated to recompute the frequencies, mode shapes, and modal participation factors.

E. The procedure was repeated until convergence was achieved.

3.8C.7.1.2 Seismic Analysis

The wall was represented by a three-degree-of-freedom simplified beam model. A response spectrum analysis was performed yielding the inertial loading to be imposed on the system.

The four types of end conditions allowed for the beam model that was used to perform the analysis are shown schematically below:



3.8C.7.1.3 Stress and Deflection Calculations

The stress calculations were performed for the final configuration of the section using working stress methods. Based on inertial loads, applied external loads, and the computed section stiffness, the beam model deflection was determined.

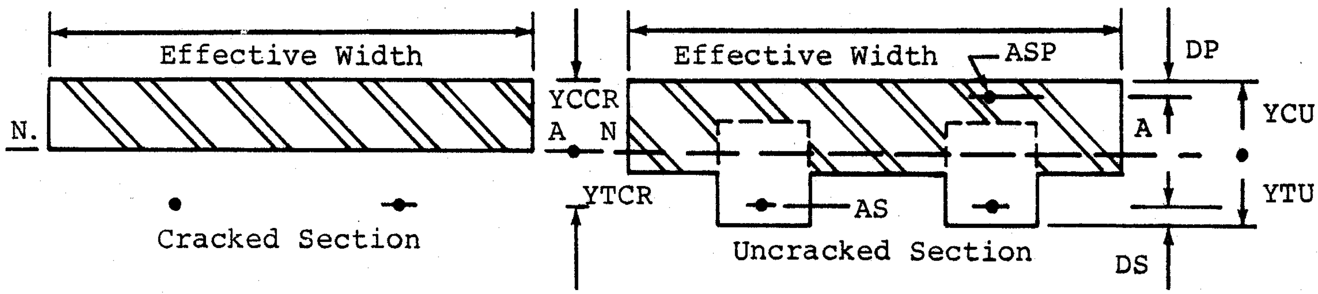
3.8C.7.1.4 Governing Codes

- ACI 531-79 and commentary.
- Uniform Building Code, 1970 edition.
- Other codes as specified.

3.8C.7.2 ANALYTICAL PROCEDURE

3.8C.7.2.1 Block-Wall Stress Calculation

The governing equations for block-wall stress calculations were developed using a working stress approach.



Idealized Section for Analysis

The section properties were calculated based on a transformed section with the block material as a base. The standard concrete analysis equilibrium concept is as follows:

Σ FORCES = 0 or tension = compression

Σ Moment = M = section internal moment

The following equations for stress calculation for bending were obtained:

Case A - Uncracked section

fMB = (M/IUCR) x YCU
fST = NSM x (M/IUCR) x (YTU-DS)
fSC = NSM x (M/IUCR) x (YCU-DP)

Case B - Cracked section

fMB = (M/ICR) x YCCR
fST = NSM x (M/ICR) x (YTCR-DS)
fSC = NSM x (M/ICR) x (YCCR-DP)

For both Case A and Case B, the axial compression stresses were calculated and interaction was checked.

$$(fMA/FMA) + (fMB/FMB) \leq 1.0$$

For axial tension, it was assumed that only the reinforcing steel carries the tension.

The definitions of the variables used in the above equations are:

M = bending moment.

FMB = allowable masonry compressive stress due to bending.

FMA	=	allowable masonry compressive stress due to axial force.
fMB	=	masonry compressive stress due to bending.
fMA	=	masonry compressive stress due to axial force.
IUCR	=	uncracked moment of inertia.
ICR	=	cracked moment of inertia.
YCU	=	distance to extreme fiber in compression (uncracked).
YTU	=	distance to extreme fiber in tension (uncracked).
YCCR	=	distance to extreme fiber in compression (cracked).
YTCR	=	distance to extreme fiber in tension (cracked).
AC	=	transformed compressive area of section.
NSM	=	modular ratio for steel.

3.8C.7.2.2 Eigenvalue Solution and Response Calculation

The following two matrices were determined based upon boundary conditions and structural properties:

Flexibility matrix = $[F]$
Mass matrix = $[\backslash M \backslash]$

Calculate transformation matrix

$$[/M*/] = [/M/^{-1/2}]$$

Using Gauss elimination technique with column pivoting, calculate the structural stiffness matrix:

$$[k] = [F^{-1}]$$

Calculate transformed stiffness matrix $[\bar{k}]$ such that:

$$[\bar{k}] = [/M*/][k][/M*/]^T$$

Tridiagonalize $\left[\bar{k}\right]$ using Householder's method, and evaluate the characteristic value equation:

$$\left[\bar{k}\right](\Phi_i) + W_i^2(\Phi_i) = 0$$

Calculate eigenvalues using Sturm sequence on the tridiagonal matrix.

Calculate eigenvectors using Wilkinson's method on the tridiagonal matrix.

(W_i) are the eigenvalues for the untransformed stiffness matrix [k]. Calculate the frequencies:

$$f_i = W_i/2\pi$$

Eigenvectors $\{\Phi_i\}$ must be transformed into the vectors $\{\Phi_i\}$ of the untransformed matrix:

$$\{\Phi_i\} = [/M^*/]\{\Phi_i\}$$

Compute modal participation factors:

$$(R_i) = \sum_j^n 1[\Phi_{ij}]^T[/M_{ij}/]$$

The modal values of the inertia forces $\{P\}_i$ at the dynamic degrees of freedom for the i^{th} mode were given by:

$$\{P\}_i = (R_i)(a_i)[/M/]\{\Phi_i\}$$

R_i = participation factor for the i^{th} mode.

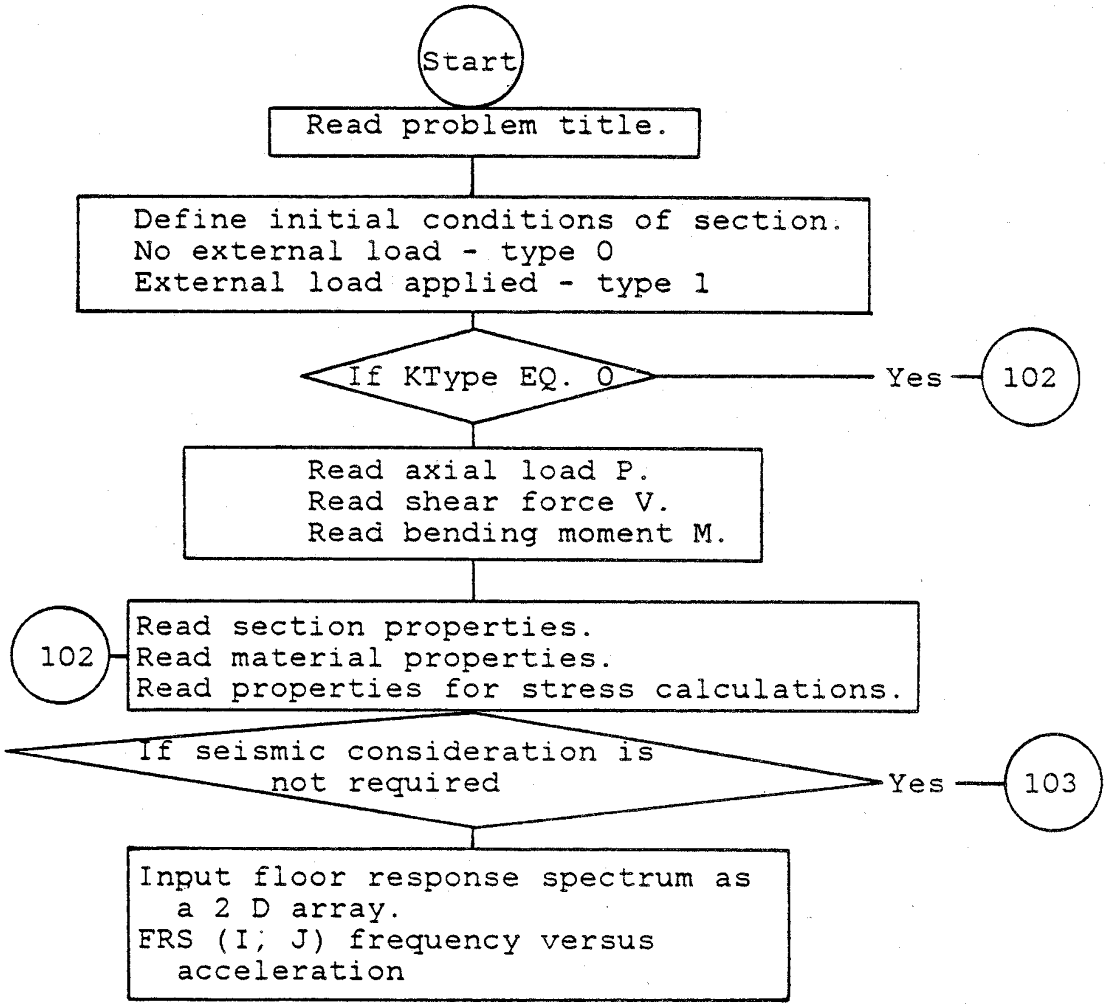
a_i = acceleration for the i^{th} mode.

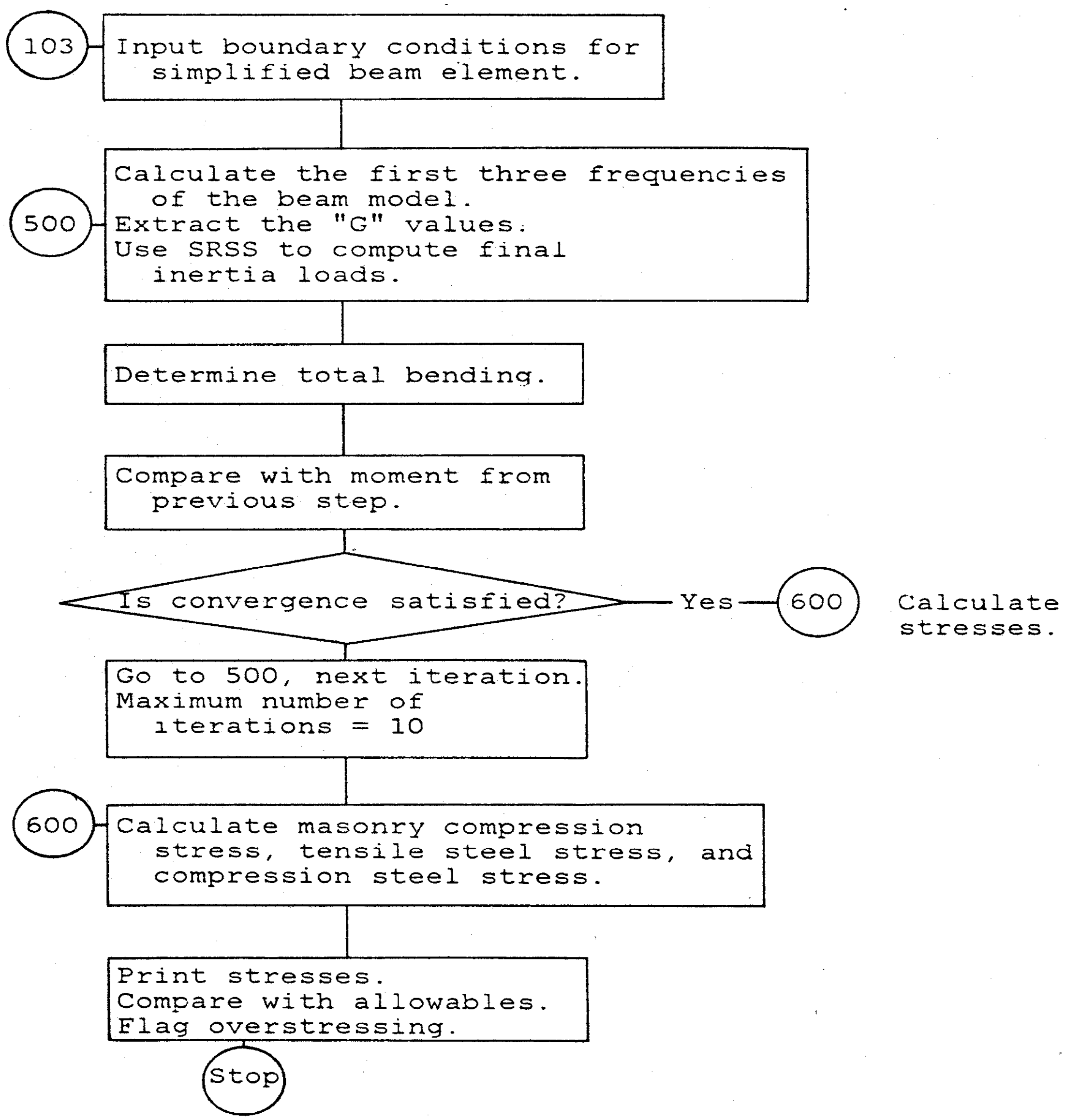
$\{\Phi_i\}$ = mode shape for the i^{th} mode.

Using the calculated inertial loads and the seismic moments, the shear and corresponding deflection were calculated using the "square root of the sum of the squares" (SRSS) method since the modes were not closely spaced.

3.8C.7.3 COMPUTER PROGRAM

3.8C.7.3.1 Flow Chart of the Block Wall Program





3.8C.7.3.2 Hand Calculation for Computer Verification

Two core masonry units were assumed to be 44% solid by volume with running bond. Nominal thickness is 12 in., with two 5 vertical reinforcing bars at 16-in. spacing. Exact dimensions were:

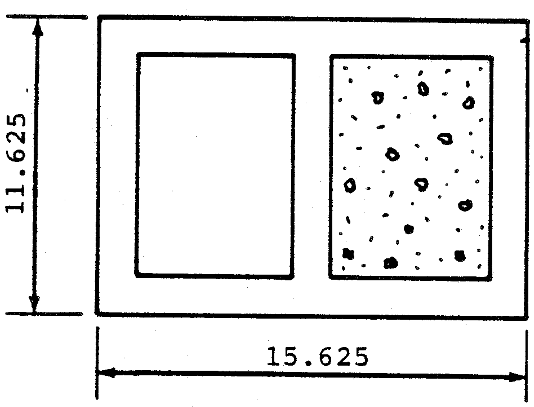
11 5/8 in. x 7 5/8 in. x 15 5/8 in.,
t_s = 1.25 in., t_w = 1.12 in.

A. Uncracked Section Properties

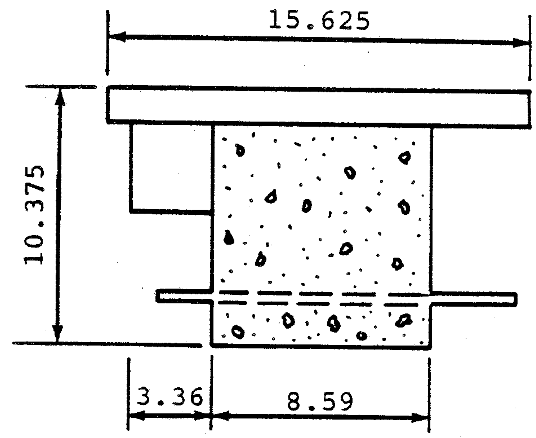
Transform all materials to block material:

$$n_1 = \frac{\text{steel } E_s}{\text{block } E_m} = \frac{29 \times 10^6}{1 \times 10^6} = 29 \qquad n_2 = \frac{\text{grout } E_c}{\text{block } E_m} = \frac{1.4 \times 10^6}{1.0 \times 10^6} = 1.4$$

Tensile steel area	A_s	=	0.31 in.2
Tension steel cover	D_s	=	3.375 in.
Thickness of the wall	H	=	11.625 in.
Effective width of beam	b_{eff}	=	15.625 in.



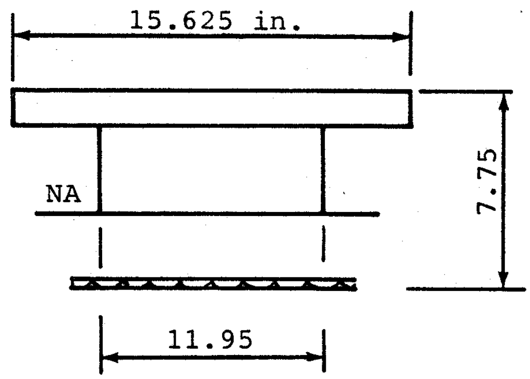
Assumed Section



Tranformed Section

Uncracked moment of inertia = I_t = 1096.2 in.^{4}

B. Cracked Section Properties



Cracked moment of inertia = I_{cr} = 326.7 in.^{4}

C. Calculation of Effective Area (Axial and Shear)

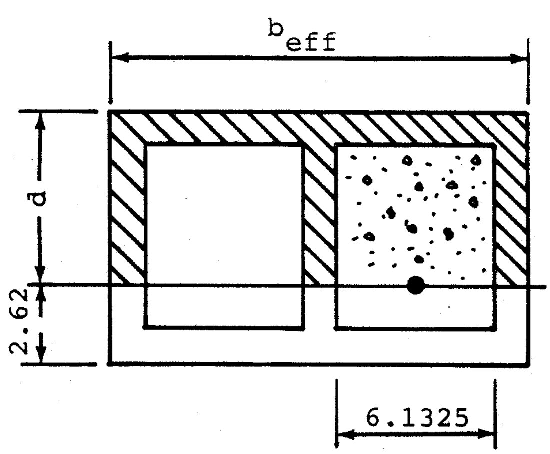
Reference ACI Code 531-79.

AXIAL = 2(1.12 + 6.1325 + 1.12) 1.25 + (9.125 x 1.12) 3 + (6.1325 x 9.125) x 1.4 + 2 x (6.1325 + 1.12) = 144.4 in.^{2}

D. Calculation of Shear Area

Reference ACI Code 531-79.

ASHEAR = (11.625 - 3.875) 1.12 x 3 + 2 x (6.1325 + 1.12) + 1.4 (11.625 - 3.875 - 1.25) 6.1325 = 26.04 + 15.33 + 55.8 = 97.2 in.^{2}



NOTE:

The above calculations are for uniform inertia loadings.

E. Dynamic Inertia Loading

For a 12-in. wall grouted at 16 in. on center, the average weight of a completed wall is 111 lb/ft^2.

Weight/unit length = $\frac{111 \times 16}{144} = 12.3 \text{ lb/in.}$

$$f = \frac{\pi}{2(L)^2}\sqrt{\frac{EIg}{A\gamma}} = \frac{\pi}{2(240)^2}$$

$$\sqrt{\frac{1.6 \times 10 \times 1096.2 \times 386.4}{12.3}} = 5.98 \text{ Hz}$$

Acceleration = 0.28 g

Inertia loading intensity W_i = acceleration times weight/unit length

= 0.28 x 0.0123

Seismic moment = $\frac{(0.28 \times 0.0123)(240)^2}{8} = 24.79 \text{ in.-kips}$

F. Determination of Maximum Bending Stress

Tension = $\frac{29 \times 24.79 \times (7.846 - 2.62)}{326.75} = 11.5 \text{ ksi}$

Compression = $\frac{(24.79 \times 2.528)}{326.75} = 0.192 \text{ ksi}$

3.8C.7.3.3 Computer Calculation

```
                         ****BLOCK WALLS PROGRAM***
                         **** VERSION 6  2/12/81


                                  **********************
                                  *                    *
                                  * UNITS KIPS INCHES  *
                                  *                    *
                                  **********************


      INPUT PROBLEM TITLE (UP TO 10 CHARACTERS)
>EXAMPLE
  DEFINE INITIAL CONDITION OF SECTION

    IF NO EXTERNAL LOAD APPLIED TYPE 0
    IF EXTERNAL LOAD IS APPLIED TYPE 1
>0


 INPUT SECTIONS PROPERTIES AS,ASP,DS,DP,H,L,BEFF,HEIGHT IN.
>.31,,2.62,,12.,240.,15.6,240.


      INPUT IUCR,ICR,YCU,YTU,YCCR,YTCR,AAXIAL,ASHEAR,AC
      WHERE:   IUCR=UNCRACKED INERTIA
               ICR=CRACKED INERTIA
               YCU=DIST. TO EXTREME FIBER IN COMP.(UNCRACKED)
               YTU=DIST. TO EXTREME FIBER IN TENSION (UNCRACKED)
               YCCR=DIST. TO EXTREME FIBER IN COMP.(CRACKED)
               YTCR=DIST. TO EXTREME FIBER IN TENSION(CRACKED)
               AAXIAL=EFFECTIVE AXIAL AREA
               ASHEAR=EFFECTIVE SHEAR AREA
               AC=TRANSFORMED COMPRESSIVE AREA OF SECTION
>1096.22,326.74.4.84,5.535.2.528,7.846,144.4,97.2,34.8
```

```
   INPUT YOUNG MODULUS , AVERAGE WT. PER UNIT LENGTH AND MODULAR RATIOS
>1400.,.0123.29.,1.4



   INPUT COMP.STRENGTH OF MASONRY , COMP. STRENGTH OF GROUT
   AND YIELD STRENGTH OF REINFORCING STEEL
>1.,1.8,40.


   DEFAULT ALLOWABLE STRESSES ARE ACI 531-79**
   IF ACCEPTABLE TYPE 0
   IF UNACCEPTABLE TYPE 1
>0



   CHECK IF SEISMIC LOADING IS TO BE CONSIDERED

   IF OBE SEISMIC CONSIDERATION IS REQUIRED TYPE 1
   IF SSE SEISMIC CONSIDERATION IS REQUIRED TYPE 2
   IF SEISMIC CONSIDERATION IS NOT REQUIRED TYPE 0
>2



   INPUT FLOOR RESPONSE SPECTRUM
   SPECTRUM INPUT IS A 2-D ARRAY DEFINING FREQUENCY INCPS VS ACCELERATION IN 6
   TYPE *N* NUMBER OF POINT USED TO DESCRIBE THE CURVE ?
>8



   INPUT  8 SET OF FREQUENCY VS ACCELERATIONS ENTRIES EACH ON A NEW LINE
>.2,.12
>1.2,.36
>2.,2.45
>2.6,2.45
>2.8,.75
>3.5,.75
>5.99,.28
>1000.,.28


   INPUT ADDITIONAL WEIGHTS AT MASS PTS. 1.2.3
>0.,0.,0.[2]
```

```
   BOUNDARY CONDITIONS ASSUMED FOR SIMPLIFIED BEAM MODEL

   S.S BOTH ENDS TYPE 1
   S.S ONE END FIXED THE OTHER TYPE 2
   BOTH ENDS FIXED TYPE 3
   SIMPLE CANTILEVER TYPE 4
1


*************************************************************************************


                                       **** DATA FROM INTERNAL STORAGE****


                                       ****BLOCK WALLS PROGRAM***
                                       **** VERSION 6  2/12/81








                                 **********************
                                 *                    *
                                 * UNITS KIPS INCHES  *
                                 *                    *
                                 **********************




                                       **** PROB. TITLE:  EXAMPLE    ****

                                        **** SECTION PROPERTIES ****
   AS=  .31          ASP=  .00         DS= 2.62     DP=  .00
   H= 12.0           L=240.0           B= 15.6      D=  7.8
```

INPUT FOR STRESS CALCULATION

IUCR=UNCRACKED INERTIA= 1096.22
ICR=CRACKED INERTIA*= 326.74
YCU=DIST. TO EXTREME FIBER IN COMP.(UNCRACKED)= 4.840
YTU=DIST. TO EXTREME FIBER IN TENSION(UNCRACKED)= 5.535
YCCR=DIST. TO EXTREME FIBER IN COMP.(CRACKED)= 2.528
YTCR=DIST. TO EXTREME FIBER IN TENSION(CRACKED)= 7.846
AAXIAL=EFFECTIVE AXIAL AREA= 144.40
ASHEAR=EFFECTIVE SHEAR AREA= 97.20
AC=TRANSFORMED COMPRESSIVE AREA OF SECTION= 34.80

**** MATERIAL PROPERTIES ****

YOUNG MODULUS= 1400.00
AVERAGE WT. PER UNIT LENGTH= .01230000
MODULAR RATIOS= 29.0 1.4
COMPRESSIVE STRENGTH OF MASONRY= 1.0
COMPRESSIVE STRENGTH OF GROUT= 1.8
YIELD OF REINFORCING STEEL= 40.0

** SSE SEISMIC CONSIDERATION FOR THIS PROBLEM **

FLOOR RESPONSE SPECTRUM DEFINITION

F	G
.20	.12
1.20	.36
2.00	2.45
2.60	2.45
2.80	.75
3.50	.75
5.99	.28
1000.00	.28

ADDITIONAL WEIGHTS AT MASS PTS. ARE:

ADDU1= .000 ADDU2= .000 ADDU3= .000

```
   **BEAM MODEL IS S.S AT BOTH ENDS**



 *** FREQUENCIES ARE ***    5.989    23.790    50.511

***MODAL PARTICIPATION FACTORS ARE***      .07         .00        .01

***ACCELERATIONS ARE ***        .280     .280         .280


***SEISMIC MOMENT=       25.9KIPS.IN


                                *****RESULTS OF ANALYSIS*****
   MASONRY COMPRESSIVE BENDING STRESS=   .2007KSI         ALLOWABLE =     .825KSI
   MASONRY AXIAL COMPRESSIVE STRESS=   .0000KSI           ALLOWABLE =     .394KSI
   TENSILE STEEL STRESS=   12.0305KSI                     ALLOWABLE =   36.000KSI
   COMPRESSIVE STEEL STRESS=     .0000 KSI                ALLOWABLE =   36.000KSI
   MASONRY SHEAR STRESS=     .0031KSI                     ALLOWABLE =     .058KSI
   MAXIMUM DEFLECTION =   .093572  IN.



   DO YOU WANT TO RUN BLOCK WALL AGAIN     YES TYPE 1     NO TYPE 0
>0
```

3.8C.7.3.4 Comparison Between Hand Calculation and Computer Calculation

	BLOCK WALL Program	Hand Calculation
Natural frequencies (Hz)	5.99, 23.79, 50.51	5.98
Seismic accelerations (g)	0.28, 0.28, 0.28	0.28
Seismic moment (in.-kips)[(a)]	25.9	24.79
Masonry compressive stress (psi)[(a)]	201	192
Reinforcing steel stress (psi)[(a)]	12,030	11,500

a. The program calculates a higher value because of the inclusion of the second and third modes.

SUPPLEMENT 3.8D

DETERMINATION OF EXPANSION ANCHOR BOLT LOADS IN PIPE SUPPORT BASE PLATES

3.8D.1 SUMMARY

This report describes a method for determining the anchor bolt loads in steel base plates supporting Seismic Category I piping systems. The anchors in question are of the expansion type. The loads are applied to the base plate through some type of attachment, usually concentric with the base plate, and could comprise of moments and forces in three directions. A review of the typical base plates used in supporting the subject piping systems indicates that the majority of them have either a four-, six-, or eight-bolt connection. The plate thicknesses usually vary from 1/2 in. to 1 1/2 in. and are not generally stiffened. The present formulation will, therefore, be devoted to base plate anchorage systems with aforementioned physical characteristics.

From an analytical standpoint the load distribution in a base plate anchorage system is fairly complex, and it is necessary, therefore, that certain simplifying assumptions be made to arrive at conservative yet practical solutions. However, such assumptions should take into consideration the following parameters, which might affect the load distribution in the anchorage system.

- Flexibility of the base plate: consideration of bending effects.
- Bolt stiffness: based on available load displacement data.
- Prying action.

For expansion anchor bolts, prying action will not be critical for the following reasons:

- Where the anchorage system capacity is governed by the concrete shear cone, the prying action would result in an application of an external compressive load on the cone and would not therefore affect the anchorage capacity.
- Where the bolt pullout determines the anchorage capacity, the additional load carried by the bolt due to the prying action will be self-limiting. With the bolt stiffness decreasing with increasing load, at higher loads the bolt extension will be such that the corners of the base plate will lift off, and the prying action will be relieved. This has been found to occur when the bolt stiffnesses in the finite element analysis were varied from a high to a low value to correspond typically to the initial stiffness and the stiffness beyond the allowable design load.

3.8D.2 METHOD OF ANALYSIS FOR ANCHOR BOLT LOADS

In general, the finite element method of analysis may be used to analyze the base plates under consideration. However, such an approach will be both time consuming and expensive considering the number of base plates involved. A quasi-analytical approach has been formulated taking into account the base plate flexibility and the bolt stiffness. The results of the quasi-analytical method have been verified with appropriate finite element solutions and have shown good correlation for the typical cases studied.

3.8D.2.1 INTRODUCTION

The purpose of this study was to develop an analytical method for determining tension loads on expansion anchors used as anchors for pipe support base plates. Finite element analysis(1) served as a data base for developing less expensive and less time consuming analytical methods. The method that is presented as a result of this study uses plate flexibility and bolt stiffness as the primary parameters. This method will be computerized for four-, six-, and eight-bolt patterns.

3.8D.2.2 ANALYSIS

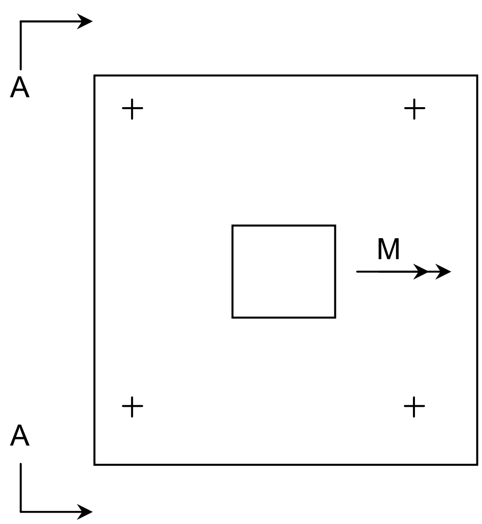
In the quasi-analytical model presented here, the plate is primarily treated as a beam on elastic springs. Base plates with three different bolt configurations were considered.

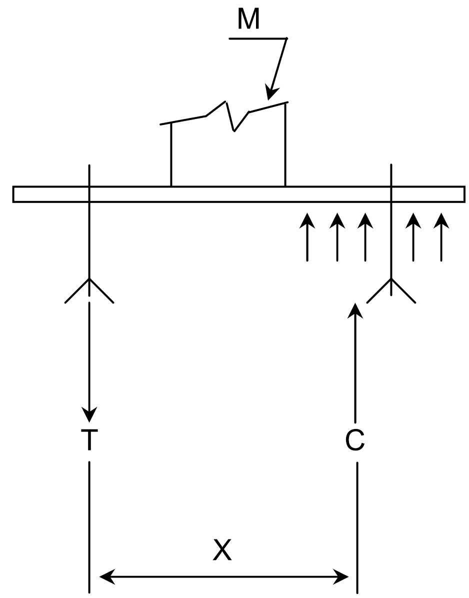
3.8D.2.3 ASSUMPTIONS

- Symmetrical bolt patterns.
- Centroidal loading.
- Attachment dimensions small compared to the plate dimensions.
- Units for all variables.
  force = kips
  length = in.

3.8D.3 FOUR-BOLT PATTERN: MOMENT- AND TENSION-LOADING CASES

Given a plate with a four-bolt pattern and a moment about one axis: This plate will be modeled as a beam.





Section A-A

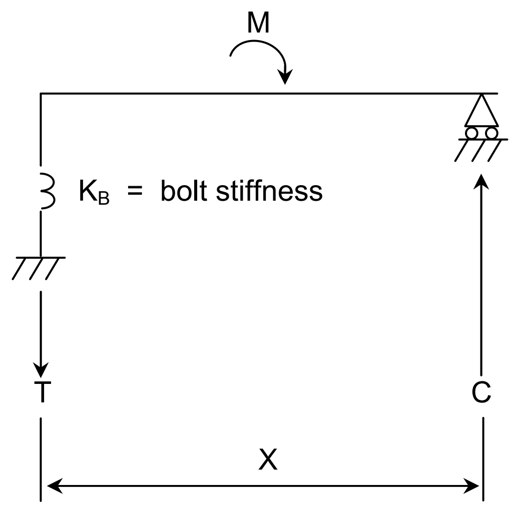
$$T(X) = C(X) = M$$

where:

T = total tension (kips).

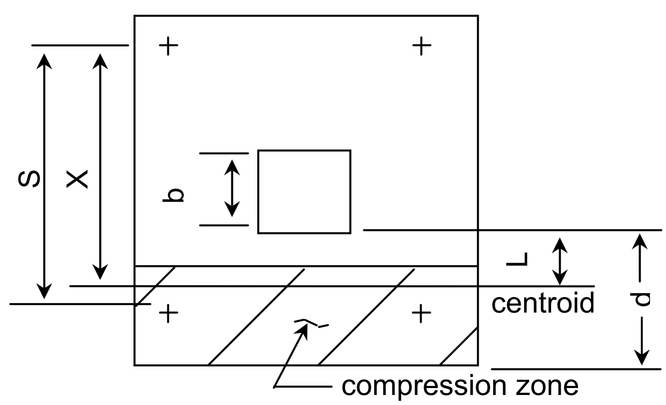
C = resultant of compressive stress block (kips).

The beam will be idealized as being supported at the location of the compressive force resultant. Therefore, if the compression centroid can be located, X becomes known, and T can be calculated.



$T \bullet (X) = M$

For a four-bolt pattern loaded centriodally:



$\therefore X = \frac{S}{2} + \frac{b}{2} + L$

conceptually, L = function (t, d, K_B)

where:

L	=	distance from edge of attachment to the center of compression (in.).
t	=	plate thickness (in.).
D	=	distance from edge of attachment to the edge of the plate (in.).
K_B	=	bolt stiffness (kips/in.).

Based on a number of finite element analysis results (i.e., varying T, d, and K_B), the following empirical relationship was derived:

$$L = 3.5\left[\left(\frac{t}{d}\right)^{2/3}\left(\frac{44}{K_B}\right)^{1/3}\right](d) \tag{1}$$

where:

$L \leq d$

Once L is calculated, total tension (T) and bolt load (F_T) can be found:

$$T = \frac{M}{\frac{S}{2} + \frac{b}{2} + L} \tag{2}$$

$$F_T = \frac{T}{2} = \frac{M}{S + b + 2L} \text{ For centriodically loaded four-bolt patterns only.} \tag{3}$$

This method can be extrapolated for use with combined loading cases.

For biaxial bending:

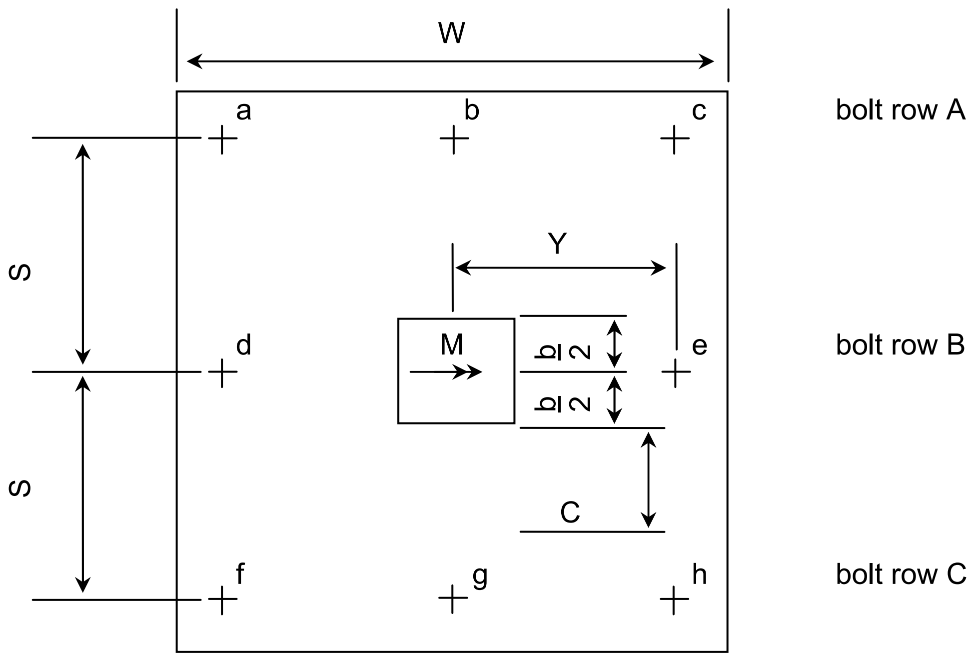
$$\text{Critical } F_T = \frac{M_X}{S_x + b_x + 2L_X} + \frac{M_Y}{S_y + b_y + 2L_Y} \tag{4}$$

For combined bending and tension:

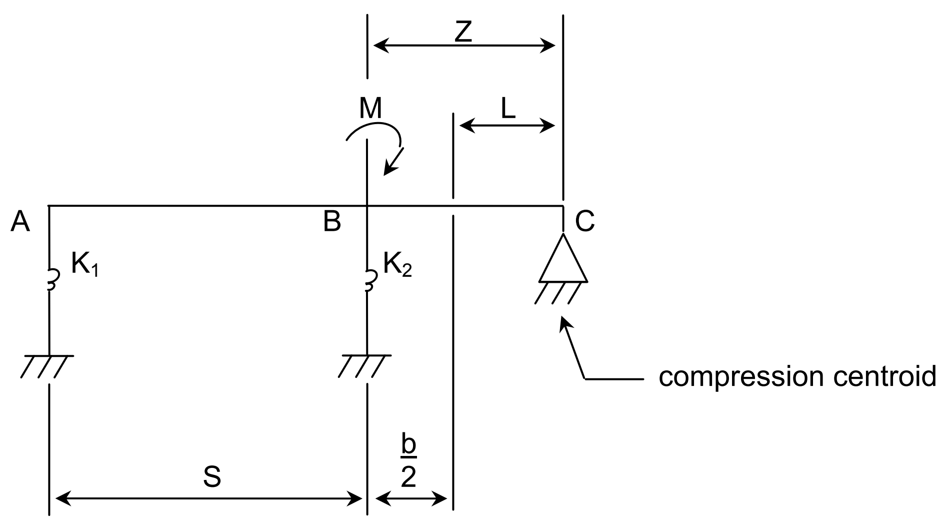
$$\text{Critical } F_T = \frac{M}{S + b + 2L} + \frac{T}{4} \tag{5}$$

Since L varies with t, d, and K_B, the method for finding L can be used for many plate and bolt patterns. Once L is known, the plate can be modeled as a beam on springs. The beam can be solved by various methods, and the total tension force for any row of bolts can be calculated. This will be demonstrated for six- and eight-bolt patterns in the following details.

3.8D.4 EIGHT-BOLT PATTERN: MOMENT-LOADING CASE



Beam model:



K_B = bolt stiffness

$$I = \frac{W\,t^3}{12}$$

The reactions for this indeterminate beam model can be solved using the principle of virtual work. The following equations were derived for eight-bolt patterns:

$$Z = \frac{b}{2} + L$$

where: L is determined from equation 1.

$$EI = 2417\ Wt^3\ (\text{kips-in.}^2)$$

Redundants are taken at C:

$$-EI\delta_{CO} = \frac{EIM(K_1 + K_2)}{S^2 K_1 K_2}\left[Z + \left(\frac{K_1}{K_1 + K_2}\right)S\right] - \frac{MZS}{3} \tag{6}$$

where: δ_{CO} = deflection at C due only to M:

$$EI\delta_{CC} = \frac{EI}{S^2 K_1 K_2}\left[K_1 S^2 + 2K_1 ZS + (K_1 + K_2)Z^2\right]\frac{Z^2}{3}[Z + S] \tag{7}$$

where: δ_{CC} = deflection due to a 1^K - force applied at C:

$$\text{Reaction at } C = R_C = -\frac{EI\delta_{CO}}{EI\delta_{CC}} \tag{8}$$

$$\therefore R_A = \frac{[M - Z(R_C)]}{S}$$

$$R_B = R_C - R_A$$

As the plate gets wider and Z becomes small compared to Y, the two middle bolts cannot be lumped together as one support with $K_2 = 2K_B$. K_2 will be something less than $2K_B$. The following expression for K_2 yielded results that were in good agreement with finite element method results:

$$K_2 = 2K_B\left(\frac{Z}{Y}\right)^2 \le 2K_B \tag{9}$$

For plate sizes generally used in pipe supports, this width effect will have negligible effect on row A; i.e., the stiffnesses of the three bolts can still be lumped together in the beam model.

The reactions in the beam model are now known. The reaction at any one support is the total tension in that row of bolts. To distribute the load to the bolts:

For row B from symmetry, tension per bolt = $F_{Td} = F_{Te} = \frac{R_B}{2}$ (10)

For row A, the relative stiffness of the plate and the bolts and the bolt distance from the attachment will affect the load distribution between the middle and corner bolts. Evidently, the bolt closest to the attachment will carry more load, and, if the attachment size is small, the bolt-to-attachment distance may be substituted by the distance of the bolt to the center line of the plate. Thus, tension in the middle bolt b:

$$F_{Tb} = \alpha\left[f\left(\frac{K_B}{\frac{EI}{\ell_1^3}}\right)\right]\left[\frac{\frac{1}{L_M}}{\frac{1}{L_M}+\frac{2}{L_C}}\right](R_A) \tag{11}$$

where:

L_M = distance from plate center to bolt b.

L_C = distance from plate center to bolts a and c.

ℓ_1 = S + Z.

α = constant.

Based on several finite element method analyses, the following expression of F_{Tb} was derived:

$$F_{Tb} = \lambda(R_A) = \frac{2}{3}\left[\frac{K_B}{\frac{EI}{\ell_1^3}}\right]^{1/4}\left[\frac{\frac{1}{L_M}}{\frac{1}{L_M}+\frac{2}{L_C}}\right](R_A) \tag{12}$$

with the limits $0.333 < \lambda < 1.0$ corresponding to very rigid and very flexible plates.

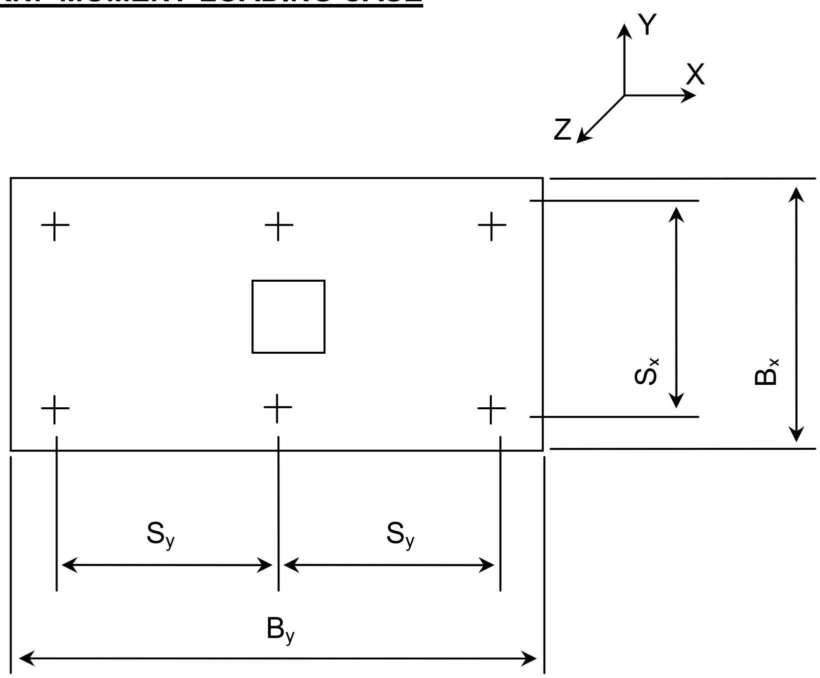
Tension in the corner bolts is given by:

$$F_{Tc} = F_{Ta} = \frac{R_A - F_{Tb}}{2} \tag{13}$$

$$F_{Tf} = F_{Tg} - F_{Th} = 0 \tag{14}$$

For biaxial bending, the resultant bolt forces will be determined by superposition.

3.8D.5 SIX-BOLT PATTERN: MOMENT-LOADING CASE



The six-bolt pattern can be solved by using a combination of the equations for four-bolt and eight-bolt patterns.

For moment about the X-X axis:

A. Use equations 1 and 2 to solve for total tension.

B. Use the eight-bolt distribution equations, 12 and 13, for solving the bolt loads with

$$\ell_1 = \frac{S_X}{2} + Z \text{ and } EI = 2417 B_Y t^3$$

For moment about the Y-Y axis:

A. Use equations 6, 7, and 8 to solve for reactions with

$$K_2 = 2K_B \left(\frac{Z}{Y}\right)^2; \; S = S_Y; \; Y = \frac{S_X}{2} \text{ and}$$

$$EI = 2417\, B_X t^3$$

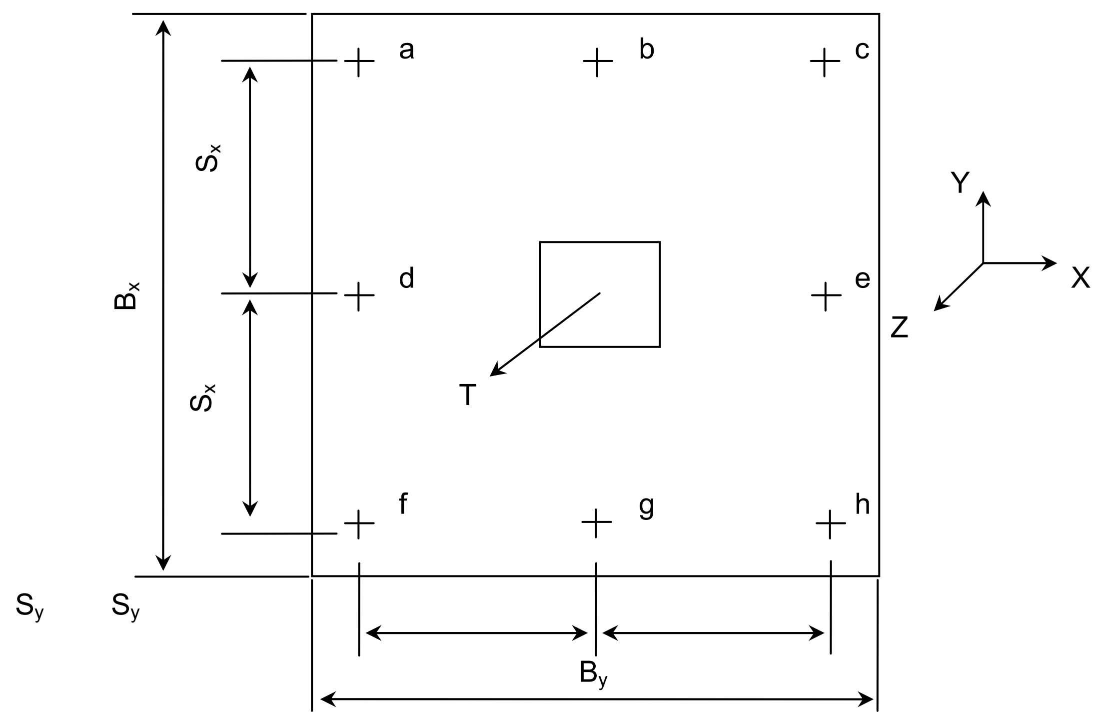
B. Divide the reactions corresponding to each bolt row by two to obtain individual bolt loads.

3.8D.6 SIX- AND EIGHT-BOLT PATTERNS: TENSION-LOADING CASES

Unlike the four-bolt pattern, for the six- and eight-bolt cases the centrally applied tension cannot be distributed equally to all the bolts due to the interplay of bolt and plate stiffnesses and to the relative distances of the bolts from the point of application of the load.

Based on the moment case, it will be assumed that the parametric variable affecting the load distribution will be of the same form as in the moment case. The constant 8/9 for the distribution factors, DFM_x and DFM_y was obtained from finite element analysis results.

3.8D.6.1 EIGHT-BOLT PATTERNS: TENSION-LOADING CASE



T = tension load.

F_T = load per bolt.

Calculate:

$$EI_1 = 2417\ B_X t^3$$

$$EI_2 = 2417\ B_Y t^3$$

$$K_X = \frac{EI_1}{2S_Y}$$

$$K_Y = \frac{EI_2}{2S_X}$$

$$T_X = \left[\frac{K_X}{K_X + K_Y}\right]T;\ T_Y = T - T_X$$

$$L_C = \left[(S_X)^2 + (S_Y)^2\right]^{1/2}$$

$$DFM_X = \frac{8}{9}\left[\frac{K_B(2S_Y)^3}{EI_1}\right]^{1/4}\left[\frac{\frac{1}{S_Y}}{\frac{1}{S_Y} + \frac{2}{L_C}}\right];\ \frac{4}{7} \le DFM_X \le 1.00$$

$$DFM_X = \frac{8}{9}\left[\frac{K_B(2S_X)^3}{EI_2}\right]^{1/4}\left[\frac{\frac{1}{S_X}}{\frac{1}{S_X} + \frac{2}{L_C}}\right];\ \frac{4}{7} \le DFM_Y \le 1.00$$

<u>NOTE</u>:

For plate stiffness varying from infinitely rigid to extremely flexible,

$$\frac{4}{8} \le DFM \le 1$$

Since a rigid plate does not exist, $4/7$ was used as a limit.

$$F_{Tb} = F_{Tg} = [DFM_Y]\left[\frac{T_Y}{2}\right]$$

$$F_{Td} = F_{Te} = [DFM_X]\left[\frac{T_X}{2}\right]$$

$$F_{Ta} = F_{Tc} = F_{Tf} = F_{Th} = \frac{T - 2(F_{Tb} + F_{Td})}{4}$$

If, by above equations, $F_{Td} < F_{Ta}$ or $F_{Tb} < F_{Ta'}$ set $F_{Td} = F_{Ta}$ or $F_{Tb} = F_{Ta}$ as limiting values for rectangular plates.

3.8D.6.2 SIX-BOLT PATTERN: TENSION-LOADING CASE

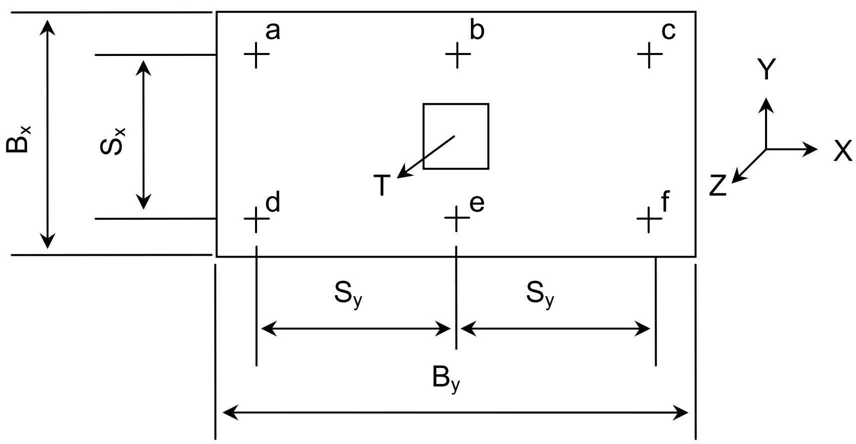
$$EI_1 = 2417\, B_X t^3$$

$$EI_2 = 2417\, B_Y t^3$$

$$K_X = \frac{EI_1}{2S_Y}$$

$$K_Y = \frac{EI_2}{S_X}$$

$$T_Y = \left[\frac{K_Y}{K_X + K_Y}\right]T$$



$$DFM_Y = \left[\frac{8K_B (S_Y)^3}{9EI_2}\right]^{1/4}\left[\frac{\frac{1}{S_Y}}{\frac{1}{S_Y} + \frac{1}{L_c}}\right]; \quad \geq \frac{4}{7} \text{ and} \leq 1.00$$

where:

$$L_C = \left[\frac{S_X^2}{2} + \frac{S_Y^2}{1}\right]^{1/2}$$

$$F_{Tb} = F_{Te}[DFM_Y]\left[\frac{T_Y}{2}\right]$$

$$F_{Ta} = F_{Tc} = F_{Td} = F_{Tf} = \left[\frac{T - 2(F_{Tb})}{4}\right]$$

Based on the above equation, if $F_{Ta}(= F_{Tc} = F_{Td} = F_{Tf}) > F_{Tb} = (F_{Te})$, as may be the case where:

$S_X \geq 2S_Y$, then

$$F_{Ta} = F_{Tc} = F_{Td} = F_{Tf} = F_{Tb} = F_{Te} = \frac{T}{6}.$$

3.8D.6.3 COMPARISON OF RESULTS

Finite Element Method Versus Bechtel Model Sketches of Base Plates Analyzed

A. Four-Bolt Pattern

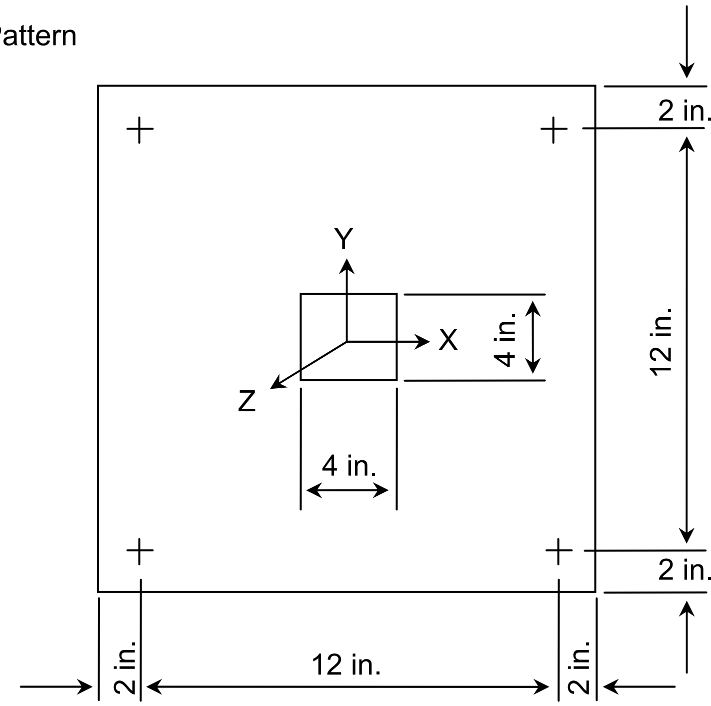
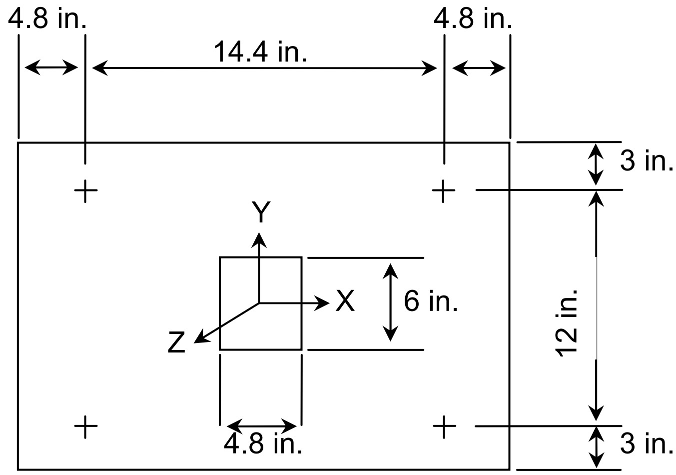


Plate	t[a]	K_B	Loading
1	1/2 in.	44	M_X = 18 kips-in.
2	1/2 in.	44	M_X = 18 kips-in., M_Y = 36 kips-in.
3	1/2 in.	44	M_X = 18 kips-in., F_Z = 4 kips-in.
4	3/4 in.	44	M_X = 18 kips-in.
5	3/4 in.	150	M_X = 18 kips-in.
6	3/4 in.	300	M_X = 18 kips-in.

a. K_B = bolt stiffness (kips/in.); t = plate thickness.



Plate[a]	t	K_B	Loading
7	3/8 in.	44	M_Y = 247.5 kips-in.
8	2 in.	44	M_Y = 247.5 kips-in.
9	1/2 in.	44	M_Y = 247.5 kips-in., M_X = 247.5 kips-in.

a. From Teledyne Engineering Report.[2]

B. Six-Bolt Pattern

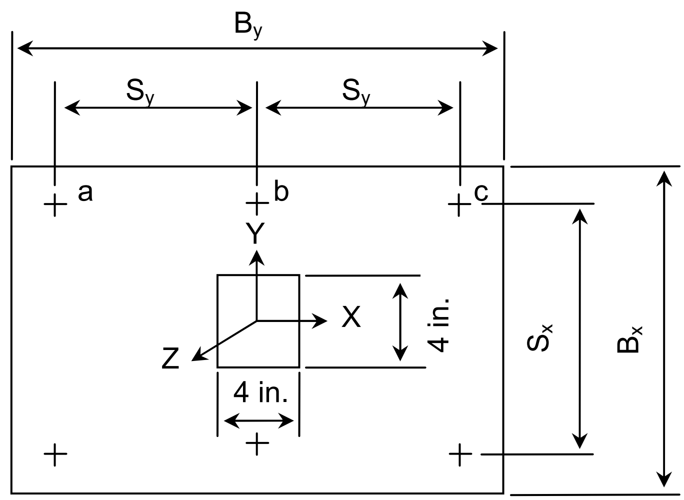


Plate	t	K_B	S_X	S_Y	B_X	B_Y	Loading
1	1/2 in.	44	12	8	16	20	M_X = 36 kips-in.
2	1 in.	440	12	8	16	20	M_X = 36 kips-in.
3	1 in.	44	22.5	4	25.5	12	F_Z = 10 kips
4	2 in.	44	22.5	4	25.5	12	F_Z = 10 kips
5	3/4 in.	44	12	6	16	16	F_Z = 10 kips
6	1 in.	44	12	6	16	16	F_Z = 9 kips

C. Eight-Bolt Pattern

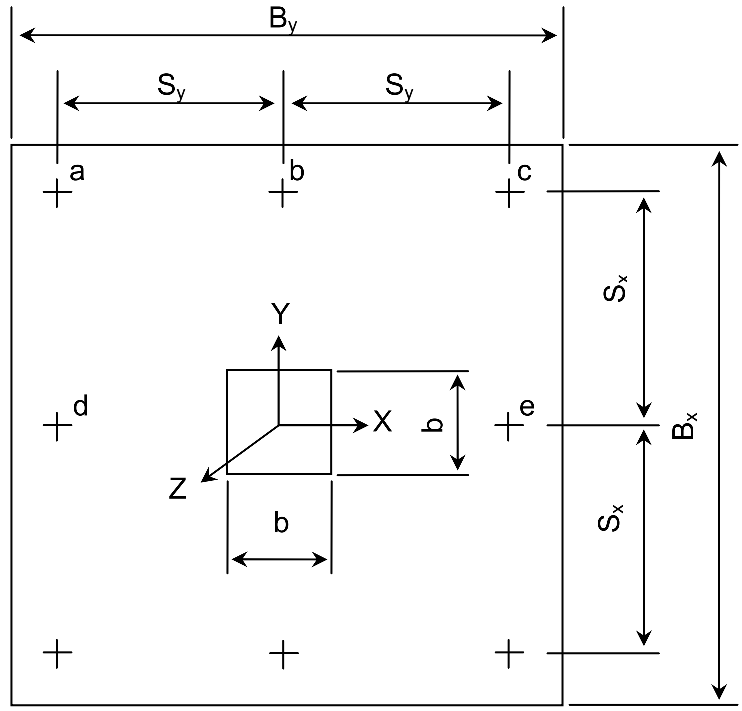


Plate	t	K_B	S_X	S_Y	B_X	B_Y	b	Loading
1	1 1/4 in.	44	12	12	28	28	6	M_X = 10 kips-in.
2	1 1/4 in.	440	12	12	28	28	6	M_X = 180 kips-in.
3	1 in.	300	8	8	20	20	4	M_X = 90 kips-in.
4	1 1/4 in.	150	12	12	28	28	6	F_Z = 16 kips
5	1 1/4 in.	44	12	12	28	28	6	F_Z = 18 kips
6	1 in.	44	6	10	16	24		F_Z = 10 kips

3.8D.6.4 TABULATED RESULTS

A. Four-Bolt Pattern

Load per Bolt (kips)			
Analysis Method Plate	Finite Element	Bechtel Analytical Model	Percent Difference
A (1)	0.75	0.75	0
A (2)	2.08	2.25	+8.2
A (3)	1.71	1.75	+2.3
A (4)	0.64	0.68	+6.3
A (5)	0.75	0.78	+4.0
A (6)	0.78	0.84	+7.7
A (7)	9.12	9.19	+0.8
A (8)	6.12	6.45	+5.4
A (9)	16.61	18.17	+9.4

B. Six-Bolt Pattern

	Tensile Load Per Bolt (kips)					
	Bolts a and c	Bolt b	Bolts a and c	Bolt b	Percent Difference	
Analysis Method Plate	Finite Element		Bechtel Analytical Model		Bolts a and c	Bolt b
B (1)	0.65	1.84	0.64	1.72	-1.5	-6.5
B (2)	0.61	1.96	0.72	1.86	+18.0	-5.1
B (3)	1.68	1.64	1.67	1.67	-0.7	+1.5
B (4)	1.67	1.66	1.67	1.67	0	+0.2
B (5)	1.55	1.89	1.67	1.67	-7.2	+13.5
B (6)	1.45	1.59	1.5	1.5	+3.2	-6.1

C. Eight-Bolt Pattern

Analysis Method Plate	Tensile Load per Bolt (kips)						Percent Difference		
	Bolt a	Bolt b	Bolt d	Bolt a	Bolt b	Bolt d			
	Finite Element			Bechtel Analytical Model			Bolt a	Bolt b	Bolt d
C (1)	1.89	2.64	0.75	1.94	2.70	0.92	+2.69	+2.3	+17.0
C (2)	1.55	5.26	1.46	1.58	5.14	1.47	+1.9	-2.3	+0.7
C (3)	1.22	3.32	0.88	1.32	3.23	0.85	+8.2	-2.6	-3.0
C (4)	1.08	2.92	1.46	1.08	2.92	1.46	0	0	0
C (5)	0.83	1.17	0.59	0.86	1.14	0.57	+3.6	-2.6	-3.5
C (6)	0.99	1.95	1.06	0.96	2.04	1.01	-3.1	+4.4	+5.2

REFERENCES

1. "ANSYS" Engineering Analysis System, developed by Swanson Analysis System, Inc.

2. Diluna, L. J. and Flaherty, J. A., "An Assessment of the Effect of Plate Flexibility on the Design of Moment-Resistant Base Plates," Teledyne Engineering Services (submitted to American Society of Mechanical Engineers for publication).

3.9 MECHANICAL SYSTEMS AND COMPONENTS

3.9.1 DYNAMIC SYSTEM ANALYSIS AND TESTING

3.9.1.1 Vibration Operational Test Program

The Hatch Nuclear Plant-Unit 2 (HNP-2) design is in conformance with Sections NB-3622, NC-3622.3, and ND-3611 of the American Society of Mechanical Engineers (ASME) Code, Section III, which requires that the piping be arranged and supported with consideration of vibration and that the designer is responsible by design and observation under startup or initial operating conditions to ensure that vibration of piping systems is within acceptable levels.

The test program is designed through observation to identify any excessive vibration anywhere on a given piping system, not just at selected points. Piping systems are monitored to verify that the piping and piping restraints will withstand dynamic effects due to normal operation; flow-induced, steady-state vibration; and anticipated transients. Also, piping vibrations are monitored to ensure that they are within acceptable limits. If an observed displacement is judged to be excessive anywhere in the system during preoperational testing or during normal operation, the displacement will be measured and corrective action taken for Class 1, 2, and 3 systems. The test program is standard practice and is a continuing program of observation and inspection to identify past or present cases of excessive vibration.

Vibration and transient response for main steam, recirculation, and feedwater piping inside the drywell is measured by the instrumentation used to measure thermal expansion in these systems. Other Class 1, 2, or 3 piping systems throughout the plant are the subject of construction acceptance testing, startup testing, or surveillance during normal operation or shutdown as dictated by accessibility considerations.

Acceptance criteria for piping system vibration include:

A. Flow-Induced, Steady-State Vibration

The measured range of displacements for the main steam, recirculation, and feedwater lines is reported to the system designer or a stress analyst for evaluation and resolution.

An evaluation of flow-induced, steady-state vibration in other piping systems shall be made as dictated by visual observation.

B. Transient Response

Data acquisition in support of the anticipated, rapid, short duration plant transients, such as turbine stop valve closure or main steam relief valve operation, is returned to the piping designer to allow verification of the conservatism of the piping analysis. Other transients which may occur during plant testing and operation are

evaluated as dictated by observation of piping systems during the transient or by visual inspection of the piping system following the transient.

If any vibration is determined to be greater than that expected in the piping design analysis for a given piping system, resultant stresses will be calculated. These stresses will be appropriately combined with the stresses caused by dead weight, earthquake, and pressure and compared to the allowable primary stresses. If the allowable code stresses are exceeded, restraints will be installed to eliminate the displacements or reduce them to acceptable levels. If during the test the piping systems restraints are determined to be inadequate or damaged, corrective restraints will be installed and the test program for identifying excessive vibration by inspection and observation will continue in order to verify that the vibration has been reduced to an acceptable level.

The transients specifically noted in paragraph 3.9.1.1.1 below are considered to be reasonable checks of system responses to reasonably severe transients. Absolute worst condition transients for which these Class 1, 2, and 3 systems were designed and analyzed are noted in the System Design Specifications. All other system transients were reviewed and determined to be insignificant. In any event, Georgia Power Company (GPC) observed and inspected Class 1, 2, and 3 system responses to all transients experienced during the preoperational test program and up to March 22, 1997, as a matter of course. Since March 22, 1997, Southern Nuclear Operating Company, as the exclusive operating licensee, has observed and inspected Class 1, 2, and 3 system responses to all transients experienced during operation.

All piping supports were installed and adjusted to proper specifications prior to testing. For any of these components whose modification may have a significant effect on a system, the magnitude of that effect was evaluated by the designer or a stress analyst and, if necessary, that portion of the system was monitored for effects of vibration or transient response during a similar event.

Any displacement which is judged to be significant will be analyzed for resultant stresses and corrections will be made, if necessary, for all systems.

3.9.1.1.1 ASME Class 1 and 2 Components

Valves, piping, and supports associated with the following components are visually checked for excessive vibration:

Component	Quantity
High-pressure coolant injection (HPCI) pump and turbine	1
Reactor core isolation cooling (RCIC) pump and turbine	1
Recirculating pumps	2
Residual heat removal (RHR) and core spray (CS) pumps	6

The specific conditions for which vibrations are checked are as follows:

- Design flowrate.
- Minimum flowrate (shutoff flow), as feasible.
- Maximum flowrate (runout flow), as feasible.
- Startup.
- Shutdown.
- Other specific transient or operating conditions (discussed below) which might be expected to produce abnormal vibration or pressure pulsations.

A. Restart Testing Following Recirculation Piping Replacement

A restart test program was performed following fuel cycle 4 after replacement of the recirculation piping due to intergranular stress corrosion cracking. The restart test program verified that system performance is satisfactory for safe operation of the station at all expected operating conditions. During the restart program, calibration of the affected systems, based on the new recirculation system configuration, was Pre-Operational and Startup Specification and Data Sheet. Plant-specific restart procedures were conducted or verified to be correct; and piping expansion and vibration were monitored to confirm design values as specified in the Recirculation completed, and a detailed schedule of testing was followed.

1. Heatup from Ambient to Rated Temperature and Pressure

Following satisfactory installation of the replacement recirculation pipe, special instruments were installed in the drywell at preselected locations on the recirculation, RHR, and the reactor water cleanup (RWC) piping to monitor piping vibration, expansion and strain. In the main control room (MCR), signal taps were installed to monitor selected process signals. The following tests were conducted during this phase of the restart program:

a. Reactor vessel process temperatures were monitored during heatup to determine that specified temperature limits were not exceeded (STI-16).

b. System expansion (STI-17) checks were made during heatup to verify freedom of motion of major recirculation system equipment and piping.

c. Strain measurements were taken at rated temperature and pressure to confirm design values.

d. Nuclear instrumentation was monitored to verify proper operation following cable replacement after the outage.

e. Safety relief valves (STI-26) were tested to verify proper discharge and that no blockage exists in the safety relief valve discharge lines.

2. From Rated Temperature to 100% Power

Reactor power was increased to 100% in a controlled fashion with two major testing plateaus:

- Along the 75% rod line.

- Along the 100% rod line.

The following tests were conducted:

a. Vibration measurements (STI-33) were performed to determine the vibration characteristics of the recirculation piping, RHR, and RWC piping as reactor power was increased.

b. Control system stability (STI-23, 29) was demonstrated for both the feedwater and recirculation system controllers.

c. The 75% and 100% load lines were reverified, and new jet pump baseline data were obtained.

d. The jet pumps were calibrated (STI-35), based upon the current jet pump riser flow distribution and current recirculation pipe design.

e. Recirculation pump trips (STI-30) were conducted to verify that vibration and system performance are within design values.

f. A cavitation search (STI-30) was conducted to verify that the low feedwater flow interlock is still adequate to prevent jet pump and recirculation pump cavitation.

g. The recirculation pump high-speed stops were reset, based upon the current recirculation system configuration.

An analysis of the recirculation system has been completed to determine the potential for damage due to water hammer. Since the recirculation system is filled with water and is self-venting by configuration, the problem area of most concern is the potential for damage due to pressure waves caused by rapid changes in flow velocity.

The recirculation system minimum valve closure time of 30 s is much too slow to cause water hammer. If instantaneous seizure of the recirculation pump should

occur, stoppage of the impeller does not result in a large instantaneous change in flow velocity as would be required for water hammer effects to occur. This is because a large open flow area still exists through the pump impeller when it is stopped.

When the pump seizes, it changes from a device which aids the flow of water to a device which impedes its flow. Two pressure waves which modify the flow are sent out from the pump. The wave that travels up the suction pipe is a compression wave, while the wave traveling down the discharge pipe is a rarefaction wave. Evaluation of the pressure waves, using equations of the form $\Delta P = C\Delta V$, results in a wave strength of < 200 psi. That is, the pressure in the suction pipe is < 200 psi above normal operating pressure, while the pressure in the discharge pipe is < 200 psi below normal operating pressure. This change in pressure is within the design capability of the piping system.

Since there is no further energy input to the system after the pump seizes, any conceivable combination of pressure wave reinforcement in the piping system caused by reflections from valves, elbow, orifices, etc., cannot exceed the strength of the original wave from which they were subdivided.

The water hammer effect in the recirculation system is, therefore, negligible.

B. For Main Steam System

Flow-induced vibrations have been measured on earlier plants (Dresden 2), where the conditions were generally similar, but where the piping configurations did not closely resemble those of HNP-2. During the startup of HNP-2, the instrumentation which exists for determining expansion and movement of the steam lines was used wherever possible to augment further the data on flow-induced vibrations.

1. Description of Mathematical Model and Analytical Technique

The mathematical model of the main steam line is constructed to simulate the physical dynamic characteristics of the piping system. The model consisted of lumped masses at discrete points connected by weightless elements. The elements were assigned cross-sectional and elastic properties identical to those of the pipe section which the element represented. Each lumped mass point included the mass of the piping and insulation in its vicinity. The masses of valves were lumped independently because of the increased local weight concentrations these items represented.

The main steam piping is subjected to two transient conditions that produce dynamic loads acting on it. These two conditions are turbine stop valve closure and relief valve lifting.

a. Safety Relief Valve

The safety relief valves discharge through an enclosed piping system which carries the steam to the suppression pool. Under the conditions of steady-state flow, the forces associated with flow acting on the piping system are self-equilibrated and do not create bending moments in the piping system. These safety relief valves do not cause large steady forces acting on the system as do the more common safety relief valves discharging through an elbow directly to the atmosphere. The safety relief valves discharging into an enclosed piping system do create momentary imbalanced forces acting on the piping system during the first few milliseconds following safety relief valve lift. These loads are caused by the following:

1.) A movement by change of force exerted on the discharge piping during the first few milliseconds when the safety relief valve has started to open and prior to the time steady-state flow has been established.

2.) A fluid transient load will be exerted on the safety relief valves and respective piping during the first few milliseconds when the valve is opening and prior to the time steady-state flow has been established. (With steady-state flow, the dynamic flow reaction forces will be self-equilibrated in the safety relief valve discharge piping).

3.) Forces are produced on the discharge piping system immediately after the relief valve lifts, due to fluid momentum changes at each elbow.

4.) These forces vary with time and with position along the discharge pipe because the flowrate through the relief valve is a function of time.

b. Turbine Stop Valve Closure

Prior to turbine stop valve closure, saturated steam flows through each main steam line at nuclear-boiler-rated pressure and mass-flowrate. Flow stops at the upstream side of the valve at the instant stop valve closure is achieved. However, flow of steam into the main steam line from the reactor vessel continues until the fluid compression wave produced by the stop valve closure reaches the vessel nozzle. Repeated reflections of this wave at the vessel end of the main steam line and at the turbine stop valve end produce time-varying, segmented forces at each elbow in the main steam line and are calculated by the TSMOOD computer program. The dynamic analysis of the main steam and relief valve piping system for turbine stop valve closure is performed using the time-history analysis by direct integration method

on the SAP4 (structural analysis program) computer program. No test data are available to compare with the results of the analysis.

A conservative dynamic analysis has been made for both of these transients to determine stress levels, rather than by actual test measurements. The moments from these loads were combined individually with earthquake moments by the square-root-of-the-sum-of-the-squares method. These combined moments will then be added to deadweight moments to determine bending stresses. The bending stresses plus the longitudinal pressure stresses will be shown to be < 2.25 S_m, in accordance with the principles and methods of ASME, Section III. A further justification for the use of the square-root-of-the-sum-of-the-squares method and for the load combinations with respect to the ASME Code, Section III criteria is provided below.

2. Square-Root-of-the-Sum-of-the-Squares Method

The square-root-of-the-sum-of-the-squares method is used when combining the operating basis earthquake (OBE) loads with the operational transients (turbine stop valve closure and safety relief valve blowdown loads.) The use of this method is justified by the fact that earthquake excitation is a random process with amplitudes increasing to a peak and then decaying, and the fact that the amplitude of the operational transients (turbine stop valve closure and safety relief valve blowdown) loads also rise to a peak and then decay. Therefore, considering that the dynamic responses of such loads possess varying frequencies, amplitudes, and random-phase relationship with respect to each other, the square-root-of-the-sum-of-the-squares method is adequate for calculating the design loads. The only piping system for which this method of load combination was used was the main steam system.

3. Load Combination

The combination of the OBE + operational transients (such as turbine stop valve closure and relief valve opening loads) is considered an emergency condition and evaluated against emergency acceptance criteria of the ASME Code, Section III, Equation 9 (2.25 S_m).

The upset condition was not selected for the following reasons:

a. The OBE loading as characterized has an encounter probability of < 10^{-2} per reactor year; thus, it is an emergency condition on the probability scale. This justifies evaluation of the consequences of an OBE event (including the operational transient which is assumed to result from the OBE) against the emergency condition criteria.

b. The turbine stop valve closure and the safety relief valve discharge loads are operative only during a very small portion of the seismic cycle; thus, there is a reduced probability of the effects combining adversely.

The number of maximum stress cycles due to the event combination of OBE + operational transients are too small to be used as a design condition (five maximum cycles per OBE for the event [OBE + relief valve] and three maximum cycles per OBE for the [OBE + turbine stop valve] event.

c. It is more reasonable to use the emergency stress limit (table 3.9-1) rather than the upset.

The probability of the combined load is conditional on the probability of the individual event, and it is reasonable to use only a probable value for the combined load.

d. The shakedown and fatigue effects of these loads are adequately considered in the ASME Code by the applications of these loads in the appropriate ASME Code equations. There is thus no doubt in the fact that these loads are considered in assessing the forces, moments, accelerations, fatigue, and stresses throughout the piping system components.

Review of the analysis work done on main steam piping on HNP-2 shows that the loading combination of the OBE and operational transients was tested as an upset condition using ASME equations 12-14. The associated stresses were below the upset intensity limit. However, with regard to this situation, OBE and operational transients should continue to be treated as an emergency condition.

The load combinations summarized in table 3.9-2 show the different event and load combinations assumed for the analysis of the main steam piping system.

C. For Other Systems

Specific attention was directed toward evaluating possible vibration problems during the performance of the following transients:

1. RHR System

Taking suction from the reactor, start the RHR pumps with the RHR system in the normal lineup. With the RHR pump running, open and close the reactor injection valves. Minimum flow bypass valves will open automatically. Stop the RHR pumps.

2. CS System

Taking suction from the condensate storage tank (CST), start the CS pumps with the CS system in the normal lineup. With the CS pumps running, open

and close the reactor injection valves. Minimum flow bypass valves will open automatically. Stop the CS pumps.

3. HPCI System

Taking suction from the CST, start the turbine-driven HPCI pump with the HPCI system in the normal lineup. With the HPCI pump running, open and close the reactor injection valves. Minimum flow bypass valve will open automatically. Stop the HPCI pump.

4. RCIC System

Taking suction from the CST, start the turbine-driven RCIC pump with the RCIC system in the normal lineup. With the RCIC pump running, open and close the reactor injection valves. Minimum flow bypass valves will open automatically. Stop the RCIC pump.

Any observed displacement of piping which is judged to be significant was measured and the resultant stresses calculated. The resultant stresses were combined with the stresses caused by dead weight, earthquake, and pressure and compared to the allowable primary stresses. If the allowable code stresses were exceeded, restraints were installed to eliminate the displacements or reduce them to acceptable levels.

3.9.1.1.2 ASME Class 3 Components

Valves, piping, and supports associated with a given system were visually checked for excessive vibration during the normal course of the preoperational test program. Any displacement observed then or thereafter during normal operation which was judged to be significant was analyzed for resultant stresses and corrections made, if necessary, as noted above for Class 1 and 2 systems.

3.9.1.2 Dynamic Testing Procedures

A description of the tests or analyses used in the design of safety-related mechanical equipment (pumps, valves, and heat exchangers) to withstand seismic loadings is given in supplement 3.7A, and sections 3.7A.2 and 3.7A.3.

Most of this mechanical equipment is physically isolated from the effects of the faulted plant condition; therefore, it will see negligible accident loadings. For equipment which is not isolated from the effects of the faulted plant condition, the following design criteria have been established:

A. Piping and components not designed to withstand the dynamic effects of pipe whip must be part of the redundant, physically separated subsystems so that single failure of one subsystem does not affect the operability of the redundant subsystem.

B. Where systems are subjected to potential vibratory loadings due to the dynamic effects of fluid momentum changes (water hammer), the following measures have been taken to avoid the causes of such changes:

1. Motor-operated valves (MOVs) in the emergency core cooling system (ECCS) are not capable of closing or opening at speeds greater than 6 in./s. Catastrophic failure is improbable for MOVs.

2. ECCS and feedwater pumps are not capable of fast starts under normal operating conditions because the lines are filled with fluid. Seizure of the prime mover (motor or turbine) is considered a single failure in the ECCS and renders the complete subsystem inoperative. Pressures and fluid velocities in the ECCS are such that a water hammer stemming from pump motor seizure can be tolerated within the ASME Code faulted limits.

3. Air and steam voids that may develop in a stagnant system due to leakage are prevented in the RHR and CS systems by providing pump discharge check valves and automatic water charging on the pump discharge piping. The HPCI (and RCIC) pump lines do not need a charging system because the CST provides the same function. The pump suction piping of the HPCI and RCIC systems is pressurized by the CST and the suction piping of the RHR and CS systems are pressurized by the static head in the suppression pool. The HPCI and RCIC systems discharge to the feedwater line from the pump. Thus, the water in the discharge piping cannot leak into the higher pressure feedwater line.

   Although system vents are located at the piping high points, air pockets, resulting from poor or inadequate system drainage, filling, and venting during and after maintenance or prior to startup, could result in severe water hammer. To preclude this, procedural control is recognized as the only available means to assure proper system venting.

3.9.1.3 Dynamic System Analysis Methods for Reactor Internals

3.9.1.3.1 Forcing Functions and Dynamic Response of Reactor Internals

The major reactor internal components within the vessel are subjected to extensive testing, coupled with dynamic system analyses, to describe properly the resulting flow-induced vibration phenomena incurred from normal reactor operation and from anticipated operational occurrences. (Refer to section 4.2.)

In general, the vibration forcing functions for operational flow transients and steady-state conditions are not predetermined by detailed analysis. Special analyses of the response signals measured from reactor internals of similar designs are performed to predict amplitude and modal contributions, and parameter studies useful for extrapolating the results from the tests of internals and components of similar designs are performed. This vibration prediction

method is appropriate where standard hydrodynamic theory cannot be applied due to complexity of the structure and flow conditions. Elements of the vibration prediction method are outlined as follows:

A. Dynamic analyses of major components and subassemblies are performed to identify natural vibration modes and frequencies. The analysis models used for Seismic Category I structures are similar to those outlined in section 3.7A.2, Seismic System Analysis.

B. Data from previous plant vibration measurements are assembled and examined to identify predominant vibration response modes of major components. In general, response modes are similar, but response amplitudes vary among boiling water reactors (BWRs) of differing sizes and design.

C. Parameters are identified which are expected to influence vibration response amplitudes among the several reference plants. These include hydraulic parameters such as velocity and steam flowrates, and structural parameters such as natural frequency and significant dimensions.

D. Correlation functions of the variable parameters are developed which, multiplied by response amplitudes, tend to minimize the statistical variability between plants. A correlation function is obtained for each major component and response mode.

E. Predicted vibration amplitudes for components of the prototype plant are obtained from these correlation functions, based on applicable values of the parameters for the prototype plant. The predicted amplitude for each dominant response mode is stated in terms of a range, taking into account the degree of statistical variability in each of the correlations. The predicted mode and frequency are obtained from the dynamic analyses of paragraph A.

The dynamic modal analysis also forms the basis for interpretation of the prototype plant preoperational and initial startup test results (paragraph 3.9.1.3.2). Modal stresses are calculated and relationships are obtained between sensor response amplitudes and peak component stresses for each of the lower normal modes. The allowable amplitude in each mode is that which produces a peak stress amplitude of ± 10,000 psi.

3.9.1.3.2 Preoperational Flow-Induced Vibration Testing of Reactor Internals

HNP-2 reactor internals were tested in accordance with provisions of Regulatory Guide 1.20, Revision 2, for nonprototype Category I plants. The inspection of reactor internals was conducted following cold preoperational functional testing of the reactor system in significant flow modes. The inspection was conducted prior to fuel loading and reactor criticality. This inspection was practical to conduct, having been performed three times previously on the Browns Ferry-Unit 1, Fitzpatrick, and Duane Arnold prototype BWRs in response to provisions of Regulatory Guide 1.20.

Flow testing included operation of the recirculation system at flows up to 100% of rated volumetric flow. The results of vibration measurements in prototype plants show that the flow condition produces vibration responses in core-support structures which are greater than those at normal operating conditions and are thus conservative for testing. The test duration of 63 h included both normal and unbalanced recirculation system operation to subject major components to a minimum of 10^6 cycles at their lowest dominant response frequencies and at maximum response amplitudes.

After completion of flow testing, the vessel and shroud heads were removed and the vessel drained. Access to the lower plenum was provided by opening a manhole in the shroud support plate. Reactor internal structures and components, including those in the lower plenum region, were given a close visual inspection to detect possible wear, cracking, loosening of bolts, and the presence of debris and loose parts. The inspection covered all components which were examined in the prototype reactor, including the following categories:

- Peripheral control rod drive and incore guide tubes, housing, and their lower joints.
- Incore guide tube stabilizer connections and stabilizer bars. Plenum region for evidence of loose and/or failed parts.
- Inside surfaces of the jet pump adapter to shroud support welds and jet pump diffuser to jet pump adapter welds.
- Liquid control and delta pressure line and bracket welds.
- The shroud-to-shroud support weld.
- Jet pump instrument lines and brackets.
- Jet pump annulus for evidence of loose parts.
- Jet pump beams, beam bolts, wedges, and locator screws.
- Jet pump riser braces and welds.
- Shroud head and shroud bolt lug welds.
- Shroud and shroud head flange locating pins for evidence of deleterious motion marks other than those caused from normal installation.
- Core support plate bolt keepers.
- Steam separators and standpipes, and shroud head bolt support ring brackets and supports.
- Feedwater sparger and attachments.

- CS lines, brackets, and CS spargers.

Reactor internals for HNP-2 are substantially the same as the internals design configurations which have been tested in prototype BWR 4 plants. Results of the prototype tests are presented in reference 3. This report also contains additional information on the confirmatory inspection program.

3.9.1.4 Correlations of Reactor Internals Vibration Tests With the Analytical Results

Prior to initiation of the instrumented vibration test program for the prototype plant, extensive dynamic analyses of the reactor and internals are performed. The results of these analyses are used to generate the allowable vibration levels during the vibration test. The vibration data obtained during the test are always analyzed in detail. The results of the data analysis, vibration amplitudes, natural frequencies, and mode shapes are then compared to those obtained from the theoretical analysis.

Such comparisons provide the analysts with added insight into the dynamic behavior of the reactor internals. The additional knowledge gained is used in the generation of dynamic models for seismic and loss-of-coolant accident (LOCA) analyses for HNP-2. The models used for HNP-2 are the same as those used for the vibration analysis of the prototype plant. A comparison of predicted internals response characteristics for HNP-2 and Cooper is provided below.

COMPARISON OF PREDICTED INTERNALS RESPONSE

Component and Mode	Predicted Amplitude: HNP-2 Low	HNP-2 High	Cooper[a] Low	Cooper[a] High	Cooper[a] Measured Amplitude
Shroud/ separator assembly	0	43 mils	0	39 mils	8 mils
Jet pump (tangential)	0.5 mils	2.8 mils	0.5 mils (19 micro-strain)	2.7 mils (112 micro-strain)	20 micro-strain
Jet pump (radial)	0.4 mils	4.0 mils	0.3 mils	3.2 mils	4 mils

The vibration test data are supplemented by data from forced oscillation tests of reactor internal components to provide the analysts with additional information concerning the dynamic behavior of the reactor internals.

3.9.1.5 Analysis Methods Under LOCA Loadings

In order to ensure that no significant dynamic amplification of load occurs as a result of the oscillatory nature of the blowdown forces, a comparison was made of the periods of the applied forces and the natural periods of the core support structures being acted upon by the applied forces. These periods were determined from a comprehensive dynamic model of the reactor pressure vessel (RPV) and internals with 27 degrees of freedom as shown on figure 3.9-1. The maximum values of the differential pressures resulting from a steam line break are provided in table 4.2-20.

Only motion in the vertical direction was considered here; therefore, each structural member (between two mass points) can have only an axial load. Besides the real masses of the RPV and core support structures, account was made for the water inside the RPV.

The time-varying pressures are applied to the dynamic model of the reactor internals described above. Except for the nature and locations of the forcing functions and the dynamic model, the dynamic analysis method is identical to that described for seismic analysis.

The reactor internals for HNP-2 were designed and analyzed in accordance with applicable regulatory requirements, considering the postulated simultaneous occurrence of LOCA and design basis earthquake (DBE) events. Square-root-of-the-sum-of-the-squares of peak

a. Cooper is a 218-in. BWR 4 that has been operated at full power. Cooper is similar to HNP-2 except that the rated recirculation flow for Cooper is 6% lower.

magnitude or absolute-sum-of-peak magnitude are used to combine response-time histories of two or more dynamic loads. The square-root-of-the-sum-of-the-squares method is applied where responses have random time phasing, response amplitudes vary in time, and the frequencies of responses are comparable. In general, the use of the square-root-of-the-sum-of-the-squares method to combine the peak-dynamic, response-time histories of different events can be justified under circumstances where there is a reasonably high probability of not exceeding the square-root-of-the-sum-of-the-squares method value. As the number of loads being combined increases, the probability of their peak values combining absolutely decreases.

Due to the low stresses calculated from the HNP-2 internals, the absolute-sum method was used to combine the DBE stresses and LOCA differential pressure stresses. The design loading combinations are described in table 3.9-4.

For piping, the LOCA loads which were considered on HNP-2 were pressure changes in the systems affected and transient wave or pressure forces transmitted from a broken pipe through the vessel and into unbroken piping systems where integrity is required to prevent LOCA escalation. Evaluation of these loads indicated the following:

A. Pressure changes were no greater than the peak pressure already considered in non-LOCA evaluations.

B. Transient wave forces transmitted through the vessel into unbroken piping systems are insignificant for the BWR.

Therefore, for piping design and analysis, the DBE load represented the only significant primary loading to be considered for the combined loading of DBE + LOCA. (See table 3.9-5.) Thus, the LOCA loading for the unbroken systems was conservatively taken to be the peak transient pressure obtained from non-LOCA evaluations, and this load was combined with the DBE plus system deadweight to obtain the total design loading combination for piping systems. As an example, this form of load combination is shown in table 3.9-6 for the recirculation system piping. The results of the snubber evaluations are tabulated in table 3.9-7.

The Class 1 components within the reactor coolant pressure boundary (RCPB) have been designed to withstand the effects of the postulated high-energy line breaks inside the primary containment postulated in section 3.6. The LOCA loads were jet impingement, differential pressure, jet thrust reactions, and pipe whip. These Class 1 components are classified Seismic Category I and, therefore, are also designed to withstand the effects of the DBE.

HNP-2 is designed so that a seismic event will not induce sufficient stresses in the piping systems to cause a pipe break in a Seismic Category I system. Therefore, the combination of LOCA + DBE is not a design basis for HNP-2. However, as an independent study, the combination of LOCA + DBE loads for the Class 1 components in the RCPB was evaluated. The LOCA loads described above were added directly to the DBE loads and compared to allowables except for the broken pipe, as discussed below. A summary of the evaluation is presented below.

For the unbroken reactor coolant piping, LOCA loads, such as jet impingement loads and differential pressure loads across the outside pipe wall, were evaluated according to Equation 9

of Subsection NB-365O of ASME Section III. These loads, when added directly to the DBE loads, were found to be below the faulted condition limits of the piping components. The load combinations are provided in table 3.9-8. For pipe supports, the combined loads due to deadweight, DBE, and LOCA were within the manufacturer's faulted condition allowables, and shock suppressor loads were within emergency allowables. For RPV nozzles, the combined loads due to thermal expansion, deadweight, DBE, and LOCA were within the manufacturer's allowables.

For the broken reactor coolant piping, the pipe was not evaluated except for the adequacy of its whip restraints because the seismic stresses were insignificant when added to the LOCA stresses. Therefore, the design of the whip restraints, as presented in section 3.6, is considered adequate.

3.9.1.6 Analytical Methods for ASME Code Class 1 Components

The analytical methods used to evaluate stresses for ASME Code Class 1 components are in conformance with Sections NB-3200 through NB-3600, and Appendix F of ASME Boiler and Pressure Vessel (B&PV) Code, Section III. As permitted by NB-3630(d) of the 1975 Summer Addenda to the ASME Code, Section III, Class 1 piping 1 in. in diameter and smaller may be alternatively analyzed according to the rules of NC-3600.

Both elastic and inelastic stress analysis techniques are used in the design of the core support and reactor internal structures to show that the stress limits, specified in tables 4.2-12 through 4.2-14, are not exceeded. When an inelastic stress analysis was performed on these components, the elastic (linear) system analysis was checked to see if it required modification. The procedure is to perform a linear analysis with the stiffness of the inelastic component reduced to the stiffness value corresponding to the inelastic displacement value. A nonlinear dynamic analysis was performed in lieu of a linear analysis if the natural frequencies of the system with reduced stiffness deviated significantly from that of the unreduced system.

Use of inelastic methods of analysis for other components is not anticipated at this time. These methods, however, may be used in those cases where it is deemed desirable and appropriate to permit significant (local) inelastic response. In these cases, if any, the system or subsystem analysis performed to establish loads which act on components and component supports is modified to include the inelastic strain compatibility in the local regions of the components and component supports at which significant (local) inelastic response is permitted.

When an elastic system analysis is employed to establish the loads which act on components and supports, elastic stress-analysis methods are also used in the design calculations to evaluate the effects of the loads on the components and supports. In particular, inelastic methods such as plastic instability and limit analysis methods, as defined in Section III of the ASME Code, Appendix F, are not used in conjunction with an elastic system analysis (except for core support and reactor internals).

Table 3.9-6 provides the load conditions for the recirculation piping system (Class 1 pipe).

3.9.1.7 Fatigue Monitoring of ASME Code, Section III Class 1 Piping

To account for the increase in operating life as a result of the renewed license, the cumulative fatigue usage factor (CFUF) for Class 1 piping is monitored. Bounding locations in the feedwater piping, primary steam condensate drainage, and the main steam piping are monitored (subsection 18.2.12) to ensure the CFUF for the Class 1 piping will not exceed 1.0 during operation of the plant.

3.9.2 ASME CODE CLASS 2 AND 3 COMPONENTS

For HNP-2, this refers to either ASME Code Class 2 and 3 components (table 3.9-9) or similar non-RCPB safety-related, pressure-retaining components designed to earlier codes. (For safety-related mechanical components not covered by the ASME Code, see table 3.9-10.)

3.9.2.1 Plant Conditions and Design Loading Combinations

ASME Code Class 2 and 3 components of fluid systems were constructed in accordance with Section III of the ASME B&PV Code. Some components (piping, pumps, and valves) ordered prior to July 1971 were designed to other industry codes (tables 3.2-1 and 3.2-2) when the effective ASME Section III was not applicable. The specific quality group classification for each principal component is provided in table 3.2-1.

Torus-attached piping (TAP) and safety relief valve discharge line (SRVDL) piping were designed and constructed per the NRC requirement for the Mark I Long-Term Program (LTP) as documented in the "Mark I Containment Long-Term Program Safety Evaluation Report," NUREG-0661.[6] Modifications made to SRVDL, TAP, and their supports due to hydrodynamic loads identified during the LTP are presented in supplement 3.8B.

Tables 3.9-4 and 3.9-11 through 3.9-28 list the design-loading combinations for the major components of each safety-related system.

Load combinations for ASME Code Class 2 and 3 piping are as follows:

A. Primary Stress

1. Normal Conditions

   The following combination of loads and stress limits is considered in satisfying the requirements of NC-3652.1 and ND-3652.2 of ASME Section III.

   - Design pressure and deadweight $\leq S_h$

2. Upset Conditions

The following combinations of loads and stress limits are considered in satisfying the requirements of equation (9) of NC-3652.2 and ND-3652.2 of ASME Section III:

- Maximum pressure + deadweight + OBE $\leq 1.2\ S_h$

- Maximum pressure + deadweight + relief valve opening or fast valve closure as applicable.[a] $\leq 1.2\ S_h$

3. Emergency Conditions

The following combination of loads and stress limits is considered in satisfying the requirements of equation (9) of NC-3652.2 and ND-3652.2 of ASME Section III:

- Maximum pressure + deadweight + OBE + fast valve closure and/or relief valve opening as applicable.[a] $\leq S_y$ or $\leq 1.8\ S_h$

4. Faulted Conditions

The following combination of loads and stress limits is considered in satisfying the requirements of equation (9) of NC-3652.2 and ND-3652.2 of ASME Section III:

- Maximum pressure + deadweight + DBE + fast valve closure and/or relief valve opening as applicable.[a] $\leq 2.4\ S_h$

B. Secondary Stress

Either of the following loading combinations may be used to satisfy the requirements of NC-3652.3 and ND-3652.3 of ASME Section III:

a. Fast valve closure in this case is applicable only to the Class 2 portion of the main steam and the Class 3 main steam relief valve discharge piping. Relief valve opening is applicable only to main steam relief valve discharge piping and to the steam piping to the RHR heat exchanger.

- Thermal expansion and OBE anchor displacement stresses[(a)(b)] $\leq [1.25\ S_c + 0.25\ S_h]$

- Design pressure + weight + maximum range of thermal expansion and OBE anchor displacement stresses[(a)] $\leq 1.25\ (S_c + S_h)$

The allowable stress limits for ASME Class 2 and 3 piping assumes that the total number of full-temperature cycles to which this piping will be exposed is < 7000 for the life of the plant.

3.9.2.2 Design Loading Combinations

The combinations of design loadings are categorized with respect to plant conditions identified as normal, upset, emergency, or faulted and are shown in tables 3.9-4 and 3.9-11 through 3.9-28 for the major components, and paragraph 3.9.2.1 for piping. For nuclear steam supply system (NSSS) design, the methods used for the various load combinations are shown in these tables. Tables 3.9-4 and 3.9-11 through 3.9-28, along with the text of sections 3.9 and 5.2, contain the required information to demonstrate conformance with criteria for ASME Class 1, 2, and 3 components or their pre-1971 code equivalents. The tables in section 3.9, when taken together with the loading combination table 3.9-29, provide the necessary information on loading combinations, design criteria, and results to demonstrate the conformance of Bechtel equipment with code requirements. The various types of loadings (design mechanical loadings, emergency condition loads, etc.) are combined by absolute sum.

Design criteria and the specific manner of combining loads for normal, upset, emergency, and faulted conditions for supports of all ASME Class 1, 2, and 3 active and inactive components and piping are now provided. Specific design criteria used for supports for active pumps and valves for both General Electric- and Bechtel-supplied equipment are presented.

Design criteria and the manner of combining loads for supports of ASME Class 1, 2 and 3 active and inactive components are provided below and in table 3.9-3.

A. Supports on Bechtel-Supplied Piping

   Standard piping supports used on Nuclear Class 1 piping are designed in accordance with NB-3674 of the 1971 ASME Code, Section III, which further references American National Standards Institute (ANSI) B31.7, 1969, Divisions 1-720 and 1-721. Class 2 and 3 standard piping supports are designed in accordance with ASME Section III, Subsection NC-3674, with references to

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a. Anchor displacements due to OBE may be deleted if considered with primary stresses in equation 9 of NC-3652.2 and ND-3652.2 or if not a design condition.

b. This formula contains a time-limited aging analysis based upon an assumed number of thermal cycles. The thermal cycles assumed for HNP are adequate to account for the period of extended operation during the renewed license term. (See sections 18.1 and 18.5.)

ANSI B31.1.0, 1967, paragraphs 120 and 121. Because the loading conditions for normal, upset, emergency, and faulted conditions are not included in these codes, the design of nonstandard linear-type supports has used the procedures of Appendix XVII-2000 of the 1974 ASME Section III. The loading combinations under the various plant conditions are presented in table 3.9-3.

Shear lugs used to hold clamps on Class 1 piping are analyzed with the Class 1 piping.

Plate and shell-type supports on Nuclear Class 1 piping are limited to the anchors and restraints at the flued heads. The effects of these supports are considered with the flued head Class 1 analyses. The supports are designed for pipe rupture loads in accordance with appendix F.

Plate and shell-type supports on Nuclear Class 2 and 3 piping are designed to the loading combinations and stress allowables provided in table 3.9-3.

In addition to the stress limits provided in table 3.9-3, a standard specified deflection of 1/16 in. is allowed under the worst combination of loadings. If deflections exceed this limit, the validity of the analysis is reviewed by a stress engineer. The deflection of whip restraints is discussed in subsection 3.6.5. The design of supplementary steel is in accordance with the American Institute of Steel Construction with the additional requirements on deflection under the worst combination of loadings.

B. Supports on Bechtel-Supplied Equipment

Vendor-supplied equipment supports purchased by Bechtel are designed in accordance with the applicable ASME Code to which the equipment is designed. The design limits specified for pumps and tanks are such as to limit the material to no gross deformation as well as to limit the deflection to assure operability.

C. Supports on GE-Supplied Piping and Equipment

Supports for GE-supplied components have been designed to various criteria depending on the application and previous experience with the particular equipment to be supported. With the exception of the supports for the jet pump instrumentation penetration seal and the RPV, supports for GE-supplied ASME Class 1, 2, and 3 components have been designed using one of the following approaches.

1. The Direct Addition of the Maximum Forces Resulting from Seismic Loading, Dead Loads, and Operating Loads (Fluid Flow Reaction, Thermal Expansion, Equipment Nozzle Loading from Connecting Piping, etc.) in the Weakest or Worst Direction to Establish Anchoring Requirements

Components for which supports were designed in this manner are:

- RHR pumps
- RHR heat exchangers
- CS pumps
- SLC pumps
- HPCI pump
- RCIC pump

The design criterion was that the stress remain within the allowable values for the holddown or anchor bolts. The results are as follows:

	Calculated Stress (psi)	Allowable Stress (psi)
RHR pumps	2335	25,000[a]
RHR heat exchangers	8400	37,800
CS pumps	1842	25,000[a]
SLC pumps (shear)	5960	16,500[a]
SLC pumps (tensile)	3730	16,500[a]
HPCI pump (shear)	3917	20,000[a]
HPCI pump (tensile)	6154	7000[a]
RCIC pump	5070	32,400[a]

2. Treated as Extensions of the Piping Systems of Which They Form an Integral Part and Designed in Accordance with the Rules Governing Supports Associated with the Codes Specifying the Design Requirements for the Piping Systems

   The GE-supplied piping systems are the main steam piping from the RPV through the second main steam isolation valve (MSIV) and the original recirculation system piping which have both been designed, fabricated, and supported in accordance with the 1971 Edition of Section III of the ASME B&PV Code. Components for which supports were designed in this manner are main steam safety relief valves, MSIVs, recirculation pumps, recirculation pump suction valves, and recirculation pump discharge valves.

a. Allowable value from ASME Code, Section VIII.

Recirculation system piping, which was replaced in 1984, was designed in accordance with the 1980 Edition of the ASME Code, Section III, Class 1 with Addenda through Winter 1981. The materials, testing, and fabrication were in accordance with the ASME Code, Section III, 1980 Edition with Addenda through Winter 1980.

As noted, two GE-supplied components do not fall into these categories. The following procedures were used to design those supports:

a. Jet Pump Instrumentation Seal

The design criteria used are those specified in Section III of the ASME B&PV Code, Subsection NB, considering pressure (A), pipe reaction (B), seismic (C), and thermal (D) loadings. The following loading combinations are considered:

- Normal and upset - A + B + 1/2C + D
- Faulted - A + B + C

b. Reactor Pressure Vessel Support Skirt

The structural integrity of the reactor vessel support skirt has been assured by the preparation of a stress report as required by Paragraph N-142, Section III, of the ASME Code. This report demonstrates that the support skirt has met all applicable ASME Code stress limits. Requested details of these calculations may be found in this report which is on file with the authorized inspector at the plant site.

D. For Bechtel Equipment

Valves are supported by the piping system of which they are a part. The stresses at the pipe valve junctions are held within the same limits as all other points in the piping system. The supports for Bechtel-provided active pumps are checked against the appropriate allowables by the pump manufacturer. Loads resulting from the connecting piping are considered in the analysis.

E. For GE Equipment

The recirculation piping suspension system was supplied by GE and provides supports for active components. Recirculation pump discharge valves were designed prior to the issue of any definitive standard to which either the analysis methods or the resultant stress levels could be judged. However, these active valves were treated as extensions of the piping systems of which they form an integral part and designed in accordance with the rules governing supports associated with the codes specifying the design requirements for the piping systems. The replaced recirculation system piping is designed in accordance with the 1980 Edition of Section III of the ASME B&PV Code, with Addenda through

Winter 1981. The materials, testing, and fabrication were in accordance with the ASME Code, Section III, 1980 Edition with Addenda through Winter 1980.

Three types of component supports have been used to support the recirculation system piping such as hangers, struts, and snubbers which are designed, fabricated, and assembled so that they cannot become disengaged by the movement of the support pipe or equipment while performing its function during the various operating conditions of the plant. The design load on each of these component supports is identified as follows:

1. Hangers

   The design load on hangers is the load caused by deadweight. The hangers are calibrated to ensure that they support the design load at both their hot and cold load settings. Hangers provide a specified down travel and up travel in excess of the specified thermal movement.

2. Struts

   The design load on struts includes those caused by deadweight, thermal expansion, primary seismic loads (OBE, DBE, and system anchor displacements, etc.).

3. Snubbers

   The design load on snubbers includes those loads caused by seismic forces (OBE and DBE), system anchor displacements, etc.

The analyses that are used for the design of these components supports to ensure that all such supports will not deform to the extent that would impair the pressure-retaining integrity of the supported components under normal, upset, emergency, and faulted plant conditions, can essentially be divided into three parts, as given below:

a. Piping Analysis to Determine Design Loads on Component Supports

   The piping analysis is performed with GE SAP4 program. SAP4 is a general structural analysis program for static and dynamic analysis of linear elastic complex structures. The finite element displacement method is used to solve for the displacements and loads and computes the stresses of each element of the structure. The loads resulting from thermal expansion, deadweight, primary seismic loading (OBE and DBE), and system anchor displacements are first determined individually and then combined under normal, upset, emergency, and faulted plant conditions to determine the design load on the respective component support. Piping supports are then designed by the load rating method, and the load combinations for the various plant operating conditions correspond to those used to design the supported pipe. Design

transient cyclic data are not applicable to piping supports as no fatigue evaluation is necessary to meet the code requirements.

The recirculation pipe replacement, RHR piping between the connections to the recirculation loop suction and discharge and the head fittings, and the portion of RWC piping between the connection to the RHR suction and the penetration, were analyzed using the General Electric PISYS-05 computer program. PISYS performs static and dynamic analyses of piping systems. The analysis modules of PISYS were taken directly from the SAP4G program.

b. Selection from Vendor Data

After determining the design load by piping analysis, component supports are selected from the vendor data that indicate loads are equal to or below the load rating of the components.

c. Analysis and/or Tests to Demonstrate Acceptability

Finally, the vendor performs analyses and/or tests to demonstrate acceptability for his load rating data on component supports.

3.9.2.3 Design Stress Limits

For safety-related ASME Code Class 1, 2, and 3 components, the design stress limits are listed in tables 3.9-4 and 3.9-11 through 3.9-28 and paragraph 3.9.2.1. Inelastic methods as permitted by ASME Code, Section III for Class 1 components (also appendix F) were not used for these components. For Code Class 1 components, the normal, upset, and emergency conditions used in the analysis are as follows:

Conditions

Normal and Upset Conditions:

- RPV boltup and unbolt.[a]
- Startup (100°F/h heatup rate).[b]
- Natural circulation startup.
- Daily reduction to 75% power.[a]

a. Applied to RPV only.
b. Bulk average vessel coolant temperature change in any 1-h period.

- Weekly reduction to 50% power.[a]
- Control rod pattern change.[a]
- Loss of feedwater heaters:
    - Turbine trip with 100% steam bypass.
    - Partial feedwater heater bypass.
- Scram.
    - Turbine-generator trip, feedwater on, isolation valves stay open.
    - Loss of ac power, natural circulation restart.
    - Other scrams.
- Reduction to 0% power, hot standby, shutdown (100°F/h cooldown rate).[b]

Emergency Conditions:

- Scram.
    - Loss of feedwater pumps, isolation valves closed (100°F/h).[b]
    - Single safety relief valve blowdown (177°F/10 min and 100°F/h cooldown rate).[b]
    - Automatic blowdown.
    - Reactor overpressure with delayed scram, feedwater stays on, isolation valves stay open.
- Improper start of cold RRS loop.
- Improper startup with recirculation system pumps off and drain shut off followed by turbine roll and increase to rated power.
- Faulted condition.
- Pipe rupture and blowdown.

a. Applied to RPV only.
b. Bulk average vessel coolant temperature change in any 1-h period.

3.9.2.4 Analytical and Empirical Methods for Design of Pumps and Valves

A combination of analysis, component testing, and operating experience is employed in the design of pumps and valves. Design criteria and the manner in which loads are combined for ASME Class 2 and 3 pumps and valves are provided below.

All components meet the specific requirements of the ASME Code to which they were designed. Except for Class 2 and 3 valves, Seismic Category I components have been analyzed for OBE loads combined with any identified concurrent operating transients by the absolute sum, and stresses were held within the upset stress allowable. Class 2 and 3 valves have not been analyzed for an equivalent OBE loading condition because they are shown to withstand a more severe DBE equivalent loading.

An evaluation of the direct combination of all LOCA + DBE loads for the Class 1 components in the RCPB was performed and is summarized in paragraph 3.9.1.5.

A. Analytical Methods

All Bechtel-supplied ASME Nuclear Class 2 or 3 active valves are designed in accordance with ANSI B16.5. Valves with extended structures which could affect pressure integrity are analyzed for a minimum of 3.0-g vertical and horizontal loading in combination with normal operating loads. These extended structures are also modeled into the piping stress analysis, and the piping stresses are held within ASME Code allowables for the loading combinations identified in paragraph 3.9.2.1.

The only Bechtel-supplied Nuclear Class 2 or 3 active pumps are the residual heat removal service water (RHRSW) and plant service water (PSW) pumps whose loading combinations are addressed in appendix A in the conformance to Regulatory Guide 1.48. Tables 3.9-30 through 3.9-32 summarize the loading combinations and the design criteria for these pumps.

A more detailed description of design criteria and the manner of combining loads for the RHR and CS pumps classified as ASME Class 2 active pumps is listed below.

Closure bolting is calculated by using Rules for Bolted Flange Connections, ASME Section VIII, Appendix II, and allowable working stresses in accordance with ASME Section VIII. The bolting loads include design pressure and temperature, design gasket load, static mass forces, nozzle loads, and seismic acceleration. The nozzle loads used are the DBE values using the worst combination of loads in the three axes of each nozzle.

Wall thickness is calculated in accordance with the rules of ASME Section VIII, Part UG, with stress limits from ASME Section VIII. Furthermore, Bijlaard analyses, using the ASME Section VIII stress limits, are performed on pressure-retaining, nozzle-shell intersections considering the loads, design pressure and temperature, static mass forces, nozzle loads, and seismic

acceleration. The nozzle loads used are the DBE values using the worst combination of loads in the three axes of each nozzle.

Four sets of calculations are performed:

- OBE seismic acceleration with maximum nozzle moments.
- OBE seismic acceleration with maximum nozzle forces.
- DBE seismic acceleration with maximum nozzle moments.
- DBE seismic acceleration with maximum nozzle forces.

The only GE-supplied ASME Class 2 or 3 active valves are the control rod hydraulic system valves. Design calculations and certifications are provided by the valve vendors to demonstrate that the seismic capability of the body-to-bonnet bolts, yoke (as operator support), and operator bolts are sufficient to withstand the seismic forces applied to the mass center of the operator. The calculations combine the stresses in the valve components due to seismic loads (horizontal and vertical acting simultaneously) and the stresses due to other live and dead loads along with the operating loads. These combined loads do not exceed 1.5 times the ASME Code allowable stresses. The valve will not fail to function during application of these forces. Calculations are also submitted to demonstrate compliance with Paragraphs 452.1a and 453.1 of the ASME Nuclear Pump and Valve Code.

B. Component Testing

Active mechanical equipment classified as Seismic Category I is shown capable of performing its function during the life of the plant under postulated plant conditions. Equipment with operating condition functional requirements includes active pumps and valves in fluid systems such as the RHR system, CS system, and the isolation systems.[a]

Operability is ensured by satisfying the requirements of the following programs. Continued operability is ensured by periodic testing.

C. Pumps

All active pumps are qualified for operability by first being subjected to rigid tests both prior to installation in the plant and after installation in the plant. The in-shop tests include hydrostatic tests of pressure-retaining parts to 1.5 times the design

a. Active equipment must perform a mechanical motion during the course of accomplishing a safety function.

pressure times the ratio of material allowable stress at room temperature to the allowable stress value at the design temperature; seal leakage tests; and performance tests, while the pump is operated with flow, to determine total developed head, minimum and maximum head, net positive suction head requirements, and other pump/motor parameters. After the pump is installed in the plant, it undergoes the cold hydro tests, functional tests, and the required periodic inservice inspection and operation. These tests demonstrate reliability of the pump for the design life of the plant.

In addition to these tests, the safety-related active pumps were analyzed for operability during a seismic condition by ensuring that the pump will not be damaged during the seismic event, and the pump will continue operating despite the addition of the seismic loads.

Performing these analyses with the conservative loads stated and with the restrictive stress limits of tables 3.9-15, 3.9-18, 3.9-20, 3.9-22, 3.9-24, and 3.9-26 as allowables ensures that critical parts of the pump will not be damaged during the seismic condition and that the reliability of the pump for post-seismic condition operation is not expected to be impaired by the seismic event.

The second criterion necessary to ensure operability is that the pump functions throughout the seismic event. The pump/motor rotor combination is designed to rotate at a constant speed under all conditions unless the rotor becomes completely seized, i.e., with no rotation. Motors are designed to withstand short periods of severe overload. Typically, the rotor can be seized 5 full seconds before a circuit breaker trips to prevent damage to the motor. However, the high rotary inertia in the operating pump rotor, and the nature of the random, short duration loading characteristics of the seismic event, will prevent the rotor from becoming seized. In actuality, the seismic loadings will cause only a slight increase, if any, in the torque (motor current) necessary to drive the pump at the constant design speed. Therefore, the pump will not shut down during the event and will operate at the design speed despite the seismic loads.

From the previous arguments, the safety-related pump/motor assemblies will not be damaged and will continue operating under seismic loadings; therefore, they will perform their intended functions. These proposed requirements take into account the complex characteristics of the pump and are sufficient to demonstrate and ensure the seismic operability of the active pumps.

The functional ability of active pumps after a seismic condition is ensured since only normal operating loads and steady-state nozzle loads exist. Since it is demonstrated that the pumps would not be damaged during the faulted condition, the post-seismic condition operating loads will be no worse than the normal plant operating limits. This is ensured by requiring that the imposed nozzle loads (steady-state loads) for normal conditions and post-seismic conditions are limited by the magnitudes of the normal condition nozzle loads. The post-seismic condition ability of the pumps to function under these applied loads is proven during the normal operating plant conditions for active pumps.

D. Design Criteria (NSSS Design)

1. Pumps

The ASME Code allowable stresses and design calculated stresses for specific components of the various pumps are tabulated below. This tabulation shows that the design values are well within the allowable elastic limits for the materials used, thus ensuring no geometric or dimensional deformation as a result of exposure to the worst case postulated loading environment. Functional capability is, therefore, preserved for the design conditions with substantial margins.

RHR and CS Systems:

Pressure Boundary Components	Stresses (psi) RHR	CS	Allowable
Suction shell	6095	3366	17500
Discharge Nozzle	15900	11995	17500
Suction nozzle	13813	9480	17500
Nozzle head bolts	22307	15423	25000
Torispherical head of shell	10148	3179	17500
Stuffing box	2200	2200	15000
Discharge head plate	7877	1818	15000
Outer column flange	10334	10162	17500
Mounting bolts	2335	1842	25000

SLC Pump - Allowable and Calculated Stresses:

Pressure Boundary Components	Stresses (psi) Calculated	Allowable
Fluid Cylinder	3640	17500
Cylinder head extensions	5280	17500
Stuffing box	10690	30000
Stuffing box studs	16770	25000
Cylinder head studs	17720	25000
Cylinder head covers	1788	15000
Gland	3910	75000

Nonpressure Boundary Components	Stresses (psi) Calculated	Allowable
Motor holddown bolting (shear)	1720	10000

Motor holddown bolting (tensile)	1230	16500
Motor foot, integral (shear)	115	4400
Pump holddown bolting (shear)	5960	16500
Pump holddown bolting (tensile)	3730	16500

Pump analysis denotes minimum metal thickness of the fluid chamber, cylinder walls, cylinder covers, and allowable stress of bolting materials. Hydrostatic test of the assembled unit was conducted at 1.5 times the design pressure. A performance test was performed on each pump assembly as proof that the unit meets specification requirements. Preoperational testing was performed prior to the startup phase of the power test program to verify and document that equipment and system combined meet the design requirements.

HPCI Pump - Allowable and Calculated Stresses:

Pressure Boundary Components	Stresses (psi) Calculated	Allowable
Closure bolting (main)	19950	20000
Closure bolting (booster)	17400	20000
Casing wall thickness (main)	12050	14000
Casing wall thickness (booster)	3650	14000

Nonpressure Boundary Components	Stresses (psi) Calculated	Allowable
Pump bolts (tensile) booster	664	7000
Pump bolts (tensile) main	996	7000
Dowel pins (shear) booster	4990	20000
Dowel pins (shear) main	7485	20000
Anchor bolt (tensile)	6154	7000
Anchor bolt (shear)	3917	20000

NOTE: Eight anchor bolts carry the stresses for both units mounted on a common baseplate.

Pump analysis also denotes minimum metal thickness of pump case and the allowable stress of bolting materials. Hydrostatic test of the assembled unit was conducted at 1.5 times the design pressure. A performance test was performed on pump for proof that the unit satisfied specification requirements by measurements of developed head flow, and vibration. Preoperational testing will be performed prior to startup phase of the power test program to verify and document that equipment and system combined meet design requirements.

RCIC Pump - Allowable and Calculated Stresses:

Pressure Boundary Components	Stresses (psi) Calculated	Allowable
Barrel	9200	17500
Suction nozzle	5348	17500
Discharge nozzle	5348	17500
End cover bolting	22600	25000

Nonpressure Boundary Components	Stresses (psi) Calculated	Allowable
Pump holddown bolting	5070	32400
Pump taper pin	9820	21000
Bearing housing pin	1390	21000

Pump analysis also denotes minimum metal thickness of barrel, end covers, and nozzle walls, and the allowable stress of bolting materials. Hydrostatic test of the assembled unit was done at 1.5 times the design pressure. Performance test was performed on pump for proof that the unit satisfied specification requirements by measurements of developed head, flow, and vibration. Preoperational testing was performed prior to startup phase of the power test program to verify and document that equipment and system combined meet design requirements.

E. Design Criteria for HPCI and RCIC Turbines and Bechtel-Supplied Components

1. HPCI and RCIC Turbines

Calculated stress levels were summarized in comparison with ASME Code allowable stress levels. Maximum values are tabulated as follows:

	Allowable Percentage HPCI	RCIC
Pressure casting	64	86
Pressure bolting	92	80
Structural	49	44

Shaft deflections under operating loads, including seismic, are < 0.015 in. for the HPCI turbine and 0.005 in. for the RCIC turbine. These deflections are substantially less than the internal clearance (0.1875 in.) between the turbine rotor and stationary parts.

The design margins noted above provide for sufficient component dimensional stability to assure their required functional capabilities.

2. For Bechtel-Supplied Components

Both active and inactive pumps and valves used in a safety-related system are designed to the applicable sections of the ASME Code, Section III, and other recognized standards. The manufacturers are also required to comply with specific requirements in the design specification concerning earthquake loadings, environment conditions, and operating transients. The most severe loading condition for each component then becomes a design condition. (See table 3.9-29.) Since the ASME Code does not address operability, pumps or valves required to perform a motion to provide their safety function are so designated in the design specification and manufacturers are required to demonstrate this by testing detailed analysis or a combination of both. If testing is not practical, a detailed stress and deflection analysis is performed. Stresses are maintained below yield so that no permanent deformation occurs, and deflections are minimized so that the potential for binding or misalignment is eliminated.

Where functional capability of a piping system is required, stress levels do not exceed yield unless the effect of permanent deformation is evaluated. It is reasoned that by eliminating permanent deformation, the component will not only maintain its pressure integrity but also retain its dimensional stability. For these systems, the use of ASME Code Case 1606 and Appendix F of the ASME Code Section III is avoided.

F. Valves

Safety-related active valves must perform their mechanical motion in times of an accident. Assurance must be supplied that these valves will operate during a seismic event. Qualification tests accompanied by analyses were conducted for all active valves. Active valves in the RCPB and other Seismic Category I systems are listed in tables 3.9-33 and 3.9-34.

The safety-related valves are subjected to a series of stringent tests prior to service and during the plant life. Prior to installation, the following tests are performed--shell hydrostatic test to the ASME Code, Section III, requirements, back seat and main seat leakage tests, disc hydrostatic test, functional tests to verify that the valve will open and close within the specified time limits when subjected to the design differential pressure and operability qualification of valve actuators.

Cold hydro qualification tests, hot functional qualification tests, and periodic inservice operation are performed <u>in-situ</u> to verify and ensure the functional ability of the valve. These tests and appropriate maintenance ensure operability of the valve for the design life of the plant. The valves are designed using either the standard or the alternate design rules of ASME Code, Section III. On all active valves, an analysis of the extended structure is also performed for static equivalent seismic loads applied at the center of gravity of the extended structure. These valves are required to have a lowest natural frequency of vibration ≥ to 20 Hz. The

natural frequency has been determined by analysis of a three lumped mass cantilever beam model of the valve. This model also enables calculation of some higher mode frequencies. The maximum stress limits allowed in these analyses show structural integrity.

Valves which are safety related but can be classified as not having an overhanging structure, such as check valves and safety relief valves, are considered separately.

Due to the particular simple characteristics of the check valves, they will be qualified as follows:

- In-shop hydrostatic test.
- In-shop seat leakage test.
- Periodic in-situ valve exercising and inspection to assure the functional capability of the valve.

The safety relief valves are qualified by the following procedures. These valves are subjected to tests and stress analyses including the seismic loads, in-shop hydrostatic seat leakage, and performance tests. In addition to these tests, periodic in-situ valve inspection, as applicable, and periodic valve removal, refurbishment, performance testing, and reinstallation are performed to ensure the functional capability of the valve.

During a seismic event, it is allowed that the seismic accelerations imposed upon the valve may cause it to open momentarily and discharge under system conditions which otherwise would not result in valve opening. This is of no real safety or other consequence.

Using the methods described, all the safety-related valves in the systems are qualified for operability during a seismic event. These methods proposed conservatively simulate the seismic event and ensure that the active valves will perform their safety-related function when necessary.

1. MSIV

   Two different valve internals designs for the valve disk (poppet) and piston, stem disk (pilot poppet), stem, and bonnet (cover) are installed in the MSIVs. The original Rockwell/Edwards design includes an oversized diameter disk, or poppet. The newer design includes a modification kit supplied by Weir Valves and Controls (WVC) which replaces the valve disk and piston, stem disk, stem, and bonnet with a new valve poppet, pilot poppet, stem, and cover. The WVC internals design functions in a similar way to the Rockwell/Edwards design but decreases valve seat leakage and wear of valve internal components. Table 3.9-65 lists which valve internals design each MSIV has installed.

To assure that deformation of the valve body does not interfere with functional capability of the MSIV, the piping reaction loads are design limited. The axial, binding, and torsional pipe loads are separately limited to 2 S_m at 500°F (41,000 psi) in the attached schedule 80 pipes.

Assuming these piping loads to be applied, the MSIV is designed in accordance with ASME B&PV Vessel Code, Section III, Class I requirements. This code limits the outer fiber stress in critical regions of the body to 1.5 S_m at 500°F (29,100 psi). Thus, the body of the valve is assumed to remain elastic and assures only small deformation and operability.

The complete analysis of the piping and valve as a system under the worst faulted condition of the piping system causes a stress in the adjoining piping of 3289 psi or 0.14 S_m. The combination of loads are those given in table 3.9-8. For this piping, the applied load stress in the critical crotch region of the valve body is 1438 psi or 0.07 S_m which is well below the yield point of 1.5 S_m.

With respect to the functional capability of the bonnet, body flange, and associated bolting, the piping-induced loads, due to dynamic motion, are limited by design to 1.5-g horizontal and 0.6-g vertical applied at the greater center of mass. These components are designed to remain below 1.5 S_m, and they are elastic under the worst faulted loading condition of the piping as described in table 3.9-8.

Under these conditions, the maximum calculated stress in studs is 45,000 psi or 65% of allowable. For the valve with the Rockwell/Edwards internals design, calculated required thickness of the bonnet is 4.34 in. For the valves with the WVC internals design, calculated required thickness of the bonnet is 4.39 in. For the valves with the Rockwell/Edwards internals design, actual thickness of bonnet is 6.33 in. or 46% greater than required. For the valves with the WVC internals design, actual thickness of bonnet is 6.52 in. or 49% greater than required. The body flange was evaluated for stress and was determined to have a maximum stress of 29,977 psi or 83% of allowable.

All the above components were analyzed by using the code method of converting moments and forces from design acceleration loads to equivalent pressure. These calculations assume that piping-induced dynamic loads are at the design limit of 1.5-g horizontal and 0.6-g vertical.

The valve actuator assembly was analyzed for design accelerations. The induced loads were resolved into a single-load vector and applied at the actuator center of gravity. Each actuator component was evaluated and stress determined. The maximum stress calculated was 33,400 psi or 80% of allowable which is 90% of yield strength. Steamline system, calculated-acceleration values were 0.64-g horizontal and 0.12-g vertical. These values are 43% and 20% of design accelerations, respectively. Finally, static deflection tests completed on this configuration determined that

the valve remained operable up to a load equivalent of 4.3 g applied perpendicular to the actuator centerline before the closure speed was effected. The valve continued to operate as the load was increased to 5.7 g. The functional capability of the operator, stem and guidance system has been demonstrated to be over 3.3 times the loads applied as calculated by the piping analysis reported in table 3.9-14.

Two of the four MSIVs are mounted at 35 degrees from the main steam pipe vertical centerline. The effects of this rotation on the pipe centerline are to decrease the seismic effect on the valve top works. This occurs due to the difference of the horizontal and vertical components. Also, the valve closing time will not be affected by this rotation since the closing force is reduced by < 1%. The variable-flow hydraulic control valves will automatically compensate for this extremely small loss of driving force.

The analysis and tests reported above assures functional capability of the MSIV to close under the worst-faulted conditions of the piping system. To assure that the disc and stationary seat of the valve seal to prevent long-term low-volume leakage, the long-term piping-applied loads (table 3.9-14) on the valve are limited to 0.75 S_m in the connecting pipe. The analysis of the piping system has shown that after the dynamic loads have ceased, the actual piping stress is 0.18 S_m.

2. Recirculation Valves

The recirculation gate valves are analyzed in accordance with the requirements of the ASME Code, Section III, for Class I valves. The code requires that the secondary stress level in the valve critical crotch area due to the individual effects of axial, bending and torsion shall be 1.5 S_m or less at 500°F for the valve body material. This requirement assures the adequacy of the valve body to safely transmit forces and moments imposed by the connecting pipe and guarantees that the valve body material will remain within the elastic range while exposed to the worst combination of pipe reaction loads.

The pressure integrity of the valve assembly involves, not only the valve body, but also the body-to-bonnet pressure boundary bolting. This pressure boundary bolting must meet ASME Code and ANSI B 31.7 requirements. These requirements are met by calculating a minimum bolting area using the actual flange dimensions and the flange design pressure, P_{FD}. The term P_{FD} includes the seismic loading components as well as valve design pressure. Thus calculated, the minimum required bolting area is 37.53 in.^{2} for the suction valve and 44.41 in.^{2} for the discharge valve. The actual bolting area in both cases is 55.86 in.^{2} which is 49% greater than required for this suction valve and 26% greater than required for the discharge valve. These margins, above already conservative bolting areas, assure the pressure integrity of the body-to-bonnet pressure boundary bolting.

Pressure integrity as described in the preceding discussion is all that is required of the suction valve. Only the discharge valve is required to operate in a LOCA condition. The required operation of the discharge valve is closure. There are three areas to be considered for discharge valve operation--the valve body, the stem, and the actuator. The valve body must not deform as a result of the loading environment so that the gates can move properly to the seating position. As previously mentioned, the design criteria of 1.5 S_m maintains the body within the elastic range and precludes deformation. Thus, evaluation of the valve body determines that it will not interfere with valve operability.

Consideration of the stem requires that it must have free movement through the packing to lower the gates to the seating position. Calculations to evaluate stem buckling prove that the stem will not buckle (deflect) under a direct summation of operator thrust and vertical seismic loads. Stem deflection was found to occur; however, due to horizontal seismic loading on the valve extended mass. The valve extended mass consists of the actuator, yoke, and stem assembly. Actuator deflection was determined by assuming that the mass of the entire assembly acts on the actuator horizontal centerline. Then, a linear deflection distribution along the stem was assumed for conservatism, and the stem deflection at the top of the packing box was determined. This deflection is only 40% of the packing box and stem-machining tolerance when using design seismic loadings.

With the seismic loadings as determined by the stress analysis using the loading combination presented in table 3.9-16, the stem deflection at the packing box is 16% of the machining tolerance. Therefore, it is assured that there will be no interference between the stem and the packing box to hamper valve operability.

Lastly, valve operability is ultimately dependent on the performance of the actuator. The suction valve actuator used is manufactured by Limitorque and has been generically qualified to Institute of Electrical and Electronics Engineers (IEEE) 382-1972 which specifically calls for IEEE 344-1971 and IEEE 323-1971, thus proving its operability by test. The suction valve actuator is not required to be qualified under 10 CFR 50.49. The test, which lasted 30 days, was conducted by the Franklin Institute Research Laboratory in September 1972. The actuator is qualified to 5.8 g at 35 Hz which is in excess of the expected seismic loadings provided in table 3.9-16. The discharge valve is qualified to IEEE 382-1972, IEEE 344-1975, and IEEE 323-1974; and is qualified to be operable to 340°F with an external pressure of 105 psig, the saturated pressure of steam at that temperature.

The three areas for consideration of valve operability have been evaluated, and the closure of the discharge valve in a LOCA condition is assured.

3. Main Steam Safety Relief Valves

The main steam safety relief valves are designed in accordance with the requirements of the ASME Code, Section III, for Class 1 components. The Code requires that the stress levels in the critical crotch area of the valve due to axial, bending, and torsional effects shall be ≤ 1.5 S_m of the valve body material at 500°F. This criterion assures the adequacy of the valve body for safety transmitting the forces and moments imposed by the connecting pipe since it assures that the valve body material will remain within the elastic limit and maintain its pressure integrity. The maximum stress level in the valve body is 10,406 psi or 0.54 S_m, which is only 38% of the permitted elastic design load limit of 1.5 S_m. Operability of the main steam safety relief valves under both normal conditions has been demonstrated by bench tests. Since the valve design consists of components that are not deflection limited and reaction during valve operation comprises the major portion of worst-case loading, it is concluded that the valve will remain operable under the worst-case loading.

G. Vital Pump and Valve Appurtenances

All appurtenances vital to the operation of active pumps and valves have been qualified by testing, analysis, or a combination of both.

The operability of certain appurtenances has been qualified by the equipment vendors through dynamic testing programs described below. Results of representative analyses for those components not qualified by testing are provided in tables 3.9-4, 3.9-11 through 3.9-28, and 3.9-35 through 3.9-64. Where analyses are performed, it is verified that deflections produced by seismic loadings do not limit the operability of the component.

1. Qualification Test of National Acme Company Snap-Lock Electric Switch D 2400X-2

A seismic qualification test program of National Acme snap-lock electric switch D 2400X-2 was conducted by Fisher Controls Company and reported in document 1529 dated November 2, 1972. Testing was conducted with the switch assembly fastened to a metal plate which in turn was attached to a shaker table. All tests were conducted with the switch in an operating condition. The following is a summary of the test procedure and results:

a. Test Procedure

Conduct a continuous frequency sweep for each of the 3 axes, from 5 to 60 Hz at an acceleration level of 1.0 g in not < 31 s.

If the resonant frequency is < 33 Hz, conduct a 4 g, 1-min dwell at the resonant frequency and at 10 and 33 Hz.

If the resonant frequency is > 33 Hz, conduct a 4 g, 1-min dwell at 10, 17, 25, and 33 Hz and at the resonant frequency if it is < 60 Hz.

b. Test Results

The snap-lock electric switch performed satisfactorily with no malfunctions noted and meets or exceeds the specifications outlined in the test procedure.

2. Qualification Testing of Valve Motor Operators

An ongoing program of seismic and environmental testing dating back to 1968 was undertaken by Limitorque Corporation on their valve motor operators.

A seismic qualification test program was conducted by Lockheed Electronics Company for Limitorque Corporation, and the results are included in reports issued by Limitorque on January 2, 1969 (Engineering Order 600198), and June 2, 1972 (Engineering Order 600374).

The 1969 report includes results of a test in which the valve operator was subjected to an exploratory scan of 5 Hz to 35 Hz. No critical resonant frequencies were observed during this scan. The unit was also subjected to a 5.3-g load at 35 Hz in each of 3 different axes a total of 2 min on, 1 min off, 3 times per axis. The unit was operated electrically to both the fully open and fully closed positions. All torque and limit switches functioned properly.

In the 1972 report, the unit was subjected to 2 exploratory scans over the frequency range of 5 to 60 Hz. These scans indicated no resonances except at 44 Hz in the Y axis, 46 Hz in the Z axis, and 39 Hz in the X axis. Two 1-min dwells were performed at the resonant frequency at a nominal input of 3 to 5.8 g with the first minute of vibration followed by 1 min of rest.

The results of these tests showed no loss of operability either during or after the loads were applied.

a. Excess Flow Check Valves

The excess flow check valves supplied by Marotta Scientific Controls, Inc., were put through a seismic qualification test program at American Environments Company, Inc., and the results are reported in Marotta's seismic test report dated October 8, 1975 (Code 99657).

The valves were subjected to a biaxial continuous-sine-sweep test over the frequency range of 1 to 35 Hz. The continuous-sine-sweep test was performed simultaneously in each of two mutually perpendicular axes (including the vertical) at an angle of 45 degrees from the horizontal

axis. The input excitation level was 1-g peak horizontal and 1-g peak vertical.

The valves were tested in three operating conditions during the seismic loading: valve open (inlet pressurized to 1100 psig with the outlet blank), valve closed (inlet pressurized to 1100 psig with the outlet open to the atmosphere), and valve closing excess flow (inlet pressurized to 1100 psig with outlet blocked then quickly opened to the atmosphere).

The results of the tests indicated that no physical damage or seat leakage occurred as a result of the seismic loading.

b. Solenoid Valves

The Y-pattern solenoid valves supplied by Target Rock Corporation (used in the RHR system) have been qualified by an extensive environmental and seismic test program conducted by East-West Technology Corporation.

The seismic test procedure consisted of exploratory scans from 1 to 33 Hz at 0.2 g to determine whether any resonant frequencies existed. These scans revealed resonances at 16.5, 20, and 26.5 Hz in the major horizontal axis, 9, 17.5, and 26.5 Hz in the minor horizontal axis, and 21 Hz in the vertical axis.

Resonance dwells were then performed at 3 g and 4.5 g at each of the resonance frequencies with the valve being operated during and after the load applications.

The results of these tests, recorded in Target Rock Corporation Report 1500, dated October 22, 1974, show no loss of operability either during or after seismic load application.

The above descriptions demonstrate a prudent operability assurance program for active pumps and valves that is a reasonable balance between testing and analytical methods.

The solenoid air valve assembly (together with the supporting brackets) used for controlling power actuation of the main steam safety relief valves was dynamically tested to demonstrate that no resonant frequency exists below 33 Hz, and the assembly does not suffer any damage when subjected to vibration input at 33 Hz of 3.0-g vertical and 4.5-g horizontal accelerations.

Recirculation suction valves operators were qualified by testing to IEEE 382-1972 (IEEE Trial-Use Guide for Type Test of Class 1 Electric Valve Operators for Nuclear Power Generating Stations) and to IEEE 344-1971 (Trial-Use Guide for Seismic Qualification of Class 1 Electric Equipment for Nuclear Power Generating Stations) by Limitorque Corporation in September 1972. The suction valve

actuator is not required to be qualified under 10 CFR 50.49. (See detail on their Franklin Institute Test Report Number F-C3441.) The discharge valve operators are qualified to IEEE 382-1972 and IEEE 344-1975.

3.9.2.5 Design and Installation Criteria, Pressure-Relieving Devices

Nuclear Class 2 and 3 system components, other than the main steam relief valve piping, are protected from overpressure by the installation of pressure-relieving devices in accordance with ASME Code, Section III, Class 2 or 3. The design of pressure-relieving devices can be generally grouped in two categories, open discharge, and closed discharge.

A. Open Discharge

An open discharge is characterized by a relief valve discharge elbow open to the atmosphere.

The design of relief valve stations includes the considerations of both local stresses at the header-to-relief valve-inlet piping junction and the stresses in the relief valve-inlet piping and header.

Forces and moments on the piping resulting from thrust developed by full opening of the relief valve are considered in the stress analysis. The reaction forces are calculated in accordance with ASME Code Case 1569 using a dynamic load factor of 2. A static analysis of the piping system is performed by applying the calculated force at the pipe discharge exit. The resulting stresses at all points in the piping system are combined with other loading as described in paragraph 3.9.2.1, and the total piping stresses are maintained within ASME Code allowables.

In lieu of the above procedure, a time-history dynamic calculation may be used.

B. Closed Discharge

Nuclear Class 2 and 3 relief valve discharge piping systems > 2 in. in diameter with long discharge piping runs or with discharges submerged in water are analyzed by using RVDFT. This program is based on finite different solutions by the methods of characteristics. The computed transient pressure, velocity and density are then used to calculate loads on the piping system. These loads will be employed to obtain the dynamic effect on the piping system using a dynamic time-history analysis. The resulting stresses are combined with other loadings as described in paragraph 3.9.2.1, and the total stresses are maintained within ASME Code allowables.

Two-in. and under closed discharge piping systems are analyzed by considering pressure and momentum effects at each change in flow direction. A dynamic time-history analysis is performed using conservative forces, and the combined stresses are held within ASME Code allowables.

3.9.2.6 Stress Levels for Seismic Category I Components

Stress analysis was used to determine structural adequacy of pressure components under the operating conditions of normal, upset, emergency, or faulted as applicable.

Significant discontinuities were considered such as nozzles, flanges, etc. In addition to the design calculations required by the ASME Code, stress analysis was performed by methods outlined in the ASME Code appendices or by other methods by reference to analogous codes or other published literature.

Tables 3.9-4 and 3.9-11 through 3.9-28 give calculated stress levels, or maximum allowable loadings at significant areas of consideration for the major components of GE supply. Stress levels for major Seismic Category I components that are not supplied by GE are presented in tables 3.9-36 through 3.9-64. Stress levels for Holtec spent-fuel storage racks are presented in table 3.9-27 (sheet 3 of 3).

Stress levels for safety-related Seismic Category I piping systems above 2 in. in diameter are provided in HNP-2 stress calculations which were reviewed and revised (if necessary) as part of the overall pipe stress reanalysis effort for NRC Inspection and Enforcement Bulletin (IEB) 79-14.

3.9.2.7 Field Run Piping Systems

Except for a few cases where space was limited during construction, all 2-in. and under piping, regardless of nuclear or seismic class, was field run.

Seismic Category I piping systems are delineated in subsection 3.2.1 and in addition all 2-in. and under branch lines from these systems are also classified Seismic Category I to the first valve or other Seismic Category I component. The piping is seismically restrained and analyzed to the first anchor beyond the valve.

A design guide for use by field personnel was written to establish simplified procedures for designing and installing 2-in. and under piping. The design guide specified minimum support lengths to keep seismic response frequencies over 20 Hz and to support weight loads. The guide specified minimum offset lengths on pipe runs so that thermal expansion stresses did not exceed code allowables. Standard pipe support designs were also established to ensure adequate support design. The as-built isometrics were then sent to the engineering office for review. Where the limits of an equivalent dynamic analysis were exceeded, a dynamic analysis was performed.

The necessary changes in routing or restraints were indicated on the isometrics which were returned to the field for implementation. The revised drawing was then returned to the engineering office for approval. Once the field obtained an approved drawing, a surveillance was conducted to ensure that the as-built condition and the drawing agreed.

3.9.2.8 Inspection and Testing

Inservice testing of ASME Code Class 1, 2, and 3 pumps and valves shall be performed in accordance with ASME OM Code and applicable Addenda as required by 10 CFR 50.55a(f), except where the NRC granted specific written relief pursuant to 10 CFR 50.55a(f)(6)(i). The preservice program is presented in subsection 5.2.8, and the inservice program is provided under separate cover.

3.9.2.9 Computer Programs

A. GE

The SAP computer program used for the main steam piping analysis is described in Appendix 5A of the 238 GESSAR.

The PISYS-05 computer program used for the replaced recirculation piping performs static and dynamic analyses of piping systems. The analysis modules of PISYS were taken directly from the SAP4G program. The SAMIS computer program was used in the dynamic analysis of the hydraulic control units control rod drive (CRD) and in the analysis of the RPV support skirt and stabilizer. This program is described in Appendix 5A of the 238 GESSAR.

The MASS computer program was the principal computer program used in the design of the core structure for HNP-2. The MASS program can be described as an elemental or lumped-parameter approach (even though distributed properties are considered) for the analysis of redundant structures. The modeling of a complex structure is possible due to the variety of elements contained in MASS such as three-dimensional straight, curved, and segmented beams, tubes, and special connectors. The inputs to MASS are mechanical loadings (including maneuver), thermal gradients, and deflections. The inputs of MASS are stresses, loads, and deflections. MASS has been used in the design of the top guide, core support, shroud and shroud head, steam dryer, and CS lines chiefly as a tool for predicting load distributions and deflections with a structure. Other uses of MASS are not known since MASS has been available for use throughout the division. The MASS program has been design reviewed in accordance with Nuclear Regulatory Commission, Standard Review Plan 3.9.1 guidelines, using the standard BWRSD design control procedure for verification of computer codes used in design analysis.

B. Bechtel

The following computer programs have been used in the analysis of Seismic Category I piping:

1. ME-632/ME-101

   a. Description

      Purpose

      The stresses and loads in piping systems due to restrained expansion, deadweight, seismic movement, and earthquake are calculated using the ME-632 computer program.

      Method of Analysis

      The stiffness method of finite element analysis has been used in this program. In this method, the displacement of the joints of a given structure are considered to be the basic unknowns. The dynamic analysis is by the modal synthesis method. The modal synthesis, in principle, exploits known maximum accelerations produced in a single degree-of-freedom model of certain frequency. The programs principal assumptions are:

      - Linearly elastic structure.

      - Response spectrum input for seismic analysis; force time-history input for time-history analysis.

      - Lumped mass model satisfactorily replaces the structure.

      - Modal synthesis is applicable for seismic analysis.

      - Rotational inertias of the masses have negligible effect.

   b. Extent of Application

      The program was used for the above purpose on all piping lines which require seismic, thermal, weight, or time-history analysis.

   c. Design Control Measures

      The following is a comparison of the ME-632 program results with the results of the previously approved Engineering Data System computer program.

      <u>Example</u>:

      The two piping systems chosen for stress checks were:

- Core Spray Piping System - Monticello Nuclear Generating Plant-Unit 1

- Lines 48223-18-HE, 50056-10-HE, 50057-10-HE-SMUD Rancho Seco-Unit 1

These two test cases were chosen because independent piping stress analyses performed by Engineering Data Systems under contract to Bechtel were available for comparison purposes. The engineering data systems analysis of the CS piping system consisted of both dead weight and thermal loading, while the SMUD Rancho Seco piping system was an earthquake response spectrum analysis.

The ME-632 piping stress analyses were performed in the period September 18-29, 1972, on the PICC Honeywell 635 computer. A relocatable binary deck of the program is stored on tape no. 8312 and will be retained indefinitely for documentation purposes.

The following stress analyses were performed using the ME-632 piping stress analysis computer program:

Monticello Nuclear Power Plant Core Spray Piping System, Unit 1	Deadweight, thermal with anchor movements
SMUD Rancho Seco, Unit 1 (Lines 48223-18-HE, 50056-10-HE, 50057-10-HE)	Earthquake

The resulting forces, moments, deflections, and stresses were compared with independent analyses performed on the same piping systems using the same loadings. A comparison of results showed that differences in the output quantities were < 5% based upon the corresponding maximum value.

Based upon these results, the ME-632 program may be used with confidence to analyze piping systems per the ASME, Section III, Nuclear Piping Code.

The purpose and method of analysis of ME101 is essentially the same as ME-632.

2. ME-660/ME-661/ME-662

a. Description

Purpose

The purpose of the ME-660, -661, -662 program was to determine the temperature and stress distribution within a body as a function of time when subjected to thermal and/or mechanical loads. The program is valid for axisymmetric or plane structures and would typically be used for gross or local discontinuity analysis as described in paragraphs NB-3213.2 and NB-3213.3 of the ASME Code, Section III.

Method of Analysis

The program consists of three parts each of which can be used separately. The first part, ME-660, calculates steady-state or transient temperature distributions due to temperature or heat flux inputs. The method used is the finite element technique coupled with a step-by-step time integration procedure. The program adopts a stepwise description of environmental temperatures and heat-transfer coefficients if they are time dependent. Transient temperature distributions are calculated from the specified initial temperature and the step function heat inputs. ME-660 is for plane and axisymmetrical structures.

The second part of the program, ME-661, is built on the displacement method of the matrix theory of structures which calculates the displacements and stresses within the solids with orthotropic, temperature-dependent, nonlinear material properties. ME-661 is also for plane and axisymmetrical structures.

The third part of the program, ME-662, calculates the steady state or transient temperature distribution due to temperature or heat flux inputs. The output of this program gives the code required parameters, i.e., Δt_1, Δt_2, T_a, and T_b, where: Δt_1 is the linear thermal gradient, Δt_2 is the nonlinear thermal gradient, and T_a and T_b are the average temperature on side a and b of a gross discontinuity. ME-662 is for straight pipe only.

b. Extent of Application

The program was used to calculate ΔT_1, ΔT_2, and $|T_a-T_b|$ terms for the analysis of ASME Nuclear Class 1 piping systems.

c. Design Control Measures

The program has been verified by solving a problem and comparing the results to the solution of an identical problem obtained by hand calculation. The results were almost identical.

3. ME-913

a. Description

Purpose

ME-913 program consists of numerical calculations of stress intensity levels for Class 1 nuclear power piping components to validate their design adequacy.

Method of Analysis

The program determines the stress intensity levels of Class 1 nuclear power piping components for equations 9 through 14 of Subarticle NB-3650, Analysis of Piping Components, Section III, ASME B&PV Code. The method is described in detail in that subarticle.

Prior to running this program, the user analyzes the piping system using flexibility analysis program ME-632 and heat transfer program ME-662. The inputs to this program are the following:

- Piping configuration.
- Piping and piping component properties.
- Moment reactions due to:
  - Thermal expansion loads
  - Weight loads
  - Earthquake loads
- The thermal response of the piping system due to the specified transients: ΔT_1, ΔT_2, T_a, and T_b values for the selected points in the system.

b. Extent of Application

The ME-913 program was used to calculate stress intensity levels and fatigue usage factors for all Bechtel-supplied ASME Nuclear Class 1 piping.

c. Design Control Measures

Stress analyses were performed using ME-913 code compliance computer program for the following examples:

- Sample Analysis of a Class 1 piping system prepared by the working group on piping of the ASME B&PV Code

- Analysis of main feedwater piping inside containment for Grand Gulf-Units 1 and 2 of Mississippi Power and Light Company

The resulting stresses were compared with the results from working group calculations and hand computations performed on the same piping systems using same loadings. A comparison of results showed that the differences in the output quantities were very conservative based on the corresponding maximum value. Judging from these results, the ME-913 program may be used with confidence to analyze Nuclear Class 1 piping systems per ASME Section III.

4. TRHEAT

a. Description

TRHEAT, which was developed by the Nuclear Service Corporation of Campbell, California, is a digital computer program which determines the temperature response of a pipe due to a temperature transient in the contained fluid. The fluid temperature transient may be described as a step change from an initial to a final temperature, a ramp change terminating in a constant temperature plateau, or a series of time versus temperature points. TRHEAT results include the equivalent linear and nonlinear pipe wall temperature gradients and the discontinuity temperature differences required for calculations of piping stresses in accordance with the requirements for Class 1 piping specified in the ASME B&PV Code, Section III, Nuclear Power Plant Components.

The method of analysis used is a closed-form solution to the basic heat transfer partial differential equation.

b. Extent of Application

The TRHEAT program was used to calculate ΔT_1, ΔT_2, and $\left|T_a - T_b\right|$ terms for the analysis of ASME Nuclear Class 1 piping systems.

c. Design Control Measures

TRHEAT has been verified by the originator of the program, Nuclear Services Corporation.

5. SAP

a. Description

The SAP is a finite element computer program that is used to perform linear, elastic analyses of three-dimensional structural systems. The program is capable of performing static analyses, modal extractions, and dynamic response analyses on structures composed of any combination of a variety of modeling elements.

b. Extent of Application

The SAP was used to analyze the main steam piping for the effects of turbine stop valve closure and relief valve lifting as described in paragraph 3.9.1.1. This analysis was done by GE.

c. Design Control Measures

The program has been in existence many years and has been used for many applications in industry.

6. RVDFT

a. Description

The program was developed to analyze the effects of transient flows resulting from actuation of relief valves in closed discharge systems.

The analysis considers the steam flow, the flow of the air originally in the pipe, and the water slug at the submerged end. The method of characteristic was adopted for the analysis of the resulting unsteady pipe flows. The steam and air are treated as ideal gases while the water is dealt with as a compressible liquid (conventional hydraulic transients approach). The analysis uses a unique procedure for modeling the motion of the air and water interface during the transient. The computer code RVDFT was developed to predict pressure, velocity and density changes with time along the pipe. The computed flow

results are used to predict piping loads (dynamic forcing functions used for the structural design of the discharge pipe). The analysis is applicable to Mark I, Mark II, and Mark III containments as well as PWR quencher tanks.

b. Extent of Application

The program was used in the analysis of the main steam safety relief valve piping in the suppression chamber and the RHR relief valve piping.

c. Design Control Measures

A comparison of the program solution and actual test results taken at Quad Cities-Unit 2 was made. There is good agreement between the pressure measurements and the values predicted by the program.

C. S. Levy, Inc.

1. ANSYS

a. Description

ANSYS, Revision 3, which was developed by the Swanson Analysis Systems, Inc., is a general purpose computer program that can be used to solve several classes of engineering analysis problems. Analysis capabilities include static and dynamic; elastic, plastic, creep, and swelling; small and large deflections; steady-state and transient heat transfer and fluid flow. The matrix displacement method of analysis based on finite element idealization is used in the code. The ANSYS program has the capability of analyzing two- and three-dimensional frame structures, piping systems, two-dimensional plane and axisymmetric solids, three-dimensional solids, flat plates, and three-dimensional shells and nonlinear problems, including interfaces and cables.

Loading on the structure may be forces, displacements, pressures, temperatures, or response spectra. Loadings may be time functions for linear and nonlinear dynamic analysis. The ANSYS program uses the wave front direct solution method for the system of simultaneous linear equations developed by the matrix displacement method.

b. Extent of Application

The ANSYS code was used in the seismic analysis performed to evaluate the use of 80-mil fuel assembly channels. The seismic analysis methodology employed used the conservative response spectrum method with the ANSYS code.

c. Design Control Measures

The ANSYS code is a publicly available code which is appropriate for this application. An independent verification was performed to confirm that the assumptions and method of analysis are reasonable and that the results are consistent with a previous independent analysis method.

D. Zentech, Inc.

1. Description

ZENPIPE is a computer program that may be used to evaluate the static and dynamic performance of piping systems under various loading conditions. The program is marketed and supported by Zentech, Inc.

ZENPIPE utilizes the stiffness method to simultaneously analyze complex piping systems for any combination of multiple thermal, pressure, uniform support displacement, and concentrated force/moment loadings. A weight analysis may also be performed. Any of these static load combinations can be intermixed with a shock loading which may be determined by response spectrum analysis, response history by mode superposition, or response history by direct integration. Stresses for each loading or loading combination are calculated in accordance with the specified code and may be checked for code compliance at the user's option. ZENPIPE generates a full code compliance report for the ANSI Code B31.1, as well as the ASME Code, Section III, Division 1, Class 2 and 3.

2. Extent of Application

ZENPIPE has been used for thermal and weight, seismic analysis on ASME Class 2 and 3, and ANSI B31.1 piping in both nuclear and balance-of-plant systems.

3. Design Control Measures

The ZENPIPE computer code has been tested against the piping benchmark problems (dynamic analysis uniform support motion response spectrum method) in NUREG/CR-1677, BNL-NUREG-51267, Vol. I. The computer program was used to model seven piping systems for spectrum analysis. Mode shapes, response displacements, and support loads were reviewed, and the outputs obtained from the program were compared to those listed in NUREG/CR-1677, BNL-NUREG-51267, Vol. I. Resulting stress and code compliance was checked against hand calculations and other widely used piping software. Overall, ZENPIPE performed as expected. The accuracy it yields is within an acceptable range for engineering calculation, and this is suitable for Quality Class Category 1 calculations.

E. SST Systems, Inc.

The following computer programs were used in the analysis of Seismic Category I piping:

- CAEPIPE.
- PS+CAEPIPE.

1. CAEPIPE

a. Description

CAEPIPE is a PC-based piping stress analysis program for the nuclear, power, and petroleum industries. The software was developed and is distributed by SST Systems, Inc. Static and dynamic analyses (calculations of loads, pipe forces, displacements, mode shapes, etc.) may be performed. The piping system can be subjected to multiple load cases of different types such as thermal and seismic loads and checked for code compliance (ANSI, ASME, etc.)

b. Extent of Application

The software is classified as a Computer Software for Safety-Related Application. This program has been used for thermal and weight, seismic analysis on ASME Class 2 and 3, and ANSI B31.1 piping in both nuclear and balance-of-plant systems.

c. Design Control Measures

The CAEPIPE computer program has been benchmarked against the three test problems in NUREG/CR-1677, BNL-NUREG-51267, Vol. I, applicable to the capabilities of the program. All outputs were comparable. Based on this review, CAEPIPE is acceptable for Quality Class Category 1 calculations. It has also been benchmarked against other computer programs (e.g., ADLPIPE, TPIPE, NUPIPE, and WESTDYN), and it produced equally acceptable results.

2. PS+CAEPIPE

a. Description

PS+CAEPIPE is a group of interrelated computer programs for performing linear elastic analysis of three-dimensional piping systems subject to a variety of loading conditions. Nuclear and conventional power generation piping systems may be investigated for compliance with piping codes and with other constraints on system response. The

software was developed by DST Computer Services and SST Systems, Inc.

PS+CAEPIPE includes the PIPESTRESS software. PIPESTRESS has advanced static and dynamic analysis capabilities including detailed uniform and multi-level response spectrum analyses, time history and fatigue calculations, and multiple load cases and load combinations. Stresses due to internal pressure are calculated according to the code. PIPESTRESS solves static problems by constructing a linear finite element model of the piping system using the load-deflection relationships based on the displacement method. Dynamic analysis calculates bound solutions or time history solutions for dynamic loads, which may be described by response spectra or time history data. The dynamic analysis methods used by PIPESTRESS are Modal Extraction, Single or Multi-level Response Analysis, Multi-modal/Multi-level Response Analysis, Generalized Response Analysis, Selective Time History Analysis, Left-Out-Force Method, and Primary and Secondary Terms involved in Multi-level Response Analysis. Thermal transient analysis can be performed using a finite difference approximation to find thermal gradients in the pipe walls, thereby determining the maximum value during the transient analysis of the various stress terms. Fluid properties are calculated as functions of instantaneous transient fluid temperatures and pressures.

b. Extent of Application

The software is classified as a Computer Software for Safety-Related Application. This program has been used for thermal and weight, seismic analysis on ASME Class 2 and 3, and ANSI B31.1 piping in both nuclear and balance-of-plant systems.

c. Design Control Measures

PS+CAEPIPE was benchmarked against all seven test problems in NUREG/CR-1677, BNL-NUREG-51267, Vol. I, and against the entire set of test problems in NUREG/CR-1677, BNL-NUREG-51267, Vol. II. These permitted the verification of the analysis methods implemented in PIPESTRESS. The program was verified under a nuclear quality assurance program established in accordance with 10 CFR 50, Appendix B, and 10 CFR 21. The verification was directed by personnel competent in the design of ASME Section III, Nuclear Power Plant Components, Class 1, 2, and 3 nuclear power plant piping under ASME nuclear quality assurance procedures. The program performs calculations in accordance with the requirements and intention of Subarticles NC/ND-3600 of ASME, Section III. PIPESTRESS has been used to analyze piping for more than 100 nuclear power plants, and for numerous conventional power plants, chemical plants, oil refineries, and other process plants.

3.9.3 COMPONENTS NOT COVERED BY ASME CODE

3.9.3.1 General

Safety-related mechanical components not covered by the ASME B&PV Code are identified in table 3.9-10. The design codes for each principal component are identified and qualification methods for such equipment are summarized herein. This subsection specifically addresses the details of the mechanical design and analytical procedures for the design of the fuel; the methods and procedures used to determine the operability of the CRD and control rod insertability under LOCA and seismic loadings; mechanical design and loading criteria for HPCI and RCIC turbines; and applicable standards, codes, and testing for heating, ventilation, and air-conditioning (HVAC) equipment.

3.9.3.2 Fuel Mechanical Design and Analytical Procedures

The fuel bundle performance history is specified by the design reference fuel cycle as defined in subsection 4.2.1. Performance of individual fuel rods is then determined from the fuel-bundle performance history coupled with the exposure-dependent design, local and axial power, and exposure-peaking factors. The most limiting fuel rods within the peak performance fuel bundle, with respect to power and exposure combination, are then analyzed to determine thermal and mechanical performance characteristics.

The performance of all fuel rods satisfies the requirements identified in paragraph 4.2.1.1. Satisfaction of these requirements for all fuel rods is demonstrated by analysis of the performance of the most limiting fuel rods with respect to power and exposure level identified in the design reference fuel cycle.

Thermal design analyses performed include, but are not limited to, the determination of clad and fuel temperatures, clad and fuel thermal expansion, fuel irradiation swelling, fuel fission gas generation and release as a function of time. Employing these thermal analysis results, the mechanical design analyses are then performed to determine the most limiting clad stress and/or strain due to such loadings as:

- Internal fuel rod pressure from gaseous fission product release to the fuel-rod plenum plus initial fill gas.
- Differential fuel-clad expansions.
- External coolant pressure.
- Flow-induced rod vibrations.

Finally, the limiting combinations of cladding stress in the categories summarized in subsection 4.2.1 are identified and compared to the cladding design stress limits. All stresses are below the defined limits.

3.9.3.3 <u>Control Rod Drive Operability and Control Rod Insertability Under LOCA and Seismic Loadings</u>

In the event of a significant seismic disturbance and/or LOCA, only the rapid insertion mode (scram) is essential. Descriptions of the CRD and the CRD system operation during scram are covered in subsection 4.2.3.

The hydraulic nature of the CRDs and their location relative to the reactor vessel provides scram operability of the control rods during seismic events is assured by the generous control rod-to-channel and control rod-to-guide tube clearances. However, LOCA produces larger than normal pressure differentials across the reactor vessel internals, thus tending to reduce these clearances. These pressure differentials are considered in determining the insertability of the control rods.

The highest pressure differentials across the RPV internals occur as a result of a postulated steam line break. To ensure adequate rod-to-guide-tube clearance, the guide tube must be capable of resisting the external to internal pressure difference without collapse. In addition, any increase of friction force due to channel bulging is shown to be small compared to the total addressed in subsections 4.2.2 and 4.2.3.

3.9.3.4 <u>HPCI and RCIC Turbines</u>

The turbine mechanical design and loading criteria are given in tables 3.9-23 and 3.9-25.

3.9.3.5 <u>HVAC Equipment (Safety-Related)</u>

Table 3.9-35 presents a list of safety-related HVAC equipment, the applicable standards and codes to which they are designed, and test report numbers and/or procedure numbers for the equipment.

3.9.4 POWER UPRATES

Evaluations to support increases in power output including extended power uprate, thermal power optimization uprate, and the reactor operating pressure increase to 1060 psia, are summarized in references 8, 9, 10, and 11. The results of the evaluations indicate that the piping and components within the scope of section 3.9 are within acceptable limits for operation at 100% RTP of 2804 MWt.

REFERENCES

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5. "80-mil Channel Change for Hatch-2 Cycle-4," GE Letter, L. M. Quintana (GE) to K. G. Turnage (SCS), May 18, 1983.

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9. “Safety Analysis Report for Edwin I. Hatch Units 1 and 2 Thermal Power Optimization,” NEDC-33085P, GE Nuclear Energy, December 2002.

10. “10-PSI Dome Pressure Increase Project Report for Edwin I. Hatch Units 1 and 2,” GE-NE-0000-0003-0634-01, Revision 1, GE Nuclear Energy, July 2003.

11. RER 03-254, Reactor Operating Pressure Increase From 1050 psia to 1060 psia, Engineering Evaluation.

TABLE 3.9-1

EVENT LOAD COMBINATION CRITERIA

Event or Load Combination	Event Category	Criteria
Normal operation	Normal	Normal
Operational transients	Upset	Upset
OBE	Emergency	Upset
OBE + normal operation	Emergency	Upset
OBE + operational transients	Emergency	Emergency
DBE + operational transients	Faulted	Faulted

TABLE 3.9-2 (SHEET 1 OF 2)

MAIN STEAM LINE PIPING SYSTEM (CLASS 1 PIPE)

Condition	Load Combination	Criteria
Design	P + W + OBE	Eq 9A $< 1.5\ S_m$
Normal and Upset	For dynamic loads, individually considering: OBE, TSV, RV with other ASME Section III Code-defined loads	Eq 9B $< 1.8\ S_m$ Eq 10 $< 3.0\ S_m$ Eq 12 $< 3.0\ S_m$ Eq 13 $< 3.0\ S_m$ Eq 14 U < 1.0
Emergency	$P_e + W + [(OBE)^2 + (TSV)^2]^{1/2}$	Eq 9C $< 2.25\ S_m$
	$P_e + W + [(OBE)^2 + (RV)^2]^{1/2}$	
Faulted	$P_e + W + [(DBE)^2 + (TSV)^2]^{1/2}$	Eq 9D $< 3.0\ S_m$
	$P_e + W + [(DBE)^2 + (RV)^2]^{1/2}$	

LEGEND

P = stresses due to design pressure.
P_e = stresses due to peak pressure.
W = stresses due to weight pressure.
RV = stresses due to safety relief valve opening.
TSV = stresses due to turbine stop valve closure.

TABLE 3.9-2 (SHEET 2 OF 2)

Criteria Per ASME Section III NB-3600	Node No.	Main Steam Line	Maximum Stress Intensities (psi): Power Uprate Stress[b]	Maximum Stress Intensities (psi): Code Allowable Stress	Maximum Stress Intensities (psi): Ratio: Power Uprate to Allowable[a]
Equation 9: Design	21	D	11,942	26,550	0.45
Normal/Upset	210	B	16,554	31,860	0.52
Emergency	210	B	16,516	39,825	0.41
Faulted	21	D	17,435	53,100	0.33
Equation 10	49F	C HPCI	57,560	53,100	1.08
Equation 12	49F	C HPCI	42,882	53,100	0.81
Equation 13	21	D	32,311	53,100	0.61
Equation 14 (Fatigue)	17	D	CUF = 0.28	CUF < 1.0	

a. Since equation 10 is not satisfied, the piping is qualified by meeting equations 12 and 13, and the maximum stress for equation 13 for any node occurs at node point 21 on line D.
b. Values reflect extended power uprate evaluation. Thermal power optimization evaluations indicate no impact and 10-psi reactor operating pressure increase evaluation indicates there are only minor increases in stresses which remain within available margins.

TABLE 3.9-3 (SHEET 1 OF 4)

LOADING COMBINATIONS UNDER VARIOUS PLANT CONDITIONS

NONSTANDARD LINEAR-TYPE SUPPORTS

	Load Combinations	Allowable Stress In Accordance With Appendix XVII-2000
I.	Design, Normal, Upset	
	Hydro weight + hydro pressure[a]	0.66 F_y bending
	Deadweight + thermal + secondary OBE + primary OBE + design pressure[a] + RVO	0.4 F_y shear
	Deadweight + thermal + RVC + design pressure[a]	0.6 F_y tensile
	Deadweight + thermal + FV + design pressure[a]	
II.	Emergency[c][1]	
	Deadweight + design pressure[a] + primary OBE + RVC	1.33 (0.66 F_y) bending
	Deadweight + design pressure[a] + primary OBE + FV	1.33 (0.4 F_y) shear
		1.33 (0.6 F_y) tensile
III.	Faulted	
	Deadweight + design pressure[a] + primary DBE + RVC or RVO + FV	1.2 F_y but < 0.7 Su
	Pipe rupture for whip restraints in contact with pipe[b]	1.2 F_y but < 0.7 Su
	Pipe rupture for whip restraints not in contact with pipe	BN-TOP-2, Rev 2

TABLE 3.9-3 (SHEET 2 OF 4)

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a. Pressure included only for checking stresses in the pipe wall.

b. The faulted condition of pipe rupture for whip restraints normally in contact with the pipe is evaluated independently of all other loading combinations. The loads used in these calculations are based on the ultimate strength of the pipe. Whip restraints not normally in contact with the pipe are discussed separately in subsection 3.6.5 and supplement 15A.

c. See paragraph 3.9.1.1.1. This load combination was used only for the turbine stop and safety relief valves. During the long-term blowdown following the establishment of steady-state flow for a closed relief valve discharge system, the reactions on the discharge piping, relief valve, and inlet piping are balanced, and no stresses are introduced as a result of relief valve blowdown. The time duration for the stresses induced during the transient preceding steady-state flow is ~ 200 ms. After this period of time, the motions are damped out.

   It may be argued that an earthquake could cause a plant trip and consequential relief valve actuation. However, the probability of the maximum stresses from these transients (in a time sense) occurring at the same location, at the same instant in time and in place, is extremely low.

   Furthermore, the number of cycles (3-10) during which both are occurring is more extremely low.

NOTES:

1. The rigid restraints for the main steam and the relief valve discharge lines are designed with adequate margin in the design stress so that the normal stress allowables can be met, using the emergency condition loading combination.

   The snubbers used on the main steam and the relief valve discharge lines are designed with adequate margin in the design stress so that normal stress allowables can be met, using loading combination I of the faulted condition loading combinations.

LEGEND

RVO - dynamic effects associated with an open relief valve discharge
FV - dynamic effects associated with fast valve closure
RVC - dynamic effects associated with a closed relief valve discharge

TABLE 3.9-3 (SHEET 3 OF 4)

PLATE- AND SHELL-TYPE SUPPORTS

Load Combinations	Allowable Stress
I. Design, Normal, Upset	
Hydro weight + hydro pressure[(a)]	
Deadweight + thermal + FV + design pressure[(a)]	S_h general membrane
Deadweight + thermal + primary OBE + secondary OBE + design pressure[(a)] + RVO	1.5 S_h general membrane plus bending
II. Emergency[(c)(1)]	
Deadweight + design pressure[(a)] + primary OBE + RVC or FV	1.2 S_h general membrane
	1.8 S_h general membrane plus bending
III. Faulted	
Deadweight + design pressure[(a)] + primary DBE + RVO or RVC + FV	1.2 F_y[(b)] but < 0.7 Su
Pipe rupture for whip restraints normally in contact with pipe	1.2 F_y[(b)] but < 0.7 Su
Pipe rupture for whip restraints not normally in contact with pipe	BN-TOP-2, Rev 2

TABLE 3.9-3 (SHEET 4 OF 4)

a. Pressure needs to be included only for checking stresses in the pipe wall.

b. F_y is the minimum yield stress of the component at the operating temperature. Normally, supports are designed elastically. If 1.2 F_y is used, the effects of plastic deformation on the elastic analysis are considered.

c. See paragraph 3.9.1.1.1. This load combination was used only for the turbine stop and safety relief valves. During the long-term blowdown following the establishment of steady-state flow for a closed relief valve discharge system, the reactions on the discharge piping, relief valve, and inlet piping are balanced, and no stresses are introduced as a result of relief valve blowdown. The time duration for the stresses induced during the transient preceding steady-state flow is ~ 200 ms. After this period of time, the motions are damped out.

It may be argued that an earthquake could cause a plant trip and consequential relief valve actuation. However, the probability of the maximum stresses from these transients (in a time sense) occurring at the same location, at the same instant in time and in place, is extremely low.

Furthermore, the number of cycles (3-10) during which both are occurring is likewise extremely low.

NOTES

1. The rigid restraints for the main steam line are designed with adequate margin in the design stress so that the normal stress allowables can be met, using the emergency condition loading combination.

   The snubbers used on the main steam line and the relief valve discharge lines are designed with adequate margin in the design stress so that normal stress allowables can be met, using loading combination I of the faulted-condition loading combinations.

LEGEND

RVO - dynamic effects associated with an open relief valve discharge
FV - dynamic effects associated with fast valve closure
RVC - dynamic effects associated with a closed relief valve discharge

TABLE 3.9-4 (SHEET 1 OF 9)

RPV INTERNALS AND ASSOCIATED EQUIPMENT

Criteria	Loading[a][h]	Primary Stress Type	Allowable Stress (psi)	Calculated Stress (psi)
Top Guide - Highest Stressed Beam				
Primary stress limit - The allowable primary membrane stress plus bending stress is based on ASME B&PV Code, Section III, for Type 304 stainless-steel plate.				
For normal and upset condition stress intensity:	Normal and upset conditions:			
$S_A = 1.5\ S_m = 1.5 \times 16{,}925$ psi $= 25{,}388$ psi	Normal ΔP Weight of structure OBE, vertical and horizontal	General membrane plus bending	25,388	15,258
	Upset ΔP due to loss-of-feedwater heaters Weight of structure	General membrane plus bending	25,388	3426
For emergency condition:	Emergency condition:			
$S_{limit} = 1.5\ S_A = 1.5 \times 25{,}388 = 38{,}081$ psi	Emergency ΔP due to ADS actuation[b] Weight of structure	General membrane plus bending	38,081	3908
For faulted condition:	Faulted condition:			
$S_{limit} = 2\ S_A = 2 \times 25{,}388 = 50{,}775$ psi	LOCA ΔP due to steam line break Weight of structure DBE, horizontal and vertical	General membrane plus bending	50,775	25,215

TABLE 3.9-4 (SHEET 2 OF 9)

Criteria	Loading[a][h]	Primary Stress Type	Allowable Stress (psi)	Calculated Stress (psi)
Top Guide Beam End Connections				
Primary stress limit - ASME B&PV Code, Section III, defines material stress limit for Type 304 stainless steel.				
For normal and upset condition stress intensity:	Normal and upset conditions:			
$S_A = 0.6\ S_m = 0.6 \times 16{,}925$ psi $= 10{,}155$ psi	Normal ΔP Weight of structure OBE, horizontal and vertical	Pure shear	10,155	6744
	Upset ΔP due to loss-of-feedwater heaters Weight of structure	Pure shear	10,155	1275
For emergency condition:	Emergency condition:			
$S_{limit} = 1.5\ S_A = 1.5 \times 10{,}155$ psi $= 15{,}232$ psi	Emergency ΔP due to ADS actuation[b] Weight of structure	Pure shear	15,232	1454
For faulted condition:	Faulted condition:			
$S_{limit} = 2\ S_A = 2 \times 10{,}155$ psi $= 20{,}310$ psi	LOCA ΔP due to steam line break Weight of structure DBE, horizontal and vertical	Pure shear	20,310	11,291

TABLE 3.9-4 (SHEET 3 OF 9)

Criteria	Loading[a][h]	Primary Stress Type	Allowable Stress (psi)	Calculated Stress (psi)
Top Guide Aligners				
Primary stress limit - The allowable primary membrane stress plus bending stress is based on ASME B&PV Code, Section III, for Type 304 stainless-steel plate.				
For normal and upset condition stress intensity:	Normal and upset conditions:			
$S_A = 1.5 \ S_m = 1.5 \times 16{,}925$ psi $= 25{,}388$ psi	Normal ΔP Weight of structure OBE, horizontal and vertical	General membrane plus bending	25,388	(c)
	Upset ΔP due to loss- of-feedwater heaters Weight of structure	General membrane plus bending	25,388	(c)
For emergency condition:	Emergency condition:			
$S_{limit} = 1.5 \ S_A = 1.5 \times 25{,}388 = 38{,}081$ psi	Emergency ΔP due to ADS actuation[b] Weight of structure	General membrane plus bending	38,081	(c)
For faulted condition:	Faulted condition:			
$S_{limit} = 2 \ S_A = 2 \times 25{,}388 = 50{,}775$ psi	LOCA ΔP due to steam line break Weight of structure DBE, horizontal and vertical	General membrane plus bending	50,775	(c)

TABLE 3.9-4 (SHEET 4 OF 9)

Criteria	Loading[a][h]	Primary Stress Type	Allowable Stress (psi)	Calculated Stress (psi)
Core Support	Normal and upset conditions:			
Primary stress limit - The allowable primary membrane stress plus bending stress is based on ASME B&PV Code, Section III, for Type 304 stainless-steel plate.	Normal ΔP Weight of structure OBE, horizontal and vertical	General membrane plus bending	25,388	18,650
For allowable stresses, see Top Guide, Longest Beam.	Upset ΔP due to loss-of-feedwater heaters Weight of structure	General membrane plus bending	25,388	10,902
	Emergency condition:			
	Emergency ΔP due to ADS actuation[b] Weight of structure	General membrane plus bending	30,081	10,921
	Faulted condition:			
	LOCA ΔP due to steam line break Weight of structure DBE, horizontal and vertical	General membrane plus bending	50,775	24,937

TABLE 3.9-4 (SHEET 5 OF 9)

Criteria	Loading[a][h]	Primary Stress Type	Allowable Stress (psi)	Calculated Stress (psi)
Core Support Aligners				
Primary stress limit - ASME B&PV Code, Section III, defines material stress limit for Type 304 stainless steel.	Normal and upset conditions:			
For allowable shear stresses, see Top Guide Beam, End Connections.	Normal ΔP Weight of structure OBE, horizontal and vertical	Pure shear	10,155	(d)
	Upset ΔP due to loss-of-feedwater heaters Weight of structure			
	Emergency condition:			
	Emergency ΔP due to ADS actuation[b] Weight of structure	Pure shear	15,232	(d)
	Faulted condition:			
	LOCA ΔP due to steam line break Weight of structure DBE, horizontal and vertical	Pure shear	20,310	2000

TABLE 3.9-4 (SHEET 6 OF 9)

Criteria	Loading[e][h]	Primary Stress Type	Allowable Stress (psi)	Calculated Stress (psi)
CRD Housing				
For normal and upset condition:	Normal and upset condition loads:			
	Design pressure Stuck rod scram loads OBE, with housing lateral support installed	Maximum membrane stress intensity occurs in tube-to-tube weld near center of housing for normal, upset, and emergency conditions.	S_m = 16,660	13,150
Primary stress limit - The allowable primary membrane stress is based on ASME B&PV Code, Section III, for Type 304 stainless steel.				
For emergency condition:				
S_{limit} = 1.2 S_m in accordance with ASME Section III.	Emergency condition loads:			
	Design pressure Stuck rod scram loads DBE, with housing lateral; lateral support installed		1.2 S_m = 20,000	13,150
	OBE 0.8-g horizontal (statically applied) OBE 0.2-g vertical (statically applied)			
	OBE 1.6-g horizontal (statically applied) OBE 0.4-g vertical (statically applied)			

TABLE 3.9-4 (SHEET 7 OF 9)

Criteria	Loading[e][h]	Primary Stress Type	Allowable Stress (psi)	Calculated Stress (psi)
CRD				
Primary stress limit - The allowable primary membrane stress plus bending is based on ASME B&PV Code, Section III.				
For normal and upset conditions:	Normal and upset condition loads:			
$S_m = 1.5 \times 17,238 = 25,860$ psi	Maximum hydraulic pressure from CRD supply pump[f]	Maximum stress intensity occurs at point on Y-Y axis of indicator tube.	25,860	20,790
CRD Guide Tube				
Primary stress limit - The allowable primary membrane stress plus bending stress is based on ASME B&PV Code, Section III for Type 304 stainless-steel tubing.				
For normal and upset condition:	Normal condition:			
$S_m = 16,925$ psi	Since calculated stresses for faulted condition are less than normal condition allowables, normal condition is satisfied and not reported.			

TABLE 3.9-4 (SHEET 8 OF 9)

Criteria	Loading[e][h]	Primary Stress Type	Allowable Stress (psi)	Calculated Stress (psi)
CRD Guide Tube (continued)				
For faulted condition:	Faulted condition loads:			
$S_{limit} = 1.5\ S_m = 1.5 \times 16{,}295 = 25{,}400$ psi	Deadweight Pressure drop across guide tube due to failure of steam line Crossflow loading Seismic loading DBE 1.2-g horizontal (statically applied) DBE 0.14-g vertical[g] (statically applied)	Maximum bending stress under faulted loading condition occurs at center of guide tube.	25,400	6061
Incore housing				
Primary stress limit - The allowable primary membrane stress is based on ASME B&PV Code, Section III, for Type 304 stainless steel.				
For normal and upset condition: $S_m = 16{,}660$ psi at 575°F	Since emergency condition stresses are less than normal condition limits, normal condition is satisfied and not reported.			
For emergency condition: $S_{limit} = 1.2\ S_m = 1.2 \times 16{,}660 = 20{,}000$ psi	Emergency condition load: Design pressure DBE DBE 1.6-g horizontal (statically applied) DBE 0.4-g vertical (statically applied)	Maximum membrane stress intensity occurs at outer surface of vessel penetration.	20,000	15,290

TABLE 3.9-4 (SHEET 9 OF 9)

a. The horizontal load is based on the results of the dynamic seismic analysis of the building for 0.08-g OBE and 0.15-g DBE free-shield ground motion. The vertical load is based on 0.1-g OBE and 0.2-g DBE times the component weight statically applied. The load combination method used was the absolute sum of the individual loads.

b. Automatic depressurization system.

c. Twenty-four wedges, which will resist the horizontal seismic top-guide shear load, are installed in the annulus between the top guide and the shroud. Therefore, there is no load on the top-guide aligners.

d. The friction force between core support and core support flange due to the preload of the studs is greater than the shear load induced by the specified earthquake.

e. These loads were directly combined.

f. Accident conditions do not increase this loading. Earthquake loads are negligible. Direct addition of all other loads is less than the hydraulic pressure load, and other loads are not additive to the hydraulic pressure load.

g. 0.14-g vertical = 70.5 psi and is considered negligible.

h. The analyses were performed assuming 100-mil-thick fuel assembly channels. The effect on seismic loads due to a design change from 100-mil to 80-mil-thick channels is negligible.[(4)]

TABLE 3.9-5

HNP-2 LOAD COMBINATIONS (SEISMIC + LOCA)

Component	Loads Combined	Method of Combination[a]	FSAR Analysis Reference	Remarks
RPV shell	Seismic, mechanical, thermal, and transient (NPC + DBE + DSL)	Direct addition	Section 3.9	Meets Article 4, Section III, ASME Code for all loads; also meets ASME Section III, N-417.11.
RPV nozzles	Seismic, mechanical, thermal, and transient (NPC + DBE + DSL)	Direct addition	Section 3.9	Meets Section III, ASME Code, N-450; also meets ASME Section III, N-417.11.
RPV skirt and stabilizers	Annulus pressurization. (See FSAR supplement 6A.)	Direct addition	Supplement 6A	DBE added by SRSS due to very low probability of combining with specific break location. (Not original design basis.) Elastic stress limit not exceeded.
RPV internals	Seismic, deadweight, LOCA (steam line break P_e + W + DBE)	Direct addition	Table 3.9-4	SRSS load combination justified (dynamic loads).
Class 1 piping (unbroken)	Seismic, deadweight, LOCA (P_o + W + DBE) or (NPC + DBE + DSL)	Direct addition except for RV and TSV operation	Tables 3.9-2, 3.9-6. Paragraph 3.9.1.5	Meets ASME Code, Section III, NB-3656.
Class 1 valves (pipe mounted)	Same as for Class 1 piping	Same as for Class 1 piping. Piping reaction loads are design limited for valves.	Tables 3.9-14 and 3.9-16. Section 3.9	Relief valves meet ASME NPVC, 1968. MSIV meets ASME Section III, 1971 (winter addenda); recirculation valves meet Article 4 of NPVC, 1968.
Class 1 pumps (inactive) recirculation	Same as for Class 1 piping	Same as for Class 1 piping. Piping reaction loads are design limited for pumps.	Table 3.9-15. Paragraph 3.9.2.2	Meets Section VIII, NPVC.

a. Dynamic loads are combined by SRSS when three or more result in cyclic dynamic responses, for DBE + LOCA, or for any two loads for which it can be demonstrated that the SRSS value has at least an 84% nonexceedence probability (NEP).

TABLE 3.9-6

RECIRCULATION PIPING SYSTEM (CLASS 1 PIPE)[a]

Condition	Load Combination	Criteria	Limit (psi)	Load (psi)	Stress Ratio	Location
Design	P_D + W + OBE_1	≤1.5 S_m	25,875	25,067	0.969	Loop A, node 181, hanger lugs
Normal and upset	TE, P_o, NO, OT, OBE	Eq 10 ≤ 3.0 S_m if Eq 10 exceeds Eq 12 ≤ 3.0 S_m	51,750	39,838	0.77	Loop A, node 220, reducer
		Eq 13 ≤ 3.0 S_m	51,750	49,724	0.961	Loop A, node 500, RHR supply tee
		U ≤ 1.0[b]	-	-	0.05[b]	
Upset	P_o + W + OBE_1	The lesser of Eq 9 ≤ 1.8 S_m and Eq 9 ≤ 1.5 S_y	29,223	25,691	0.879	Loop A, node 181, hanger lugs
Faulted	P_o + W + DBE_1	The lesser of Eq 9 ≤ 3.0 S_m and Eq 9 ≤ 2.0 S_y	38,964	26,060	0.669	Loop A, node 181, hanger lugs

LEGEND

I = inertia
NO = normal operating loads
OT = operating transient loads
P = design pressure stresses
P_o = operating pressure stresses
TE = thermal expansion stresses

a. The HNP-2 recirculation piping was evaluated for the effects of power uprate and shown to satisfy the applicable Code requirements. Reference 7 provides a summary of the results. Effect of extended power uprate, thermal power optimization uprate, and 10-psi reactor operating pressure increase on results summarized in reference 7 is insignificant.

b. Usage factor.

TABLE 3.9-7

SNUBBER EVALUATION FOR LOCA AND DBE

Loop A

Snubber Description[b]	DBE Load (kips)	LOCA Load[a] (kips)	$\sqrt{(LOCA^2 + DBE^2)}$	Faulted Rating (kips)	Ratio
SA1	6.2	3.5	7.1	132	0.05
SA2	31.4	1.6	31.4	132	0.24
SA3	30.2	3.5	30.4	132	0.23
SA4	26.0	3.2	26.2	132	0.20
SA5	23.1	1.1	23.1	132	0.18
SA6	7.8	1.1	7.9	132	0.06
SA14	8.5	3.6	9.2	67.5	0.14
SA19	19.3	17.2	25.9	36.4	0.71
SA20	20.2	15.5	25.5	36.4	0.70
SA21	20.7	3.0	20.9	36.4	0.57
SA22	12.5	0.0	12.5	36.4	0.34

Loop B

Snubber Description[b]	DBE Load (kips)	LOCA Load[a] (kips)	$\sqrt{(LOCA^2 + DBE^2)}$	Faulted Rating (kips)	Ratio
SB1	7.0	3.4	7.8	75	0.10
SB2	40.6	6.4	41.1	75	0.55
SB3	39.1	4.7	39.4	75	0.53
SB4	32.1	3.2	32.3	75	0.43
SB5	28.9	2.4	29.0	75	0.39
SB6	7.9	1.4	8.0	75	0.11
SB12	17.3	7.8	19.0	30	0.63
SB14	8.9	4.2	9.8	45	0.22
SB19	17.2	15.7	23.3	30	0.78

a. Jet impingement only.
b. Snubbers SA7, SA8, SA12, SA13, SA17, SB7, SB8, SB13, SB17, SB20, SB21, and SB22 were deleted from the recirculation piping system during the snubber reduction program.

TABLE 3.9-8 (SHEET 1 OF 2)

LOAD COMBINATIONS FOR LOCA[a] + DBE FOR CLASS 1 RCPB COMPONENTS

Component	Plant Condition	Combination Loading[a]	Stress Limit	ASME Section III Reference
Class 1 piping (See note 1.)	Normal and upset	NPC or UPC + OBE	1.5 S_m (primary)	NB-3652
			3 S_m (primary + secondary)	NB-3653 NB-3654
	Emergency	EPC (weight + maximum pressure)	2.25 S_m (primary)	NB-3655
	Faulted	NPC (weight + pressure + DBE)	3 S_m (primary)	NB-3656 (See Note 4.)
Class 1 valves (inactive) by standard or alternative design rules	Normal and upset	NPC or UPC	S_m (primary) 3 S_m (primary + secondary)	NB-3500
	Emergency	EPC	S_m (primary) 3 S_m (primary + secondary)	NB-3500
	Faulted	NPC (weight + stem thrust + maximum service pressure + DBE (See Note 2.)	S_m (primary)	NB-3524 (See Note 2.)
Class 1 valves (active) by analysis	Normal and upset	NPC or UPC	U.F. < 1	NB-3222.4
	Emergency	EPC	U.F. < 1	NB-3224
	Faulted	NA (See Note 3.)	NA (See Note 3.)	NA (See Note 3.)

TABLE 3.9-8 (SHEET 2 OF 2)

Component	Plant Condition	Combination Loading[a]	Stress Limit	ASME Section III Reference
Class 1 valves (active) by standard or alternative design rules	Normal and upset	NPC or UPC	S_m (primary) 3 S_m (primary + secondary)	NB-3500
	Emergency	EPC	S_m (primary) 3 S_m (primary + secondary)	NB-3500
	Faulted	NPC (weight + stem thrust + maximum service pressure) + DBE (See Note 2.)	S_m (primary)	NB-3524 (See Note 2.)

NOTES:

1. Bechtel analyzed General Electric-supplied main steam and the original recirculation systems for jet impingement and differential pressure.
2. Reference Note 3 of table 3.9-29. LOCA effects are evaluated for the piping; because the valve body is thicker, the piping stress is considered limiting.
3. Design by analysis per NB-3200 is used for fatigue evaluation only.
4. General Electric evaluated the replacement recirculation piping for Bechtel-supplied jet impingement loads.

LEGEND

NPC - normal plant condition
UPC - upset plant condition
EPC - emergency plant condition
NA - not applicable

a. LOCA - jet impingement and differential pressure across components.

TABLE 3.9-9 (SHEET 1 OF 4)

ASME CODE CLASS 2 AND 3 COMPONENTS

	Code Class	Design Pressure (psi)	Design Temperature (°F)	Active
Reactor system				
Power range detector pressure containment parts	2	1250	600	NO
Nuclear boiler system				
Vessels, air accumulators	2	180	340	NO
Piping, relief valve discharge	3	500	470	NO
CRD hydraulic system				
Valves, scram discharge volume lines	2	1250	280	YES
Valves, insert and withdraw lines	2	1750	575	NO
Piping, scram discharge volume lines	2	1250	280	NO
Piping, insert and withdraw lines	2	1750	575	NO
Standby liquid control system (SLCS)				
SLC tank	2	ATM[(a)]	< 250	NO
Pump	2	1400	150	NO
Valves beyond isolation valves	2	1400	150	NO
Piping beyond isolation valves	2	1400	150	NO
Neutron monitoring system				
Piping, TIP[(b)] (reactor pressure, containment)	2	100	340	NO
Valves, isolation TIP[(b)] system	2	100	340	NO

TABLE 3.9-9 (SHEET 2 OF 4)

		Code Class	Design Pressure (psi)	Design Temperature (°F)	Active
RHR system					
Heat exchangers, primary side		3	450	470	NO
Heat exchangers, secondary side		Section VIII	450	470	NO
Piping, beyond outermost isolation valves		2	450	358	NO
Pumps		2	500	40 to 360	YES
Valves, beyond isolation valves		2	300	BW[c] ANSI	YES
CS					
Piping, beyond outermost isolation valves to pump discharge		2	460	225	NO
Pumps		2	500	40 to 212	YES
Valves, beyond outermost isolation valves to pump discharge		2	460	225	YES
HPCI					
Piping beyond outermost isolation valve	Steam	2	1250	575	NO
	Water	2	1330	170	
Pump		2	1500	40 to 140	YES
Valves (other)	Steam	2	1250	575	YES
	Water	2	1330	170	
RCIC system					
Piping beyond outermost isolation valve	Steam	2	1250	575	NO
	Water	2	1300	170	
Pumps		2	1500	40 to 140	YES
Valves (other)	Steam	2	1250	575	YES
	Water	2	1300	170	
Radwaste system					
Valves, containment isolation		2	150	212	YES

TABLE 3.9-9 (SHEET 3 OF 4)

	Code Class	Design Pressure (psi)	Design Temperature (°F)	Active
RWC system				
Filter-demineralizer unit	3	1400	150	NO
Piping, beyond outermost isolation valves	3	1300	575	NO
Pumps	3	1400	575	YES
Valves (other)	3	1300	575	YES
Heat exchangers (regenerative)	3	1400	575	NO
Heat exchangers (nonregenerative)	3	1400 tube 150 shell	575 tube 370 shell	NO
Fuel pool cooling and cleanup (FPCC) system				
Vessels, filter-demineralizers	3	150	150	NO
Vessels (other)	3	150	150	NO
Piping	3	150	150	NO
Pumps	3	150	150	NO
Valves	3	150	150	NO
RHRSW system				
Piping	3	525	125	NO
Pumps	3	595	125	YES
Valves	3	525	125	YES
PSW system				
Pumps	3	180	125	YES
Piping to the reactor building	3	180	125	NO
Valves to the reactor building	3	180	125	YES
Piping in the reactor building	3	185	125	NO
Valves in the reactor building	3	185	125	YES

TABLE 3.9-9 (SHEET 4 OF 4)

	Code Class	Design Pressure (psi)	Design Temperature (°F)	Active
Drywell pneumatic system				
Piping and valves	2	150	353/150	NO
Diesel generator system				
Day tanks	3	atmospheric pressure	105	NO
Piping for diesel service water system	3	180	125	NO
Valves for diesel service water system	3	180	125	YES
Pumps, fuel oil system	3	15	70	YES
Primary containment	MC	56	340	NO
Standby gas treatment system (SGTS)				
Filter train housing	2	+2 to -2	150	NO
Valves	2	150	150	YES
Piping	2 and 3	150	150	NO
Fans	-	+2 to -2	150	YES

a. Atmosphere, standard.
b. Traversing incore probe.
c. Bingham Willamette Company.

TABLE 3.9-10 (SHEET 1 OF 2)

SAFETY-RELATED MECHANICAL COMPONENTS NOT COVERED BY ASME CODE

Principal Components	FSAR Location[a]	Design Code	Qualification Method
Reactor system			
CRD housing supports	4.5	AISC	Analytical
Reactor internal structures, engineered safety features	4.2.2	NA	Analytical and empirical
Control rods	4.2.3.1	NA	Prototype tests
CRD system	4.2.3.2	NA	Analytical and prototype tests
Core support structures	4.2.2	NA	Analytical
RPV stabilizer	5.4.6.3.3.2	AISC	Analytical
Fuel assemblies	4.2.1	NA	Analytical, prototype tests, and operating experience
Recirculation system			
Pipe restraints, recirculation line	3.9.2.1 - 3.9.2.2	AISC	Analytical and tests
CRDH system			
Hydraulic control unit	4.2.3.2	ASME ANSI	Analytical and prototype tests
SLCS			
Atmospheric storage tank	6.3	API-620 API-650	Seismic analyses
HPCI system			
Turbine	6.3	ASME Section VIII[a]	Analytical
RCIC system			
Turbine	6.3	ASME Section VIII[a]	Analytical

TABLE 3.9-10 (SHEET 2 OF 2)

Principal Components	FSAR Location[a]	Design Code	Qualification Method
RHRSW system			
Mechanical draft cooling towers	9.2.7	AISC ACI[c]	Analytical
Diesel generator systems			
Diesel generators	9.5	DEMA ANSI IEEE NEMA[d]	Analytical
SGTS			
Filters, exhaust fans, drivers	6.2.4	AMCA SMACNA[e] ORNL[f] NSIC-65	Analytical and prototype tests
Housing, valves, piping	table 3.2-1	ASME III-2	Seismic calculations
Reactor building ventilation			
All components with safety functions	9.4.2	AMCA SMACNA[e]	Analytical
Emergency equipment area cooling units	9.4	AMCA SMACNA[e]	Analytical

a. ASME Code, Section VIII, used as a design guide.
b. Location of summary of stress and dynamic calculations or experimental testing.
c. American Concrete Institute.
d. National Electric Manufacturers' Association.
e. Sheet Metal and Air Conditioning Contractors National Association.
f. Oak Ridge National Laboratory.

TABLE 3.9-11

FUEL ASSEMBLY WITH 100-mil CHANNELS

	Horizontal Seismic Loadings					
	OBE			DBE		
	80-mil Calculated[b]	100-mil Calculated	Seismic Design Basis[a]	80-mil Calculated[b]	100-mil Calculated	Seismic Design Basis[a]
Shear at top of fuel (lb)	335	300	446	630	563	892
Shear at bottom of fuel (lb)	350	279	436	660	523	871
Maximum fuel moment (lb-in.)	19,200	13,800	21,600	35,800	25,900	43,200
Maximum fuel acceleration (g)		0.834	1.5		1.56	3.0

a. The seismic design basis loading allowances were determined after considering normal loads for the OBE case and accident loads for the DBE case. Design basis loadings are the same for 80-mil and 100-mil channels.
b. See reference 4.

TABLE 3.9-12 (SHEET 1 OF 2)

RPV SUPPORT EQUIPMENT

Criteria	Loading	Location	Allowable Stress (psi)	Calculated Stress (psi)
RPV stabilizer				
Primary stress limit:				
AISC specification for the construction, fabrication, and erection of structural steel for buildings. For normal and upset conditions, AISC allowable stresses, but without the usual increase for earthquake loads.	Upset condition:(b) Spring preload OBE	Rod	84,000	f_t = 78,200(a)
		Bracket	22,000	f_b = 20,900
			14,000	f_v = 13,220
For emergency conditions, 1.5 x AISC allowable stresses.	Emergency condition:(b) Spring preload DBE	Bracket	33,000	f_b = 24,500
			21,000	f_v = 15,510
For faulted conditions, material yield strength.	Faulted condition:(b) Spring preload DBE Jet reactor load	Bracket	36,000	f_b = 26,100
			21,500	f_v = 16,510
CRD housing support				
Primary stress limit:				
AISC specification for the design, fabrication, and erection of structural steel for buildings.	Faulted condition:(c) loads Deadweight Impact force from failure of CRD housing	Beams (top cord)	33,000	f_a = 12,200
			33,000	f_b = 16,500
		Beams (bottom cord)	33,000	f_a = 10,300
			33,000	f_b = 11,700

TABLE 3.9-12 (SHEET 2 OF 2)

Criteria	Loading	Location	Allowable Stress (psi)	Calculated Stress (psi)
For normal and upset conditions:	(Deadweights and earthquake loads are very small as compared to jet force and are not considered.)	Grid structure	41,500	f_b = 40,700
$F_a = 0.60\ F_y$ (tension)			27,500	f_v = 11,100
$F_b = 0.60\ F_y$ (bending)				
$F_v = 0.40\ F_y$ (shear)				
For faulted conditions:				
F_a limit = 1.5 F_a (tension)				
F_b limit = 1.5 F_b (bending)				
F_v limit = 1.5 F_v (shear)				
F_y = material yield strength				

a. The ratio maximum stress limit is highest for upset loading conditions.
b. These loads were directly combined.
c. The only loading condition considered was faulted which assumes the instantaneous circumferential separation of a CRD housing.

TABLE 3.9-13 (SHEET 1 OF 1)

MAIN STEAM RELIEF VALVES (TARGET ROCK)

Note: Target Rock three-stage SRV model 0867F/09G-001 is qualified per S-63182 and S-63848. The excessive detail concerning the design of two-stage SRVs has been removed from this table in accordance with NEI 98-03.

TABLE 3.9-14 (SHEET 1 OF 14)

MAIN STEAM ISOLATION VALVES

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
	All references are made to ASME B&PV Code, Section III, Nuclear Power Plant Components, 1971 Edition, as addended by Summer 1971, Winter 1971, unless otherwise specified. Reference Reference the same code for explanation of the symbols used.		
Body minimum wall thickness	Reference paragraph NB-3543, Nonstandard Pressure-Rated Valve, Table NB-3542-1.		
	For design condition of 1250 psig and 575°F, the primary service rating = 495 based on a core diameter of 21.83 in. t_m = 1.48 in. (including a corrosion allowance of 0.12 in.).	1.48	1.79
Body shape rule	Reference paragraph NB-3544, Body Shape Rules.		
Radius of crotch	Reference paragraph NB-3544.1(a), Radius of Crotch.		
	criterion $r_2 \leq 0.3\ t_m$ as r_2 = 0.88 in., t_m = 1.48 in. $\rightarrow$ 0.88 $\geq$ 0.3 x 1.480 = 0.4044 criterion satisfied.		
Corner radii on internal surfaces	Reference paragraph NB-3544.1(b), Corner Radii on Internal Surfaces.		
	criterion $r_4 < r_2$; r_4 = 0.62 in., r_2 = 0.88 in. $\rightarrow$ 0.62 < 0.88 criterion satisfied.		
Out of roundness	Reference paragraph NB-3544.5, Out of Roundness, Figure NB-3545.1-2.		
	criterion $\frac{b}{t_b}+\frac{3}{4}\left(\frac{3b^2-2ab-a^2}{t_b^2}\right)+1 \leq 1.5\,\frac{S_m}{P_s}$ where:		
	a = 6.87 in., b = 11.67 in., t_b = 3.21 in., S_m = 19,400 psi at 500°F for ASME SA 216 WCB $\rightarrow$ 19.27 $\leq$ 21.56 criterion satisfied.		
Longitudinal curvature	Reference paragraph NB-3544.6, Longitudinal Curvature.		
	criterion $\frac{1}{r_{long}}+\frac{1}{r_{lat}} \geq \frac{4}{3d_m}$ where: r_{long} = 37.63 in., r_{lat} = 11.67 in., d_m = 21.83 in. $\rightarrow$ 0.11 $\geq$ 0.06 criterion satisfied		

TABLE 3.9-14 (SHEET 2 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
Flat wall limitation	Reference paragraph NB-3544.7, Flat Wall Limitation. criterion $\frac{d}{t} \le \frac{3}{2}\frac{d_m}{t_m}$ where: d_m = 21.83 in., t_m = 1.48 in., d = 32.75 in., t = 3.70 in. $\rightarrow 8.85 \le 22.13$ criterion satisfied.		
Minimum wall at weld end	Reference paragraph NB-3544.8, Minimum Wall at Weld End. Actual thickness at 1 x 1 in. (i.e., 1.48 in.) measured alone. The run direction is 2.05 in.	1.48	2.05
Primary crotch stress due to internal pressure	Reference paragraph NB-3545.1. criterion $P_m = \left(\frac{A_p}{A_m} + 0.5\right)P_s < S_m$ where: A_p = 404.53 in.^{2}, A_m = 77.59 in.^{2}, P_s = 1350 psig, P_m = 7713 psi, S_m = 19,400 psi $\rightarrow S_m > P_m$ criterion satisfied.	19,400	7713
Valve body secondary stress	Reference paragraph NB-3545.2.		
Primary plus secondary stress due to internal pressure	Reference paragraphs NB-3545.2(a)(1), NB-3545.2(a)(2). $Q_p = C_p\left(\frac{r_i}{t_e} + 0.5\right)P_s$ where: C_p = 3, r_i = 9.1 in., P_s = 1350 psi, t_e = 2.58, Q_p = 16,310 psi for wye-type valve $Q'_p = C_a Q_p$ where: C_a = 1.33 $\rightarrow$ $Q'_p =$ 21,692 psi		
Secondary stress due to pipe reaction(1)	Reference paragraphs NB-3545.2(b) and NB-3524; Figures NB-3545.2-3, NB-3545.2-5, and NB-3545.2-6.		
Direct or axial load effect	$P_{ed} = \frac{F_d S}{G_d}$ where: S = 41,000, F_d = 27 in.^{2}, G_d = 144 in.^{2} $\rightarrow$ P_{ed} = 7688 psi	29,100	7688 1162(3)

TABLE 3.9-14 (SHEET 3 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
Bending load effect	$P_{eb} = C_b \frac{F_b S}{G_b}$ where: S = 41,000, F_b = 295 in.^{3}, i.d. = 21.83 in., r_i = 9.10, t_e = 2.58, $\bar{r} = 10.39$ in. $\frac{t_e}{\bar{r}} = 0.25 > 0.19 \rightarrow C_b = 1$ $G_b = \frac{I}{r_i + t_e}$ where: I = 7889 in.^{4}, r_i = 9.10 in., t_e = 2.58 in. $\rightarrow G_b$ = 675 in.^{3} $\rightarrow P_{eb} = 1 \times \frac{295 \times (41{,}000)}{675} = 17{,}919$ psi	29,100	17,919 1437$^{(3)}$
Torsion load effect	Reference paragraphs NB-3545.2(b)(1), NB-3545.2(b)(6)(c). $P_{et} = 2\frac{F_b S}{G_t}$ where: F_b = 295 in.^{3}, S = 41,000 psi $G_t = C_t \bar{A}\,\bar{t}$ where: C_t = 1.75, $\bar{A} = 345$ in.^{2}, $\bar{t} = 2.26$ in. $\rightarrow G_t$ = 1364 in.^{3} $\rightarrow P_{et}$ = 17,734 psi	29,100	17,734 160$^{(3)}$
Thermal secondary stress at crotch region	Reference paragraph NB-3545.2(c); Figures NB-3545.2(c)-2, NB-3545.2(c)(2), NB-3545.2(c)-3, NB-3545.2(c)-3, and NB-3545.2(c)-4. $Q_T = Q_{T_1} + Q_{T_2}$ where: T_{e_1} = 4.20 in., $Q_{T_1} = 2100$ $Q_{T_2} = C_6 C_2 \Delta T_2$ where: C_2 = 0.53, C_6 = 220, and ΔT_2 = 5°F $\rightarrow Q_{T_2}$ = 583 psi		

TABLE 3.9-14 (SHEET 4 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
	criterion $S_N = Q'_p + P_{ed} + 2Q_{T_2} \leq 3\ S_m$ where: $Q'_p = 21{,}692$, $P_{ed} = 7688$, $Q_{T_2} = 583$ $\rightarrow 30{,}546 \leq 58{,}200$ criterion satisfied.	58,200	30,546
Normal duty valve fatigue requirements	Reference paragraphs NB-3545.3, NB-3545.3(a), and NB-3545.3a; Figure 1-9-1. criterion $N_a \geq 2000$ cycles $S_{p_1} = \frac{2}{3}Q'_p + \frac{P_{eb}}{2} + Q_{T_3} + 1.3\ Q_{T_1}$, $S_{P_2} = 0.4Q'_p + \frac{K}{2}\left(P_{eb} + 2Q_{T_3}\right)$ where: $Q'_P = 21{,}692$, $P_{eb} = 17{,}919$, $K = 2$, $Q_{T_1} = 2100$, $Q_{T_3} = 682$ psi $\rightarrow S_{p_1} = 26{,}822$, $S_{p_2} = 27{,}938$, S_a = to larger of S_{p_1} and S_{p_2} $\rightarrow S_a = 27{,}938$ $\rightarrow N_a = 25{,}000 \geq 2000$ criterion satisfied.		
Cyclic loading requirements at valve crotch	Reference paragraph NB-3550. For the largest temperature change range criterion $Q'_p + P_{ed} + C_6C_2C_4\Delta T_{f\,max} \leq 3\,S_m$ where: $Q'_P = 21{,}692$ psi, $P_{ed} = 7688$, $C_6 = 220$ at $\Delta T_{f\ max}$ of 342°F, $C_2 = 0.52$, $C_4 = 0.23$, $S_m = 19{,}400$ $\rightarrow 38{,}379 \leq 58{,}200$ criterion satisfied.	58,200	38,379
	Thermal transients not excluded by Code: criterion $\sum \frac{N_{ri}}{N_i} < 1$ Calculate the fatigue usage factor (I_t) as follows: $C_3 = 0.61$ $S_{n\,max} = Q'_p + P_{eb} + C_6C_3C_4\Delta T_{f\,max} \rightarrow S_{n\,max} = 50{,}167$ psi		

TABLE 3.9-14 (SHEET 5 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
	Since $S_{n\,max} < 3\,S_m$ (= 58,200), the following equation is used: $S_i = \frac{4}{3}Q'_p + P_{eb} + C_6(C_3C_4 + C_5)\Delta T_{fi}$		
	for ΔT_{fi} = 342°F, N_{ri} = 8, S_i = 140,162 psi, N_i = 1500, N_{ri} / N_i = 0.005		
	ΔT_{fi} = 122°F, N_{ri} = 10, S_i = 80,131 psi, N_i = 8000, N_{ri} / N_i = 0.001		
	ΔT_{fi} = 90°F, N_{ri} = 120, S_i = 71,400 psi, N_i = 15,000, N_{ri} / N_i 0.008		
	where: $I_t = \sum \frac{N_{ri}}{N_i} = 0.014 < 1$ criterion satisfied.		
Disk design calculation Refer to Note 5.	Reference paragraph NB-3546.3, Table I-1.1, Roark, 4th Edition, pp 220 and 222.		
	Disk design conditions, P_S = 1350 psi at 500°F, S_m = 20,800 psi at 500°F		
	Case No. 13: $S = \frac{3W}{4mt^2(a^2 - b^2)}\left[a^4(3m+1) + b^4(m-1) - 4m\,a^2b^2 - 4\,(m+1)a^2b^2\,(\ln(a/b))\right]$		
	where: W = 1350 psi, m = 10/3, t = 4.63 in., a = 9.25 in., b = 2.28 in., → S_t = 11,261 psi		
	Case No. 14: $S = \frac{3W}{2\pi mt^2}\left[\frac{2a^2\,(m+1)}{a^2 - b^2}\ln\left(\frac{a}{b}\right) + (m-1)\right]$		
	where: W = 60,134 lb_f, t = 4.63 in., m = 10/3 , a = 9.25 in., b = 2.28 in., → S_t = 6130 psi		
	$S_t = S_{t\,Case\,No.\,13} + S_{t\,Case\,No.\,14} = 17{,}391 \le 20{,}800$	20,800	17,391
	Case No. 21: $S_r = \frac{3W}{4t^2}\left[\frac{4a^4\,(m+1)\ln\left(\frac{a}{b}\right) - a^4\,(m+3) + b^4\,(m-1) + 4a^2\,b^2}{a^2\,(m+1) + b^2\,(m\text{-}1)}\right]$		
	where: W = 1350, m = 10/3, t = 1.80 in., a = 9.25, b = 7.75 in. → S_r = 3102 psi		

TABLE 3.9-14 (SHEET 6 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
	Case No. 22: $S_r = \frac{3W}{2\pi t^2}\left[\frac{2a^2(m+1)\ln\left(\frac{a}{b}\right)+a^2(m-1)-b^2(m-1)}{a^2(m+1)+b^2(m-1)}\right]$		
	where: W = 288,993, m = 10/3, t = 1.80, a = 9.25, b = 7.75 $\rightarrow S_r$ = 15,885 psi		
	Total Stress $= S_{r_{21}} + S_{r_{22}}$ = 18,987 psi, allowable stress 20,800 psi	20,800	18,987
	S_{shear} at inner edge disk $S_{shear} = \frac{F}{A}$ where: F = 60,134 lb, A = 64.9 in.^{2} $\rightarrow S_{shear}$ = 927 psi	12,480	927
	S_{shear} at seat bore $S_{shear} = \frac{F}{A}$ where: F = 397,142 lb, A = 79 in.^{2} $\rightarrow S_{shear}$ = 5027 psi	12,480	5027
	Allowable shear stress = 0.6 x allowable stress = 0.6 x 20,800 = 12,480 psi		
	Hub Tensile Stress $S = \frac{F}{A}$ where: F = 281,566 lb, A = 44.6 in.^{2} $\rightarrow$ S = 6313 psi		
	Allowable stress = 20,800 psi	20,800	6313
Stem disk calculation Refer to Note 5.	Reference Roark, 4th Edition, p 216, Table I-1.1.		
	Design condition, P_S = 1350 psi at 500°F, allowable stress = 20,800 psi		
Tensile and shear	Case No. 1: $S_r = S_t = \frac{3W_p}{8\pi m t^2}(3m+1)$		
Refer to Note 5.	where: W_p = P x A = 26,085 lb, m = 10/3, t = 1.39 in. $\rightarrow S_t$ = 5319 psi		

TABLE 3.9-14 (SHEET 7 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
	Case No. 3: $S_r = S_t = \frac{3W}{2\pi m t^2}\left[\frac{1}{2}(m-1)+(m+1)\ln\left(\frac{a}{r_o}\right)-(m-1)\frac{r_o^2}{2a^2}\right]$		
	where: W = 38,500, t = 1.39, a = 2.48, r_o = 1.31 in., m = 10/3 $\rightarrow S_t$ = 10,285 psi		
	$S = S_{t\ Case\ 1} + S_{t\ Case\ 3}$ = 15,604 psi	20,800	15,604
	Shear stress above seat		
	$S_{shear} = \frac{F_s}{A}$ where: F_S = 64,585 lb_f, A = 16.51 in.^{2} $\rightarrow S_{shear}$ = 3910 psi		
	Allowable stress = 0.6 S_m = 12,480	12,480	3910
Thread strength Refer to Note 5.	1 7/8 - 12 UN - 2 Thread Reference Federal Thread Standard Part No. 1, page 5, (1957 Edition).		
	Thread Shear Area		
	$AS_n = \pi n L_e D_{s\ min}\left[\frac{1}{2n}+0.57735\left(D_{s\ min}-E_{n\ max}\right)\right]$		
	where: n = 12 threads/in., $D_{S\ min}$ = 1.8618 in., $E_{n\ max}$ = 1.8287 in., L_e = 1.00 in.		
	$\rightarrow AS_n$ = 4.26 in.^{2}		
	$\rightarrow$ Shear stress = $\frac{F}{AS_n}$ = 6593 psi, where: F = 28,085 lb, AS_n = 4.26 in.^{2}		
	Allowable stress = 0.6 S_m = 12,480	12,480	6593
Piston design calculation Refer to Note 5.	Design condition, P_S = 1350 psi at 500°F, S_m = 19,400 psi, ultimate tensile stress = 70,000		
Thread strength Refer to Note 5.	Thread Shear Area		
	$AS_S = \pi n L_e K_{n\ max}\left[\frac{1}{2n}+0.57735\left(E_{s\ min}-K_{n\ max}\right)\right]$		
	where: $K_{n\ max}$ = 8.5147, L_e = 1.95 in. $\rightarrow AS_s$ = 35.24 in.^{2}, $E_{s\ min}$ = 8.5527, n = 8		

TABLE 3.9-14 (SHEET 8 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
	Actual shear stress = $\frac{F}{AS_s}$ where: F = 271,574 lb, AS_s = 35.24 in.^{2} $\rightarrow S_a$ = 7707 psi		
	Allowable stress = 0.6 S_n = 11,640	11,640	7707
Hoop stress Refer to Note 5.	Reference Roark, 4th Edition, p 308, Case No. 34. $S = P\left(\frac{2b^2}{b^2 - a^2}\right)$ where: b = 8.68 in., P = 19,400 psi, a = 7.55 in. S = 11,092 psi		
	where: 11,092 ≤ 19,400 condition acceptable	19,400	11,092
Tensile stress at thread relief Refer to Note 5.	$S_m = \frac{F}{\Delta A_t}$ where: F = 283,132 lb, $\Delta A_t = A_1 - A_2$ = 21.39 in.^{2}, S_m = 19,400 lb $S_m = \frac{283{,}132}{21.39} \rightarrow S_m$ = 13,237 psi		
	$\rightarrow$ 13,237 < 19,400 condition acceptable	19,400	13,237
Bonnet design calculation[(2)] Refer to Note 5.	Reference paragraph NB-3647.1(a), paragraph UG-34(K)(2) of ASME Section VIII, Division 1, 1971 Edition.		
Minimum thickness[(2)] Refer to Note 5.	$P_{fd} = P + P_{eg};\ P_{eg} = \frac{16M}{\pi G^3} + \frac{4F}{\pi G^2}$ $\left.\begin{matrix} M = 455{,}937 \\ M = 189{,}729^{(4)} \end{matrix}\right\}$ in.-lb, F = 38,500 lb, G = 19.25 in. $\rightarrow P_{eg}$ = 458 psi, P_{fd} = 1808 psi $t = d\sqrt{\frac{CP}{S} + \frac{1.78\ W\ hg}{S_d^3}}$ M = (combined seismic coefficient) (moment arm) (operator weight) where: C = 0.3, P = 1808 psi, S = 19,400 psi, hg = 2.375 in., W = 748,666 lb, d = 19.25 in. $\rightarrow$ t = 4.34 in.		

TABLE 3.9-14 (SHEET 9 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
Reinforcement[(2)] Refer to Note 5.	Reference paragraph UG-39(a)(2) of ASME Section VIII, Division 1, 1971 Edition (to account for opening for stem in the bonnet).		
	$t = \sqrt{2}\left(d \sqrt{\frac{CP}{S} + \frac{1.78\,W\,hg}{S_d^3}} \right)$		
	$\rightarrow$ t = 6.21 in., t = 6.21 + 0.12 = 6.33 in. (corrosion allowance = 0.120 in.)	6.33	6.62
Bonnet studs design calculation[(2)]	Reference paragraph 3232.1 and Article E-1000.		
	Bolt used 20 pieces of 1 3/8-UNC bolts.		
	Total bolt area = 24.6 in.^{2}		
Normal operation[(2)]	Pressure stress at operating condition		
	$S_1 = \frac{W_{m_1}}{A_b}$ = 29,642 lb/in.^{2} where: $W_{m_1} = 729{,}201$ lb , A_b = 24.6 in.^{2}		
	Gasket load at ambient condition with no internal pressure		
	$S_2 = \frac{W_{m_2}}{A_b}$ = 3430 lb/in.^{2} where: $W_{m_2} = 84{,}320\ lb_f$, A_b = 24.6 in.^{2}		
	Maximum tensile stress = 29,642 lb/in.^{2}		
	Thermal stress is assumed negligible, because the coefficient of thermal expansion of bonnet plate and stud is the same.		
	Standard preload = 45,000 psi.		
	Higher stress condition = the standard preload.		
	Allowable stress is 69,400 psi. Condition acceptable.	69,400	45,000
Body flange design calculation[(2)]	Reference paragraph NB-3647.1 and ASME Section VIII, Division 1 of ASME B&PV Code, 1971 Edition		

TABLE 3.9-14 (SHEET 10 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
	Total flange moment under operating conditions		

$M_O = M_D + M_G + M_T$

$M_D = H_D\ h_D,\ H_D = 0.785\ B^2P,\ h_D = R + 0.591$

where: $B = 18.5$ in., $P = 1808$ psi $\rightarrow H_D = 485{,}749\ lb_f$, $h_D = 1.69$ in., $M_D = 820{,}915$ in. - lb

$$M_G = H_G\ h_G,\ H_G = W - H,\ h_G = \frac{C - G}{2}$$

where: W is the higher of W_{m_1} and W_{m_2}

$$W_{m_1} = 0.785\ G^2P + (2b \times 3.14\ G\,m\,P)$$

$$W_{m_2} = 3.14\ Gby$$

where: $G = 19.25$ in., $b = 0.31$ in., $m = 3$, $y = 4500 \rightarrow W_{m_1} = 729{,}201$ lb, $W_{m_2} = 84{,}320$ lb

$H_G = 203{,}269$ lb, $h_G = 2.375$ in. $\rightarrow M_G = 480{,}389$ in-lb

$$M_T = H_T\ h_T,\ H_T = H - H_D,\ h_T = \frac{R + g_1 + h_G}{2}$$

where: $H = 525{,}932$, $H_D = 485{,}749$, $R = 0.62$ in., $g_1 = 2.13$ in., $h_G = 2.38$ in.

$\rightarrow H_T = 40{,}183$ lb, $h_T = 2.57$ in., $M_T = 103{,}270$ in.-lb_f

$M_O = 1{,}404{,}574$ in.-lb where: $M_D = 820{,}915$ in.-lb, $M_G = 480{,}389$ in.-lb,

$M_T = 103{,}270$ in.-lb

Total flange moment under gasket seating condition

$$M_o = W\frac{(C - G)}{2},\ W = \frac{(A_m + A_b)}{2}s_a$$

where: $C = 24$ in., $A_b = 24.8$ in.^{2}, $G = 19.25$ in., $A_m = 21.01$ in.^{2}, $s_a = 40{,}000$ psi at 100°F

$\rightarrow W = 912{,}200$ lb $\rightarrow M_O = 2{,}166{,}475$ lb/in.

TABLE 3.9-14 (SHEET 11 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
Longitudinal hub stress[2]	Reference Paragraph NB-3647.1(c). $S_H = \frac{f M_0}{L g_1^2 B} + \frac{PB}{4g_0}$		
	at operating condition S_H = 22,015 psi	29,100	22,015 (14,481) [4]
	at atmospheric condition S_H = 29,977 psi	34,950	29,977 (21,572) [4]
Radial stress[2]	Reference UA-51(1), Equation (7) of Section VIII of ASME B&PV Code, 1971 Edition. $S_R = \frac{(1.33t_e + 1)M_0}{Lt^2B}$		
	at operating condition S_R = 6605 psi	29,100	6605 (5708) [4]
	at atmospheric condition S_R = 10,188 psi	34,950	10,188 (9,318) [4]
Tangential stress[2]	Reference UA-51(1), Equation (8) of Section VIII of ASME B&PV Code, 1971 Edition $S_T = \frac{(YM_0)}{t^2B} - ZS_R$ where: Y = 5.0, t = 4.25 in., Z = 2.60, B = 18.50 in.		
	at operating condition S_T = 3844 psi	29,100	3844 (4067) [4]
	at atmospheric condition S_T = 5928 psi	34,950	5928 (6640) [4]
Flange stress criteria[2]	Reference paragraph UA-52 of Section VIII of ASME B&PV Code, 1971 Edition.		

TABLE 3.9-14 (SHEET 12 OF 14)

Criteria	Method of Analysis		Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
	criteria $\frac{S_H + S_R}{2} < S_m$; $\frac{S_H + S_T}{2} < S_m$			
	at operating condition	$\frac{S_H + S_R}{2} = 14{,}310$ psi	19,400	14,310 (10,096) (4)
		$\frac{S_H + S_T}{2} = 12{,}930$ psi	19,400	12,930 (9,276) (4)
	at atmospheric condition	$\frac{S_H + S_R}{2} = 20{,}083$ psi	23,300	20,080 (15,445) (4)
		$\frac{S_H + S_T}{2} = 17{,}953$ psi	23,300	17,953 (14,106) (4)
Stem calculation Refer to Note 5.				
Back-seated stress	$S = \frac{F}{A}$			
Refer to Note 5.	where:	F = 9616 lb net upward force		
		A = 2.268 in.^{2}, smallest cross-sectional area on the stem		
		S = 4240 psi < 26,700 psi	26,700	4240
Valve close stem stress	$S = \frac{F}{A}$			
Refer to Note 5.	where:	F = 38,500 lb net down force		
		A = 2.268 in.^{2}, smallest cross-sectional area on the stem		
		S = 16,975 psi < 26,700 psi	26,700	16,975
Disk entering seat Refer to Note 5.	$S = \frac{F}{A}$			

TABLE 3.9-14 (SHEET 13 OF 14)

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness (in.)	Calculated Stress or Actual Thickness (in.)
	where: F = 26,085 disk load		
	A = 2.268 in.^{2}, smallest cross-sectional area on the stem		
	S = 11,501 psi < 26,700 psi	26,700	11,501
Stem thread strength Refer to Note 5.	Reference Federal Thread Standard.		
	Stem Thread Mating with Disk		
	Thread 1.875 in. - 12 UN - 2 Thread		
	$A_{S_1} = \pi n L_e K_{n\,max}\left[\frac{1}{2n} + 0.57735\left(E_{s\,min} - K_{n\,max}\right)\right]$		
	where: n = 12, E_{smin} = 1.8131, L_e = 1.25 in., K_{nmax} = 1.8030 in. A_{S1} = 4.04 in.^{2}		
	$\tau = \frac{F}{A_{S_1}}$ where: F = 38,500 lb_f, A_{S1} = 4.04 in.^{2} $\rightarrow$ τ_{sd} = 9540 psi	16,020	9540
	Stem Thread Mating with Air Pneumatic Cylinder		
	Thread 2 in. - 12 UN - 2		
	$A_{S2} = \pi n L_e\ K_{nmax}\left[\frac{1}{2n} + 0.57735\left(E_{s\,min} - K_{n\,max}\right)\right]$		
	where: n = 12, E_{smin} = 1.9380 in., L_e = 1.14 in., $K_{n\,max}$ = 1.928 in. $\rightarrow A_{S2}$ = 3.93 in.^{2}		
	$\tau = \frac{F}{A_{S_2}}$ where: F = 31,400 lb_f, A_{S2} = 3.93 in.^{2} $\rightarrow \tau$ = 7990 psi	16,020	7990

TABLE 3.9-14 (SHEET 14 OF 14)

NOTES:

1. Secondary stresses due to pipe reaction are limited in the adjoining pipe by specification to 2 S_m at faulted conditions [peak pressure, thermal, deadweight, DBE, relief valve lift, and TSV closure].

$$(S)_{torsion} = \frac{M_A}{2\,Z_{pipe}}$$

$$(S)_{bending} = \frac{\sqrt{M_B^2 + M_C^2}}{Z_{pipe}}$$

$$(S)_{axial} = \frac{F_A + P_A}{A_{pipe}}$$

where: $M_{A,B,C}$ are moments due to

$$\left[\text{peak pressure} + \text{thermal} + \text{deadweight} + \sqrt{DBE^2 + R/V_{lift} \text{ or } TSV_{closure}{}^2}\right]_{A,B,C}$$

F_A and P_A are axial loads due to pipe reaction and peak pressure. The piping stress report includes those calculations to assure that the stresses are within the acceptable limit (table 3.9-14).
2. Calculations relating to the body flange, bonnet, and bolting design include the effects of the DBE plus valve actuation forces as converted per code (NB-3647) to an effective pressure. Stresses are then calculated to assure conformance to stress limits.
3. Calculated values based on maximum actual stress (S) determined from pipe system analysis.
4. Calculated values based on the maximum actual accelerations determined from pipe system analysis.
5. Only applicable to MSIVs with the oversized Rockwell/Edwards disk installed. For valves with the WVC modification kit installed, refer to Addendum 3 of vendor document S-60776 for these design calculations. Table 3.9-65 specifies which MSIVs have the oversized Rockwell/Edwards disk installed or the WVC modification kit installed.

TABLE 3.9-15 (SHEET 1 OF 3)

RECIRCULATION PUMPS

Criteria	Method of Analysis	Analytical Results	Allowable Stress or Actual Thickness
Casing minimum wall thickness	$t = \frac{PR}{SE - 0.6P} + C$	$t = 2.75$ in.	$S_{allow} = 15{,}114$ psi
Loads:	where:		
	t = minimum required thickness (in.) P = design pressure (psig)		$t_{act} = 3.00$ in.
Normal and upset condition	R = maximum internal radius (in.) S = allowable working stress (psi) E = joint efficiency		
Design pressure and temperature	C = corrosion allowance (in.)		
Primary membrane stress limit:			
Allowable working stress per ASME Section III, Class C			
Casing cover minimum thickness	$S_s = \frac{F}{A}$	$S_s = 3370$ psi	$S_{allow} = 8740$ psi
Loads:	F = force A = area at shear point		$t_{act} = 3.5$ in.
Normal and upset condition			
Design pressure and temperature	$S_b = \frac{Kqa^2}{h^2}$		
Primary bending and shear stress limit:	q = pressure load a = radius of OD	$S_b = 5950$ psi	$S_{allow} = 15{,}075$ psi
1.5 S_m per ASME Code for Pumps and Valves for Nuclear Power Class I	b = radius of ID h = plate thickness		$t_{act} = 7$ in.

TABLE 3.9-15 (SHEET 2 OF 3)

Criteria	Method of Analysis	Analytical Results	Allowable Stress or Actual Thickness
Seal cover			
Loads:			
Normal and upset condition Design pressure and temperature Design gasket load	Flange thickness shall be calculated in accordance with ASME Section VIII, Paragraph UG 34, Unstayed Flat Heads and Covers.	t = 2.59 in.	t_{act} = 2.625 in.
Seal chamber minimum wall thickness	$t = \frac{PR}{SE - 0.6P} + C$	t = 0.741 in.	S_{allow} = 15,075 psi
Loads:	where:		t_{act} = 1.375 in.
Normal and upset condition Design pressure and temperature Piping reactions during normal operation	t = minimum required thickness (in.) P = design pressure (psig) R = maximum internal radius (in.) S = allowable working stress (psi) E = joint efficiency C = corrosion allowance (in.)		
Combined Stress Limit:			
1.5 S_m per ASME Code for Pumps and Valves for Nuclear Power Class 1.			
Mounting bracket combined stress	Bracket vertical loads are determined by summing the equipment, fluid weights, and vertical seismic forces. Bracket horizontal loads are determined by applying the specified seismic force at mass center of pump-motor assembly (flooded).	Combined stress (shear plus tensile)	S_m = 15,150 psi
Loads:		Lug #1 S_C = 6506 psi	S_y = 30,000 psi
Flooded weight		Lug #2 S_C = 7976 psi	
DBE horizontal seismic force = 1.5 g		Lug #3 S_C = 10,762 psi	
DBE vertical seismic force = 0.144 g			

TABLE 3.9-15 (SHEET 3 OF 3)

Criteria	Method of Analysis	Analytical Results	Allowable Stress or Actual Thickness
Combined stress limit: Yield stress	Horizontal and vertical loads are applied simultaneously to determine tensile, shear, and bending stresses in the brackets. Tensile, shear, and bending stresses are combined to determine maximum combined stresses.		
Stresses due to the seismic loads Loads: Operation pressure and temperature DBE horizontal seismic force = 1.5 g DBE vertical seismic force = 0.144 g Combined stress limit: Yield stress	The flooded pump-motor assembly is analyzed as a free body supported by constant support hangers from the pump brackets. Horizontal and vertical seismic forces are applied at mass center of assembly, and equilibrium reactions are determined for the motor and pump brackets. Loads, shear, and moment diagrams are constructed using live loads, dead loads, and calculated snubber reactions. Combined bending, tension, and shear stresses are determined for each major component of the assembly including motor support barrel, bolting, and pump casing. The maximum combined tensile stress in the cover bolting is calculated using tensile stresses determined from loading diagram plus tensile stress from operating pressure.	Motor bolt tensile stress: S_{act} = 7500 psi Pump cover bolt tensile stress: S_{act} = 19,000 psi Motor support barrel combined stress: S_{act} = 1546 psi	 S_{allow} = 11,200 psi S_{allow} = 32,000 psi S_{allow} = 22,400 psi

TABLE 3.9-16 (SHEET 1 OF 8)

STRUCTURAL AND MECHANICAL LOADING CRITERIA

Reactor Recirculation Gate Valve (28-in. Discharge)

The discharge valve is designed to accommodate loads transmitted by the piping system so that the maximum stress in the pipe, at the point of attachment to the valve, is 25,000 psi. The discharge valve and operator are designed to accommodate static seismic loadings of 1.5-g and 2.65-g horizontal, respectively, and 0.8-g and 1.0-g vertical, respectively, acting at the operator's center of mass.

The valve and operator were analyzed as a system using the loading combination of deadweight, peak pressure, and the absolute value of the DBE loading as an algebraic sum.

These calculated seismic loadings and stress are seen to be less than the design values in all cases. The following table establishes that the valve can accept the design piping and pressure loads.

Component Loads Design	Design Procedure	Required Design Value	Actual Design Value
Body and bonnet			
Loads:			
Design pressure	System requirement	1525 psi	1525 psi
Design temperature	System requirement	575°F	575°F
Peak pressure	System requirement	1572 psi	1650 psi
Pressure rating (psi)	Used Tables 451.4 and 451.5 of NPVC	P_r = 799 psi	P_r = 800 psi
Minimum wall thickness (in.)	Used Table 452.1 of NPVC (dm = 22)	$t_m \geq$ 2.114 in.	t_m = 2.114 min
Primary membrane stress (psi)	Used Paragraph 452.3 of NPVC	$P_m \leq S_m$ (500°F) = 19,600 psi	P_m = 10,293 psi
Secondary stress due to pipe reaction	Used Paragraph 452.4b of NPVC (S = 25,000)	P_e = greatest value of P_{ed}	P_{ed} = 5580 psi
		P_{eb} and $P_{et} \leq 1.5\ S_m$ (500°F)	P_{eb} = 12,702 psi
		1.5 (19,600) = 29,400 psi	P_{et} = 12,277 psi
			P_e = 12,702 psi

TABLE 3.9-16 (SHEET 2 OF 8)

Component Loads Design	Design Procedure	Required Design Value	Actual Design Value
Primary plus secondary stress due to internal pressure	Used Paragraph 452.4a of NPVC.	$S_n \leq 3\ S_m$ (500°F) = 58,800 psi	Q_p = 24,284 psi
Thermal secondary stress	Used Paragraph 452.4c of NPVC.	Same as above	Q_T = 5591 psi
Sum of primary plus secondary stress	Used Paragraph 452.4 of NPVC.	Same as above	S_n = 32,642 psi
Fatigue requirements	Used Paragraph 452.5 of NPVC.	$N_a \geq 2000$ cycles	$N_a >> 10^5$ cycles
Cyclic rating	Used Paragraph 454 of NPVC.	$I_t \leq 1$	I_t = .004 (normal duty)
<u>Body-to-bonnet bolting</u>			
Loads:			
Design pressure and temperature, gasket loads, stem operational load, and seismic load (DBE)	USAS B31.7, Paragraph 1-704.5.1. Used ASME Section VIII, 1968, Paragraphs UA-47 to UA-51, as required by Paragraph 453.1 of NPVC.		
1.5-g horizontal (statically applied) 0.8-g vertical (statically applied)			
Bolt area	Same as above.	$A_b \geq 44.41$ in.^{2}	A_b = 55.86 in.^{2}
		$S_b \leq 28,800$ psi	S_b = 22,899 psi
Body flange stresses	USAS B31.7, Paragraph 1-704.5.1 Used ASME Section VIII, 1968, Paragraphs UA-47 to UA-51, as required by Paragraph 453.1 of NPVC.		

TABLE 3.9-16 (SHEET 3 OF 8)

Component Loads Design	Design Procedure	Required Design Value	Actual Design Value
Operating condition	USAS B31.7, Paragraph 1-704.5.1 Used ASME Section VIII, 1968, Paragraphs UA-47 to UA-51, as required by Paragraph 453.1 of NPVC.	$S_H \leq 1.5\ S_m$ (500°F) = 29,400 psi $S_R \leq 1.5\ S_m$ (500°F) = 29,400 psi $S_T \leq 1.5\ S_m$ (500°F) = 29,400 psi	S_H = 26,021 psi S_R = 8113 psi S_T = 9019 psi
Gasket seating condition	USAS B31.7, Paragraph 1-704.5.1 Used ASME Section VIII, 1968, Paragraphs UA-47 to UA-51, as required by Paragraph 453.1 of NPVC.	$S_H \leq 1.5\ S_m$ (100°F) = 30,000 psi $S_R \leq 1.5\ S_m$ (100°F) = 30,000 psi $S_T \leq 1.5\ S_m$ (100°F) = 30,000 psi	S_H = 29,981 psi S_R = 11,671 psi S_T = 12,972 psi
Bonnet flange			
Operating condition	Calculate bonnet flange thickness according to rules of ASME Section VIII, Art. UA-6, Figure UA-6c.	$S_{max} \leq S_m$ (500°F) = 19,600 psi	S = 6232 psi
<u>Stresses in Stem</u>			
Loads:			
Operator thrust and torque			
Stem thrust stress	Calculate stress due to operator thrust in critical cross-section.	$S_T \leq 0.8\ S_m$ = 35,280 psi	S_T = 28,512 psi
Stem torque stress	Calculate shear stress due to operator torque in critical cross-section.	$S_S \leq 0.6\ S_m$ = 26,460 psi	S_S = 17,369 psi

TABLE 3.9-16 (SHEET 4 OF 8)

Component Loads Design	Design Procedure	Required Design Value	Actual Design Value
Disc Analysis			
Loads:			
Maximum differential pressure			
Maximum stress in disc	Calculate maximum stress according to Table 10 of Roark's "Formula for Stress and Strain."	$S_{max} \leq 1.5\ S_m$ (500°F) = 28,500 psi	Maximum stress = 22,885 psi
Yoke and yoke connections			
Loads:			
Stem operational load	Calculate stresses in the yoke and yoke connections to acceptable structural analysis methods.		
Maximum stress in yoke		$S_{max} \leq S_m$ = 19,400 psi	Maximum stress = 14,662 psi

TABLE 3.9-16 (SHEET 5 OF 8)

Reactor Recirculation Gate Valve (28-in. Suction)

The suction valve is designed to accommodate loads transmitted by the piping system so that the maximum stress in the pipe at the point of attachment to the valve is 25,000 psi. The suction valve and operator are designed to accommodate static seismic loadings of 1.5-g horizontal and 0.8-g vertical, acting at the operator's center of mass.

The valve and operator were analyzed as a system, using the loading combination of deadweight, peak pressure, and the absolute value of the DBE loading as an algebraic sum.

These calculated seismic loadings and stress are seen to be less than the design values in all cases. The following table establishes that the valve can accept the design piping and pressure loads.

Component Loads Design	Design Procedure	Required Design Value	Actual Design Value
Body and bonnet			
Loads:			
Design pressure	System requirement	1275 psi	1275 psi
Design temperature	System requirement	575°F	575°F
Peak pressure	System requirement	1362 psi	1400 psi
Pressure rating (psi)	Used Tables 451.4 and 451.5 of NPVC.	$P_r = 668$ psi	$P_r = 668$ psi
Maximum wall thickness (in.)	Used Table 452.1 of NPVC. (dm = 22)	$t_m \geq 1.77$ in.	$t_m = 1.77$ in.
Primary membrane stress (psi)	Used Paragraph 452.3 of NPVC.	$P_m \leq S_m$ (500°F) = 19,600 psi	$P_m = 8606$ psi
Secondary stress due to pipe reaction	Used Paragraph 452.4b of NPVC. (S = 25,000 psi)	P_e = greatest value of P_{ed}	$P_{ed} = 5318$ psi
		P_{eb} and $P_{et} \leq 1.5\ S_m$ (500°F)	$P_{eb} = 11{,}980$ psi
		1.5 (19,600) = 29,400 psi	$P_{et} = 11{,}575$ psi
			$P_e = P_{eb} = 11{,}980$ psi

TABLE 3.9-16 (SHEET 6 OF 8)

Component Loads Design	Design Procedure	Required Design Value	Actual Design Value
Primary plus secondary stress due to internal pressure	Used Paragraph 452.4a of NPVC.	$S_n \leq 3S_m$ (500°F) = 58,800 psi	Q_p = 20,580 psi
Thermal secondary stress	Used Paragraph 452.4c of NPVC.	Same as above	Q_T = 5704 psi
Sum of primary plus secondary stress	Used Paragraph 452.4 of NPVC.	Same as above	S_n = 28,478 psi
Fatigue requirements	Used Paragraph 452.5 of NPVC.	$N_a \geq$ 2000 cycles	$N_a \geq 7 \times 10^5$ cycles
Cyclic rating	Used Paragraph 454 of NPVC.	$I_t \leq 1$	I_t = 0.003 (normal duty)
<u>Body-to-bonnet bolting</u>			
Loads:			
Design pressure and temperature gasket loads, stem operational load, seismic load (DBE):	USAS B31.7, Paragraph 1-704.5.1. Used ASME Section VIII, 1968, Paragraphs UA-47 to UA-51, as required by Paragraph 453.1 of NPVC.		
1.5-g horizontal (statically applied) 0.8-g vertical (statically applied)			
Bolt area	Same as above.	$A_b \geq 37.53$ in.^{2}	A_b = 55.86 in.^{2}
		$S_b \leq$ 28,800 psi	S_b = 19,350 psi
Body flange stresses	USAS B31.7, Paragraph 1-704.5.1. Used ASME Section VIII, 1968, Paragraphs UA-47 to UA-51, as required by Paragraph 453.1 of NPVC.		

TABLE 3.9-16 (SHEET 7 OF 8)

Component Loads Design	Design Procedure	Required Design Value	Actual Design Value
Operating condition	USAS B31.7, Paragraph 1-704.5.1. Used ASME Section VIII, 1968, Paragraphs UA-47 to UA-51, as required by Paragraph 453.1 of NPVC.	$S_H \le 1.5\ S_m$ (500°F) = 29,400 psi $S_R \le 1.5\ S_m$ (500°F) = 29,400 psi $S_T \le 1.5\ S_m$ (500°F) = 29,400 psi	S_H = 23,456 psi S_R = 6531 psi S_T = 8703 psi
Gasket seating condition	USAS B31.7, Paragraph 1-704.5.1. Used ASME Section VIII, 1968, Paragraphs UA-47 to UA-51, as required by Paragraph 453.1 of NPVC.	$S_H \le 1.5\ S_m$ (500°F) = 30,000 psi $S_R \le 1.5\ S_m$ (500°F) = 30,000 psi $S_T \le 1.5\ S_m$ (500°F) = 30,000 psi	S_H = 28,945 S_R = 10,253 S_T = 13,619 psi
Bonnet flange	USAS B31.7, Paragraph 1-704.5.1. Used ASME Section VIII, 1968, Paragraphs UA-47 to UA-51, as required by Paragraph 453.1 of NPVC.		
Operating condition	Calculate bonnet flange thickness according to rules of ASME Section VIII, Art. UA-6, figure UA-6(c).	$S_{max} \le S_m$ (500°F) = 19,600 psi	S = 5294 psi
<u>Stresses in stem</u>			
Loads:			
Operator thrust and torque			
Stem thrust stress	Calculate stress due to operator thrust in critical cross-section.	$S_T \le 0.8\ S_m$ = 35,280 psi	S_T = 24,792 psi
Stem torque stress	Calculate shear stress due to operator torque in critical cross-section.	$S_S \le 0.6\ S_m$ = 26,460 psi	S_S = 15,241 psi

TABLE 3.9-16 (SHEET 8 OF 8)

Component Loads Design	Design Procedure	Required Design Value	Actual Design Value
Disc analysis			
Loads:			
Maximum differential pressure			
Maximum stress in disc	Calculate maximum stress according to Table 10 of Roark's "Formula for Stress and Strain."	$S_{max} \leq 1.5\ S_m\ (500°F) = 28,500$ psi	Maximum stress = 19,418 psi
Yoke and yoke connections			
Loads:			
Stem operational load			
Maximum stress in yoke	Calculate stresses in the yoke and yoke connections to acceptable structural analysis method.	$S_{max} \leq S_m = 19,400$ psi	Maximum stress = 14,662 psi

LEGEND

NPVC = draft ASME Code for Pumps and Valves for Nuclear Power, 1978, including March 1970 Addendum.

TABLE 3.9-17

HYDRAULIC CONTROL UNIT PIPING

Criteria	Loading	Location	Allowable Stress (psi)	Calculated Stress (psi)
For normal condition:				
From ANSI B31.1, Code for Power Piping	Normal condition load Maximum normal hydraulic system pump pressure	3/4-in. drive withdraw piping	S_h = 15,000	14,596
For normal condition:				
S_h = 15,000 psi				
For upset and emergency condition: When upset or emergency condition exists for < 1% of the time, the code allows 20% increase in stress.	Upset condition[a] load Shutoff pump pressure OBE (negligible load)	3/4-in. drive withdraw piping	1.2 S_h = 18,000	17,056
S_a = 1.2 S_h = 18,000 psi	Emergency condition[a] Shutoff pump pressure DBE (negligible load) OBE: 1-g horizontal[b] (statically applied) DBE: 2-g horizontal[b] (statically applied)	3/4-in. drive withdraw piping	1.2 S_h = 18,000	17,162

a. Vertical earthquake is not defined since only relatively short vertical piping runs are involved.
b. These loads were directly combined.

TABLE 3.9-18 (SHEET 1 OF 3)

SLC PUMP

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
1. Closure bolting Design condition[a] analyzed is operating condition and gasket seating condition resulting from: Design pressure Design temperature Design gasket load	Bolting loads and stresses calculated per "Rules for Bolted Flange Connections," ASME Section VIII, Appendix II.	Stuffing box bolts = 25,000 Cylinder head bolts = 25,000	18,150 19,600
Bolting stress limit Allowable working stress per ASME Section VIII.	Pressure area method maximum stress point on fluid cylinder.	16,500	9000
2. Wall thickness Stress[a] is calculated at areas of thinnest wall section under operating conditions resulting from load combinations of design pressure and temperature. Stress limit ASME Section VIII			
3. Motor mount bolts Design condition[a] to show bolting stressed by seismic loads are within the allowable limits.	Seismic forces acting on motor subject bolts to tension and shear.	Tension = 16,500 Shear = 10,000	860 1220

TABLE 3.9-18 (SHEET 2 OF 3)

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
Loads:			
Design basis earthquake 1.5-g horizontal, and 0.14-g vertical, plus deadweight.	For the maximum moment due to pipe reaction, the maximum force shall not exceed the allowable.		
Stress limit			
0.9-yield tension and twice allowable shear per ASME Section VIII.			
		Allowable Nozzle Forces and Moments, Force (lb), Moment (ft-lb)	Calculated Nozzle Forces and Moments
4. Nozzle Loads			
The design condition[a] analyzed is an uninterrupted operation during operating conditions resulting from design pressure, temperature, deadweight, thermal expansion, and OBE (horizontal = 0.75 g; vertical = 0.07 g).	Total nozzle stress with this criteria does not exceed stress limits. Mount bolts do not exceed stress limits.	Suction M = 4.59 (711-F) not to exceed 1385 ft-lb	Pump A F_x = 58 lb M_x = 24 ft-lb F_y = 64 lb M_y = 173 ft-lb F_z = 131 lb M_z = 43 ft-lb Pump B F_x = 55 lb M_x = 16 ft-lb F_y = 24 lb M_y = 283 ft-lb F_z = 164 lb M_z = 13 ft-lb
		Discharge M = 2.3 (342-F) not to exceed 283 ft-lb	Pump A F_x = 196 lb M_x = 23 ft-lb F_y = 216 lb M_y = 50 ft-lb F_z = 66 lb M_z = 153 ft-lb Pump B F_x = 228 lb M_x = 12 ft-lb F_y = 163 lb M_y = 102 ft-lb F_z = 129 lb M_z = 77 ft-lb

TABLE 3.9-18 (SHEET 3 OF 3)

Criteria	Method of Analysis	Allowable Nozzle Forces and Moments, Force (lb), Moment (ft-lb)	Calculated Nozzle Forces and Moments
The design condition[a] analyzed is no functional failure resulting from design pressure, temperature, deadweight, thermal expansion, and DBE (horizontal = 1.5 g; vertical = 0.14 g).		Suction M = 4.59 (1422-F) not to exceed 2060 ft-lb	Pump A F_X = 59 lb M_X = 27 ft-lb F_Y = 68 lb M_Y = 185 ft-lb F_Z = 143 lb M_Z = 45 ft-lb Pump B F_X = 83 lb M_X = 20 ft-lb F_Y = 33 lb M_Y = 292 ft-lb F_Z = 176 lb M_Z = 20 ft-lb
		Discharge M = 2.3 (684-F) not to exceed 444 ft-lb	Pump A F_X = 198 lb M_X = 23 ft-lb F_Y = 216 lb M_Y = 50 ft-lb F_Z = 66 lb M_Z = 153 ft-lb Pump B F_X = 228 lb M_X = 12 ft-lb F_Y = 163 lb M_Y = 102 ft-lb F_Z = 129 lb M_Z = 77 ft-lb
Stress limit ASME Section VIII, for normal and upset condition; 1.5 of allowable stress for emergency. Mount bolts 0.9-yield for tension and twice allowable shear for emergency.			

a. These loads are combined directly.

TABLE 3.9-19 (SHEET 1 OF 2)

SLC TANK

Criteria	Method of Analysis	Allowable Stress or Minimum Thickness Required (psi)	Calculated (psi)
Shell thickness	Minimum thickness	3/16 in.	0.015 in.
Loads: normal and upset[a] Design pressure and temperature	$t = \frac{2.6D(H-1)G\text{ in.}}{SE}$		
Stress limit:	D = nominal ID H = tank height G = specific gravity S = allowable stress E = joint efficiency		
Allowable working stress per ASME Section VIII.	Not < 3/16 in.		
Shell stress:	Loads will not produce excessive tensile or compressive (buckling) stresses.	Tensile	
Loads: Emergency[a]		10,000	685
OBE nozzle load		Compressive	
		5190	2190
Stress limit:			
ASME Section VIII, compression 1/3 yield			

TABLE 3.9-19 (SHEET 2 OF 2)

Criteria	Method of Analysis	Allowable Stress and Moments Force (lb), Moment (ft-lb)		Calculated Forces and Moments	
Allowable nozzle loads Application of forces and moments by attaching pipe on outlet nozzle under combined maximum thermal expansion, deadweight, and DBE loading reaction. Stress due to internal pressure shall not produce an equivalent bending and torsional stress in the nozzles or shell in excess of the allowable stress as defined by the ASME Boiler and Pressure Vessel Code, Section VIII. [(a)]	Stresses will not be excessive if piping loads do not exceed the allowables.	F_C = 235 lb F_L = 235 lb F_R = 105 lb	M_C = 366 in.-lb M_L = 366 in.-lb M_T = 1050 in.-lb	F_X = 347 lb F_Y = 229 lb F_Z = 435 lb	M_X = 106 ft-lb M_Y = 82 ft-lb M_Z = 8 ft-lb

a. These loads were directly combined.

TABLE 3.9-20 (SHEET 1 OF 3)

RHR PUMP

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
1. Closure bolting[a]	Bolting loads and stresses calculated per "Rules for Bolted Flange Connections," ASME Section VIII, Appendix II.	Maximum allowable stress = 25,000 using the OBE seismic acceleration values	Maximum calculated = 21,090
The design condition analyzed considers the following loads:			
Design pressure and temperature			
Design gasket load			
Seismic acceleration, nozzle forces, and/or moments, static mass forces. The seismic acceleration is applied in two directions:		Maximum allowable stress = 94,500 using the DBE seismic acceleration values	Maximum calculated = 22,300
DBE (1.5-g horizontal) (0.14-g vertical)			
OBE (0.75-g horizontal) (0.07-g vertical)			
Bolting stress limit:			
Allowable working stress per ASME Section VIII	Per rules of Part UG, Section VIII	Maximum allowable stress main pump = 17,500	Maximum calculated = 11,360
2. Wall thickness			
The design condition[a] analyzed considers the following loads:			
Design pressure and temperature			
Stress limit:			
ASME Section VIII			

TABLE 3.9-20 (SHEET 2 OF 3)

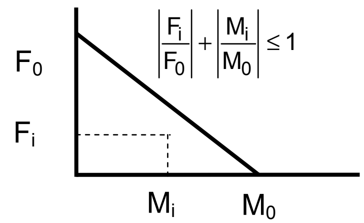
Criteria	Method of Analysis	Allowable Nozzle Forces and Moments, Force (lb), Moment (ft-lb)	Calculated Nozzle Forces and Moments
3. Nozzle loads The design conditions[a] analyzed consider the following: Deadweight, thermal expansion, force and/or moment, and seismic acceleration applied in two directions: DBE (1.5-g horizontal) (0.14-g vertical) OBE (0.75-g horizontal) (0.07-g vertical) Stress limit: ASME Section VIII, primary local membrane stress 1.5 of allowable stress when using OBE values 1.8 of allowable stress when using DBE values	For maximum stresses due to maximum loads	The following expression relates allowable combination of forces and moments:  $\left\|\frac{F_i}{F_0}\right\| + \left\|\frac{M_i}{M_0}\right\| \leq 1$ where: F_i = largest of three actual external orthogonal forces (F_x, F_y, and F_z) that may be imposed by the pipe. M_i = largest of three actual external orthogonal moments (M_x, M_y, and M_z) permitted from the pipe when they are combined simultaneously for any condition. F_0 = allowable of F_i when all moments are zero. M_0 = allowable value of M_i when all forces are zero. The values of F_0 and M_0 are given below: Using the DBE seismic values: Suction: F_0 = 10,610 M_0 = 42,060 Discharge: F_0 = 11,050 M_0 = 35,140	See following page.

TABLE 3.9-20 (SHEET 3 OF 3)

Criteria	Method of Analysis	Allowable Nozzle Forces and Moments, Force (lb), Moment (ft-lb)	Calculated Nozzle Forces and Moments
			Using the DBE seismic values:
			Pump A
			Suction: F_i = 5915 lb
			M_i = 19,523 ft-lb
			Discharge: F_i = 4540 lb
			M_i = 18,014 ft-lb
			Pump B
			Suction: F_i = 5001 lb
			M_i = 16,564 ft-lb
			Discharge: F_i = 4540 lb
			M_i = 18,014 ft-lb
			Pump C
			Suction: F_i = 9005 lb
			M_i = 28,891 ft-lb
			Discharge: F_i = 6496 lb
			M_i = 12,847 ft-lb
			Pump D
			Suction: F_i = 9174 lb
			M_i = 26,478 ft-lb
			Discharge: F_i = 6496 lb
			M_i = 12,847 ft-lb

a. These loads were directly combined.

TABLE 3.9-21 (SHEET 1 OF 2)

RHR HEAT EXCHANGER

Criteria	Method of Analysis	Minimum Thickness Required (in.)	Actual Thickness (in.)
1. Closure Bolting	Bolting loads and stresses calculated per "Rules for Bolted Flange Connections" ASME Section VIII, Appendix II.		
Loads: Normal and upset[a]			
Design pressure and temperature			
Design gasket load			
Bolting Stress Limit	Shell cover bolts	1 1/4 (diameter)	1 1/4
Allowable working stress per ASME Section VIII	Channel cover bolts	1 1/4 (diameter)	1 1/4
2. Wall Thickness	Shell side ASME Section III, TEMA[b] Class C		
Loads: Normal and upset[a]			
Design pressure and temperature	Tube side ASME Section VIII and TEMA[b] Class C		
Stress Limit			
ASME Section VIII	Shell	1.121	1.250
	Shell cover	1.168	1.218
	Channel ring	1.181	1.250
	Tubes: 17 BWG	0.0492	0.052
	Tubes: 18 BWG	0.0439	0.044
	Channel cover	7.403	7.75
	Tube sheet	6.047	6.0625

TABLE 3.9-21 (SHEET 2 OF 2)

Criteria	Method of Analysis	Allowable Nozzle Forces and Moments, Force (lb) Moment (ft-lb)	Calculated Nozzle Forces and Moments
3. Nozzle Loads	Maximum moments due to pipe reaction and maximum forces shall not exceed the allowable limits.	See below.	
Design pressure and temperature[a]			
Deadweight, thermal expansion and DBE:	Primary stress < 1.8 of allowable per ASME Section VIII.		
1.5-g horizontal (statically applied) 0.5 g vertical (statically applied)			

Allowable limits
Design basis

	N1	N2	N3	N4
F_x =	29,400 lb	28,200 lb	45,800 lb	28,400 lb
F_y =	28,400 lb	33,700 lb	47,500 lb	28,400 lb
F_z =	28,400 lb	27,400 lb	45,800 lb	29,200 lb
M_x =	460,000 lb-in.	450,000 lb-in.	746,000 lb-in.	520,000 lb-in.
M_y =	520,000 lb-in.	524,000 lb-in.	885,000 lb-in.	520,000 lb-in.
M_z =	520,000 lb-in.	498,000 lb-in.	746,000 lb-in.	560,000 lb-in.

	B001 A&B Inlet	B001 A&B Outlet	B001 A&B SW Inlet	B001 A&B SW Outlet
F_x =	2986 lb	682 lb	1984 lb	4304 lb
F_y =	13,287 lb	2898 lb	4149 lb	3766 lb
F_z =	9028 lb	3213 lb	2131 lb	740 lb
M_x =	24,845 ft-lb	17,214 ft-lb	15,519 ft-lb	6125 ft-lb
M_y =	10,079 ft-lb	10,516 ft-lb	8100 ft-lb	11,223 ft-lb
M_z =	20,039 ft-lb	2777 ft-lb	5948 ft-lb	8475 ft-lb

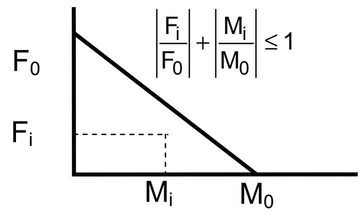
a. These loads are directly combined.
b. Tubular Exchanger Manufacturers Association.

TABLE 3.9-22 (SHEET 1 OF 2)

CORE SPRAY PUMP

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
1. Closure bolting The design condition[a] analyzed considers the following loads: Design pressure and temperature Design gasket load	Bolting loads and stresses calculated per "Rules for Bolted Flange Connections," ASME Section VIII, Appendix II.	Maximum allowable stress = 25,000 Using the OBE seismic acceleration values	Maximum calculated = 13,500
Seismic acceleration, nozzle forces, and/or moments, static mass forces. Seismic acceleration is applied in two directions: DBE: (1.5-g horizontal) (0.14-g vertical) OBE: (0.75-g horizontal) (0.07-g vertical) Bolting stress limit Allowable working stress per ASME Section VIII		Maximum allowable stress = 94,500 Using the DBE seismic acceleration values	Maximum calculated = 15,420
2. Wall thickness The design condition[a] analyzed considers the following loads: Design pressure and temperature Stress limit ASME Section VIII	Per rules of ASME Section VIII, Part UG.	Maximum allowable stress main pump = 17,500	Maximum calculated = 9230

TABLE 3.9-22 (SHEET 2 OF 2)

Criteria	Method of Analysis	Allowable Nozzle Forces and Moments, Force (lb) Moment (ft-lb)	Calculated Nozzle Forces and Moments
3. Nozzle loads The design conditions[a] analyzed consider the following: Deadweight, thermal expansion, force and/or moment, and seismic acceleration applied in two directions: DBE: (1.5-g horizontal) (0.14-g vertical) OBE: (0.75-g horizontal) (0.07-g vertical) Stress Limit ASME Section VIII, primary local membrane stress 1.5 of allowable stress when using the OBE values 1.8 of allowable stress when using the DBE values	For maximum stresses due to maximum loads	The following expression relates allowable combination of forces and moments:  $\left\|\frac{F_i}{F_0}\right\| + \left\|\frac{M_i}{M_0}\right\| \leq 1$ where: F_i = largest of three actual external orthogonal forces (F_x, F_y, and F_z) that may be imposed by the pipe. M_i = largest of three actual external orthogonal moments (M_x, M_y, and M_z) permitted from the pipe when they are combined simultaneously for any condition. F_0 = allowable value of F_i when all moments are zero. M_0 = allowable value of M_i when all forces are zero. The values of F_0 and M_0 are given below. Using the DBE seismic values: Suction: F_0 = 6380 M_0 = 20,290 Discharge: F_0 = 4460 M_0 = 12,640	A / B F_I = 4651 / 4651 M_I = 11,323 / 11,323 F_I = 4894 / 3000 M_I = 5191 / 3100

a. These loads were directly combined.

TABLE 3.9-23 (SHEET 1 OF 3)

HPCI TURBINE

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
1. Closure bolting Loads: Normal and upset[(a)] Design pressure and temperature Design gasket load OBE[(b)] Bolting stress limit Allowable working stress per ASME Section VIII	Bolting loads and stresses calculated per "Rules for Bolted Flange Connections," ASME Section VIII, Appendix II.	Maximum allowable stress = 20,000	Maximum calculated = 18,290
2. Casing wall thickness Loads: Normal and upset[(a)] Design pressure and temperature Design gasket load OBE[(b)] Stress limit ASME Section VIII	Per rules of ASME Section VIII, Part UG.	Maximum allowable stress = 14,000	Maximum calculated = 8975

TABLE 3.9-23 (SHEET 2 OF 3)

Criteria	Method of Analysis	Allowable Nozzle Forces and Moments Force (lb) Moment (ft-lb)	Calculated Nozzle Forces and Moments
3. Nozzle loads	For the resultant moment due to pipe reaction, the resultant force shall not exceed the allowable.	Inlet	
Loads: normal[a]		F = (7570-M)/3	F_R = 1405 lb M_R = 2933 ft-lb
	Design analysis has demonstrated the acceptability of these values.	Exhaust	
Deadweight and thermal expansion		F = (9930-M)/3	F_R = 2645 lb M_R = 11,768 ft-lb
Loads: normal plus upset[a]		Inlet	
		F = (20,000-M)/2.5 but not to exceed 5000 lb	F_R = 1480 lb M_R = 5660 ft-lb
Deadweight thermal expansion, and OBE[b]		Exhaust	
		F = (20,000-M)/0.8 but not to exceed 11,500 lb	F_R = 2759 lb M_R = 12,162 ft-lb
Loads: emergency[a]		Inlet	
		F = (30,000-M)/2.5 but not to exceed 7500 lb	F_R = 2004 lb M_R = 6354 ft-lb
Deadweight, thermal expansion, and DBE[a]		Exhaust	
		F = (30,000-M)/0.8 but not to exceed 17,250 lb	F_R = 3020 lb M_R = 12,558 ft-lb
Stress limits			
Specified by vendor for normal; ASME Section VIII for upset; increased 20% for emergency.			

TABLE 3.9-23 (SHEET 3 OF 3)

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
4. Turbine mounting bolts (turbine to base plate) Loads: normal and upset[(a)] OBE[(b)] Nozzle loads for OBE, dead weight, and thermal expansion Loads: emergency[(a)] DBE[(b)] Nozzle loads for DBE, dead weight, and thermal expansion Stress limits ASME Section VIII allowables for normal and upset. For emergency 0.9-yield tension and twice allowable shear.	Vertical and horizontal forces on mounting bolts and dowel pins calculated as the sum of seismic accelerations on the turbine and pipe reaction forces and moments of the nozzles.	61,100 Tensile and shear stress for bolting materials as specified in ASME Section VIII.	29,800 By meeting the nozzle load criteria of nozzle loads, the detailed seismic analysis indicates that mounting bolts and dowel pins satisfy allowable stress requirements.

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a. These loads were directly combined.
b. OBE = 0.75-g horizontal, 0.24-g vertical (statically applied).
   DBE = 1.5-g horizontal, 0.48-g vertical (statically applied).

TABLE 3.9-24 (SHEET 1 OF 2)

HPCI PUMP

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
1. Closure bolting Design condition[a] analyzed is operating condition and gasket seating condition resulting from design pressure and temperature. Design gasket load resulting from design pressure plus gasket factor. Bolting stress limit Allowable working stress per ASME Section VIII	Bolting loads and stresses calculated per "Rules for Bolted Flange Connections," ASME Section VIII, Appendix II.	Maximum allowable stress main pump = 20,000 booster pump = 20,000	Maximum calculated main pump = 19,950 booster pump = 17,400
2. Casing wall thickness Design condition[a] analyzed is stress due to pressure loading resulting from design pressure and temperature. Stress limit ASME Section VIII	Per rules of ASME Section VIII, Part UG, nozzle stress maximum case stress.	Maximum allowable stress main pump = 14,000 booster pump = 14,000	Maximum calculated main pump = 12,050 booster pump = 3650

TABLE 3.9-24 (SHEET 2 OF 2)

Criteria	Method of Analysis	Allowable Nozzle Forces and Moments Force (lb) Moment (ft-lb)	Calculated Nozzle Forces and Moments
3. Nozzle loads	For maximum resultant moment due to pipe reaction, maximum resultant force shall not exceed allowable. Total nozzle stress with this criteria does not exceed stress limits.		
Design condition[a] analyzed in uninterrupted operation during operating condition resulting from design pressure, temperature, deadweight, thermal expansion, and OBE: (horizontal = 0.75 g) (vertical = 0.07 g)		Suction	
		F = 33,000-0.79M	F_R = 9265 lb M_R = 16,702 ft-lb
		Discharge	
		F = 32,000-1.54M	F_R = 2691 lb M_R = 13,898 ft-lb
Design condition[a] analyzed is no functional failure resulting from design from design pressure, temperature, deadweight, thermal expansion, and DBE: (horizontal = 1.5 g) (vertical = 0.14 g)		Suction	
		F = 43,000-0.74M	F_R = 11,600 lb M_R = 20,900 ft-lb
		Discharge	
		F = 47,000-1.23M	F_R = 3850 lb M_R = 18,100 ft-lb
Stress limit			
ASME Section VIII, for normal and upset; 1.5 of allowable stress for emergency.			

a. These loads were directly combined.

TABLE 3.9-25 (SHEET 1 OF 3)

RCIC TURBINE

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
1. Closure bolting Loads: normal and upset[(a)] Design pressure and temperature Design gasket load OBE[(b)] Bolting stress limit: Allowable working stress per ASME Section VIII	Bolting loads and stresses calculated per "Rules for Bolted Flange Connections," ASME Section VIII, Appendix II.	Maximum allowable stress = 25,000	Maximum calculated = 20,100
2. Casing wall thickness Loads: normal and upset[(a)] Design pressure and temperature OBE[(b)] Stress limit: ASME Section VIII	Per rules of ASME Section VIII, Part UG.	Maximum allowable stress = 17,500	Maximum calculated = 12,700

TABLE 3.9-25 (SHEET 2 OF 3)

Criteria	Method of Analysis	Allowable Nozzles Forces and Moments Force (lb) Moment (ft-lb)	Calculated Nozzle Forces and Moments
3. Nozzle loads	For resultant moment due to pipe reaction, resultant force shall not exceed the allowable.	Inlet	
Loads: normal[(a)]		F = (2620-M)/3	F_R = 765 lb M_R = 1730 ft-lb
Deadweight and thermal expansion	Detailed design analysis has demonstrated acceptability of these values.	Exhaust	
		F = (6000-M)/3	F_R = 535 lb M_R = 4170 ft-lb
Loads: normal plus upset[(a)]		Inlet	
		F = (3000-M)/2.5	F_R = 1000 lb M_R = 2600 ft-lb
Deadweight, thermal expansion, and OBE[(b)]		Exhaust	
		F = 3(6000-M) but not to exceed 8370 lb	F_R = 600 lb M_R = 5000 ft-lb
Loads: emergency		Inlet	
		F = (4500-M)/2.5	F_R = 1090 lb M_R = 2780 ft-lb
Deadweight, thermal expansion, and DBE[(b)]		Exhaust	
		F = 3(9000-M) but not to exceed 12,555 lb	F_R = 925 lb M_R = 4860 ft-lb
Stress limits:			
Specified by vendor for normal; ASME Section VIII for upset; increased by 20%			

TABLE 3.9-25 (SHEET 3 OF 3)

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
4. Turbine mounting bolts (turbine to base plate)	Vertical and horizontal forces on mounting bolts and dowel pins calculated as sum of seismic accelerations on turbine and pipe reaction forces and moments on nozzles.		
Loads: normal and upset[a]		61,100	26,800
OBE[c]		Tensile and shear stress for bolting materials as specified in ASME Section VIII.	By meeting nozzle load criteria, detailed seismic analysis indicates that mounting bolts and dowel pins satisfy allowable stress requirements.
Nozzle loads for OBE, deadweight and thermal expansion			
Loads: emergency		Tensile stress less than 0.9-yield and shear stress less than twice allowable of ASME Section VIII.	
DBE[c]			
Nozzle loads for DBE, deadweight and thermal expansion			
Stress limits:			
ASME Section VIII allowable for normal and upset. For emergency 0.9-yield tension and twice allowable shear.			

a. These loads were directly combined.
b. OBE = 0.75-g horizontal, 0.24-g vertical (statically applied).
   DBE = 1.5-g horizontal, 0.48-g vertical (statically applied).
c. OBE = 1.75-g horizontal, 0.24-g vertical (statically applied).
   DBE = 1.5-g horizontal, 0.48-g vertical (statically applied).

TABLE 3.9-26 (SHEET 1 OF 2)

RCIC PUMP

Criteria	Method of Analysis	Allowable Stress (psi)	Calculated Stress (psi)
1. Closure bolting Design condition[a] analyzed is operating condition and gasket seating condition resulting from design pressure, temperature and design gasket load. Bolting stress limit: Allowable working stress per ASME Section VIII	Bolting loads and stresses calculated per "Rules for Bolted Flange Connections," ASME Section VIII, Appendix II.	Maximum allowable stress = 25,000	Maximum calculated = 22,600
2. Casing wall thickness Design condition[a] analyzed is stress due to pressure loading resulting from design pressure and temperature. Stress limit: Per ASME Section III	Per rules of ASME Section VIII, Part UG, nozzle stress barrel.	Maximum allowable stress Main pump = 17,500	Maximum calculated = 9200

TABLE 3.9-26 (SHEET 2 OF 2)

Criteria	Method of Analysis	Allowable Nozzle Forces and Moments Force (lb) Moment (ft-lb)	Calculated Nozzle Forces and Moments
3. Nozzle loads	For maximum moment due to pipe reaction, maximum force shall not exceed allowable.		
Design condition[a] analyzed is uninterrupted operation during operating conditions resulting from design pressure, temperature, deadweight, thermal expansion, and OBE:	Total nozzle stress with this criteria does not exceed stress limits.	Suction	
		F = 9400-2.50M	F_R = 708 lb M_R = 1050 ft-lb
(horizontal = 0.75 g) (vertical = 0.07 g)		Discharge	
		F = 9400-4.33M	F_R = 585 lb M_R = 1970 ft-lb
Design condition analyzed is no functional failure resulting from design pressure, temperature, deadweight, thermal expansion, and DBE:		Suction	
		F = 19,000-2.42M	F_R = 708 lb M_R = 1050 ft-lb
		Discharge	
(horizontal = 1.5 g) (vertical = 0.14 g)		F = 19,000-5.05M	F_R = 585 lb M_R = 1970 ft-lb
Stress limit:			
ASME Section VIII for normal and upset; 1.5 of allowable stress for emergency.			

a. These loads were directly combined.

TABLE 3.9-27 (SHEET 1 OF 3)

FUEL STORAGE RACKS

A. General Electric

Criteria	Loading	Location	Allowable Stress (psi)	Calculated Stress (psi)
New-fuel storage racks				
Stresses due to normal, upset, or emergency loading shall not cause racks to fail so as to result in a critical fuel array.	Emergency condition	Column	16,000	2950
	Dead loads Full fuel load in rack DBE	Base-to-column welds	11,000	1100
Primary stress limit				
Paper Nos. 3341 and 3342, Proceedings of American Society of Civil Engineers, Structural Division, December 1962 (task committee on light-weight alloys).		Channel	20,000	3150
		Support channel-to-column weld	6000	2650
Spent-fuel storage racks				
Stresses due to normal, upset, emergency, or faulted loading shall not cause racks to fail so as to result in a critical fuel array.	Loads: Dead loads Full fuel load in rack DBE Live loads			

<u>Allowable Stresses</u>

Criteria	Stress Type		D+L + OBE	D+L + SEE
Allowable stresses for each loading combination follow ASME Boiler and Pressure Vessel Code, Section III, Subsection NF, per "Operating Technical Position for Review and Acceptance of Spent Fuel Storage and Handling Applications." Only an elastic analysis was considered. The two controlling loading combination equations were found to be: D + L + OBE and D + L + SSE. D + L + T + SSE was also considered to check for elastic buckling per ASME Section III, Subsection NF. Allowable	Tension	(w/o pin hole)	0.6 Sy	
		(w/pin hole)	0.45 Sy	Increased by $1.2 \frac{Sy}{F}$
	Shear		0.4 Sy	
	Bending Stress		0.66 Sy	
	Bearing		0.9 Sy	

TABLE 3.9-27 (SHEET 2 OF 3)

Allowable Stresses (continued)

stresses are given in Table 4-1 based on the following equations, and they are consistent with the requirements specified in Regulatory Guide 1.124.

NOTE: Sy and F are specified minimum yield strength and allowable tensile stress, respectively.

Emergency Condition B

Loading:

In addition to the loading conditions given on sheet 1 of 3, the racks were tested and analyzed to determine their capability to safely withstand the accidental, uncontrolled drop of the fuel grapple from its fully retracted position into the weakest portion of the rack.

Results of Analysis:

All criteria were met.

Analysis showed that the grapple would shear the welds in the area where the impact occurred. The longitudinal structural member bends but does not fail in shear. Grapple penetration into the rack is not sufficient to cause the vertical columns to deflect the fuel into a critical array. Static load testing showed that forces in excess of those resulting from a grapple drop are required to cause the columns to deflect to the extent that the criteria are violated.

	Comparison of Calculated Stress vs Allowables (psi)			
	OBE Condition		SSE Condition	
Location/Type	Calculated Stress	Allowable[(a)]	Calculated Stress	Allowable[(a)]
Tube wall shear	6040	11,000	7400	22,000
Tube wall compression	7180	14,880	8400	29,760
Tube weld throat shear	8540	11,000	10,400	22,000
Angle, weld throat shear	8540	11,000	10,400	22,000
Casting wall shear	6240	11,000	9210	22,000
Casting wall compression	11,600	16,500	12,500	33,000
Casting base weld shear	4920	11,000	7250	22,000
Support plate weld throat shear	3400	11,000	7250	22,000
Closure plate compression	6570	14,880	7460	29,760
Closure plate shear	6840	11,000	8450	22,000
Closure plate weld shear	9120	11,000	11,300	22,000
Corner tube local compressive - stress check for local buckling	--	--	6900	17,224

TABLE 3.9-27 (SHEET 3 OF 3)

B. Holtec

Criteria	Loading	Location	Allowable Stress (psi)	Calculated Stress (psi)
Spent-fuel storage racks:				
Stresses due to normal, upset, emergency, or faulted loading shall not cause racks to fail so as to result in a critical fuel array.	Fully loaded Half loaded (diagonally) Half loaded (along long axis) Half loaded (along short axis)			
Allowable stresses:				
Stress limits are derived from ASME Code, Section III, Appendix F, faulted values. Parameters and terminology are in accordance with the ASME Code. Calculated values were derived by nonlinear dynamic analysis.	Normal and Upset Conditions (Level A or Level B) Level D Service Limits	At column-to-base welds	38,340	12,450

Comparison of Bounding Calculated Loads/Allowables at Impact Locations and at Welds

	OBE Condition		DBE Condition	
Location/Type	Calculated Stress	Allowable	Calculated Stress	Allowable
Fuel assembly/cell wall impact (lbf)	210	8272[b]	411	8272[b]
Rack/baseplate weld (psi)	3967	21,300	6934	38,340
Baseplate/pedestal weld (psi)	1492	21,300	12,450	38,340
Cell/cell welds (psi)			2511[c]	10,000[d]

a. Allowable stresses are referenced in ASME Code, Section III, Subsection NF.
b. Based on the limit load for a cell.
c. Cell-to-cell weld stresses, including consideration of shear.
d. Conservatively based on OBE allowable stresses.

TABLE 3.9-28

RECIRCULATION PIPE AND PUMP RESTRAINTS

Component	Loading	Locations	Design Limits	Calculated Limits
Restraint frame	Reaction force from pipe break	Multiple on recirculation piping	50% of uniform ultimate strain	< 50% of all restraints
Stainless-steel bar	Reaction force from pipe break	Multiple on recirculation piping	50% of uniform ultimate strain	< 50% of all restraints
Carbon-steel cable	Reaction force from pipe break	Multiple on recirculation piping	90% of guaranteed minimum breaking strength	< 90% of all cables
Pump restraint	Reaction force from pipe break	One on each pump	Primary membrane stress $\sigma_T \leq 1.0 (\sigma_y)$	$\sigma_T < 1.0 (\sigma_y)$
Attachment welds	Reaction force from pipe break	At piping and pump restraint locations	Weld shear stress $\sigma_{SH} \leq 1.5 (\sigma_{AWS})$	$\sigma_{SH} < 1.5 (\sigma_{AWS})$

NOTES:

σ_y = Minimum yield strength by testing or from American Society of Testing Materials (ASTM) specifications. Uniform ultimate strain determined by testing or from ASTM specifications.

σ_{AWS} = Allowable weld stress from American Welding Society Welding Code or AISC structural code.

TABLE 3.9-29 (SHEET 1 OF 2)

DESIGN CRITERIA FOR ASME CLASS 2 AND 3 COMPONENTS[5]

Loading Condition[1]	Piping	Valves	Pumps	Vessels	Mechanical Snubbers	Expansion Bellows
Design			ASME Section III, ASME Section VIII. Performance testing in accordance with the Hydraulic Institute procedures	ASME Section III; ASME Section VII, Division I	See Note 7, typical.	ASME Section III
Design pressure and temperature plus (expansion for bellows)	See paragraph 3.9.2.1 (typical for all loading conditions).	ASME Section III, ANSI B16.5				
Normal						
Operating pressure plus deadweight plus stem thrust for valves plus (nozzle loads for pumps and vessels)		ASME Section III, ANSI B16.5	ASME Section III, ASME Section VIII Performance testing in accordance with the Hydraulic Institute procedures	ASME Section III ASME Section VIII	-	NA[4]
Upset						
Operating pressure plus deadweight plus OBE plus (nozzle loads for pumps and vessels)	-	NA[4]	ASME Section III, ASME Section VIII stress allowable S_h and 1.5 S_m	ASME Section III ASME Section VIII, Division I tension 0.6 S_y shear 0.45 S_y bending 0.66 S_y (See Note 2.)	-	NA[4]
Emergency						
Operating pressure plus deadweight	-	NA[4]	NA	NA	-	NA[4]
Faulted						
Operating pressure plus deadweight plus DBE plus (stem thrust for valves) plus (nozzle loads for pumps and vessels) (See Note 8.)	-	Structure analysis < S_y testing (See Notes 3 and 6.)	ASME Section III, stress < 0.9 S_y testing (See Note 2.)	ASME Section III, ASME, Section VIII, Division 1 stress < 0.9 S_y (See Note 2.)	-	IEEE 344, 1971 stress < 0.9 S_y or by fatigue life in accordance with ASME Paper 61-WA-18

TABLE 3.9-29 (SHEET 2 OF 2)

NOTES:

1. Loading conditions of design, normal, upset, emergency, and faulted are for reference.

2. Seismic loads for pumps and vessels are included only for those designated as Seismic Category I. See appendix A, Compliance with Regulatory Guide 1.48, for details on the RHR and PSW active pumps.

3. Valves designated as Seismic Category I with extended structures are modeled into the piping system analyses as eccentric masses; piping stresses are held within the ASME Code, Section III allowables, as specified in paragraph 3.9.2.1. In addition, a structural analysis is performed by the valve manufacturer in which a static 3-g loading is applied to the operator center of gravity. The 3-g loading is the maximum response that the valve would experience as installed in the piping system under a DBE. The valve assembly is shown not to amplify this response by maintaining natural frequencies in the rigid range. Operability is assured by a combination of analysis and testing. Structural analyses show that deflections are small so as not to cause binding, and motor-operator testing verifies that the motor cycles freely during a minimum acceleration of 3 g.

4. NA means that the particular loading condition was not analyzed for that component.

5. Piping reactions on Class 2 and 3 valves need not be evaluated, because wall thicknesses are at least 10% thicker than the pipe to which they are attached when designed in accordance with ANSI B16.5.

6. Yield stress is used as an allowable for evaluation of the most severe loading condition on the basis of eliminating permanent deformation, thus maintaining dimensional stability. Where active components must remain functional during the event, operability is demonstrated by testing or analysis, or a combination of both. Analyses verify that only elastic deformation occurs so that deflections are small; thus, operability is not impaired.

7. Mechanical snubbers purchased in accordance with ASME Code Section III, (NF) are designed by analysis and generically tested to confirm functional capability. DBE loads are directly combined with dynamic loads due to fast valve opening or closing, and the resulting load is designated as the normal load for the snubber.

8. For the Plant-Unique Analysis, the effects of simultaneous loads due to the DBE and the torus displacement LOCA loads were evaluated for external piping connected to the torus.

TABLE 3.9-30 (SHEET 1 OF 2)

RHRSW 14-in. PUMPS[a]

Location	Load Conditions	Criteria	Allowable Stress (psi)	Calculated Stress (psi)
Discharge head shell	Weight + OBE + nozzle load	Combined stress ($\sigma < S_H$)	12,600	Not calculated because DBE stress < OBE allowable.
	Weight + DBE + nozzle load	$\sigma < 0.9\ S_Y$	29,700	2668
Discharge head base	Weight + OBE + nozzle load	$\sigma < S_H$	13,700	5269
	Weight + DBE + nozzle load	$\sigma < 0.9\ S_Y$	27,000	7281
Sub base	Weight + OBE + nozzle load	$\sigma < S_H$	12,600	4465
	Weight + DBE + nozzle load	$\sigma < 0.9\ S_Y$	29,700	6459
Column pipe	P + weight + OBE	$\sigma < S_H$	20,000	4453
	P + weight + DBE	$\sigma < 0.9\ S_Y$	31,500	5457
Pipe hub	P + weight + OBE	$\sigma < 1.5\ S_H$	30,000	14,723
	P + weight + DBE	$\sigma < 0.9\ S_Y$	31,500	16,172
Column flange	P + weight + OBE	$\sigma < 1.5\ S_H$	20,550	9286
	P + weight + DBE	$\sigma < 0.9\ S_Y$	27,000	12,287
Lineshaft	Torque + thrust + DBE	$\sigma < 0.3\ S_Y$ per ANSI B58.1	12,000	Not calculated because DBE stress < OBE allowable.
	Torque + thrust + DBE	$\sigma < 0.9\ S_Y$	36,000	10,414

TABLE 3.9-30 (SHEET 2 OF 2)

Location	Load Conditions	Criteria	Allowable Stress (psi)	Calculated Stress (psi)
Bolts				
Motor to head	OBE	$\sigma < S_H$	35,500	Not calculated because DBE stress < OBE allowable.
	DBE	$\sigma < 0.9\ S_Y$	94,500	4353
Head-to-sub base	OBE + nozzle load	$\sigma < S_H$	35,000	18,072
	DBE + nozzle load	$\sigma < 0.9\ S_Y$	94,500	26,167
Column flange	OBE	$\sigma < 2\ S_H$ per NB 3232.1	20,000	9863
	DBE	$\sigma < 0.9\ S_Y$	27,000	12,790
Anchor bolts	OBE + nozzle load		Material not specified; stress is below allowables for common bolt materials.	5959
	DBE + nozzle load			8640

a. All allowables are in accordance with ASME B&PV Code, Section III, 1971 Edition, except as noted.

TABLE 3.9-31 (SHEET 1 OF 2)

STANDBY SERVICE WATER PUMP[a]

Location	Load Conditions	Criteria	Allowable Stress (psi)	Calculated Stress (psi)
Discharge head shell	P + weight + OBE	Combined stress ($\sigma < S_H$)	15,000	Not calculated because DBE stress < OBE allowable.
	P + weight + DBE	$\sigma < 0.9\ S_Y$	31,500	1354
Discharge head base	P + weight + OBE	$\sigma < S_H$	13,700	Not calculated because DBE stress < OBE allowable.
	P + weight + DBE	$\sigma < 0.9\ S_Y$	27,000	9818
Sub base	OBE	$\sigma < S_H$	12,600	Not calculated because DBE stress < OBE allowable.
	DBE	$\sigma < 0.9\ S_Y$	29,700	7358
Column pipe	Weight + OBE	$\sigma < S_H$	15,000	Not calculated because DBE stress < OBE allowable.
	Weight + DBE	$\sigma < 0.9\ S_Y$	31,500	12,760
Column flange	P + weight + OBE	$\sigma < 1.5\ S_H$	20,550	10,562
	P + weight + DBE	$\sigma < 0.9\ S_Y$	27,000	17,541
Flange hub	P + weight + OBE	$\sigma < 1.5\ S_H$	22,500	8597
	P + weight + DBE	$\sigma < 0.9\ S_Y$	31,500	11,753

TABLE 3.9-31 (SHEET 2 OF 2)

Location	Load Conditions	Criteria	Allowable Stress (psi)	Calculated Stress (psi)
Lineshaft	Thrust + torque + OBE	$\sigma < 0.3\ S_Y$ per ANSI B58.1	12,000	Not calculated because DBE stress < OBE allowable.
	Thrust + torque + DBE	$\sigma < 0.9\ S_Y$	36,000	8206
Top bowl	P + weight + OBE	$\sigma < S_H$	13,700	Not calculated because DBE stress < OBE allowable.
	P + weight + DBE	$\sigma < 0.9\ S_Y$	27,000	6085
Bolts				
Motor to head	OBE	$\sigma < S_H$	35,000	Not calculated because DBE stress < OBE allowable.
	DBE	$\sigma < 0.9\ S_Y$	94,500	1253
Head to sub base	OBE	$\sigma < S_H$	35,000	Not calculated because DBE stress < OBE allowable.
	DBE	$\sigma < 0.9\ S_Y$	94,500	15,419
Column flange	OBE	$\sigma < S_H$	35,000	21,272
	DBE	$\sigma < 0.9\ S_Y$	94,500	31,648
Anchor bolts	OBE	$\sigma < S_H$	Material not specified; stresses are below allowables for common bolt materials.	6596
	DBE	$\sigma < 0.9\ S_Y$		11,209

a. All allowables are in accordance with ASME B&PV Code, Section III, 1971 Edition, except as noted.

TABLE 3.9-32 (SHEET 1 OF 2)

PSW PUMPS[a]

Item	Loading Combination	Criteria	Allowable Stress (psi)	Calculated Stress (psi)
Discharge head shell	Weight + OBE	Combined stress $\sigma < S_H$	12,000	Not calculated since DBE stress < OBE allowable.
	Weight + DBE	$\sigma < 0.9\ S_y$	29,700	580
Discharge head base	Weight + OBE	$\sigma < S_H$	13,700	10,986
	Weight + DBE	$\sigma < 0.9\ S_y$	27,000	16,637
Sub base	Weight + OBE	$\sigma < S_H$	12,600	11,823
	Weight + DBE	$\sigma < 0.9\ S_y$	29,700	18,164
Column pipe	P + weight + OBE	$\sigma < S_H$	15,000	4562
	P + weight + DBE	$\sigma < 0.9\ S_y$	31,500	6549
Pipe-hub stress	P + weight + OBE	$\sigma < 1.5\ S_H$	22,500	10,792
	P + weight + DBE	$\sigma < 0.9\ S_y$	31,500	12,997
Column flange	P + weight + OBE	$\sigma < 1.5\ S_H$	20,550	16,592
	P + weight + DBE	$\sigma < 0.9\ S_y$	27,000	23,959
Lineshaft	Thrust + torque + OBE	Combined shear stress $< 0.30\ S_y$ per ANSI B58.1	12,000	Not calculated because DBE stress < OBE allowable.
	Thrust + torque + DBE	$\sigma < 0.9\ S_y$	36,000	11,069
Bolts				
Motor-to-head	OBE	$\sigma < S_H$	35,000	Not calculated because DBE stress < OBE allowable.
	DBE	$\sigma < 0.9\ S_y$	94,500	3477

TABLE 3.9-32 (SHEET 2 OF 2)

Item	Loading Combination	Criteria	Allowable Stress (psi)	Calculated Stress (psi)
Head-to-sub base	OBE	$\sigma < S_H$	35,000	17,155
	DBE	$\sigma < 0.9\ S_y$	94,500	26,255
Column flange	OBE	$\sigma_{ave} < 2\ S_H$ per NB-3232.1	20,000	15,577
	DBE	$\sigma < 0.9\ S_y$	27,000	22,398
Anchor bolts	OBE	$\sigma < S_H$	Material not specified; stresses are below allowables for common bolt materials.	8147
	DBE	$\sigma < 0.9\ S_y$		17,812

a. All allowables are in accordance with ASME B&PV Code, Section III, 1971 Edition, except as noted.

TABLE 3.9-33 (SHEET 1 OF 16)

ACTIVE VALVES IN RCPB AND OTHER SEISMIC CATEGORY I SYSTEMS (BECHTEL SUPPLIED)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [(a)]
RCPB (See drawing nos. H-26000, H-26001, H-26003, H-26014, H-26015, H-26018, H-26020, H-26021, H-26023, H-26024, H-26036, H-26037, and H-26189)					
2B21-F010A,B	Feedwater line inside containment	Check	18	None	A, F
2B21-F016	Steam line drain isolation valve inside containment	Gate	3	Motor	A, F[(e)]
2B21-F019	Steam line drain isolation valve outside containment	Gate	3	Motor	A, F[(e)]
2B21-F024A-D	Inboard MSIV air supply check valve	Check	1 1/2	None	C
2B21-F029A-D	Outboard MSIV air supply check valve	Check	1 1/2	None	C
2B21-F036A-H, K,L,M	Main steam safety/relief air inlet check valve	Check	1	None	C
2B21-F037A-H, K,L,M	Main steam safety/relief discharge line vacuum breaker	Check	6	None	C
2B21-F041	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F043A,B	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F045A,B	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F047A,B	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F049A,B	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E

TABLE 3.9-33 (SHEET 2 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
RCPB (cont)					
2B21-F051A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F053A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F055	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F057	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F059A-H, L,M,N,P,R-U	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F061	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F070A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F071A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F072A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F073A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B21-F076A,B	Feedwater line isolation valve outside containment	Check	18	Air	B, G[e]
2B21-F077A,B	Feedwater line isolation valve outside containment	Check	18	Air	B, G[e]
2B31-F003A,B	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E

TABLE 3.9-33 (SHEET 3 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
RCPB (cont)					
2B31-F004A,B	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B31-F009A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B31-F010A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B31-F011A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B31-F012A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B31-F013A,B	Recirculation pump seal supply line inside containment	Check	3/4	None	C
2B31-F017A,B	Recirculation pump seal supply line outside containment	Check	3/4	None	C
2B31-F019	Recirculation sample line isolation inside containment	Globe	1	Air	F[e]
2B31-F020	Recirculation sample line isolation outside containment	Globe	1	Air	F[e]
2B31-F040A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2B31-F057A,B	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2E11-F008	RHR shutdown suction outside containment	Gate	20	Motor	A, F[e]
2E11-F009	RHR shutdown suction inside containment	Gate	20	Motor	A, F[e]
2E11-F015A,B	RHR system LPCI line outside containment	Gate	24	Motor	A, F[e]

TABLE 3.9-33 (SHEET 4 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
RCPB (cont)					
2E11-F022	RHR vessel head spray inside containment[f]				
2E11-F023	RHR vessel head spray outside containment[f]				
2E11-F050A,B	RHR system LPCI check valve inside containment	Check	24	None	A, F
2E11-F122A,B	Bypass for check valves 2E11-F050A,B	Globe	1	Air	B[b][e]
2E21-F005A,B	CS pump discharge outside containment	Gate	10	Motor	A, F[e]
2E21-F006A,B	CS pump discharge inside containment	Check	10	None	A, F
2E21-F018A-C	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2E21-F037A,B	Bypass for check valves 2E21-F006A,B	Globe	1	Air	B[b][e]
2E41-F002	HPCI steam line isolation inside containment	Gate	10	Motor	A, F[e]
2E41-F003	HPCI steam line isolation outside containment	Gate	10	Motor	A, F[e]
RCIC System (See drawing nos. H-26023 and H-26024.)					
2E41-F024A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2E51-F007	RCIC steam line isolation inside containment	Gate	4	Motor	A, F[e]
2E51-F008	RCIC steam line isolation outside containment	Gate	4	Motor	A, F[e]
2E51-F044A-D	Instrumentation line excess flow check valve	Excess flow check	1	Self/solenoid reset	E
2E51-F001	Exhaust line isolation valve	Stop check	10	None	B, G
2E51-F002	Exhaust drain isolation valve	Stop check	2	None	C

TABLE 3.9-33 (SHEET 5 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
RCIC (cont)					
2E51-F003	Suction from suppression pool isolation	Butterfly	6	Air	C[c][e]
2E51-F004	Condensate pump discharge drain line	Globe	1	Air	B[b]
2E51-F005	Condensate pump discharge drain line	Globe	1	Air	B[b]
2E51-F010	Suction from CST isolation	Gate	6	Motor	B, G
2E51-F011	Suction from CST	Check	6	None	B, G
2E51-F012	RCIC pump discharge isolation	Gate	4	Motor	B, G
2E51-F013	RCIC pump discharge isolation	Gate	4	Motor	B, G[e]
2E51-F014	RCIC pump discharge line	Check	4	None	B, G
2E51-F015	Cooling water pressure regulator	Pressure check	2	Process fluid	B[b]
2E51-F017	Pump suction relief valve	Relief	1 1/2	Self	C
2E51-F018	Cooling water relief valve	Relief	1 1/2	Self	C
2E51-F019	Minimum flow isolation	Globe	2	Motor	C
2E51-F021	Minimum flow line	Check	2	None	C
2E51-F025	Drain line to main condenser	Globe	1	Air	B[b]
2E51-F026	Drain line to main condenser	Globe	1	Air	B[b]
2E51-F028	Vacuum pump discharge line	Check	2	None	C
2E51-F029	Suction from suppression pool isolation	Gate	6	Motor	B, G
2E51-F030	Suction from suppression pool	Check	6	None	B, G
2E51-F031	Suction from suppression pool isolation	Gate	6	Motor	B, G[e]
2E51-F040	Turbine exhaust line	Check	10	None	B, G

TABLE 3.9-33 (SHEET 6 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
RCIC (cont)					
2E51-F045	Turbine inlet isolation	Globe	4	Motor	B, G
2E51-F046	Cooling water isolation	Globe	2	Motor	C
2E51-F047	Condensate pump discharge	Check	2	None	C
2E51-F102	Exhaust line vacuum breaker line	Check	1 1/2	None	C
2E51-F103	Exhaust line vacuum breaker line	Check	1 1/2	None	C
2E51-F104	Exhaust line vacuum breaker line isolation	Gate	1 1/2	Motor	C[e]
2E51-F105	Exhaust line vacuum breaker line isolation	Gate	1 1/2	Motor	C[e]
RWC System					
2G31-F001	RWC line isolation inside containment	Gate	6	Motor	A, F[e]
2G31-F004	RWC line isolation outside containment	Gate	6	Motor	A, F[e]
RHR System (See drawing nos. H-26014 and H-26015.)					
2E11-F003A,B	Heat exchanger discharge	Gate	16	Motor	B, G[e]
2E11-F004A,-D	Pump suction	Gate	24	Motor	B, G[e]
2E11-F006A-D	Pump suction	Gate	20	Motor	B, G[e]
2E11-F007A,B	Minimum flow line	Gate	4	Motor	B, G[e]
2E11-F010	Crosstie line	Gate	20	Motor	B, G
2E11-F011A,B	Minimum flow line	Gate	4	Motor	B, G[e]
2E11-F016A,B	Containment spray	Globe	16	Motor	B, G[e]
2E11-F017A,B	LPCI line	Angle	24	Motor	B, G[e]
2E11-F021A,B	Containment spray	Gate	16	Motor	B, G[e]

TABLE 3.9-33 (SHEET 7 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
RHR System (cont)					
2E11-F024A,B	Test line	Globe	16	Motor	B, G[e]
2E11-F026A,B	Heat exchanger to RCIC	Gate	4	Motor	B, G[e][g]
2E11-F027A,B	Torus spray	Globe	16	Motor	B, G[e]
2E11-F028A,B	Torus spray	Gate	16	Motor	B, G[e]
2E11-F040	Discharge to radwaste system	Globe	4	Motor	B, G[e]
2E11-F041A-D	Pressure sensing line isolation valve	Globe	1	Air	B[b]
2E11-F047A,B	Discharge to heat exchanger	Gate	16	Motor	B, G[e]
2E11-F048A,B	Heat exchanger bypass	Globe	24	Motor	B, G[e]
2E11-F049	Discharge to radwaste	Gate	4	Motor	B, G[e]
2E11-F053A,B	Heat exchanger to RCIC	Globe	3	Air	B[b][e]
2E11-F065A-D	Pump suction	Butterfly	24	Air	B, G[e]
2E11-F073A,B	Service water crosstie	Gate	10	Motor	B, G[e]
2E11-F074A,B	Service water in heat exchanger A to DRW	Globe	1	Solenoid	B[b][e]
2E11-F075A,B	Service water crosstie	Gate	10	Motor	B, G[e]
2E11-F079A,B	Process sample line	Globe	3/4	Solenoid	B[b]
2E11-F080A,B	Process sample line	Globe	3/4	Solenoid	B[b]
SBGT System (See drawing no. H-26078.)					
2T46-F001A,B	SGTS inlet isolation valve	Butterfly	18	Air	C[d][e]
2T46-F002A,B	SGTS exhaust isloation valve	Butterfly	18	Air	C[d]
2T46-F003A,B	SGTS inlet isolation valve	Butterfly	18	Air	C[d]

TABLE 3.9-33 (SHEET 8 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
Primary Containment Purge System (Reference drawing no. H-26084.)					
2T48-F307	Drywell purge inlet	Butterfly	18	Air	C[d][e]
2T48-F308	Drywell purge inlet	Butterfly	18	Air	C[d][e]
2T48-F309	Torus purge inlet	Butterfly	18	Air	C[d][e]
2T48-F310	Torus isolation valve before vacuum breaker	Butterfly	20	Air	C[d][e]
2T48-F311	Torus isolation valve before vacuum breaker	Butterfly	20	Air	C[d][e]
2T48-F318	Torus purge valve	Butterfly	18	Air	C[d][e]
2T48-F319	Drywell purge valve	Butterfly	18	Air	C[d][e]
2T48-F320	Drywell purge valve	Butterfly	18	Air	C[d][e]
2T48-F324	Torus purge inlet	Butterfly	18	Air	C[d][e]
2T48-F326	Torus purge valve	Butterfly	18	Air	C[d][e]
2T48-F328A,B	Torus to secondary containment vacuum breakers	Check	20	Air	D
2T48-F332A,B	Torus vent valve	Globe	2	Diaphragm	B[b][e]
2T48-F333A,B	Torus vent valve	Globe	2	Diaphragm	B[b][e]
2T48-F335A,B	Drywell vent valve	Globe	2	Diaphragm	B[b][e]
2T48-F336A,B	Drywell vent valve	Globe	2	Diaphragm	B[b]
2T48-F337A,B	Torus vent valve	Globe	2	Diaphragm	B[b]
2T48-F338	Torus bypass	Globe	2	Diaphragm	B[b][e]
2T48-F339	Torus bypass	Globe	2	Diaphragm	B[b][e]
2T48-F340	Drywell bypass	Globe	2	Diaphragm	B[b][e]
2T48-F341	Drywell bypass	Globe	2	Diaphragm	B[b][e]

TABLE 3.9-33 (SHEET 9 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
Primary Containment Purge System (cont)					
2T48-F342A-L	Vacuum breaker solenoid valves	Globe	1/2	Solenoid	B[b]
2T48-F361A,B	Instrument sensing line isolation valve	Globe	1	Air	B[b]
2T48-F362A,B	Instrument sensing line isolation valve	Globe	1	Air	B[b]
2T48-F363A,B	Instrument sensing line isolation valve	Globe	1	Air	B[b]
2T48-F364A,B	Instrument sensing line isolation valve	Globe	1	Air	B[b]
2T48-F334A,B	Drywell vent valve	Globe	2	Diaphragm	B[b][e]
H_2O_2 Analyzer System					
2P33-F002	Inboard drywell sample A	Globe	1	Air	B[b][e]
2P33-F003	Inboard drywell sample B	Globe	1	Air	B[b][e]
2P33-F004	Inboard drywell return A	Globe	1	Air	B[b][e]
2P33-F005	Inboard drywell return B	Globe	1	Air	B[b][e]
2P33-F006	Inboard torus sample A	Globe	1	Air	B[b][e]
2P33-F007	Inboard torus sample B	Globe	1	Air	B[b][e]
2P33-F010	Outboard drywell sample A	Globe	1	Air	B[b][e]
2P33-F011	Outboard drywell sample B	Globe	1	Air	B[b][e]
2P33-F012	Outboard drywell return A	Globe	1	Air	B[b][e]
2P33-F013	Outboard drywell return B	Globe	1	Air	B[b][e]
2P33-F014	Outboard torus sample A	Globe	1	Air	B[b][e]
2P33-F015	Outboard torus sample B	Globe	1	Air	B[b][e]

TABLE 3.9-33 (SHEET 10 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
HPCI System (See drawing nos. H-26020 and H-26021.)					
2E41-F001	HPCI turbine inlet isolation valve	Gate	10	Motor	B[(b)(e)]
2E41-F004	Pump suction from CST isolation	Gate	16	Motor	B[(b)(e)]
2E41-F006	Pump discharge to feedwater line isolation	Gate	14	Motor	B[(b)(e)]
2E41-F007	Pump discharge to feedwater line isolation	Gate	14	Motor	B, G[(e)]
2E41-F019	Pump suction from CST	Check	16	None	B, G
2E41-F020	Pump suction relief valve	Relief	1	None	C
2E41-F035	Cooling water pressure regulator	Pressure check	2	Process fluid	B[(b)]
2E41-F041	Suction from suppression pool isolation	Gate	16	Motor	B, G[(e)]
2E41-F042	Suction from suppression pool isolation	Gate	16	Motor	B, G[(e)]
2E41-F045	Suction from suppression pool	Check	16	None	B, G
2E41-F046	Minimum flow line	Check	4	None	B, G
2E41-F048	Barometric condensate pump discharge line	Check	2	None	C
2E41-F049	HPCI turbine exhaust	Check	20	None	B, G
2E41-F050	Cooling water relief valve	Relief	2	None	C[(c)]
2E41-F051	Suppression pool suction isolation	Butterfly	16	Air	C[(e)]
2E41-F053	Exhaust drain pot drain isolation	Globe	1	Solenoid	B[(b)(e)]
2E41-F057	Oil cooler outlet	Check	2	None	C
2E41-F059	Cooling water isolation valve	Globe	2	Motor	C[(e)]
2E41-F102	Exhaust line vacuum breaker line	Check	2	None	C

TABLE 3.9-33 (SHEET 11 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
HPCI System (cont)					
2E41-F103	Exhaust line vacuum breaker line	Check	2	None	C
2E41-F104	Exhaust line vacuum breaker line isolation	Gate	2	Motor	C[e]
2E41-F111	Exhaust line vacuum breaker line isolation	Gate	2	Motor	C[e]
CS System (See drawing no. H-26018.)					
2E21-F001A,B	Pump suction line from torus	Gate	20	Motor	B, G[e]
2E21-F003A,B	Pump discharge line	Check	12	None	B, G
2E21-F004A,B	Pump discharge line	Gate	10	Motor	A, F[e]
2E21-F015A,B	Test line	Globe	10	Motor	B,G[e]
2E21-F019A,B	Pump suction line from torus	Butterfly	20	Air	C[c][e]
2E21-F031A,B	Minimum flow bypass	Gate	3	Motor	B, G[e]
Jockey Pump System (See drawing no. H-26019.)					
2E11-F123A,B	RHR fill line check valve	Check	2	None	C
2E11-F124A,B	RHR fill line isolation	Globe	2	None	C
2E21-F039A,B	CS fill line check valve	Check	1 1/2	None	C
2E21-F040A,B	CS fill line isolation	Globe	1 1/2	None	C
2E21-F044A,B	Discharge to CS test line isolation	Stop check	1 1/2	None	C
2E21-F050A,B	Pump suction line	Check	2	None	C
2E21-F052A,B	Pump discharge line	Check	2	None	C
2E21-F053A,B	Pump discharge line	Check	2	None	C

TABLE 3.9-33 (SHEET 12 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
Jockey Pump System (cont)					
2E21-F056A,B	Pump suction line	Check	2	None	C
2E21-F061A,B	Pump recirculation line	Check	2	None	C
2E21-F063A,B	Pump recirculation line	Check	2	None	C
FPCC System (See drawing no. H-26039.)					
2G41-F017	RHR to fuel pool cooling isolation	Gate	6	Handwheel	B
2G41-F034	RHR to fuel pool cooling isolation	Gate	6	Handwheel	B
2G41-F038	Diffuser check valve	Check	6	None	B
2G41-F039	Diffuser check valve	Check	6	None	B
PSW System (See drawing nos. H-21033, H-26050 and H-26051.)					
2P41-F035A,B	Inlet to HPCI pump room coolers	Globe	2	Air	B[(b)(e)]
2P41-F036A,B	Inlet to RHR and CS pump room coolers	Globe	3	Air	B[(b)(e)]
2P41-F037A-D	Cooling water to RHR pumps	Globe	1 1/2	Air	B[(b)(e)]
2P41-F039A,B	Inlet to RHR and CS pump room coolers	Globe	3	Air	B[(b)(e)]
2P41-F040A,B	Inlet to RCIC pump room coolers	Globe	2	Air	B[(b)(e)]
2P41-F042A,B	Inlet to CRD pump room coolers	Globe	3	Air	B[(b)]
2P41-F310	Dilution line	Butterfly	30	Motor	C[(d)]
2P41-F316A-D	Turbine building isolation	Butterfly	30	Motor	C[(d)]
2P41-F320A-D	Minimum flow valve	Globe	3	Air	B[(b)]

TABLE 3.9-33 (SHEET 13 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
PSW System (cont)					
2P41-F334A,B	Pump motor cooling water	Globe	1	Process fluid	B[b]
2P41-F339A,B	Diesel generator cooler outlet	Butterfly	6	Air	C[d]
2P41-F340	Diesel generator cooler outlet	Butterfly	6	Air	C[d]
Reactor Building Closed Cooling Water System (See drawing nos. H-26054 and H-26055.)					
2P42-F051	Isolation valve outside containment	Gate	6	Motor	B, G[e]
2P42-F052	Isolation valve outside containment	Gate	6	Motor	B, G[e]
RHR Service Water System (See drawing no. H-21039.)					
2E11-F068A,B	Heat exchanger outlet control	Globe	18	Motor	B[b][e]
2E11-F119A,B	Crosstie line	Gate	18	Motor	B[e]
2E11-F207A-D	Minimum flow line	Globe	2	Air	B[b]
Drywell Pneumatic System (See drawing nos. H-26066 and H-28023.)					
2P70-F001A,B	Drywell pneumatic nitrogen backup valve	Globe	2	Air	B[b][e]
2P70-F002	Drywell pneumatic isolation valve	Globe	1	Air	B[b][e]
2P70-F003	Drywell pneumatic isolation valve	Globe	1	Air	B[b][e]
2P70-F004	Drywell pneumatic isolation valve	Globe	2	Air	B[b][e]
2P70-F005	Drywell pneumatic isolation valve	Globe	2	Air	B[b][e]
2P70-F044	Drain from receiver 2P70-A001	Solenoid	1/2	Electric	B[b]

TABLE 3.9-33 (SHEET 14 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [(a)]
Drywell Pneumatic System (cont)					
2P70-F103A,B	Outlet from filters 2P70-D009A,B	Globe	1	Process fluid	B[(b)]
Torus Drainage and Purification System (See drawing no. H-26042.)					
2G51-F011	Condensate pump suction from torus	Gate	3	Air	B, G[(e)]
2G51-F012	Condensate pump suction from torus	Gate	3	Air	B, G[(e)]
Radwaste System (See drawing nos. H-26026 through H-26032.)					
2G11-F003	Drywell floor drain sump first isolation valve	Gate	3	Air	B[(e)]
2G11-F004	Drywell floor drain sump second isolation valve	Gate	3	Air	B[(e)]
2G11-F019	Drywell equipment drain sump first isolation valve	Gate	3	Air	B[(e)]
2G11-F020	Drywell equipment drain sump second isolation valve	Gate	3	Air	B[(e)]
Drywell Cooling and Chilled Water System					
2P64-F045	Chilled water line isolation	Globe	6	Motor	B, G[(e)]
2P64-F047	Chilled water line isolation	Globe	6	Motor	B, G[(e)]
Instrument Air System (See drawing nos. H-21028, H-21077, H-26064, H-26070, H-26260 and H-26261.)					
2P52-F565	Nitrogen backup to instrument air	Globe	2	Motor	C
Fission Product Monitoring System (See drawing nos. H-16173 and H16274.)					
2D11-F050	Fission product sample	Globe	1	Air	B[(b)(e)]

TABLE 3.9-33 (SHEET 15 OF 16)

Valve No.	Service Description	Valve Type	Size Line (in.)	Actuator Type	Environmental Design Conditions [a]
Fission Product Monitoring System (cont)					
2D11-F051	Fission product sample	Globe	1	Air	B[b][e]
2D11-F052	Fission product sample	Globe	1	Air	B[b][e]
2D11-F053	Fission product sample	Globe	1	Air	B[b][e]

TABLE 3.9-34 (SHEET 1 OF 2)

ACTIVE VALVES IN RCPB AND OTHER SEISMIC CATEGORY I SYSTEMS (GE SUPPLIED)

MPL No.	Type	Line Size (in.)	System Installed	Operator Actuator	Environmental Design Condition	Remarks
B21-F013A,B C,D,E,F,G,H,K, L,M (e)		6	Nuclear boiler system in main steam lines	Solenoid controlled air valve and pneumatic diaphragm operator	(a)(d)	Solenoid valves qualified by Target Rock to IEEE Std 323-1974
	Safety relief, 3-stage, pilot-operated Target Rock 0867F/09G-001					Qualified per S-63182 and S-63848 by Target Rock
B21-F022A,B,C,D (inboard) B21-F028A,B,C,D (outboard)	MSIV, Rockwell	24	Nuclear boiler system in main steam lines	Spring and pneumatic cylinder-air-to open, air and/or spring-to-close	(b)(d)	
B31-F031A,B	Block (discharge)	28	Recirculation system	Motor-operated operator Limitorque	(c)(d)	Operators from Limitorque qualified to IEEE Std 382-1972
B31-F023A,B	Block (suction)	28				

a. Ambient Conditions - Valves are exposed to the following ambient conditions within a pressure-retaining enclosure:

	Normal	Emergency A	Emergency B	Emergency C	Emergency D	Emergency E
Temperature (°F)	150	340	340	320	250	200
Pressure (psig)	0 to 2	65 (maximum)	45 (maximum)	45 (maximum)	25 (maximum)	20 (maximum)
Relative humidity (%)	100	100	100	100	100	100
Duration	Continuous	< 60 s	3 h	3 h	24 h	100 days

(Total duration is the sum of the separate durations.)

TABLE 3.9-34 (SHEET 2 OF 2)

Incident radiation: Continuous for design life

Gamma: 65 R/h

Gamma and neutron: 75 R/h

The valves are operable, within specification limits, during normal and emergency conditions A, B, C, D, and E, except that valves need not be operable in the power-actuated, pressure-relieving mode during exposure to emergency condition A but must survive exposure to emergency condition A without a detrimental effect. Variance in set pressure, during the automatic pressure-relieving mode of operation, due to ambient superimposed back pressure is permissible.

b. Valves operate as specified at the following ambient conditions within the pressure-retaining enclosure:

	Normal	Emergency				
		A	B	C	D	E
Temperature (°F) (maximum)	150	340	340	320	250	200
Pressure (psig)	0 to 2	65 (maximum)	45 (maximum)	45 (maximum)	25 (maximum)	20 (maximum)
Relative humidity (%)	100	100	100	100	100	100
Duration	Continuous	< 60 s	3 h	3 h	24 h	100 days

(Total duration is the sum of the separate durations.)

Incident radiation: Continuous for design life

Gamma: 15 R/h

Gamma and neutron: 25 R/h

The valves are capable of operation, within specified limits, except regarding variance in closing speed due to superimposed back pressure resulting from emergency ambient conditions during 1-h (total) exposure to emergency ambient conditions A and B, and remain closed during the continuance of emergency ambient conditions.

c. Valves are inaccessible for periods of up to 1 year and are designed to operate over the specified design life in an atmosphere of air or nitrogen at 150°F with 100% humidity. Valves are also designed to operate satisfactorily when exposed to a saturated steam atmosphere or mixture of nitrogen and steam under the following conditions:

- 340°F for 3 h at 100% humidity between -2 and 45 psig.
- 320°F for 4 1/2 h at 100% humidity between -2 and 20 psig.

d. Refer to Plant Hatch Central File for the Environmental Qualification of Electrical Equipment for the environmental design conditions of certain aspects of this equipment.

TABLE 3.9-34 (SHEET 2 OF 2)

Incident radiation: Continuous for design life

Gamma: 65 R/h

Gamma and neutron: 75 R/h

The valves are operable, within specification limits, during normal and emergency conditions A, B, C, D, and E, except that valves need not be operable in the power-actuated, pressure-relieving mode during exposure to emergency condition A but must survive exposure to emergency condition A without a detrimental effect. Variance in set pressure, during the automatic pressure-relieving mode of operation, due to ambient superimposed back pressure is permissible.

b. Valves operate as specified at the following ambient conditions within the pressure-retaining enclosure:

	Normal	Emergency				
		A	B	C	D	E
Temperature (°F) (maximum)	150	340	340	320	250	200
Pressure (psig)	0 to 2	65 (maximum)	45 (maximum)	45 (maximum)	25 (maximum)	20 (maximum)
Relative humidity (%)	100	100	100	100	100	100
Duration	Continuous	< 60 s	3 h	3 h	24 h	100 days

(Total duration is the sum of the separate durations.)

Incident radiation: Continuous for design life

Gamma: 15 R/h

Gamma and neutron: 25 R/h

The valves are capable of operation, within specified limits, except regarding variance in closing speed due to superimposed back pressure resulting from emergency ambient conditions during 1-h (total) exposure to emergency ambient conditions A and B, and remain closed during the continuance of emergency ambient conditions.

c. Valves are inaccessible for periods of up to 1 year and are designed to operate over the specified design life in an atmosphere of air or nitrogen at 150°F with 100% humidity. Valves are also designed to operate satisfactorily when exposed to a saturated steam atmosphere or mixture of nitrogen and steam under the following conditions:

- 340°F for 3 h at 100% humidity between -2 and 45 psig.
- 320°F for 4 1/2 h at 100% humidity between -2 and 20 psig.

d. Refer to Plant Hatch Central File for the Environmental Qualification of Electrical Equipment for the environmental design conditions of certain aspects of this equipment.

TABLE 3.9-35 (SHEET 1 OF 5)

DESIGN CRITERIA FOR HVAC COMPONENTS NOT COVERED BY ASME CODE

System Components	Controlling Standards and/or Codes	Test Report No. Test Procedure No.
SBGT system train		
Exhaust fans and drivers	Applicable seismic response curves per attachment to Specifications	Seismic calculations
	American Society for Testing and Materials (ASTM) Standards D2862, A366, 1056, D2866, & D28	
	Air Moving and Conditioning Association Incorporated, Test Codes 300-67, & 210-67	Certified fan performance curves
	IEEE Standard 323-71, Guide for Qualification of Class 1E Equipment for Nuclear Power Generating Stations	Equipment not required for rulemaking, 10 CFR 50.49
	Military Specifications MIL-51079 and MIL-R6130	
Filter housing		Seismic calculations
	ANSI B16.5 (1968), for steel pipe flanges and flanged fittings	Inspection test documentation (filter media tests, etc.) per Farr Company Procedure QC-10
	ANSI B16.11 (1966), for forged-steel fittings, socket welded and threaded	In-place leak test of HEPA filter banks per Farr Company Procedure L53460
	ANSI N45.8, Testing of Nuclear Air Cleaning Systems	In-place leak test of carbon bank per Farr Company Procedure L53577
		Certified fan performance curves
Filter elements	Underwriters' Laboratories Standard UL-900, Air Filter Units	Seismic calculations
	NRC Health and Safety Information Bulletin, Issue 306, March 31, 1971	Inspection test documentation (filter media tests, etc.) per Farr Company Procedure QC-10

TABLE 3.9-35 (SHEET 2 OF 5)

System Components	Controlling Standards and/or Codes	Test Report No. Test Procedure No.
SBGT system train (cont)		
	Military Specification MIL-F-51068C, Filter, Particulate, High Efficiency, and Fire Resistance	In-place leak test of HEPA filter banks per Farr Company Procedure L53460
	National Bureau of Standards (NBS) Bulletin A, Test Method for Air Filters, by A.S. Dill, 1966	In-place leak test of carbon bank per Farr Company Procedure L53577
	ASTM Standards, D2862, A366, 1056, D2866, D28	Certified fan performance curves
	ANSI N101.1, 1972, Efficiency Testing of Air Cleaning Systems Containing Devices for Removal of Particles	
	Technical, Unimpregnated	
	NRC Division of Reactor Development and Technology Standard RDTM16-IT, "Gas-Phase Adsorbents for Trapping Radio Active Iodine Compounds," including Amendment dated March 7, 1973	
	ANSI N45.8, Testing of Nuclear Air Cleaning Systems	
Instrumentation	IEEE 279 and IEEE 336	Seismic calculations
RHR pump room cooling units and HPCI pump room cooling unit		
Fan and driver		Seismic Analysis Report, CVI A 905-9913
		ARI calculations, CVI A 905-9927
	Air Moving and Conditioning Association Incorporated Test Code 300-67, 210-67	Liquid penetrant test per CVI Procedure 38-1002

TABLE 3.9-35 (SHEET 3 OF 5)

System Components	Controlling Standards and/or Codes	Test Report No. Test Procedure No.
RHR pump room cooling units and HPCI pump room cooling unit (cont)		
Fan and driver (cont)	ASTM Standards	Hydrostatic coil test per CVI Procedure A 905-9903
		Certified fan performance curves
Plenum	American Welding Society (AWS) Standard D1.0-69	Seismic Analysis Report, CVI A 905-9913
		ARI calculations, CVI A 905-9927
		Liquid penetrant test per CVI Procedure 38-1002
		Hydrostatic coil test per CVI Procedure A 905-9903
		Certified fan performance curves
Battery room		
Emergency exhaust fan and driver		Seismic Analysis Report, CVI A 905-9913
		Seismic Calculations
	Air Moving and Conditioning Association Incorporated Test Codes 300-67 and 210-67	Certified performance curves
	ASTM Standard	

TABLE 3.9-35 (SHEET 4 OF 5)

System Components	Controlling Standards and/or Codes	Test Report No. Test Procedure No.
Control room environmental control (MCREC) system		
Fans	Air Moving and Conditioning Association	Seismic calculations
		Certified fan performance curves
Filters (HEPA and charcoal)	American Filter Institute Artificial dust weight test (Section 1)	Seismic calculations
	United States Army Edgewood Arsenal Instruction Manuals: 136-300-195A and 136-300-175A	In-place leak test of HEPA filter banks per Farr Company Procedure L41656
	Military Specification MIL-F-51068A, as amended and modified to meet particular needs of national atomic energy program per NRC Health and Safety Information Bulletin Issue 212, June 25, 1965	In-place leak test of carbon bed per Farr Company Procedure L47634 Certified fan performance curves
	American Society of Heating, Refrigerating and Air Conditioning Engineers	
	Sheet Metal and Air Conditioning Contractors National Association, Incorporated	
	NRC Bulletin 306	
	NBS Standard (dust spot)	
	Underwriters' Laboratories UL-900 and UL-586	
	ASTM D1056	
	NRC DP1075	
Air-conditioning unit	Associated Air Balance Council	Seismic calculations
	Air Moving and Conditioning Association	Leak test for cooling coils
		Certified fan performance curves

TABLE 3.9-35 (SHEET 5 OF 5)

System Components	Controlling Standards and/or Codes	Test Report No. Test Procedure No.
(MCREC) system (cont)		
Ductwork and insulation		Seismic calculations
	Sheet Metal and Air Conditioning Contractors National Association	Leak test for cooling coils
	National Building Code, Section 200	In-place leak test of HEPA filter banks per Farr Company Procedure L41656
	Underwriters' Laboratories	In-place leak test of carbon bed per Farr Company Procedure L47634
		Certified fan performance curves
Controls and instrumentation	National Electric Manufacturers Association	Seismic calculations
	IEEE Standard	Leak test for cooling coils
Refrigerant piping and condensing units	ANSI B31.5	Seismic calculations
	American Society of Heating, Refrigerating and Air-Conditioning Engineers	Leak test for cooling coils
	ASME Section VIII, Division I	

LEGEND

HEPA - high-efficiency particulate air.

TABLE 3.9-36 (SHEET 1 OF 5)

CORE SPRAY 10-in. GATE VALVE

1. Seismic analysis (NB-3524)

Valve Critical Sections	Method of Analysis	Limit	Minimum Required (G/f_n)	Calculated (G_{sm}/f_n)
Motor/yoke bolting	Calculate static acceleration (G_{sm}) that will make sum of bending stress plus thrust stress at section = S_m.	G_{sm} > 3.0 g	3.0	38.3
Yoke arm			3.0	9.46
Yoke/bonnet bolting			3.0	12.75
Body neck			3.0	60.95
Natural frequency	Three lumped-mass cantilever beam analyzed by matrix iteration method.	f_n > 20	20.0	117.0

TABLE 3.9-36 (SHEET 2 OF 5)

2. Design of pressure-retaining parts

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required	Calculated Stress or Actual Dimension
Minimum wall thickness	NB-3541		
$t_m \leq t_e$	NB-3542.1	t_m = 0.772 in.	t_e = 0.937 in.
for $d'_m \geq 1.5\ d_m$	NB-3542.2	1.5 d_m = 14.625 in.	$d'_m = 13.0$ in.
Radius of crotch			
$r_2 \geq 0.3\ t_m$	r_2 = NB-3544.1(a)	0.3 t_m = 0.2316 in.	r_2 = 1.5 in.
$r_3 \geq \begin{Bmatrix} 0.05t_m \\ 0.1h \end{Bmatrix}$ whichever is greater		0.10 H = 0.0625 in.	r_3 = 0.125 in.
Body primary and secondary stress	NB-3545		
$P_m = \left(\frac{A_f}{A_m} + 0.5\right)P_s$	NB-3545.1(a)(2)		
Inlet end	$S_m \geq P_m$ at 500°F	S_m = 19,400 psi at 500°F	P_m = 8377 psi
Outlet end		S_m = 19,400 psi at 500°F	P_m = 8377 psi
Disc and seat ring analysis	NB-3546.2		
$P_m \leq 1.0\ S_m$	Acceptable stress analysis method	1.0 S_m = 19,400 psi at 500°F	P_m = 11,433 psi
$S_{max} \leq 1.5\ S_m$		1.5 S_m = 29,100 psi at 500°F	S_{max} = 19,357 psi

TABLE 3.9-36 (SHEET 3 OF 5)

3. Structural analysis for other valve parts (NB-3546.3)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Design Value (psi)	Calculated Stress or Actual Value (psi)
Stem thrust stress	Calculate stress due to operator thrust	S_m = 26,700 at 500°F	S_T = 10,904
Stem torque stress	Calculate stress due to operator torque	0.6 S_m = 16,020 at 500°F	S_S = 453
Gasket seating stress	Acceptable stress analysis method	1.5 S_m = 28,300 at 500°F	S = 9590
Bonnet	Calculate stress due to pressure	S_m = 19,400 at 500°F	S = 13,410
Protective or thrust ring	Calculate stress due to pressure	S_m = 44,100 at 500°F	S = 11,554
Eyebolt	Calculate stress due to pressure	S_m = 7000 at 500°F	S = 3097

TABLE 3.9-36 (SHEET 4 OF 5)

4. Cyclic loading requirements

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Secondary stress due to pipe reaction	NB-3545.2(b)		
$P_{ed} = \frac{F_d S}{G_d}$	NB-3545.2(b)(1) Figures NB-3545.2-2, 3 NB-3545.2(b)(1)	1.5 S_m = 29,100 at 500°F	P_{ed} = 9440
$P_{eb} = C_b \frac{F_b S}{G_b}$	Figures NB-3545.2-4, 5 Figure NB-3545.2-6 NB-3545.2(b)(5)	1.5 S_m = 29,100 at 500°F	P_{eb} = 18,608
$P_{et} = 2 \frac{F_b S}{G_t}$	NB-3545.2(b)(1) NB-3545.2(b)(6)(c)	1.5 S_m = 29,100 at 500°F	P_{et} = 17,721
Primary plus secondary stress due to internal pressure	NB-3545.2(a)(1)		
$Q_P = C_P \left(\frac{r_i}{t_e} + 0.5 \right) P_s$	Figure NB-3545.1-1 Figure NB-3545.1(a)-1	No limit required	Q_P = 27,630
Thermal secondary stress at inlet and outlet crotch	NB-3545.2(c)		
Q_{T_1}	Figures	No limit required	Q_{T_1} = NA[(a)]
Q_{T_2}	NB-3545.2(c)-2, 3	No limit required	Q_{T_2} = 177
Q_{T_3}	NB-3545.2(c)-4, 5	No limit required	Q_{T_3} = 271

TABLE 3.9-36 (SHEET 5 OF 5)

4. Cyclic loading requirements (cont)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Valve body secondary stress criteria at crotch region	NB-3545.2		
$S_n = Q_p + P_{ed} + 2Q_{T_2} \leq 3S_m$		3 S_m = 58,200 at 500°F	S_n = 37,425
Normal-duty valve fatigue requirements	NB-3545.3 NB-3550		
$S_{P_1} = \frac{2}{3}Q_p + \frac{P_{eb}}{2} + Q_{T_3} + 1.3Q_{T_1}$	NB-3545.3(a)	No limit required	S_{P_1} = 28,618
$S_{P_2} = 0.4\,Q_P + P_{eb} + 2Q_{T_3}$	NB-3545.3(a)	No limit required	S_{P_2} = 30,201 S_A = 30,201
$N_a \geq 2000$ cycles	N_a = Figure I-9-1	2000 cycles	N_a = 10,900 cycles
Cyclic stress calculation	NB-3554		The analysis complies with NB-3222.4(c). Therefore, fatigue analysis and usage factor are not required.
$Q_P + P_{ed} + C_6C_2C_4\Delta T_{f\,(max)} \leq 3S_m$	NB-3554(a)	3 S_m = 58,200 at 500°F	
$Q_P + P_{eb} + C_6C_3C_4\Delta T_{f\,(max)} \leq 3S_m$	NB-3554(b)	3 S_m = 58,200 at 500°F	
$S_i = \frac{4}{3}Q_p + P_{eb} + C_6(C_3C_4 + C_5)\Delta T_{fi}$	N_i = Figure I-9-1	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	$\sum \frac{N_{ri}}{N_i} = NA$ [a]

a. NA - not applicable.

TABLE 3.9-37 (SHEET 1 OF 5)

HPCI AND CORE SPRAY 10-in. GATE VALVE

1. Seismic analysis (NB-3524)

Valve Critical Sections	Method of Analysis	Limit	Minimum Required (G/f_n)	Calculated (G_{sm}/f_n)
Motor/yoke bolting	Calculate static acceleration (G_{sm}) that will make sum of bending stress plus thrust stress at section = S_m.	G_{sm} > 3.0 g	3.0	97.7
Yoke arm			3.0	65.7
Yoke/bonnet bolting			3.0	24.9
Body neck			3.0	169.6
Natural frequency	Three lumped-mass cantilever beam analyzed by matrix iteration method.	f_n > 20	20.0	141.0

TABLE 3.9-37 (SHEET 2 OF 5)

2. Design of pressure-retaining parts

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required	Calculated Stress or Actual Dimension
Minimum wall thickness	NB-3541		
$t_m \le t_e$	NB-3542.1	t_m = 1.061 in.	t_e = 1.84 in.
for $d'_m \ge 1.5\,d_m$	NB-3542.2	1.5 d_m = 13.968 in.	d'_m = 11.625 in.
Radius of crotch			
$r_2 \ge 0.3\ t_m$	r_2 = NB-3544.1(a)	0.3 t_m = 0.3183 in.	r_2 = 1.50 in.
$r_3 \ge \left\{\begin{matrix} 0.05t_m \\ 0.1h \end{matrix}\right\}$ whichever is greater		0.05 t_m = 0.0594 in.	r_3 = 0.125 in.
Body primary and secondary stress	NB-3545		
$P_m = \left(\frac{A_f}{A_m} + 0.5\right)P_s$	NB-3545.1(a)(2)		
Inlet end	$S_m \ge P_m$ at 500°F	S_m = 19,400 psi at 500°F	P_m = 8443 psi
Outlet end		S_m = 19,400 psi at 500°F	P_m = 8443 psi
Disc and seat ring analysis	NB-3546.2		
$P_m \le 1.0\ S_m$	Acceptable stress analysis method	1.0 S_m = 19,400 psi at 500°F	P_m = 14,448 psi (due to thrust)
$S_{max} \le 1.5\ S_m$		1.5 S_m = 29,100 psi at 500°F	S_{max} = 16,443 psi

TABLE 3.9-37 (SHEET 3 OF 5)

3. Structural analysis for other valve parts (NB-3546.3)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Design Value (psi)	Calculated Stress or Actual Value (psi)
Stem thrust stress	Calculate stress due to operator thrust	S_m = 26,700 at 500°F	S_T = 11,670
Stem torque stress	Calculate stress due to operator torque	0.6 S_m = 16,020 at 500°F	S_S = 533
Gasket seating stress	Acceptable stress analysis method	1.5 S_m = 28,300 at 500°F	S = 11,661
Bonnet	Calculate stress due to pressure	S_m = 19,400 at 500°F	S = 13,263
Protective or thrust ring	Calculate stress due to pressure	S_m = 26,700 at 500°F	S = 13,735
Eyebolt	Calculate stress due to pressure	S_m = 7000 at 500°F	S = 6073

TABLE 3.9-37 (SHEET 4 OF 5)

4. Cyclic loading requirements

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Secondary stress due to pipe reaction	NB-3545.2(b)		
$P_{ed} = \frac{F_d S}{G_d}$	NB-3545.2(b)(1) Figures NB-3545.2-2, 3 NB-3545.2(b)(1)	1.5 S_m = 29,100 at 500°F	P_{ed} = 10,084
$P_{eb} = C_b \frac{F_b S}{G_b}$	Figures NB-3545.2-4, 5 Figure NB-3545.2-6 NB-3545.2(b)(5)	1.5 S_m = 29,100 at 500°F	P_{eb} = 21,282
$P_{et} = 2 \frac{F_b S}{G_t}$	NB-3545.2(b)(1) NB-3545.2(b)(6)(c)	1.5 S_m = 29,100 at 500°F	P_{et} = 21,289
Primary plus secondary stress due to internal pressure	NB-3545.2(a)(1)		
$Q_P = C_P \left(\frac{r_i}{t_e} + 0.5 \right) P_s$	Figure NB-3545.1-1 Figure NB-3545.1(a)-1	No limit required	Q_P = 20,322
Thermal secondary stress at inlet and outlet crotch	NB-3545.2(c)		
Q_{T_1}	Figures	No limit required	Q_{T_1} = NA[(a)]
Q_{T_2}	NB-3545.2(c)-2, 3	No limit required	Q_{T_2} = 177
Q_{T_3}	NB-3545.2(c)-4, 5	No limit required	Q_{T_3} = 271

TABLE 3.9-37 (SHEET 5 OF 5)

4. Cyclic loading requirements (cont)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Valve body secondary stress criteria at crotch region	NB-3545.2		
$S_n = Q_p + P_{ed} + 2Q_{T_2} \leq 3S_m$		$3\ S_m = 58{,}200$ at 500°F	$S_n = 30{,}718$
Normal-duty valve fatigue requirements	NB-3545.3 NB-3550		
$S_{P_1} = \frac{2}{3}Q_p + \frac{P_{eb}}{2} + Q_{T_3} + 1.3Q_{T_1}$	NB-3545.3(a)	No limit required	$S_{P_1} = 25{,}571$
$S_{P_2} = 0.4\,Q_P + P_{eb} + 2Q_{T_3}$	NB-3545.3(a)	No limit required	$S_{P_2} = 29{,}836$ $S_A = 29{,}836$
$N_a \geq 2000$ cycles	N_a = Figure I-9-1	2000 cycles	$N_a = 20{,}100$ cycles
Cyclic stress calculation	NB-3554		
$Q_P + P_{ed} + C_6C_2C_4\Delta T_{f\,(max)} \leq 3S_m$	NB-3554(a)	$3\ S_m = 58{,}200$ at 500°F	$S_{nc} = 40{,}736$
$Q_P + P_{eb} + C_6C_3C_4\Delta T_{f\,(max)} \leq 3S_m$	NB-3554(b)	$3\ S_m = 58{,}200$ at 500°F	$S_{n\,max} = 55{,}680$
$S_i = \frac{4}{3}Q_p + P_{eb} + C_6(C_3C_4 + C_5)\Delta T_{fi}$	N_i = Figure I-9-1	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	$\sum \frac{N_{ri}}{N_i} = 0.2458$

TABLE 3.9-38 (SHEET 1 OF 5)

FEEDWATER 18-in. GATE VALVE

1. Seismic analysis (NB-3524)

Valve Critical Sections	Method of Analysis	Limit	Minimum Required (G/f_n)	Calculated (G_{sm}/f_n)
Motor/yoke bolting	Calculate static acceleration (G_{sm}) that will make sum of bending stress plus thrust stress at section = S_m.	$G_{sm} > 3.0$ g	3.0	5.851
Yoke arm			3.0	80.69
Yoke/bonnet bolting			3.0	37.935
Body neck			3.0	90.839
Natural frequency	Three lumped-mass cantilever beam analyzed by matrix iteration method.	$f_n > 20$	20.0	36.0

TABLE 3.9-38 (SHEET 2 OF 5)

2. Design of pressure-retaining parts

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required	Calculated Stress or Actual Dimension
Minimum wall thickness	NB-3541		
$t_m \leq t_e$	NB-3542.1	t_m = 1.739 in.	t_e = 3.235 in.
for $d'_m \geq 1.5\, d_m$	NB-3542.2	1.5 d_m = 23.53 in.	d'_m = 19.50 in.
Radius of crotch			
$r_2 \geq 0.3\, t_m$	r_2 = NB-3544.1(a)	0.3 t_m = 0.5217 in.	r_2 = 1.250 in.
$r_3 \geq \begin{Bmatrix} 0.05t_m \\ 0.1h \end{Bmatrix}$ whichever is greater	Figure NB-5544.1(c)-1	0.05 t_m = 0.087 in. 0.01 h = 0.0871	r_3 = 0.187 in.
Body primary and secondary stress	NB-3545		
$P_m = \left(\frac{A_f}{A_m} + 0.5\right)P_s$	NB-3545.1(a)(2)		
Inlet end	$S_m \geq P_m$ at 500°F	S_m = 18,900 psi at 500°F	P_m = 11,374 psi
Outlet end		S_m = 18,900 psi at 500°F	P_m = 11,374 psi
Valve disc seat ring analysis	NB-3546.2		
$P_m \leq 1.0\, S_m$	Acceptable structural analysis method	1.0 S_m = 18,900 psi at 500°F	P_m = 16,517 psi
$S_{max} \leq 1.5\, S_m$		1.5 S_m = 28,350 psi at 500°F	S_{max} = 18,675 psi

TABLE 3.9-38 (SHEET 3 OF 5)

3. Structural analysis for other valve parts (NB-3546.3)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Design Value (psi)	Calculated Stress or Actual Value (psi)
Stem thrust stress	Calculate stress due to operator thrust	S_m = 26,700 at 500°F	S_T = 20,300
Stem torque stress	Calculate stress due to operator torque	0.6 S_m = 16,020 at 500°F	S_S = 10,600
Gasket seating stress	Acceptable stress analysis method	1.5 S_m = 28,300 at 500°F	S = 13,500
Bonnet	Calculate stress due to pressure	S_m = 16,200 at 500°F	S = 11,005
Protective ring	Calculate stress due to pressure	S_m = 44,100 at 500°F	S = 9030
Eyebolt	Calculate stress due to pressure	S_m = 7000 at 500°F	S = 6821

TABLE 3.9-38 (SHEET 4 OF 5)

4. Cyclic loading requirements

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Secondary stress due to pipe reaction	NB-3545.2(b)		
$P_{ed} = \frac{F_d S}{G_d}$	NB-3545.2(b)(1) Figures NB-3545.2-2, 3 NB-3545.2(b)(1)	1.5 S_m = 29,100 at 500°F	P_{ed} = 4231
$P_{eb} = C_b \frac{F_b S}{G_b}$	Figures NB-3545.2-4, 5 Figure NB-3545.2-6 NB-3545.2(b)(5)	1.5 S_m = 29,100 at 500°F	P_{eb} = 8605
$P_{et} = 2\frac{F_b S}{G_t}$	NB-3545.2(b)(1) NB-3545.2(b)(6)(c)	1.5 S_m = 29,100 at 500°F	P_{et} = 8605
Primary plus secondary stress due to internal pressure	NB-3545.2(a)(1)		
$Q_P = C_P\left(\frac{r_i}{t_e} + 0.5\right)P_s$	Figure NB-3545.1-1 Figure NB-3545.1(a)-1	No limit required	Q_P = 10,838
Thermal secondary stress at inlet and outlet crotch	NB-3545.2(c)		
Q_{T_1}	Figures	No limit required	Q_{T_1} = NA[(a)]
Q_{T_2}	NB-3545.2(c)-2, 3	No limit required	Q_{T_2} = 141.69
Q_{T_3}	NB-3545.2(c)-4, 5	No limit required	Q_{T_3} = 163.73

TABLE 3.9-38 (SHEET 5 OF 5)

4. Cyclic loading requirements (cont)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Valve body secondary stress criteria at crotch region	NB-3545.2		
$S_n = Q_p + P_{ed} + 2Q_{T_2} \leq 3S_m$		$3\ S_m$ = 56,700 at 500°F	S_n = 24,912
Normal-duty valve fatigue requirements	NB-3545.3 NB-3550		
$S_{P_1} = \frac{2}{3} Q_p + \frac{P_{eb}}{2} + Q_{T_3} + 1.3 Q_{T_1}$	NB-3545.3(a)	No limit required	S_{P_1} = 19,690
$S_{P_2} = 0.4\, Q_P + P_{eb} + 2Q_{T_3}$	NB-3545.3(a)	No limit required	S_{P_2} = 17,090
	S = Figure I-9-1		S_A = 19,690
$N_a \geq 2000$ cycles	N_a = Figure I-9-1	2000 cycles	N_a = 100,000 cycles
Cyclic stress calculation	NB-3554		
$Q_P + P_{ed} + C_6 C_2 C_4 \Delta T_{f\,(max)} \leq 3S_m$	NB-3554(a)	$3\ S_m$ = 56,700 at 500°F	S_{nc}= 24,629
$Q_P + P_{eb} + C_6 C_3 C_4 \Delta T_{f\,(max)} \leq 3S_m$	NB-3554(b)	$3\ S_m$ = 56,700 at 500°F	$S_{n\ max}$ = 29,003
$S_i = \frac{4}{3} Q_p + P_{eb} + C_6 (C_3 C_4 + C_5) \Delta T_{fi}$	N_i = Figure I-9-1	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	$\sum \frac{N_{ri}}{N_i} = 0.8458$

a. Not applicable.

TABLE 3.9-39 (SHEET 1 OF 5)

RHR PUMP SUCTION 24-in. ANGLE VALVE

1. Seismic analysis (NB-3524)

Valve Critical Sections	Method of Analysis	Limit	Minimum Required (G/f_n)	Calculated (G_{sm}/f_n)
Motor/yoke bolting	Calculate static acceleration (G_{sm}) that will make sum of bending stress plus thrust stress at section = S_m.	G_{sm} > 3.0 g	3.0	41.0
Yoke arm			3.0	Upper - 308.0 Lower - 47.4
Yoke/bonnet bolting			3.0	15.1
Body neck			3.0	30.0
Natural frequency	Three lumped-mass cantilever beam analyzed by matrix iteration method.	f_n > 20	20.0	28.0

TABLE 3.9-39 (SHEET 2 OF 5)

2. Design of pressure-retaining parts

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required	Calculated Stress or Actual Dimension
Minimum wall thickness	NB-3541		
$t_m \leq t_e$	NB-3542.1	t_m = 1.54 in.	t_e = 2.375 in.
for $d'_m \geq 1.5\, d_m$	NB-3542.2	1.5 d_m = 31.875 in.	d'_m = 22.252 in.
Radius of crotch			
$r_2 \geq 0.3\, t_m$	r_2 = NB-3544.1(a)	0.3 t_m = 0.462 in.	r_2 = 4.0 in.
$r_3 \geq \begin{Bmatrix} 0.05t_m \\ 0.1h \end{Bmatrix}$ whichever is greater		0.05 t_m = 0.077 in. 0.10 h = 0.1625	r_3 = 0.187 in.
Body primary and secondary stress	NB-3545		
$P_m = \left(\frac{A_f}{A_m} + 0.5\right)P_s$	NB-3545.1(a)(2)		
Inlet end	$S_m \geq P_m$ at 500°F	S_m = 18,900 psi at 500°F	P_m = 9264 psi
Outlet end		S_m = 18,900 psi at 500°F	P_m = 9264 psi
Valve disc seat ring analysis	NB-3546.2		
$P_m \leq 1.0\, S_m$	Acceptable structural analysis method	1.0 S_m = 17,900 psi at 500°F	P_m = 7966 psi
$S_{max} \leq 1.5\, S_m$		1.5 S_m = 26,850 psi at 500°F	S_{max} = 16,010 psi

TABLE 3.9-39 (SHEET 3 OF 5)

3. Structural analysis for other valve parts (NB-3546.3)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Design Value (psi)	Calculated Stress or Actual Value (psi)
Stem thrust stress	Calculate stress due to operator thrust	S_m = 17,900 at 500°F	S_T = 17,900
Stem torque stress	Calculate stress due to operator torque	0.6 S_m = 10,740 at 500°F	S_S = 7639
Gasket seating stress	Acceptable stress analysis method	1.5 S_m = 35,000 at 500°F	S = 8710
Bonnet stress at gasket bearing area	Calculate stress due to pressure	S_m = 16,200 at 500°F	S = 10,874
Protective ring	Calculate stress due to pressure	S_m = 44,100 at 500°F	S = 5914
Eyebolt	Calculate stress due to pressure	S_m = 15,000 at 500°F	S = 11,692

TABLE 3.9-39 (SHEET 4 OF 5)

4. Cyclic loading requirements

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Secondary stress due to pipe reaction	NB-3545.2(b)		
$P_{ed} = \frac{F_d S}{G_d}$	NB-3545.2(b)(1) Figures NB-3545.2-2, 3 NB-3545.2(b)(1)	1.5 S_m = 28,350 at 500°F	P_{ed} = 4275
$P_{eb} = C_b \frac{F_b S}{G_b}$	Figures NB-3545.2-4, 5 Figure NB-3545.2-6 NB-3545.2(b)(5)	1.5 S_m = 28,350 at 500°F	P_{eb} = 8105
$P_{et} = 2 \frac{F_b S}{G_t}$	NB-3545.2(b)(1) NB-3545.2(b)(6)(c)	1.5 S_m = 28,350 at 500°F	P_{et} = 8601
Primary plus secondary stress due to internal pressure	NB-3545.2(a)(1)		
$Q_P = C_P \left(\frac{r_i}{t_e} + 0.5 \right) P_s$	Figure NB-3545.1-1 Figure NB-3545.1(a)-1	No limit required	Q_P = 22,568
Thermal secondary stress at inlet and outlet crotch	NB-3545.2(c)		
Q_{T_1}	Figures	No limit required	Q_{T_1} = 1600
Q_{T_2}	NB-3545.2(c)-2, 3	No limit required	Q_{T_2} = 334
Q_{T_3}	NB-3545.2(c)-4, 5	No limit required	Q_{T_3} = 460

TABLE 3.9-39 (SHEET 5 OF 5)

4. Cyclic loading requirements (cont)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Valve body secondary stress criteria at crotch region	NB-3545.2		
$S_n = Q_p + P_{ed} + 2Q_{T_2} \le 3S_m$		3 S_m = 56,700 at 500°F	S_n = 27,510
Normal-duty valve fatigue requirements	NB-3545.3 NB-3550		
$S_{P_1} = \frac{2}{3}Q_p + \frac{P_{eb}}{2} + Q_{T_3} + 1.3Q_{T_1}$	NB-3545.3(a)	No limit required	S_{P_1} = 21,630
$S_{P_2} = 0.4\,Q_P + P_{eb} + 2Q_{T_3}$	NB-3545.3(a)	No limit required	S_{P_2} = 18,052
			S_A = 21,638
$N_a \ge 2000$ cycles	N_a = Figure I-9-1	2000 cycles	N_a = 70,000 cycles
Cyclic stress calculation	NB-3554		
$Q_P + P_{ed} + C_6C_2C_4\Delta T_{f\,(max)} \le 3S_m$	NB-3554(a)	3 S_m = 56,500 at 500°F	S_{nc}= 44,016
$Q_P + P_{eb} + C_6C_3C_4\Delta T_{f\,(max)} \le 3S_m$	NB-3554(b)	3 S_m = 56,700 at 500°F	$S_{n\,max}$ = 54,327
$S_i = \frac{4}{3}Q_p + P_{eb} + C_6(C_3C_4 + C_5)\Delta T_{fi}$	N_i = Figure I-9-1	$\sum \frac{N_{ri}}{N_i} \le 1.0$	$\sum \frac{N_{ri}}{N_i} = 0.4992$

TABLE 3.9-40 (SHEET 1 OF 5)

RHR PUMP SUCTION 24-in. GATE VALVE

1. Seismic analysis (NB-3524)

Valve Critical Sections	Method of Analysis	Limit	Minimum Required (G/f_n)	Calculated (G_{sm}/f_n)
Motor/yoke bolting	Calculate static acceleration (G_{sm}) that will make sum of bending stress plus thrust stress at section = S_m.	$G_{sm} > 3.0$ g	3.0	4.53
Yoke arm			3.0	56.7
Motor yoke bolting			3.0	8.7
Yoke/bonnet bolting			3.0	18.4
Body neck			3.0	91.7
Natural frequency	Three lumped-mass cantilever beam analyzed by matrix iteration method.	$f_n > 20$	20.0	29.9

TABLE 3.9-40 (SHEET 2 OF 5)

2. Design of pressure-retaining parts

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required	Calculated Stress or Actual Dimension
Minimum wall thickness	NB-3541		
$t_m \le t_e$	NB-3542.1	t_m = 2.272 in.	t_e = 3.157 in.
for $d'_m \ge 1.5\, d_m$	NB-3542.2	1.5 d_m = 31.4 in.	d'_m = 26.625 in.
Radius of crotch			
$r_2 \ge 0.3\, t_m$	r_2 = NB-3544.1(a)	0.3 t_m = 0.602 in.	r_2 = 6.0 in.
$r_3 \ge \begin{Bmatrix} 0.05t_m \\ 0.1h \end{Bmatrix}$ whichever is greater		0.05 t_m = 0.1137 in. 0.10 h = 0.1888	r_3 = 0.25 in.
Body primary and secondary stress	NB-3545		
$P_m = \left(\frac{A_f}{A_m} + 0.5\right) P_s$	NB-3545.1(a)(2)		
Inlet end	$S_m \ge P_m$ at 500°F	S_m = 19,400 psi at 500°F	P_m = 9541 psi
Outlet end		S_m = 19,400 psi at 500°F	P_m = 9541 psi
Valve disc seat ring analysis	NB-3546.2		
$P_m \le 1.0\, S_m$	Acceptable structural analysis method	1.0 S_m = 19,400 psi at 500°F	P_m = 16,332 psi
$S_{max} \le 1.5\, S_m$		1.5 S_m = 29,100 psi at 500°F	S_{max} = 13,478 psi

TABLE 3.9-40 (SHEET 3 OF 5)

3. Structural analysis for
other valve parts (NB-3546.3)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Design Value (psi)	Calculated Stress or Actual Value (psi)
Stem thrust stress	Calculate stress due to operator thrust	S_m = 26,700 at 500°F	S_T = 22,845
Stem torque stress	Calculate stress due to operator torque	0.6 S_m = 16,020 at 500°F	S_S = 825
Gasket seating stress	Acceptable stress analysis method	1.5 S_m = 28,300 at 500°F	S = 17,666
Bonnet	Calculate stress due to pressure	S_m = 19,400 at 500°F	S = 3110
Protective ring	Calculate stress due to pressure	S_m = 26,700 at 500°F	S = 17,837
Eyebolt	Calculate stress due to pressure	S_m = 7000 at 500°F	S = 6147

TABLE 3.9-40 (SHEET 4 OF 5)

4. Cyclic loading requirements

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Secondary stress due to pipe reaction	NB-3545.2(b)		
$P_{ed} = \frac{F_d S}{G_d}$	NB-3545.2(b)(1) Figures NB-3545.2-2, 3 NB-3545.2(b)(1)	1.5 S_m = 29,100 at 500°F	P_{ed} = 5553
$P_{eb} = C_b \frac{F_b S}{G_b}$	Figures NB-3545.2-4, 5 Figure NB-3545.2-6 NB-3545.2(b)(5)	1.5 S_m = 29,100 at 500°F	P_{eb} = 12,124
$P_{et} = 2\frac{F_b S}{G_t}$	NB-3545.2(b)(1) NB-3545.2(b)(6)(c)	1.5 S_m = 29,100 at 500°F	P_{et} = 12,144
Primary plus secondary stress due to internal pressure	NB-3545.2(a)(1)		
$Q_P = C_P\left(\frac{r_i}{t_e} + 0.5\right)P_s$	Figure NB-3545.1-1 Figure NB-3545.1(a)-1	No limit required	Q_P = 26,774
Thermal secondary stress at inlet and outlet crotch	NB-3545.2(c)		
Q_{T_1}	Figures	No limit required	Q_{T_1} = 2650
Q_{T_2}	NB-3545.2(c)-2, 3	No limit required	Q_{T_2} = 561
Q_{T_3}	NB-3545.2(c)-4, 5	No limit required	Q_{T_3} = 822

TABLE 3.9-40 (SHEET 5 OF 5)

4. Cyclic loading requirements (cont)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Valve body secondary stress criteria at crotch region	NB-3545.2		
$S_n = Q_p + P_{ed} + 2Q_{T_2} \leq 3S_m$		3 S_m = 58,200 at 500°F	S_n = 33,450
Normal-duty valve fatigue requirements	NB-3545.3 NB-3550		
$S_{P_1} = \frac{2}{3}Q_p + \frac{P_{eb}}{2} + Q_{T_3} + 1.3Q_{T_1}$	NB-3545.3(a)	No limit required	S_{P_1} = 28,178
$S_{P_2} = 0.4\,Q_P + P_{eb} + 2Q_{T_3}$	NB-3545.3(a)	No limit required	S_{P_2} = 24,478 S_A = 28,178
$N_a \geq 2000$ cycles	N_a = Figure I-9-1	2000 cycles	N_a = 30,000 cycles
Cyclic stress calculation	NB-3554		
$Q_P + P_{ed} + C_6C_2C_4\Delta T_{f\,(max)} \leq 3S_m$	NB-3554(a)	3 S_m = 58,200 at 500°F	S_{nc}= 44,199
$Q_P + P_{eb} + C_6C_3C_4\Delta T_{f\,(max)} \leq 3S_m$	NB-3554(b)	3 S_m = 58,200 at 500°F	$S_{n\ max}$ = 56,293
$S_i = \frac{4}{3}Q_p + P_{eb} + C_6(C_3C_4 + C_5)\Delta T_{fi}$	N_i = Figure I-9-1	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	$\sum \frac{N_{ri}}{N_i} = 0.3246$

TABLE 3.9-41 (SHEET 1 OF 5)

RWC 6-in. GATE VALVE

1. Seismic analysis (NB-3524)

Valve Critical Sections	Method of Analysis	Limit	Minimum Required (G/f_n)	Calculated (G_{sm}/f_n)
Motor/yoke bolting	Calculate static acceleration (G_{sm}) that will make sum of bending stress plus thrust stress at section = S_m.	G_{sm} > 3.0 g	3.0	93.5
Yoke arm			3.0	79.2
Yoke/bonnet bolting			3.0	29.2
Body neck			3.0	92.9
Natural frequency	Three lumped-mass cantilever beam analyzed by matrix iteration method.	f_n > 20	20.0	18.97

TABLE 3.9-41 (SHEET 2 OF 5)

2. Design of pressure-retaining parts

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required	Calculated Stress or Actual Dimension
Minimum wall thickness	NB-3541		
$t_m \leq t_e$	NB-3542.1	t_m = 0.71 in.	t_e = 1.063 in.
for $d'_m \geq 1.5\, d_m$	NB-3542.2	1.5 d_m = 8.625 in.	d'_m = 7.75 in.
Radius of crotch			
$r_2 \geq 0.3\, t_m$	r_2 = NB-3544.1(a)	0.3 t_m = 0.213 in.	r_2 = 2.0 in.
$r_3 \geq \begin{Bmatrix} 0.05t_m \\ 0.1h \end{Bmatrix}$ whichever is greater		0.05 t_m = 0.05 in.	r_3 = 0.125 in.
Body primary and secondary stress	NB-3545		
$P_m = \left(\frac{A_f}{A_m} + 0.5\right) P_s$	NB-3545.1(a)(2)		
Inlet end	$S_m \geq P_m$ at 500°F	S_m = 19,600 psi at 500°F	P_m = 4510 psi
Outlet end		S_m = 19,600 psi at 500°F	P_m = 4510 psi
Valve disc seat ring analysis	NB-3546.2		
$P_m \leq 1.0\, S_m$	Acceptable structural analysis method	1.0 S_m = 19,600 psi at 500°F	P_m = 8489 psi
$S_{max} \leq 1.5\, S_m$		1.5 S_m = 29,400 psi at 500°F	S_{max} = NA[(a)]

TABLE 3.9-41 (SHEET 3 OF 5)

3. Structural analysis for other valve parts (NB-3546.3)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Design Value (psi)	Calculated Stress or Actual Value (psi)
Stem thrust stress	Calculate stress due to operator thrust	S_m = 26,700 at 500°F	S_T = 5330
Stem torque stress	Calculate stress due to operator torque	0.6 S_m = 16,020 at 500°F	S_S = 255
Gasket seating stress	Acceptable stress analysis method	1.5 S_m = 28,300 at 500°F	S = 7525
Bonnet gasket bearing area	Calculate stress due to pressure	S_m = 17,900 at 500°F	S = 2445
Protective ring	Calculate stress due to pressure	S_m = 26,700 at 500°F	S = 9549
Eyebolt	Calculate stress due to pressure	S_m = 15,000 at 500°F	S = 6297

TABLE 3.9-41 (SHEET 4 OF 5)

4. Cyclic loading requirements

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Secondary stress due to pipe reaction	NB-3545.2(b)		
$P_{ed} = \frac{F_d S}{G_d}$	NB-3545.2(b)(1) Figures NB-3545.2-2, 3 NB-3545.2(b)(1)	1.5 S_m = 29,400 at 500°F	P_{ed} = 4435
$P_{eb} = C_b \frac{F_b S}{G_b}$	Figures NB-3545.2-4, 5 Figure NB-3545.2-6 NB-3545.2(b)(5)	1.5 S_m = 29,400 at 500°F	P_{eb} = 8445
$P_{et} = 2 \frac{F_b S}{G_t}$	NB-3545.2(b)(1) NB-3545.2(b)(6)(c)	1.5 S_m = 29,400 at 500°F	P_{et} = 9334
Primary plus secondary stress due to internal pressure	NB-3545.2(a)(1)		
$Q_P = C_P\left(\frac{r_i}{t_e} + 0.5\right)P_s$	Figure NB-3545.1-1 Figure NB-3545.1(a)-1	No limit required	Q_P = 14,887
Thermal secondary stress at inlet and outlet crotch	NB-3545.2(c)		
Q_{T_1}	Figures	No limit required	Q_{T_1} = NA[(a)]
Q_{T_2}	NB-3545.2(c)-2, 3	No limit required	Q_{T_2} = 587
Q_{T_3}	NB-3545.2(c)-4, 5	No limit required	Q_{T_3} = 979

TABLE 3.9-41 (SHEET 5 OF 5)

4. Cyclic loading requirements (cont)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Valve body secondary stress criteria at crotch region	NB-3545.2		
$S_n = Q_p + P_{ed} + 2Q_{T_2} \leq 3S_m$		$3\ S_m = 58{,}800$ at 500°F	$S_n = 20{,}497$
Normal-duty valve fatigue requirements	NB-3545.3 NB-3550		
$S_{P_1} = \frac{2}{3} Q_p + \frac{P_{eb}}{2} + Q_{T_3} + 1.3 Q_{T_1}$	NB-3545.3(a)	No limit required	$S_{P_1} = 17{,}726$
$S_{P_2} = 0.4\,Q_P + P_{eb} + 2Q_{T_3}$	NB-3545.3(a)	No limit required	$S_{P_2} = 16{,}359$
			$S_A = 17{,}726$
$N_a \geq 2000$ cycles	N_a = Figure I-9-1	2000 cycles	$N_a = 300{,}000$ cycles
Cyclic stress calculation	NB-3554		
$Q_P + P_{ed} + C_6 C_2 C_4 \Delta T_{f\,(max)} \leq 3S_m$	NB-3554(a)	$3\ S_m = 58{,}800$ at 500°F	$S_{nc} = 47{,}828$
$Q_P + P_{eb} + C_6 C_3 C_4 \Delta T_{f\,(max)} \leq 3S_m$	NB-3554(b)	$3\ S_m = 58{,}800$ at 500°F	$S_{n\,max} = 70{,}843$
$S_i = \frac{4}{3} Q_p + P_{eb} + C_6 (C_3 C_4 + C_5) \Delta T_{fi}$	N_i = Figure I-9-1	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	$\sum \frac{N_{ri}}{N_i} = NA^{(a)}$

a. NA - not applicable.

TABLE 3.9-42 (SHEET 1 OF 5)

RHR PUMP DISCHARGE 20-in. GATE VALVE

1. Seismic analysis (NB-3524)

Valve Critical Sections	Method of Analysis	Limit	Minimum Required (G/f_n)	Calculated (G_{sm}/f_n)
Motor/yoke bolting	Calculate static acceleration (G_{sm}) that will make sum of bending stress plus thrust stress at section = S_m.	$G_{sm} > 3.0$ g	3.0	48.5
Yoke arm			3.0	139.6
Yoke/bonnet bolting			3.0	45.3
Body neck			3.0	145.4
Natural frequency	Three lumped-mass cantilever beam analyzed by matrix iteration method.	$f_n > 20$	20.0	44.8

TABLE 3.9-42 (SHEET 2 OF 5)

2. Design of pressure-retaining parts

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required	Calculated Stress or Actual Dimension
Minimum wall thickness	NB-3541		
$t_m \leq t_e$	NB-3542.1	t_m = 1.898 in.	t_e = 2.91 in.
for $d'_m \geq 1.5\ d_m$	NB-3542.2	1.5 d_m = 26.06 in.	d'_m = 22.5 in.
Radius of crotch			
$r_2 \geq 0.3\ t_m$	r_2 = NB-3544.1(a)	0.3 t_m = 0.57 in.	r_2 = 5.0 in.
$r_3 \geq \begin{Bmatrix} 0.05t_m \\ 0.1h \end{Bmatrix}$ whichever is greater		0.05 t_m = 0.095 in. 0.1 h = 0.132	r_3 = 0.25 in.
Body primary and secondary stress	NB-3545		
$P_m = \left(\frac{A_f}{A_m} + 0.5\right) P_s$	NB-3545.1(a)(2)		
Inlet end	$S_m \geq P_m$ at 500°F	S_m = 18,900 psi at 500°F	P_m = 8067 psi
Outlet end		S_m = 18,900 psi at 500°F	P_m = 8067 psi
Valve disc seat ring analysis	NB-3546.2		
$P_m \leq 1.0\ S_m$	Acceptable structural analysis method	1.0 S_m = 19,400 psi at 500°F	P_m = 9278 psi
$S_{max} \geq 1.5\ S_m$		1.5 S_m = 29,100 psi at 500°F	S_{max} = 12,712

TABLE 3.9-42 (SHEET 3 OF 5)

3. Structural analysis for
other valve parts(NB-3546.3)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Design Value (psi)	Calculated Stress or Actual Value (psi)
Stem thrust stress	Calculate stress due to operator thrust	S_m = 26,700 at 500°F	S_T = 21,047
Stem torque stress	Calculate stress due to operator torque	0.6 S_m = 16,020 at 500°F	S_S = 887
Gasket seating stress	Acceptable stress analysis method	1.5 S_m = 28,700 at 500°F	S = 13,853
Bonnet	Calculate stress due to pressure (radial)	1.5 S_m = 24,300 at 500°F	S = 17,258
Protective (thrust ring)	Calculate stress due to pressure	S_m = 44,100 at 500°F	S = 16,761
Eyebolt	Calculate stress due to pressure	S_m = 7000 at 500°F	S = 5950

TABLE 3.9-42 (SHEET 4 OF 5)

4. Cyclic loading requirements

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Secondary stress due to pipe reaction	NB-3545.2(b)		
$P_{ed} = \frac{F_d S}{G_d}$	NB-3545.2(b)(1) Figures NB-3545.2-2, 3 NB-3545.2(b)(1)	1.5 S_m = 28,350 at 500°F	P_{ed} = 5640
$P_{eb} = C_b \frac{F_b S}{G_b}$	Figures NB-3545.2-4, 5 Figure NB-3545.2-6 NB-3545.2(b)(5)	1.5 S_m = 28,350 at 500°F	P_{eb} = 11,214
$P_{et} = 2 \frac{F_b S}{G_t}$	NB-3545.2(b)(1) NB-3545.2(b)(6)(c)	1.5 S_m = 28,350 at 500°F	P_{et} = 11,211
Primary plus secondary stress due to internal pressure	NB-3545.2(a)(1)		
$Q_P = C_P \left(\frac{r_i}{t_e} + 0.5 \right) P_s$	Figure NB-3545.1-1 Figure NB-3545.1(a)-1	No limit required	Q_P = 24,028
Thermal secondary stress at inlet and outlet crotch	NB-3545.2(c)		
Q_{T_1}	Figures	No limit required	Q_{T_1} = 2500
Q_{T_2}	NB-3545.2(c)-2, 3	No limit required	Q_{T_2} = 545
Q_{T_3}	NB-3545.2(c)-4, 5	No limit required	Q_{T_3} = 845

TABLE 3.9-42 (SHEET 5 OF 5)

4. Cyclic loading requirements (cont)

Stress Category	ASME Section III Reference Method of Analysis	Allowable Stress or Minimum Dimension Required (psi)	Calculated Stress or Actual Dimension (psi)
Valve body secondary stress criteria at crotch region	NB-3545.2		
$S_n = Q_p + P_{ed} + 2Q_{T_2} \leq 3S_m$		3 S_m = 56,700 at 500°F	S_n = 30,757
Normal-duty valve fatigue requirements	NB-3545.3 NB-3550		
$S_{P_1} = \frac{2}{3} Q_p + \frac{P_{eb}}{2} + Q_{T_3} + 1.3Q_{T_1}$	NB-3545.3(a)	No limit required	S_{P_1} = 25,720
$S_{P_2} = 0.4\,Q_P + P_{eb} + 2Q_{T_3}$	NB-3545.3(a)	No limit required	S_{P_2} = 22,514
			S_A = 25,720
$N_a \geq 2000$ cycles	N_a = Figure I-9-1	2000 cycles	N_a = 40,000 cycles
Cyclic stress calculation	NB-3554		
$Q_P + P_{ed} + C_6 C_2 C_4 \Delta T_{f\,(max)} \leq 3S_m$	NB-3554(a)	3 S_m = 56,700 at 500°F	S_{nc}= 43,410
$Q_P + P_{eb} + C_6 C_3 C_4 \Delta T_{f\,(max)} \leq 3S_m$	NB-3554(b)	3 S_m = 56,700 at 500°F	$S_{n\,max}$ = 56,555
$S_i = \frac{4}{3} Q_p + P_{eb} + C_6 (C_3 C_4 + C_5) \Delta T_{fi}$	N_i = Figure I-9-1	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	$\sum \frac{N_{ri}}{N_i} = 0.295$

TABLE 3.9-43

RCIC PIPING 4-in. GATE VALVE AND RHR HEAD SPRAY[a] 4-in. VALVE

Design of Pressure-Retaining Parts - (NB-3541)

Stress Category	ASME Section III Reference	Minimum Dimension Required	Calculated or Actual Dimension
Port wall thickness			
t_m	NB-3542-1	Based on: d_m = 3.4375 in. P_r = 900 lb	t_m = 0.46 in.
Neck wall thickness			
$t'_m \geq t_m$	NB-3542.2	t_m = 0.46 in.	t'_m = 0.513 in.
$d'_m \geq 1.5\, d_m$		1.5 d_m = 5.156 in.	d'_m = 5.75 in.
Minimum thickness of valve for nonstandard pressure ratings	NB-3543	NA[b]	NA[b]

a. RHR head spray is deactivated.
b. NA - not applicable.

TABLE 3.9-44

MAIN STEAM DRAIN AND CRD RETURN PIPING 3-in. GATE VALVE

Design of Pressure-Retaining Parts - (NB-3541)

Stress Category	ASME Section III Reference	Minimum Dimension Required	Calculated or Actual Dimension
Port wall thickness[a]			
	NB-3542-1	$t_m = \frac{13}{32}$ in.[b]	$t_m = \frac{13}{32}$ in.
Neck wall thickness			
$t'_m \geq t_m$	NB-3542.2	$t_m = \frac{13}{32}$ in.	$t'_m = \frac{13}{32}$ in.
$d'_m \geq 1.5\, d_m$	NA[c]		
Minimum thickness of valve for nonstandard pressure ratings	NB-3543	NA[c]	NA[c]

a. See ANSI B16.5.
b. Taken from ANSI B16.5.
c. NA - not applicable.

TABLE 3.9-45

STANDBY SERVICE WATER 6-in. STRAINER

Design of Pressure-Retaining Parts

Component Part Analyzed	Method of Analysis	Allowable Stress or Minimum Thickness		Calculated Stress or Actual Thickness
Shell	UG-27 ASME Code, Section VIII	Circumferential Longitudinal	0.146 in. 0.104 in.	0.365 in. 0.365 in.
Inlet and outlet nozzle flange	UA-49 to UA-52	Longitudinal hub Radial flange Tang flange	26,250 17,500 17,500	16,518 4964 4099
Inlet nozzle	Welding Council, Bulletin 107, Johns and Orange Paper (including pressure stress)	$S_{max} (P_m)$ $S_{max} (P_m + P_b)$	12,000 18,000	6334 10,970
Outlet nozzle	Welding Council, Bulletin 107, Johns and Orange Paper (including pressure stress)	$S_{max} (P_m)$ $S_{max} (P_m + P_b)$	12,000 18,000	5240 11,727
Support	Supported by nozzle connected to piping	NA[a]		NA[a]

a. NA - not applicable.

TABLE 3.9-46

RHRSW 18-in. STRAINER

Design of Pressure-Retaining Parts

Component Part Analyzed	Method of Analysis		Allowable Stress or Minimum Thickness		Calculated Stress or Actual Thickness
Shell	ASME Code, Section VIII, UG-27		Circumferential Longitudinal	0.748 in. 0.356 in.	1 1/4 in. 1 1/4 in.
Inlet and outlet nozzle flange	UA-49 to UA-52		Longitudinal hub Radial flange Tangential flange	26,250 17,500 17,500	14,285 3730 3757
Inlet nozzle	Welding Council, Bulletin 107, Johns and Orange Paper		S_{max} (P_m) S_{max} ($P_m + P_b$)	17,500 26,250	12,550 21,529
Outlet nozzle	Welding 107, Johns and Orange Paper	< 1.0 S < 1.5 S	S_{max} (P_m) S_{max} ($P_m + P_b$)	17,500 26,250	13,976 22,120
Support bolts	Hand calculation	< 0.9 S_y < 0.45 S_y	σ_T τ	29,970 14,985	19,823 6499
Support base flange	Hand calculation	< 1.5 S < 0.6 S	σ_x τ	26,250 10,500	8687 1351
Support weld	Hand calculation	< 0.6 S	τ	10,500	4956
Frequency				> 20 Hz	30 Hz

TABLE 3.9-47

STRESS SUMMARY OF PSW 30-in. STRAINER

Design of Pressure-Retaining Parts

Component Part Analyzed	Method of Analysis	Allowable Stress or Minimum Thickness	Calculated Stress or Actual Thickness
Shell	ASME Section III and Regulatory Guide 1.48	15,500 psi/0.05 in.	1908 psi/0.625 in.[a]
Outlet nozzle	Structural dynamic computer program	17,000 psi/0.0464 in.	1576 psi/0.75 in.[a]
Inlet nozzle	Structural dynamic computer program	17,000 psi/0.0464 in.	1576 psi/0.75 in.[a]
Base plate	ASME Section III, Appendix XVII-2214.3	28,500 psi	4000 psi[a]
Anchor bolts	ASME Section III, Appendix XVIII-2461.3	35,340 psi	21,842 psi (maximum)
Frequency		> 20 Hz	22.9 Hz

a. Actual stress due to external loads plus seismic load.

TABLE 3.9-48 (SHEET 1 OF 2)

RHR AND CORE SPRAY SYSTEMS JOCKEY PUMPS

Component	Normal Steady-State Load (psi)		Normal + OBE (psi)		Normal + OBE (psi)	
Pump holddown pump stress	Actual	Allowable	Actual	Allowable	Actual	Allowable
shear	232	10,000	1845	10,000	2499	16,200
tensile	3304	20,000	7562	20,000	8370	32,400
Anchor bolt stress						
shear	373	10,000	2430	10,000	3211	16,200
tensile	1021	20,000	10,563	20,000	14,317	32,400
Shaft stress	2363	17,500	2603	17,500	2792	30,000
Support frame stress	80	23,760	1953	23,760	2617	32,400
Thrust retainer bolt stress	1232	20,000	1299	20,000	1364	32,400
Pump frame bolt stress						
shear	1000	10,000	1897	10,000	2184	16,200
tensile	502	20,000	1528	20,000	1834	32,400
Frame adapter flange stress	12,387	26,250	13,127	26,250	13,352	52,500
Adapter flange bolt stress	12,338	25,000	13,075	25,000	13,299	37,500
Maximum nozzle stress						
suction	7977	17,500	11,966	27,720	11,966	27,720
discharge	6652	17,500	9771	27,720	9771	27,720
Discharge flange stress	26,151	26,250	37,253	52,500	37,253	52,500
Suction flange stress	23,388	26,250	32,534	52,500	32,534	52,500
Frame-to-cover bolt stress	1400	20,000	4443	20,000	5432	32,400

TABLE 3.9-48 (SHEET 2 OF 2)

Component	Normal Steady-State Load (psi)		Normal + OBE (psi)		Normal + OBE (psi)	
Pump bearing loads	Actual	Allowable	Actual	Allowable	Actual	Allowable
outboard	208	3440	286	3440	335	3440
inboard	1169	5750	1618	5750	1813	5750
Impeller key stress	640	9000	640	9000	640	9000
Pump frame foot stress	69	12,600	402	12,600	489	27,000
Heat exchanger holddown bolts stress	585	20,000	5437	20,000	7043	32,400
Heat exchanger piping stress	1835	15,000	2689	18,000	3364	18,000
Motor holddown bolts						
shear	444	10,000	2033	10,000	2816	16,200
tensile	0	20,000	2908	20,000	4248	32,400
Motor bearing loads						
outboard	17	850	53	850	70	850
inboard	17	1430	77	1430	97	1430

TABLE 3.9-49 (SHEET 1 OF 3)

DIESEL ENGINE GENERATING UNIT

Skid assembly			
Combined stress in skid holddown bolts	3092 psi	Proof strength	28,000 psi
Shear stress in generator holddown bolts	2434 psi	Shear strength	33,000 psi
Combined stress in sub-base middle section holddown bolts	6428 psi	Proof strength	28,000 psi
Combined stress in sub-base forward section holddown bolts	5789 psi	Proof strength	28,000 psi
Load on last two crankshaft main bearings	3167 lb 1156 lb	Standard practice allows additional load above normal operating load.	6000 lb
Load on generator bearings	11,430 lb 8775 lb	For particular bearings at 900 rpm for 100,000-h rated radial load	18,000 lb
Combined stress in turbo-charger mounting bolts	22,915 psi	Proof strength	52,000 psi
Combined stress in turbo-charger foot bracket	22,511 psi	Yield strength	33,000 psi
Overspeed governor and shutdown system			
Acceleration that would cause shutdown	2.24 g	Maximum acceleration produced at governor by earthquake	0.554 g

TABLE 3.9-49 (SHEET 2 OF 3)

Heat exchanger stack assembly			
Bending stress in support bracket	11,940 psi	Allowable stress	21,600 psi
Shear stress in bracket bolts	11,726 psi	Allowable bolt shear	38,000 psi
Stress in brace assembly bars	5775 psi	Allowable stress	21,600 psi
Weld load at foot of support post	7507 lb	Allowable weld load	14,400 lb
Combined stress in bolts in feet	9120 psi	Proof strength	52,000 psi
Bending stress in feet	4569 psi	Allowable stress	21,600 psi
Lube oil filter			
Load on bolts in base	556 lb	Proof load	17,400 lb
Weld load at base	1372 lb	Allowable weld load	20,400 lb
Lube oil strainer			
Tension load on bolts	153 lb	Proof load	11,750 lb
Stress in base weld	1166 psi	Allowable stress	21,600 psi
Weld load at base	439 lb	Allowable weld load	20,400 lb
Fuel oil day tank			
Shear stress in holddown bolts	2537 psi	Shear strength	38,000 psi
Bending stress in feet	5387 psi	Allowable stress	21,600 psi

TABLE 3.9-49 (SHEET 3 OF 3)

Jacket water expansion tank			
Combined stress in bolt	1938 psi	Allowable stress	31,200 psi
Bending stress in feet	1006 psi	Allowable stress	21,600 psi
Weld load at attachment to tank	744 lb	Allowable weld load	23,850 lb
Air compressor skid			
Shear stress in holddown bolts	194 psi	Shear strength	33,000 psi
Air receiver tank			
Combined stress in bolts	4992 psi	Proof strength	52,000 psi
Bending stress in feet	17,611 psi	Yield strength	30,000 psi
Stress in weld at feet	907 psi	Allowable weld stress	2400 psi
Static exciter components cabinet			
Shear stress in holddown bolts	1808 psi	Shear strength	33,000 psi
Shear stress in high-voltage chassis holddown bolts	1484 psi	Shear strength	33,000 psi
Shear stress in current transformer bolts	174 psi	Shear strength	33,000 psi

NOTE:

1. Relay operation confirmed by test by Basler Electric Company and A. O. Smith Corporation.

TABLE 3.9-50 (SHEET 1 OF 2)

RHR DISCHARGE PIPING 24-in. CHECK VALVE

Section	Criteria	Calculated Value Versus Allowable
Seismic Analysis (NB-3524) Static Analysis Method Used		
Base of operator support leg	$P_m \leq S_m$ at 500°F	2480 ≤ 15,000 psi
Bolts	$P_m \leq S_m$ at 500°F	3416 ≤ 30,000 psi
Natural frequency of operator	$f_n \geq$ design specified limit	251 ≥ 20 Hz
End of valve	$P_m \leq S_m$ at 500°F	6424 ≤ 19,400 psi
Middle of valve	$P_m \leq S_m$ at 500°F	6080 ≤ 19,400 psi

ASME Section III Reference	Criteria	Calculated Value Versus Allowable
Design of pressure-retaining parts (NB-3540)		
NB-3544.1(a) Radius of crotch	$R_2 \geq 0.3\ t_m$	3.0 ≥ 0.67 in.
NB-3544.1(b)	$R_4 < R_2$	1.5 < 3.0 in.
NB-3544.5	Out of roundness < 5%	3.4% < 5%
NB-3544.6 Body curved section	$\frac{1}{r_{long}} + \frac{1}{r_{lat}} \geq \frac{4}{3d_m}$	0.096 ≥ 0.069
NB-3545.1 Primary crotch stress due to internal pressure	$P_m \leq S_m$ at 500°F	Inlet: 17,124 ≤ 21,600 psi Outlet: 18,639 ≤ 21,600 psi
NB-3545.2(b) Direct or axial load effect	$P_{ed} \leq 1.5\ S_m$ at 500°F	Inlet: 8129 ≤ 32,400 psi Outlet: 9493 ≤ 32,400 psi

TABLE 3.9-50 (SHEET 2 OF 2)

ASME Section III Reference	Criteria	Calculated Value Versus Allowable
Bending load effect	$P_{eb} \leq 1.5\ S_m$ at 500°F	Inlet: 16,778 ≤ 32,400 psi Outlet: 19,462 ≤ 32,400 psi
Torsional load effect	$P_{et} \leq 1.5\ S_m$ at 500°F	Inlet: 16,783 ≤ 32,400 psi Outlet: 18,972 ≤ 2,400 psi
NB-3545.2 Valve body - primary plus secondary stress at crotch	$S_n \leq 3\ S_m$ at 500°F	Outlet is worst case: 44,537 ≤ 4,800 psi
NB-3545.3	$N_a \geq 2000$	11,000 ≥ 2000 cycles
NB-3553 Fatigue usage	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	0.265 ≤ .0
NB-3554(a) Cyclic stress	$Q_P + P_{eb} + C_6 C_3 C_4 \Delta T_{f\,(max)} \leq 3S_m$	56,091 ≤ 4,800 psi

Section	Criteria	Calculated Value Versus Allowable
Structural analysis (NB-3546)		
Disc (due to pressure)	$S_{max} \leq S_m$ at 500°F	17,606 ≤ 20,800 psi
	$S_{shear} \leq 0.6\ S_m$ at 500°F	10,688 ≤ 12,480 psi
Cover	$S_{max} \leq 1.0\ S_m$ at 500°F	18,350 ≤ 20,800 psi
Retainer gasket	$S_{shear} \leq 0.6\ S_m$ at 500°F	9929 ≤ 25,980 psi
Retainer hinge pin	$S_{max} \leq S_m$ at 500°F	17,147 ≤ 20,800 psi
Body-to-cover joint	$P_L + P_B \leq 1.5\ S_m$ at 500°F	28,678 ≤ 32,400 psi
Indicator housing	$P_L + P_B \leq 1.5\ S_m$ at 500°F	21,000 ≤ 29,580 psi
Indicator retainer	$S_{max} \leq S_m$ at 500°F	17,405 ≤ 17,900 psi
Indicator housing and pin retainer bolting	$S_{max} \leq 2.0\ S_m$ at 500°F	45,000 ≤ 69,400 psi

TABLE 3.9-51

RHR AND CRD HYDRAULIC SYSTEMS
3-in. AND 4-in. CHECK VALVES

Seismic Analysis (NB-3524)-Static Analysis Method Used

Section	Criteria	Calculated Value Versus Allowable
Valve ends	$P_m \le S_m$	3 in. 3102 ≤ 18,400 psi
		4 in. 3102 ≤ 17,300 psi
Valve middle	$P_m \le S_m$	3 in. 3232 ≤ 18,400 psi
		4 in. 3232 ≤ 17,300 psi

Thickness requirement (NB-3541)

Location	Criteria	Actual Value Versus Minimum Required
Near welding ends	$t \ge t_m$	3 in. 1.03 ≥ 0.43 in.
		4 in. 1.03 ≥ 0.55 in.

TABLE 3.9-52 (SHEET 1 OF 3)

HPCI STEAM PIPING 10-in. CHECK VALVE

Valve Section	Criteria	Calculated Value Versus Allowable
1. Seismic analysis-static analysis method (NB-3524)		
Support base	$P_m \leq S_m$ at 500°F	6046 ≤ 16,200 psi
Shear in bolts	$S_s \leq 0.6\ S_m$ at 500°F	643 ≤ 17,880 psi
Tension in bolts	$S_T \leq S_m$ at 500°F	1905 ≤ 28,800 psi
2. Structural analysis		
Disc calculations due to pressure		
Disc thickness	$S_1 \leq S_m$ at 500°F	12,436 ≤ 20,800 psi
Disc shear at edge	$S_s \leq 0.6\ S_m$ at 500°F	8911 ≤ 12,480 psi
Cover calculations		
Cover thickness	$S_{max} \leq S_m$ at 500°F	9734 ≤ 20,800 psi
Retainer (gasket calculations)	$S_s \leq 0.6\ S_m$ at 500°F	9800 ≤ 25,980 psi
Retainer (hinge pin calculations)	$S_t \leq S_m$ at 500°F	20,274 ≤ 20,800 psi
Indicator housing and pin retainer bolting	$S_1 \leq 2\ S_m$ at 500°F	45,000 ≤ 57,600 psi
Body-to-cover analysis		
Axial stress at junction	$S_H \leq 1.5\ S_m$ Maximum	20,999 ≤ 32,400 psi
Tangential stress at junction	$S_T \leq 1.55\ S_m$ Maximum	14,549 ≤ 32,400 psi

TABLE 3.9-52 (SHEET 2 OF 3)

Valve Section	Criteria	Calculated Value Versus Allowable
Radial stress at junction	$S_R \leq 1.5\ S_m$ Maximum	$-2425 \leq 32,400$ psi
Stress intensity (S)	$S \leq 1.5\ S_m$	$25,241 \leq 32,400$ psi
3. Design of pressure-retaining parts - cyclic requirements		
NB-3544.1(a) Radius of crotch	$R_2 \geq 0.3\ t_m$	$3.75 \geq 0.339$ in.
NB.3544.6 Doubly curved section	$\frac{1}{r_{long}} + \frac{1}{r_{lat}} \geq \frac{4}{3d_m}$	$0.23 \geq 0.15$ in.
NB-3544.8 Weld ends		
Inlet	$t_i \geq t_m$	$2.54 \geq 1.13$ in.
Outlet	$t_o \geq t_m$	$1.38 \geq 1.13$ in.
NB-3545.1 Primary crotch stress due to internal pressure	$P_m \leq S_m$ at 500°F	$11,724 \leq 21,600$ psi (inlet critical)
NB-3545.2(b) Secondary stress due to pipe reaction		
Direct or axial load effect	$P_{ed} \leq 1.5\ S_m$ at 500°F	$11,176 \leq 32,400$ psi
Bending load effect	$P_{eb} \leq 1.5\ S_m$ at 500°F	$20,746 \leq 32,400$ psi
Torsional load effect	$P_{et} \leq 1.5\ S_m$ at 500°F	$20,746 \leq 32,400$ psi
NB-3545.2 Valve body - primary plus secondary stress at crotch	$S_n \leq 3\ S_m$ at 500°F	$38,848 \leq 64,800$ psi

TABLE 3.9-52 (SHEET 3 OF 3)

Valve Section	Criteria	Calculated Value Versus Allowable
NB.3545.3 Normal-duty fatigue requirement	$N_a \geq 2000$ cycles	13,000 ≥ 2000 cycles
NB-3553 Fatigue usage	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	0.466 ≤ 1.0
NB-3554(a) Cyclic stress	$Q_P + P_{ed} + C_6C_2C_4\Delta T_{f\,(max)} \leq 3S_m$	51,418 ≤ 64,800 psi
NB-3554(b) Cyclic stress	$Q_P + P_{eb} + C_6C_3C_4\Delta T_{f\,(max)} \leq 3S_m$	64,901 ≰ 64,800 psi

TABLE 3.9-53

RHR 4-in. GLOBE VALVE

Seismic Analysis (NB-3524) Static Analysis Method Used

Section	Criteria	Calculated Value Versus Allowable
Operator adapter plate interface	$P_m \leq S_m$	9035 ≤ 28,800 psi
Base of yoke	$P_m \leq S_m$	7933 ≤ 19,400 psi
Yoke body bolted joint	$P_m \leq S_m$	23,567 ≤ 28,800 psi
Body bonnet interface	$P_m \leq S_m$	16,190 ≤ 19,400 psi
Natural frequency of valve superstructure	$f_n \geq$ design specification limit	220 ≥ 20 Hz

Thickness Requirements (NB-3541)

Section	Criteria	Actual Value Versus Required Minimum
Near welding ends	$t \geq t_m$	0.65 ≥ 0.60 in.

TABLE 3.9-54

CRD HYDRAULIC SYSTEM 3-in. CHECK VALVE

Seismic Analysis-Static Method of Analysis Used

Section	Criteria	Calculated Value Versus Allowable
Cylinder-to-body connection bolting (worse case)	$\sigma_{max} \leq \sigma_{allow}$	7955 ≤ 28,800 psi
	$\tau_{max} \leq 0.6\ \sigma_{allow}$	4150 ≤ 17,280 psi
Cylinder-to-body spacer bracket (worst case)	$\sigma_{max} \leq \sigma_{allow}$ (plate)	550 ≤ 17,500 psi
	$\sigma_{max} \leq \sigma_{allow}$ (around bolt holes)	3630 ≤ 17,500 psi
Cylinder cap bracket extension (worst case)	$\sigma_{max} \leq \sigma_{allow}$	1440 ≤ 17,500 psi
Natural frequency of appurtenances	$f_n \geq$ design specification limit	320 ≥ 20 Hz

Thickness Requirements

Section	Criteria	Actual Value Versus Required Minimum
Near weld ends	$t \geq t_m$	0.9375 ≥ 0.51 in.

TABLE 3.9-55 (SHEET 1 OF 4)

FEEDWATER 18-in. CHECK VALVE

Valve Section	Criteria	Calculated Value Versus Allowable
1. Seismic analysis - static analysis method (NB-3524)		
a. Cylinder-to-valve body connection bolting		
Vertical direction		
Tension in bolts	$S_{max} \le S_{allow}$	12,640 ≤ 25,000 psi
Shear in bolts	$\tau_{max} \le 0.6\ S_{allow}$	1950 ≤ 15,000 psi
Horizontal direction		
Tension in bolts	$S_{max} \le S_{allow}$	8297 ≤ 25,000 psi
Shear in bolts	$\tau_{max} \le 0.6\ S_{allow}$	2615 ≤ 15,000 psi
b. Cylinder-to-body spacer		
X-direction tension in bolts	$S_{max} \le S_{allow}$	1045 ≤ 17,500 psi
Z-direction tension in bolts	$S_{max} \le S_{allow}$	550 ≤ 17,500 psi
c. Cylinder cap bracket		
X-direction tension in bolts	$S_{max} \le S_{allow}$	1780 ≤ 16,200 psi
Z-direction tension in bolts	$S_{max} \le S_{allow}$	6885 ≤ 16,200 psi
d. Natural frequency (cylinder appurtenance)		
Vertical direction	$20 \le f_n$ Hz	20 ≤ 699 Hz
Horizontal X-direction	$20 \le f_n$ Hz	20 ≤ 781 Hz
Horizontal Z-direction	$20 \le f_n$ Hz	20 ≤ 1398 Hz

TABLE 3.9-55 (SHEET 2 OF 4)

Valve Section	Criteria	Calculated Value Versus Allowable
2. Structural analysis		
a. Cover stress ($P_m + P_b$) calculations	$S_{max} \leq 1.5\ S_m$ at 500°F	20,575 ≤ 30,750 psi
b. Body stress calculations		
Bearing stress	$S_b \leq S_y$ at 500°F	9463 ≤ 28,300 psi
Shearing stress	$S_s \leq 0.6\ S_m$ at 500°F	5116 ≤ 11,340 psi
Bending stress total	$S_T \leq 1.5\ S_m$ at 500°F	24,810 ≤ 28,350 psi
Stress range from pressure effect	$S_{alt} \leq S_A$ at N_a = 2000 cycles	62,025 ≤ 65,000 psi
c. Neck to closure flange area of body		
Maximum stress intensity	$S_n \leq 3\ S_m$ at 500°F	50,418 ≤ 56,700 psi
Maximum stress range for fatigue	$S_{alt} \leq S_A$ at N_a= 2000 cycles	31,362 ≤ 65,000 psi
d. Load key		
Vertical shear stress	$S_s \leq 0.6\ S_m$ at 500°F	7840 ≤ 13,980 psi
Average bearing stress	$S_b \leq 1.5\ S_m$ at 500°F	9814 ≤ 34,950 psi
e. Stuffing box flange		
Stuffing box bolting areas	$A_{m_1} \leq A_{actual}$ $A_{m_2} \leq A_{actual}$	2.07 ≤ 3.31 in.^{2} 0.28 ≤ 3.31 in.^{2}
f. Disc thickness calculation	$S_r \leq S_m$ at 500°F	16,770 ≤ 18,900 psi

TABLE 3.9-55 (SHEET 3 OF 4)

Valve Section	Criteria	Calculated Value Versus Allowable
3. Design of pressure-retaining parts and cyclic requirements		
NB-3542.1 Body wall and neck thickness	$t_m \le t_e$	1.63 ≤ 1.71 in.
NB-3544.1(a) Radius of crotch	$0.3\ t_m \le r_2$	0.49 ≤ 1.5 in.
Figure NB-3544.1(c) Radius of crotch	$\left.\begin{matrix} 0.05t_m \\ \text{or } 0.1h \end{matrix}\right\} \le r_3$	0.138 ≤ 0.156 in.
NB-3544.6 Doubly curved section	$\frac{1}{r_{long}} + \frac{1}{r_{lat}} \ge \frac{4}{3d_m}$	0.158 in. ≥ 0.091
NB-3545.1 Primary crotch membrane stress due to internal pressure	$P_m \le S_m$ at 500°F $P_m \le S_m$ at 500°F	Inlet: 16,755 ≤ 18,900 psi Outlet: 14,675 ≤ 18,900 psi
NB-3545.2(b) Valve body secondary stresses due to pipe		
Direct or axial load effect	$P_{ed} \le 1.5\ S_m$ at 500°F	5055 ≤ 28,350 psi
Bending load effect	$P_{eb} \le 1.5\ S_m$ at 500°F	10,778 ≤ 28,350 psi
Torsional load effect	$P_{et} \le 1.5\ S_m$ at 500°F	10,210 ≤ 28,350 psi
NB-3545.2 Valve body primary plus secondary at stress crotch	$S_n \le 3\ S_m$ at 500°F	Inlet: 27,716 ≤ 56,700 psi Outlet: 27,814 ≤ 56,700 psi
NB-3545.3 Normal-duty fatigue requirements	$2000 \le N_a$ Hz	2000 ≤ 80,000 Hz

TABLE 3.9-55 (SHEET 4 OF 4)

Section III NB - Paragraph	Criteria	Calculated Versus Allowable
NB-3554(b) Cyclic stress	$S_{n\ max} \leq 3\ S_m$	109,850 $\nleq$ 56,700 psi
NB-3554(c) K_c calculation	If $S_{n\ max} > 3\ S_m$	Allowed per NB-3554(c)
NB-3553 Fatigue usage	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	0.9102 ≤ 1.0

TABLE 3.9-56 (SHEET 1 OF 3)

FEEDWATER 18-in. CHECK VALVE

Valve Section	Criteria	Calculated Value Versus Allowable
1. Structural analysis		
a. Disc design calculations due to pressure		
Disc thickness - tensile	$S_t \leq S_m$ at 500°F	13,426 ≤ 20,800 psi
Disc thickness - shear	$S_s \leq 0.6\ S_m$ at 500°F	11,157 ≤ 12,480 psi
b. Cover design calculations	$S_{max} \leq S_m$ at 500°F	7281 ≤ 20,800 psi
c. Retainer, gasket design calculations	$S_s \leq 0.6\ S_m$ at 500°F	8133 ≤ 25,980 psi
d. Retainer, hinge pin design calculations	$S_t \leq S_m$ at 500°F	17,103 ≤ 20,800 psi
e. Body-to-cover joint design calculations		
Maximum axial stress	Max $S_H \leq 1.5\ S_m$ at 500°F	13,859 ≤ 29,100 psi
Maximum tangential stress	Max $S_T \leq 1.5\ S_m$ at 500°F	11,429 ≤ 29,100 psi
Maximum radial stress	Max. $S_R \leq 1.5\ S_m$ at 500°F	-2425 ≤ 29,100 psi
Stress intensity (S)	$S \leq 1.5\ S_m$ at 500°F	10,558 ≤ 29,100 psi
f. Hinge pin retainer bolting	$S \leq 2\ S_m$ at 500°F	45,000 ≤ 57,600 psi
2. Seismic analysis - static analysis method used		
a. Valve ends	$S_{max} \leq 0.5\ S_y$ at 500°F	6698 ≤ 14,550 psi
b. Valve middle	$S_{max} \leq 0.5\ S_y$ at 500°F	8115 ≤ 14,550 psi

TABLE 3.9-56 (SHEET 2 OF 3)

Section III NB - Paragraph	Criteria	Calculated Value Versus Allowable
3. Design of pressure-retaining parts and cyclic requirements		
NB-3542 Minimum wall thickness of standard pressure-rated valves	Based on d_m = 13.25 in.	t_m = 1.62 in.
NB-3544.1(a) Radius of crotch	$0.3\ t_m \le r_2$	$0.486 \le 2.12$ in.
NB-3544.5 Out of roundness		
$\frac{b}{t_b} + \frac{3}{4}\left(\frac{3b^2 - 2ab - a^2}{t_b^2}\right) + 1 \le 1.5\frac{S_m}{P_s}$	Compensated by reinforcement	$20.39 \not\le 13.36$
NB-3544.6 Doubly curved section	$\frac{1}{r_{long}} + \frac{1}{r_{lat}} \ge \frac{4}{3d_m}$	$0.101 \ge 0.100$ in.
NB-3544.8 Body contours at welding ends	$1 \times t_m \le t_{inlet}$	$1.62 \le 2.98$ in.
	$1 \times t_m \le t_{outlet}$	$1.62 \le 2.80$ in.
NB-3545.1 Primary membrane stress due to internal pressure	$P_m \le S_m$ at 500°F	Inlet: 11,297 ≤ 21,600 psi Outlet: 13,176 ≤ 21,600 psi
NB-3545.2(b) Secondary stress due to pipe reaction		
Direct or axial load effect	$P_{ed} \le 1.5\ S_m$ at 500°F	4455 ≤ 32,400 psi
Bending load effect	$P_{eb} \le 1.5\ S_m$ at 500°F	7254 ≤ 32,400 psi
Torsional load effect	$P_{et} \le 1.5\ S_m$ at 500°F	7260 ≤ 32,400 psi
NB-3545.2 Valve body primary plus secondary stress at crotch	$S_n \le 3\ S_m$ at 500°F	$\left.\begin{array}{c}\text{Inlet}\\ \text{Outlet}\end{array}\right\\} 26{,}102 \le 64{,}800$
NB-3545.3 Normal-duty fatigue requirements	$2000 \le N_a$ Hz	2000 ≤ 90,000 Hz

TABLE 3.9-56 (SHEET 3 OF 3)

Section III NB - Paragraph	Criteria	Calculated Value Versus Allowable
NB-3554(b) Cyclic stress	$S_{n_{(max)}} \leq 3S_m$	(a)
NB-3554(c) K_c calculation	If $S_{n_{(max)}} > 3\, S_m$	(a)
NB-3553 Fatigue usage	$\sum \frac{N_{ri}}{N_i} \leq 1.0$	(a)

a. Data to be supplied when final calculation is submitted.

TABLE 3.9-57

DRYWELL PNEUMATIC SYSTEM FILTER ASSEMBLY

Seismic Analysis - Static Analysis Method Used

Section	Condition	Criteria	Calculated Value Versus Allowable
Upper section of base legs	Normal + OBE	$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1.0$	0.0528 < 1.0
	Normal + DBE	$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1.0$	0.0572 < 1.0
Lower section of base legs	Normal + OBE	$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1.0$	0.0386 < 1.0
	Normal + DBE	$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1.0$	0.0418 < 1.0
Shell head attachment	Normal + OBE	$P_m + P_L + P_b < 1.5\, S_m$	1539 < 22,500 psi
		$P_m + P_L + P_b + Q < 3.0\, S_m$	5293 < 45,000 psi
	Normal + DBE	$P_m + P_L + P_b < 1.5\, S_m$	1591 < 22,500 psi
		$P_m + P_L + P_b + Q < 3.0\, S_m$	5343 < 45,000 psi
Mounting bolts	Normal + DBE	$\frac{f_t}{F_t} < 1.0$	0.042 < 1.0

TABLE 3.9-58 (SHEET 1 OF 3)

FLUED HEAD XB-12

Loading Condition	Load Combination	Temperature (°F)	Code Stress Category	Allowable Stress[a] Basis	Allowable Stress[a] Value (ksi)	Maximum Stress Intensity Value (ksi)	Maximum Stress Intensity Element	Maximum Stress Intensity Theta
Design	$P + P_c + Wt + OBE$	562	P_m	S	17.5	12.1[c]	354	0°
			P_L	1.5 S	26.3			
			$P_L + P_b$					
Normal and upset	P + therm + 2(OBE) + 2(AM) + temperature (328 and 100)	562	$P_L + P_b + P_c + Q$	3 S_m	55.2	27.6	82	180°
				y' (S_y)	147.0[b]			
	P	----	P_m	----	----	9.7	1-3	----
Emergency	P + Pc + Wt + therm + DBE + EQ	358	P_m	S_y	31.3	13.1[c]	356	0°
			P_L	1.5 S_y	46.9			
			$P_L + P_b$					

TABLE 3.9-58 (SHEET 2 OF 3)

Loading Condition	Load Combination	Temperature (°F)	Allowable Stress[a] Basis	Allowable Stress[a] Value (ksi)	$(\sigma_1 + \sigma_2 + \sigma_3)$ Maximum Value (ksi)	$(\sigma_1 + \sigma_2 + \sigma_3)$ Maximum Element	$(\sigma_1 + \sigma_2 + \sigma_3)$ Maximum Theta
Design	$P + P_c$ + Wt + OBE	562	4.0 S_m	73.6	12.6	347	180°
Emergency	$P + P_c$ + Wt + Therm + DBE + EQ	358	4.6S_m	96.1	18.8	347	180°

TABLE 3.9-58 (SHEET 3 OF 3)

Faulted Condition		Maximum Stress Intensities[d] in Critical Areas		
Load Location	Load Type	Value (ksi)	Element	Theta
Outside containment	Axial force	< 8.5	At any point	-
	Transverse force	< 8.5	At any point	Any position
	Torsional moment	25.2	88	-
	Bending moment	30.9	200	90°
Inside containment	Axial force	< 8.5	At any point	-
	Transverse force	< 8.5	At any point	Any position
	Torsional moment	19.6	288	-
	Bending moment	30.7	155	0°

a. Flued-head material is ASME SA-105, Gr II.

b. $X = \frac{\text{maximum pressure stress}}{S_y} = \frac{9700}{31{,}600} = 0.307$, $\therefore y'(S_y) \sim 4.65\ (31{,}600) = 147$ ksi (Note: S_y value is for 328°F.)

c. Averaging is not required to satisfy P_m limits.

d. Values shown must not exceed 2.4 S_m in critical areas indicated in HNP-2 stress calculations. Flued-head material is ASME SA-105, Gr II. 2.4 S_m = 45.4 ksi at coincident temperature of 530°F.

TABLE 3.9-59 (SHEET 1 OF 3)

FLUED HEADS XB-16A AND B

Loading Condition	Load Combination	Temperature (°F)	Code Stress Category	Allowable Stress[a] Basis	Allowable Stress[a] Value (ksi)	Maximum Stress Intensity Value (ksi)	Maximum Stress Intensity Element	Maximum Stress Intensity Theta
Design	$P + P_c + Wt + OBE$	560	P_m	S	17.5	14.6[c]	1	180°
			P_L	1.5 S	26.3			
			$P_L + P_b$					
Normal and upset	P + therm + 2(OBE) + 2(AM) + temperature (125 and 50)	560	$P_L + P_b + P_c + Q$	3 S_m	55.3	34.1	410	0°
			----	y′ (S_y)	125.0[b]			
	P	-----	P_m	----	----	12.0	1	----
Emergency	$P + P_c + Wt + OBE$	195	P_m	S_y	33.0	6.8[c]	3	180°
			P_L	1.5 S_y	49.5			
			$P_L + P_b$					

TABLE 3.9-59 (SHEET 2 OF 3)

Loading Condition	Load Combination	Temperature (°F)	Allowable Stress[a] Basis	Allowable Stress[a] Value (ksi)	($\sigma_1 + \sigma_2 + \sigma_3$) Maximum Value (ksi)	Element	Theta
Design	$P + P_c + Wt + OBE$	560	$4S_m$	73.8	14.3	12	0°
Emergency	$P + P_c + Wt + OBE$	195	$4.6S_m$	101.2	7.1	12	0°

TABLE 3.9-59 (SHEET 3 OF 3)

Faulted Condition		Maximum Stress Intensities[d] in Critical Areas		
Load Location	Load Type	Value (ksi)	Element	Theta
Outside containment	Axial force	< 5.0	At any point	-
	Transverse force	< 8.0	At any point	Any position
	Torsional moment	21.9	138	-
	Bending moment	27.4	220	90°
Inside containment	Axial force	< 5.0	At any point	-
	Transverse force	< 9.0	At any point	Any position
	Torsional moment	20.4	306	-
	Bending moment	26.6	236	90°

a. Flued-head material is ASME SA-105, Gr II.
b. $X = \frac{\text{maximum pressure stress}}{S_y} = \frac{12,000}{35,200} = 0.341$, $\therefore y' (S_y) \sim 3.55 (35,200) = 125$ ksi - - (Note: S_y value is for 125°F.)
c. Averaging is not required to satisfy P_m limits.
d. Values shown must not exceed 0.7 S_y or 2.4 S_m, whichever is less, in critical areas indicated in HNP-2 stress calculations. Flued-head material is ASME SA-105, Gr II. 2.4 S_m = 44.8 ksi at coincident temperature of 546°F.

TABLE 3.9-60 (SHEET 1 OF 3)

FLUED HEADS XB-36

Loading Condition	Load Combination	Temperature (°F)	Code Stress Category	Allowable Stress[a] Basis	Allowable Stress[a] Value (ksi)	Maximum Stress Intensity Value (ksi)	Maximum Stress Intensity Element	Maximum Stress Intensity Theta
			P_m	S	15.3			
Design	$P + P_c + Wt + OBE$	340	P_L	1.5 S	23.0	9.2[b]	1	180°
			$P_L + P_b$					
Normal and upset	P + therm + 2(OBE) + 2(AM) + temperature	150	$P_L + P_b + P_c + Q$	$3\ S_m$	60.0	21.8	3	180°
			P_m	S_y	27.5			
Emergency	$P + P_c$ + Wt + therm + DBE + EQ	150	P_L			21.2	3	180°
			$P_L + P_b$	$1.5\ S_y$	41.2			

TABLE 3.9-60 (SHEET 2 OF 3)

Loading Condition	Load Combination	Temperature (°F)	Allowable Stress[a] Basis	Allowable Stress[a] Value (ksi)	($\sigma_1 + \sigma_2 + \sigma_3$) Maximum Value (ksi)	($\sigma_1 + \sigma_2 + \sigma_3$) Maximum Element	($\sigma_1 + \sigma_2 + \sigma_3$) Maximum Theta
Design	$P + P_c + Wt + OBE$	340	$4\ S_m$	78	9.8	351	180°
Emergency	$P + P_c + Wt + therm + DBE + EQ$	150	$4.6\ S_m$	92	26.9	12	0°

TABLE 3.9-60 (SHEET 3 OF 3)

Faulted Condition		Maximum Stress Intensities[d] in Critical Areas		
Load Location	Load Type	Value (ksi)	Element	Theta
Outside containment	Axial force	< 5.0	At any point	
	Transverse force	< 5.0	At any point	Any position
	Torsional moment	12.6	96	
	Bending moment	14.1	176	90°
Inside containment	Axial force	< 5.0	At any point	
	Transverse force	< 5.0	At any point	Any position
	Torsional moment	12.5	292	
	Bending moment	13.8	192	90°

a. Flued-head material is ASME SA-182, F-304.
b. Averaging is not required to satisfy P_m limits.
c. Values shown must not exceed 2.4 S_m in critical areas indicated in HNP-2 stress calculations. Flued-head material is ASME SA-182, F-304. 2.4 S_m = 48.0 ksi at coincident temperature of 80°F.

TABLE 3.9-61 (SHEET 1 OF 2)

LIQUID NITROGEN STORAGE TANK AND RELATED PIPING

Component	Method of Analysis	Type of Stress	Maximum Stress (psi)	Allowable Stress (psi)
Control piping (size)	ASME Section III	Primary (weight, pressure, DBE)		
1/2 in.			2733	18,800
3/4 in.			5191	18,800
1 in.			2817	18,800
1 1/2 in.			2220	18,800
1/2 in.		Secondary (thermal)	17,627	28,200
3/4 in.			23,090	28,200
1 in.			6678	28,200
1 1/2 in.		Primary plus secondary (weight, pressure, DBE, thermal)	31,239	46,000
Ambient vaporizer piping (size)	ASME Section III	Primary (weight, pressure, DBE)		
1/2-in. tube			10,600	18,800
1/2-in. pipe			400	18,800
1 1/2-in. pipe			16,940	18,800
fin			62	18,800
		Secondary (thermal)	15,128	28,200
			5509	28,200
			Negligible	28,200
			91	28,200
Inner vessel (shell)	ASME Section VIII	Primary (pressure, weight, OBE)	7112	18,800
		Primary (pressure, weight, DBE)	5348	27,000

TABLE 3.9-61 (SHEET 2 OF 2)

Component	Method of Analysis	Type of Stress	Maximum Stress (psi)	Allowable Stress (psi)
Inner vessel flange	ASME Section VIII	Primary (pressure, weight, OBE)	13,546	18,800
		Primary (pressure, weight, DBE)	16,366	27,000
Annular space piping	ASME Section III	Primary (pressure, weight, DBE)		
lines A			2360	18,800
B			2520	18,800
C			3000	18,800
D			4980	18,800
E			1796	18,800
F			2495	18,800
lines A	Cantilever method	Secondary (thermal)	Actual	< 28,200
B			Actual	< 28,200
C			Actual	< 28,200
D			Actual	< 28,200
E			Actual	< 28,200
F			Actual	< 28,200
Jacketed pressure vessel rings	Zick method	Primary (pressure, weight, OBE)	17,569	19,800
		Primary (pressure, weight, DBE)	28,453	29,700
Saddle support	"Process Equipment Design," Brownell and Young	Bending (weight + DBE)	8600	32,400
		Circumferential (weight + DBE)	3700	21,520

TABLE 3.9-62

REACTOR BUILDING SAFEGUARD SYSTEM COOLING UNITS

Component	Method of Analysis	Type of Stress	Maximum Stress	Allowable Stress (AISC)
Fan (Unit 2T41-B001)	Modal spectrum	Deflection combined (weight, pressure, DBE)	0.0035 in.	0.025 in
Plenum/cooler walls			3340 psi	21,600 psi
Plenum/cooler top			4830 psi	21,600 psi
Fan guide vanes			760 psi	21,000 psi
Motor-mounting disc			1310 psi	21,000 psi
Motor shaft			2260 psi	32,000 psi
Fan shaft			5850 psi	32,000 psi
Intermediate tube sheet			410 psi	18,000 psi
Foundation bolts			950 psi	21,000 psi
Cooling coil tubes			790 psi	13,200 psi
Fan (Unit 2T41-B002)	Modal spectrum	Deflection	0.0021 in.	0.050 in
Motor shaft			1630 psi	32,000 psi
Fan shaft			2460 psi	32,000 psi
Guide vanes			150 psi	21,000 psi
Motor-mounting disc			1230 psi	21,000 psi
Fan brackets			10 psi	21,000 psi
Mounting bolts			1500 psi	21,000 psi
Fan (Unit 2T41-B005)	Modal spectrum	Deflection combined (weight, pressure, DBE)	0.0040 in.	0.025 in
Plenum/cooler walls			2200 psi	21,600 psi
Plenum/cooler top			4940 psi	21,600 psi
Fan guide vanes			770 psi	21,000 psi
Motor-mounting disc			1120 psi	21,000 psi
Motor shaft			1650 psi	32,000 psi
Fan shaft			2470 psi	32,000 psi
Tube support			915 psi	18,550 psi
Foundation bolts			920 psi	21,000 psi
Cooling coil tubes			3103 psi	18,550 psi

TABLE 3.9-63

SGTS BLOWER

Component	Method of Analysis	Type of Stress	Maximum Stress	Allowable Stress (AISC)
Fan shaft	Static	Shear (weight + DBE)	2350	12,400
		Normal (weight + DBE)	4700	20,700
Fan shaft bearings	Local rating	NA[a]	(2.91 safety factor)	
Bearing support angles	Standard structural	Bending	6596	32,400

a. NA - not applicable.

TABLE 3.9-64

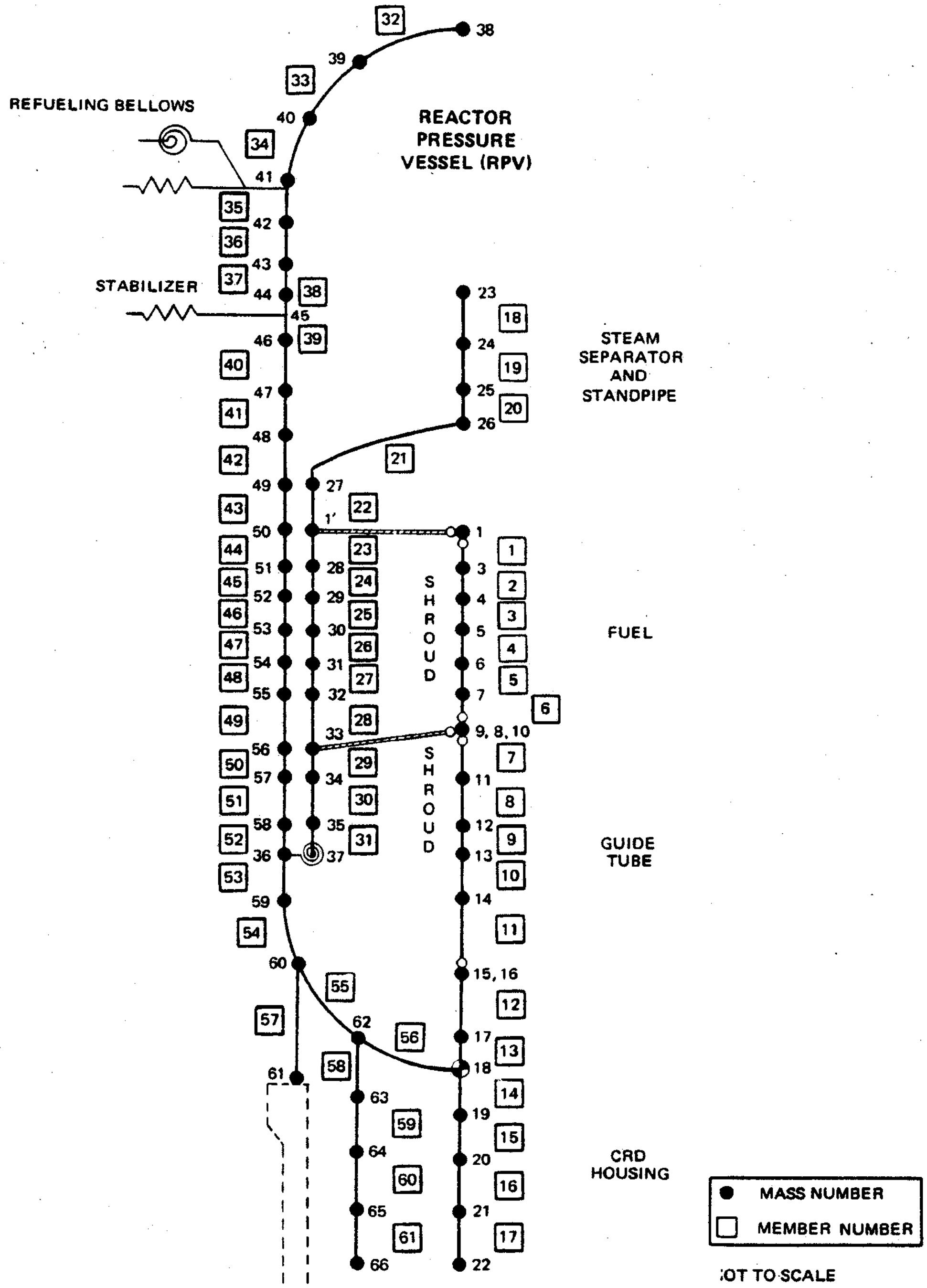
SERVICE WATER TO DIESEL EXPANSION JOINTS

Component	Method of Analysis	Type of Stress	Maximum Stress	Allowable Stress (AISC)
Bellows	ASME, Section III	Primary (DBE + Pressure)	22,500	27,000

TABLE 3.9-65

MAIN STEAM ISOLATION VALVE MODIFICATIONS

Valve No.	Oversized Rockwell/Edwards Disk Installed	Weir Modification Kit Installed
2B21-F022A	☑	☐
2B21-F022B	☑	☐
2B21-F022C	☐	☑
2B21-F022D	☑	☐
2B21-F028A	☐	☑
2B21-F028B	☑	☐
2B21-F028C	☑	☐
2B21-F028D	☑	☐



REV 19 7/01

SOUTHERN COMPANY
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SOUTHERN NUCLEAR OPERATING COMPANY
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

DYNAMIC MODEL OF RPV AND INTERNALS

FIGURE 3.9-1

3.10 SEISMIC DESIGN OF SEISMIC CATEGORY I INSTRUMENTATION AND ELECTRICAL EQUIPMENT

3.10.1 SEISMIC DESIGN CRITERIA

Subsection 3.2.1 and tables 3.10-1 and 3.10-21 provide a listing of Seismic Category I mechanical, instrumentation, and electrical equipment requiring seismic qualifications.

3.10.1.1 General

The Seismic Category I mechanical, instrumentation, and electrical equipment is designed to withstand the effects of the design basis earthquake (DBE) and to remain functional during normal and accident conditions.

The parameters used to develop seismic loadings and criteria for nonnuclear steam supply system and nuclear steam supply system (NSSS) Category I structures, systems, and components are described in supplements 3.7A and 3.7B, respectively. The performance requirements of Seismic Category I items and their respective supports are structural as well as functional. The actual service mounting of the equipment was considered in establishing seismic functional capability.

The seismic criterion used in the design and subsequent qualification of all Class 1E instrumentation and electrical equipment supplied by General Electric (GE) is applicable when required for any safety design basis and is as follows:

> The Class 1E equipment is capable of performing all safety-related functions during normal plant operation, anticipated transients, design basis accidents (DBAs), and post-accident operation, while being subjected to and after the cessation of the accelerations resulting from the DBE at the point of attachment of the equipment to the building or supporting structure.

The specific criteria for each of the many Class 1E systems are covered in chapter 7. The criteria for each of the devices used in the many Class 1E systems depend on the use in a given system; e.g., a relay in one system may have as its safety function to deenergize and open its contacts within a certain time, while in another system it must energize and close its contacts. Since GE supplies many devices for numerous applications, the approach taken was to test the device in all modes that might be used. In this way, the capability of protective action initiation and the proper operation of safety-feature circuits are assured.

3.10.1.2 Emergency Power System and Engineered Safety Features Actuation System

In addition to the general criteria described in paragraph 3.10.1.1, the standby power system and Seismic Category I instrumentation and electrical equipment associated with engineered safety features (ESFs) are designed to withstand seismic disturbances having the intensity of the DBE during post-accident operation.

3.10.1.3 Compliance With Institute of Electrical and Electronic Engineers (IEEE) Standard 344-1971

Qualification and documentation procedures used for Seismic Category I equipment and/or systems other than the NSSS meet the provisions of either IEEE Standard 344-1971, as amended by supplements 3.7A and 3.7A.A, or IEEE 344-1975.

GE-supplied Class 1E equipment meets the requirement that the seismic qualification should demonstrate the capability to perform the required function during and after the DBE. Both analysis and testing were used, but most equipment was tested. Analysis was used to determine the adequacy of mechanical strength (mounting bolts, etc.) after operating capability was established by testing.

GE-supplied Class 1E equipment with primarily mechanical safety functions (pressure boundary devices, etc.) was analyzed since the passing nature of its critical safety role usually made testing impractical. Analytical methods sanctioned by IEEE-344-1971 were used in such cases. (See table 3.10-1 for indication of the items that were qualified by analysis.)

GE-supplied Class 1E equipment having primarily active electrical safety function was tested in compliance with IEEE-344-1971, Section 3.2.

The documentation verifying the seismic qualification of GE-supplied Class 1E equipment is in accordance with the requirements of IEEE-344-1971, Section 4.

3.10.2 SEISMIC ANALYSIS, TESTING PROCEDURES, AND RESTRAINT MEASURES

3.10.2.1 Equipment Other Than NSSS

Seismic Category I mechanical, instrumentation, and electrical equipment and components other than the NSSS are designed to ensure functional integrity for the specific operation requirements categorizing them as Seismic Category I. An investigation of the equipment is required to demonstrate its ability to withstand seismic forces without loss of function.

Non-NSSS Seismic Category I equipment installed at HNP-2 was seismically qualified in accordance with either IEEE 344-1971, as amended by supplements 3.7A and 3.7A.A, or IEEE 344-1975. In addition, the qualification of some non-NSSS equipment was established using the criteria developed by the Seismic Qualification Utility Group (SQUG) and documented in the Generic Implementation Procedure (GIP), Revision 2.

There are no control systems components with natural frequencies below 5 Hz.

Various phases and/or processes are associated with the specification, procurement, and acceptance of Seismic Category I mechanical, instrumentation, and electrical equipment supplied for HNP-2. Beginning with the recognized need that the equipment or component must provide, an inquiry specification certified by a registered professional engineer is developed and issued. This specification describes the equipment or component required in terms of design

code applicability, seismic requirements, and other pertinent parameters to be applied during fabrication and design.

The inquiry specification issued, dealing with the seismic requirements for the equipment or component, details the essentials associated with the HNP-2 DBE and operating basis earthquake (OBE), as applicable. The following excerpt from the inquiry specification for the plant service water (PSW) pumps is a typical example of such specification requirements:

EXAMPLE

5.2 SEISMIC REQUIREMENTS

5.2.1 The RHR service water and plant service water pumps are designated as seismic Class 1 equipment and shall be designed in accordance with the following criteria.

5.2.2 OPERATING BASIS EARTHQUAKE (OBE) - The pumps and motors shall be designed to withstand, without exceeding normal allowable working stresses and without loss of function, the combination of normal loads plus the forces resulting from the "operating basis earthquake" (OBE), caused by a maximum horizontal ground acceleration of .08 g and maximum vertical ground acceleration of .054 g. The final OBE seismic response spectra curves are enclosed.

5.2.2.1 It is believed that a multi-degree-of-freedom system will be required to adequately model the equipment. A modal analysis using the response spectra to be furnished and the appropriate damping factor from Table 1 [not supplied] should be performed.

5.2.2.2 A single-degree-of-freedom system may be acceptable if it can be justified by the pump manufacturer. In this case, determine the natural frequency of the equipment and from each curve of response spectra select the acceleration corresponding to the natural frequency. If it is not practical to determine the natural frequency of the equipment, use the maximum acceleration of the response spectra curves.

5.2.2.3 The forces resulting from the vertical acceleration of the equipment shall be assumed to act simultaneously with the forces resulting from the horizontal accelerations.

5.2.2.4 The stresses resulting from the horizontal and vertical accelerations shall be combined directly, linearly, and in the most unfavorable direction with the stresses resulting from other loading conditions.

EXAMPLE (continued)

5.2.3 DESIGN BASIS EARTHQUAKE (DBE) - The pumps and motors shall be designed to withstand, without exceeding 90% of yield stresses, and without loss of function, the normal loads plus the forces resulting from the "design basis earthquake" (DBE) caused by a maximum horizontal ground acceleration of .15 g and a maximum vertical ground acceleration of .10 g. The procedure used for the analysis shall be as given in Paragraph 5.2; however, response spectra and percent damping for the DBE shall be used. The final DBE seismic response spectra curves are enclosed.

5.2.4 Certification - The pump and motor manufacturer must furnish certification that his equipment is designed in accordance with the above seismic requirements. Certification may consist of either:

(a) Calculations checked by an engineer knowledgeable in the design of such equipment, or

(b) Written certification that equipment has successfully passed the tests of equal or higher forces and more severe vibration exposure than stated in the above seismic requirement, with a description and the results of such tests clearly stated.

5.2.5 Complete calculations and/or test data for certification of the equipment furnished shall be submitted by the manufacturer. All calculations submitted should include the description and justification of the method used and the results of the calculations or test.

5.2.6 Response Spectra Data Furnished [not supplied]

Figure 1 - Horizontal Response Spectra el. 111′-0",
Figure 2 - Service Water Intake Structure

Figure 3 - Horizontal Response Spectra, el. 88′-9",
Figure 4 - River Intake Structure

Figure 5 - Vertical Response Spectra, el. 111′ -0",
Service Water Intake Structure
Figure 6 - Vertical Response Spectra, el. 88′-9",
River Intake Structure

Table 1 - Percentage of Critical Damping

EXAMPLE (continued)

5.2.7 Reference

AEC publication TID 7024 "Nuclear Reactor and Earthquakes" is used as the basic design guide for seismic analysis.

END OF EXAMPLE

As stated in paragraph 5.2.4 of the specification excerpt, the vendor must provide certification, either by a calculation or test report, that his equipment is designed in accordance with the seismic requirements specified. Upon receipt of the vendor's certification, an engineer familiar with the methods employed reviews the certification submitted with the objective being to concur with calculational models, assumptions, and conclusions or testing methods and results. This review is based on the vendor's conformance to the requirements specified in the inquiry specification. Formal written notification of concurrence/nonconcurrence is forwarded to the vendor upon completion of the review.

Table 3.10-4 provides a list of the major balance-of-plant (BOP) Category I equipment that the NRC Seismic Qualification Review Team reviewed during their seismic audit of HNP-2. This table provides the reasons for determining whether test or analysis or a combination of the two was chosen as the acceptable means for seismic qualification for each item, as well as provides the seismic qualification method(s) used.

A discussion of the parameters used for functional and structural seismic capability verification and the qualification levels and acceptance criteria for each item in table 3.10-4 is as follows:

600-V Station Service Switchgear

1. Method of qualification - testing[a]

   - Sinusoidal vibration tests.

   - Inclined shock tests.

   - San Fernando Valley earthquake experience.

2. Summary of results

   - Sinusoidal vibration tests.

     - Input frequency Single

a. Three copies of the GE Seismic Qualification Report 72LSP-1 were submitted to the NRC for evaluation.

- Input motion — Sinusoidal sweep and dwell

- Single-axis tests — Three axes independently

a. AKD-5 Switchgear

   - Natural frequency - 9 Hz

   - Input horizontal acceleration - 0.5 g

   - Input vertical acceleration - 0.5 g

   - Test response spectrum (TRS) versus required response spectrum (RRS) (See figure 3.10-1.)

b. AK-50 Breaker

AK-50 Breaker	Horizontal	Vertical
- Natural frequency	29 Hz	30 Hz
- Input maximum acceleration	5 g	2.5 g
- Maximum acceleration at mounting location	2.2 g	2.3 g

c. AK-25 Breaker

AK-25 Breaker	Horizontal	Vertical
- Natural frequency	44 Hz	60 Hz
- Input maximum acceleration	3 g	10 g
- Maximum acceleration at mounting location	2.2 g	2.3 g

- Inclined shock tests.

  a. AKD-5 switchgear - biaxially

     - Shock input - 40 g

  b. AK-50 and AK-25 breakers

     - Shock input - 15 g

- San Fernando Valley earthquake experience.

  The location of installation was Sylmar, California, southern terminal of the West Coast HVDC transmission line.
  The estimated ground acceleration was 0.3 g to 0.5 g by Dr. G. Housner.

3. Functional verifications

   - No structural damage to the switchgear was observed.

   - Breakers normally in the closed position remain closed due to vibration.

4. Justifications of single-axis testing

   - Equipment has minimum cross coupling in axis orthogonal to the axis of applied vibration.

   - The TRS exceeded the RRS by a significant margin.

5. Justification of single-frequency testing

   - The RRS for the equipment shows a predominant peak response at the fundamental frequency of the structure.

   - The lowest frequency of the switchgear was measured to be 9 Hz. The transmissibility value at the breaker position was very low, < 1.3 at that frequency. The lowest frequency of 9 Hz is in the portion of the response spectrum in which there is no significant change in spectral acceleration with increasing frequencies. Therefore, the equipment can be classified as rigid, and higher-mode contributions are insignificant.

   - The TRS exceeded the RRS by a significant margin.

<u>600 V-ac Motor Control Center (MCC)</u>

1. Method of qualification - testing

2. Summary of results

• Input frequency (Hz)	Single
• Input acceleration (g)	See table 3.10-5.
• Input motion	Sinusoidal dwell
• RRS versus TRS	See table 3.10-5.
• Single-axis tests	Three axes independently
• TRS versus RRS	See figures 3.10-2 through 3.10-7.

3. Functional verifications

   Circuits were monitored to verify that normal operating positions were unchanged.

Battery Charger (new)

1. Method of qualification - testing - IEEE 344-1975

2. Summary of results

   - Input motion.

     - Random multifrequency test

     - Random-wave-form motion (30-s duration)

   - Axis of test.

     - Side to side and vertical

     - Front to front and vertical

     - (Simultaneous horizontal and vertical)

   - Input acceleration (See table 3.10-6.).

   - TRS versus RRS (See figure 3.10-8.).

3. Functional verifications

   Three channels of electrical monitoring were recorded on an oscillograph recorder during the test. These channels were used to ascertain any change in the input voltage, output current, and output voltage prior to, during, and after the test. It was demonstrated that the specimen possessed sufficient integrity to withstand, without compromise of electrical function, the prescribed simulated seismic environment.

Large Induction Motors	Calculated Stress	
	OBE	DBE
1. Method of qualification - dynamic analysis		

	Calculated Stress OBE	Calculated Stress DBE
2. Summary of results		
• Rotor-shaft assembly and bearings.		
- Shaft deflection	-	0.273
- Shaft stresses	0.735	-
- Bearing loading		
❖ Guide bearings	-	0.675
❖ Guide bearing minimum oil film thickness	-	-
❖ Guide bearing contact stresses at motor standstill	-	0.174
- Thrust bearing loading	-	0.996
- Bearing loading for hydraulic upthrust conditions	-	1.000
- Journal sleeve locknuts	-	-
• Endshield assemblies.		
- Lower endshield assembly		
❖ Bearing housing stresses	0.140	0.084
❖ Stator frame - lower endshield joint fasteners	0.846	0.149
❖ Stresses at rabbit fit between stator and lower endshield	0.525	0.315
❖ Maximum stress in body of lower endshield	0.140	0.084

	Calculated Stress	
	OBE	DBE
- Upper endshield assembly		
❖ Bearing housing stresses	0.251	0.151
❖ Bearing housing fasteners	-	0.397
❖ Stator frame - upper endshield joint fasteners	0.647	0.114
❖ Discussion regarding maximum stress in upper endshield	0.772	0.463
• Stator.		
- Maximum stress in body of stator frame	-	0.639
- Stator core supports	0.108	0.033
- End turn support system	-	-
• Motor base.		
- Base fasteners	0.658	0.116
- Maximum stress in base	0.170	0.102
• Conduit box and miscellaneous components.		
- Conduit box	0.091	0.016
- Screens	0.080	0.024
- Top cap	0.020	0.003

3. Structural verifications

Stresses at critical locations were verified to be less than the allowable stresses. Structural integrity of the equipment was maintained under the seismic environment.

4. Conservatisms used in the seismic analysis are as follows:

   - The motors were analyzed for horizontal accelerations of 0.48 g (OBE) and 0.85 g (DBE). The seismic analysis for the intake structure where these motors are located indicates that the required horizontal accelerations are 0.15 g (OBE) and 0.39 g (DBE).

   - The allowables for thrust bearing loading and for bearing loading for hydraulic upthrust conditions were based on a constant loading throughout the operation of the pump motors. The maximum loadings of the bearings occurring during the entire duration of the earthquake were compared with allowables established for the constant loading for motor operation.

<u>Power Transformers</u>

1. Method of qualification - analysis

2. Summary of results

   - Natural frequency (Hz)

-	Core and coil	28.1
-	Tank	66
-	Low-voltage bar	43.6

   - Input horizontal acceleration (g) 0.37

   - Input vertical acceleration (g) 0.21

   - Comparison of actual and allowable stresses.

Component	Material GE Spec No.	Actual Stress (psi)	Minimum Yield (psi)	Actual Minimum-Yield Stress
Tank plate ends	Steel B8A3X	21,587	30,000	0.720
Tank plate fronts	Steel B8A3X	25,184	30,000	0.839
Tank braces	Steel B8A3X	17,811	30,000	0.594
Base-plate	Steel B50P520	12,853	38,000	0.338
Top clamp	Steel B50P517	33,091	38,000	0.871
Bottom clamp	Steel B50P520	11,165	38,000	0.294
Clamp stud	Steel B50P519	863	25,000	0.035
X bar bolt	Steel C1L2	3458	34,000	0.102
LV bars	CU B11B5	707	10,000	0.071

3. Structural verifications

   Stresses at critical locations were verified to be less than the allowable stresses. Structural integrity was maintained under the seismic environment.

100-kW Inverters

1. Method of qualification - testing in accordance with IEEE-344-1975

2. Summary of results

   - Input motion.

     - Random multifrequency test

     - Random-wave-form motion (30-s duration)

- Axis of test.
    - Side to side and vertical
    - Front to front and vertical
    - (Simultaneous horizontal and vertical)
- Damping - 3%.
- Input acceleration. (See table 3.10-7.)
- TRS versus RRS. (See figure 3.10-9.)

3. Functional verifications

   Three channels of electrical monitoring were recorded on an oscillograph recorder during the test. These channels were used to monitor the three 575 V-ac, 60-Hz, 3-phase outputs of the specimen. The specimen was tested in the energized no-load condition. No abnormal voltage levels or spurious operations were indicated by the electrical monitoring during the seismic tests. The specimen continued to function at the proper output voltage when the short-circuit fault was applied and removed.

Diesel Engine Generating Unit[a]

12-cylinder 8 1/8 x 10 turbocharged engine

3250 kW at 900 rpm

1. Method of qualification - combined testing and dynamic analysis
2. Summary of results
    - The following components were analyzed for seismic requirements:
        - Engine generator - skid assembly
        - Heat exchanger stack assembly
        - Lube oil filter
        - Fuel oil day tank (1000 gal)
        - Jacket water expansion tank

a. Three copies of the Colt Industries Seismic Calculation Report were submitted to the NRC for evaluation.

  - Air compressor skid
  - Air receiver tank

- The following components have been qualified by tests:
  - Basley relays
  - Clark control relays

Engine Generator Skid Assembly

Horizontal Frequencies (Hz)	Vertical Frequencies (Hz)
f_1 = 9.47	f_1 = 18.18
f_2 = 13.83	f_2 = 32.00
f_3 = 14.78	f_3 = 68.44
f_4 = 30.36	f_4 = 68.67
	f_5 = 72.13

Stress Summary

Component	Calculated S	Allowable S
Foundation holddown bolts - skid	3092	33,000
Holddown bolts - subbase/skid	2434	33,000
Holddown bolts - subbase - middle section	4363	28,000
Turbocharger bracket mounting bolts	25,915	52,000
Turbocharger mounting feet	11,528	33,000

Component	Calculated (lb)	Allowable (lb)
Engine bearings No. 14	3167	6000
Engine bearings No. 13	1156	6000
Engine bearings No. 1	11,430	18,000
Engine bearings No. 2	8775	18,000

Overspeed Governor and Shutdown System

- Linear acceleration of rotating-weight system = 2.24 g[(a)]
- Linear acceleration of latch-and-lever system = 51.90 g(a)
- Linear acceleration of manual shutdown = 6.57 g(a)
- Linear acceleration of plunger and latch unhooking = 128 g
- Applied seismic acceleration = $\frac{2.24}{0.554}$ = 0.554 g
- Applied seismic safety factor = 4.04

Heat Exchanger Stack

Horizontal Mode

Parallel-to-Engine Crankshaft (Hz)	90° to Crankshaft Centerline (Hz)
f_1 = 10.31	f_1 = 11.10
f_2 = 40.65	
f_3 = 137.25	
f_4 = 151.76	

a. Lowest linear acceleration is 2.24 g due to movement of the rotating governor weight.

Vertical Mode

f_1 = 20.2

f_2 = 55.1

f^3 = 75.5

Stress Summary

- Fore and aft stack bracing stress = 11,940 psi; okay for mild steel.
- Shear stress for bolt = 11,726 psi < allowable of 28,000 psi.
- Stress in tubing = 5775 psi, okay for mild steel.
- Weld capacity at foot of post = 14,400 lb > actual load of 7507 lb.
- Maximum combined shear and tensile principal stress = 9120 psi < allowable proof load = 52,000 psi.

Lube Oil Filter

f_1 = 31.10 Hz (horizontal)

f_1 = 148.000 Hz (vertical)

- Proof load on bolt = 17,400 lb, > actual load of 556 lb.
- Calculated bending stress = 3644 psi, < allowable bending stress = 33,000 psi.
- Weld capacity = 20,400 lb, > actual load of 1372 lb.

1000-gal Fuel Tank

f_1 (horizontal) = 13.94 Hz

f_1 (vertical) = 35.80 Hz

1. Case A - full tank
   - Combined tension and shear principal stress = 2537 psi
   - Bending stress = 5387 psi
   - Acceptable for mild steel with f_y = 33,000 psi

2. Case B - half-full tank (effect of sloshing included)

    - Maximum shear = 604 < 2537 psi for full tank.

    - Maximum bending stress = 1307 psi < 5387 psi for full tank.

    - No-bolt tension - Case A governs.

Jacket Water Expansion Tank (100 gal)

f_1 (horizontal) = 53.80 Hz (longitudinal)

f_1 (horizontal) = 43.80 Hz (lateral)

f_1 (vertical) = 45.60 Hz (lateral)

- Maximum bolt shear stress = 1012 psi.

- Maximum bolt tension stress = 1410 psi.

- Combined stress = 1936 psi, okay for mild steel.

- Bending stress = 1006 psi, okay for mild steel.

- Maximum load at weld = 744 lb < capacity of weld = 2385 lb.

- Safety factor = 3.2.

Air Compressor Skid Assembly

f_1 (horizontal) = 14.48 Hz

f_2 (horizontal) = 38.08 Hz

f_1 (vertical) = 75.40 Hz

f_2 (vertical) = 114.21 Hz

- Overturning moment = 1992 in. lb < resisting moment = 2180 in. lb.

- Shear stress in bolt = 194 psi < 33,000 psi.

Air Receiver Tank

f_1 = 6.61 Hz (horizontal)

f_2 = 33.84 Hz (vertical)

- Maximum principal stress = 4992 psi < 52,000 psi (proof load of bolt).
- Bending stress = 17,611 psi < 30,000 psi.
- Maximum stress in weld = 907 lb/in. < 2400 lb/in.

Generator Control Board

1. Structural rigidity of cabinet - qualification by analysis

   - Cabinet.

Horizontal, Hz	Vertical, Hz
f_1 = 8.237	f_1 = 13.70
f_2 = 8.41	f_2 = 73.72
f_3 = 73.42	f_3 = 95.66
f_4 = 95.67	f_4 = 98.47
f_5 = 98.97	f_5 = 333

     - Resisting moment = 27,517.5 in. lb
     - Overturning moment = 49,576 in. lb

   - Holddown bolts.
     - Tensile force in bolt = 1747 psi
     - Shearing force = 1808 psi < 33,000 psi

   - High-voltage chassis holddown bolts.
     - Tensile stress = 986 psi
     - Shear stress = 1484 psi < 33,000 psi

   - Holddown bolts of the current transformer.
     - Tensile stress = 1029 psi
     - Shear stress = 174 psi < 33,000 psi

2. Basler relays - qualification by tests

    - Input frequency — Single
    - Input acceleration — See table 3.10-8.
    - Input motion — Sine dwell
    - Maximum acceleration at mounting locations — 0.94 g (horizontal)

      0.23 g (vertical)
    - Single-axis tests — Three axes independently

3. Clark control relays - qualification by tests

    - Input frequency — Single
    - Input acceleration — 3.6 g from 5 Hz to 20 Hz
    - Maximum acceleration at mounting locations — 0.94 g (horizontal)

      0.23 g (vertical)
    - Single-axis tests — Three axes independently

4. Functional and structural verifications

    Stresses at critical locations of components essential for continuous operation were verified to be less than the allowable stresses. Structural integrity was maintained under the seismic environment.

    Relays were monitored in the energized and deenergized armature positions, and no control chatter was noted.

<u>Main Control Room Environmental Control (MCREC) System</u>

1. Method of qualification - dynamic analysis

2. Summary of results

    - Rectangular

Damper size (width x height)	36 in. by 24 in.	26 in. by 28 in.
Natural frequency (Hz)	79	153
Percent damping	5	5
Maximum horizontal acceleration (g)	0.40	0.40
Maximum vertical acceleration (g)	0.20	0.20
Maximum stress in blade (psi)	842	438
Allowable stress in blade (psi)	20,000	20,000
Maximum force on bearing (lb)	102	86
Allowable force on bearing (lb)	1990	1990
Maximum shear on anchor bolt (lb)	13	13
Allowable shear on anchor bolt (lb)	940	940
Maximum tension on anchor bolt (lb)	8	8
Allowable tension on anchor bolt (lb)	1260	1260

Damper size (width x height)	60 in. by 20 in.	20 in. by 16 in.	60 in. by 24 in.	36 in. by 24 in.
Natural frequency (Hz)	45	329	44	108
Percent damping	5	5	5	5
Maximum horizontal acceleration (g)	0.40	0.60	0.40	0.40
Maximum vertical acceleration (g)	0.20	0.20	0.20	0.20
Maximum stress in blade (psi)	1100	232	792	563
Allowable stress in blade (psi)	20,000	20,000	20,000	20,000
Maximum force on bearing (lb)	168	76	219	115
Allowable force on bearing (lb)	4980	4980	4980	4980
	60 in.	20 in.	60 in.	36 in.

Damper size (width x height)	by 20 in.	by 16 in.	by 24 in.	by 24 in.
Maximum shear on anchor bolt (lb)	23	21	12	25
Allowable shear on anchor bolt (lb)	940	940	940	940
Maximum tension on anchor bolt (lb)	10	25	19	11
Allowable tension on anchor bolt (lb)	1260	1260	1260	1260

- Round

Damper size - diameter (in.)	36	18	28
Natural frequency (Hz)	36	119	64
Percent damping	5	5	5
Maximum horizontal acceleration (g)	0.60	0.60	0.60
Maximum vertical acceleration (g)	0.20	0.20	0.20
Maximum stress in blade (psi)	738	232	387
Allowable stress in blade (psi)	20,000	20,000	20,000
Maximum force on bearing (lb)	175	59	117
Allowable force on bearing (lb)	4980	3150	4980
Maximum shear on anchor bolt (lb)	23	29	23
Allowable shear on anchor bolt (lb)	510	510	510
Maximum tension on anchor bolt (lb)	13	13	11
Allowable tension on anchor bolt (lb)	680	680	680

3. Structural verifications

   Stresses at critical locations were verified to be less than the allowable stresses. Structural integrity was maintained under the seismic environment.

Fisher Air Operator Nuclear Control Valves

1. Method of qualification - combined analysis and testing

2. Summary of results

- Valve and actuator assemblies by dynamic analysis

	Calculated	Allowable
Natural frequency (Hz)	70	-
Maximum horizontal acceleration (g)	3	-
Maximum vertical acceleration (g)	3	-
Maximum yoke leg stress (psi)	3820	23,300
Maximum bonnet stress (psi)	2060	17,340
Maximum yoke boss stress (psi)	1150	17,340

- Instruments by testing

Input frequency	Single
Input acceleration (g)	4 g at frequencies of 10, 17, 25, and 33 and at resonant frequencies
Input motion	Sine dwell for 1 min
Single-axis tests	Three axes independently

3. Functional and structural verifications

Stresses at critical locations were verified to be less than the allowable stresses. Structural integrity was maintained under the seismic environment. Solenoid valve, snap-lock electric switch, I/P transducer, and air set were at their normal operating conditions during tests. The outputs of the instruments were monitored. No malfunctions were indicated.

<u>WKM Air-Operated Nuclear Control Valves</u>

1. Method of qualification - dynamic analysis and testing

2. Summary of results

- Analysis

-	Valve size (in.)	1	3
-	Natural frequency (Hz)	61	36.6

- Instruments by testing
  - Type of input motion — Sinusoidal
  - Input frequency — Single
  - Single-axis tests — Three axes independently

3. Functional verifications

   Solenoid valve, snap-lock electric switch, I/P transducer, and air set were tested at their normal operating conditions. The outputs of the instruments were monitored. No malfunctions were indicated.

<u>Excess-Flow Check Valves</u>

1. Method of qualification - testing

2. Summary of results

   - Input frequency — Single
   - Input acceleration (g) — See table 3.10-9.
   - Input motion — Sinusoidal sweep at 1 Hz/s
   - TRS versus RRS — See figure 3.10-10.
   - Multi- or single-axis tests — Biaxial

3. Functional verifications

   The valve was energized and pressurized before, during, and after testing. Recordings and observations indicate that no circuit interruptions greater than 10 ms, malfunctions, changes or indications, degradation of performance, or physical damage were observed.

<u>Standby Gas Treatment System (SGTS) Current-to-Current Converter</u>

1. Method of qualification - testing

2. Summary of results

   - Input frequency — Single
   - Input acceleration (g) — See table 3.10-10.
   - Input motion — Sinusoidal dwell

- TRS versus RRS — See figure 3.10-11.
- Single-axis tests — Three axes independently

3. Functional verifications

   All the switches and relays showed no malfunctions under the seismic environment.

Q Panels in Diesel Generator Building

1. Method of qualification - dynamic analysis
2. Summary of results
   - Natural frequency (Hz) 50
   - Maximum horizontal acceleration (g) 0.40
   - Maximum vertical acceleration (g) 0.20
   - Maximum stress in beam (psi) 122
   - Maximum stress in plate (psi) 26
3. Structural verifications

   The calculated stresses in all structural elements for all the cases were very low. These results indicated that the panels are capable of withstanding the prescribed seismic environment.

Nuclear Service Power-Operated Valves and Power-Operated Butterfly Valves

1. Method of qualification - combined testing and analysis
2. Summary of results

- Valve assembly - dynamic analysis — See table 3.10-11.
  - Natural frequency (Hz) — > 20
  - Input motion (g) — 3
- Motor operator - testing
  - Input frequency — Single and multiple
  - Input acceleration (g) — See table 3.10-12.
  - Input motion — Sine dwell
  - Single or biaxial testing — Both

3. Functional and structural verifications

   Stresses at critical locations were verified to be less than the allowable stresses. Structural integrity was maintained under the seismic environment. The motor was monitored electrically and performed all control and indicating functions. There was no evidence of contact chatter.

Nuclear Service Air-Operated Valves

1. Method of qualification - dynamic analysis

2. Summary of results - 900 lb feedwater check valve by Atwood & Morrill Company

   - Natural frequency (Hz) — 775
   - Input acceleration (g) — 3

	Calculated	Allowable
Cylinder connection bolt	12,640	25,000
Brackets spacer	1030	17,500
Bracket extension	6860	16,200

3. Structural verifications

   Stresses at critical locations were verified to be less than the allowable stresses. Structural integrity was maintained under the seismic environment.

Nuclear Service Air-Operated Butterfly Valves[a]

1. Method of qualification - dynamic analysis and testing

2. Summary of results

   - 18-in. valve - See tables 3.10-13 and 3.10-14.

   - 6-in. valve - See tables 3.10-15 and 3.10-16.

   - Solenoid qualification.

     - Natural frequency (Hz) 6(H), 11.5(H), > 21(V)[b]

     - Input acceleration (g) 3

     - Input motion Sine beat
10 cycles/beat
5 beats/frequency

     - Single-axis tests Three axes independently

   - 10-in. valve.

     The 10-in. Minitork air-operated butterfly valves were seismically qualified by analysis of the valve-actuator assembly and testing of the solenoid valve and limit switch. The analysis calculates stress levels at critical areas in the valve-actuator assembly. Seismic stresses in other sections of the unit are not calculated because of their insignificance when compared to stress levels incurred during normal operation of the valve.

3. Functional and structural verifications

   Stresses at critical locations were verified to be less than the allowable stresses. Structural integrity was maintained under the seismic environment. The solenoid valve performed all control functions.

Service Air System Accumulators for Outboard Main Steam Isolation Valves (MSIVs)

1. Method of qualification - dynamic analysis

2. Summary of Results

a. Three copies each of the Masoneilan International, Inc. seismic calculations for 2P41-F066, F067, and 2T46-F005, in addition to the Environmental Testing Corporation test reports 10696 and 1021G-2 were provided to the NRC for evaluation.
b. H = horizontal frequencies in north-south and east-west directions. V = vertical frequencies.

	Calculated	Allowable
Natural frequency (Hz)	339	-
Percent damping	2	-
Maximum horizontal acceleration (g)	0.39	-
Maximum vertical acceleration (g)	0.30	-
Maximum longitudinal component stress in cylinder (psi)	1830	11,450
Maximum longitudinal tension stress in cylinder (psi)	3650	18,200
Maximum shear on anchor bolt (psi)	95	10,000
Maximum circumferential stress at lug attachment (psi)	8307	18,200
Maximum longitudinal stress at lug attachment (psi)	4595	18,200

3. Structural verifications

   Stresses at critical locations were verified to be less than the allowable stresses. Structural integrity was maintained under the seismic environment.

PSW System Pumps

1. Method of qualification - dynamic analysis

2. Summary of results

   Natural frequency (Hz) and Input accelerations (g)

		Mode 1	Mode 2	Mode 3	Mode 4	Vertical
Head and Motor	f	25.7	295	1155	-	178
	g-OBE	-	-	-	-	-
	g-DBE	0.374	0.374	0.374	-	0.11
Column	f	5.29	17.25	36.39	62.67	61
	g-OBE	0.70	0.30	0.22	0.22	0.11
	g-DBE	1.05	0.51	0.37	0.37	0.187
Shaft	f	30.2	-	-	-	60.5
	g-OBE	0.22	-	-	-	0.11
	g-DBE	0.374	-	-	-	0.187

A 3% damping curve was used for OBE.

A 5% damping curve was used for DBE.

Maximum of north-south or east-west values was used in all cases.

3. Structural verifications

   Stresses at critical locations were verified to be less than the allowable stresses. (See table 3.10-17.) Structural integrity was maintained under the seismic environment.

H_2 Recombiners

1. Method of qualification - testing - IEEE 344-1975

2. Summary of results

   The following test results were obtained for the recombiners, the recombiner control console, and the recombiner power cabinet, previously qualified for a seismic qualification level in excess of the RRS for HNP-2 as shown in figures 7 and 8 of the proprietary Fukushima report provided separately to the NRC.

   - Input frequency (Hz) — Random multifrequency superimposed with sine beat

   - Axis of test — Two axes simultaneously

3. Functional verifications

   For the recombiner the reaction chamber and coils were inspected, and no damage was observed after the test. The pressure transducers and transmitters were monitored. All functional anomalies observed during the tests were corrected.

For the recombiner control console, electrical functions were monitored and yielded satisfactory results. No significant visible structural anomalies occurred as a result of seismic testing.

For the recombiner power cabinet, relays were monitored and functioned satisfactorily. The whole cabinet was functioning normally after the test.

<u>Remote Shutdown Panels, Post-Accident Monitors, and Associated Components</u>

1. Method of qualification - combined analysis and testing

2. Summary of results

   - Panels by dynamic analysis.

- Natural frequency (Hz)	108
- Maximum horizontal acceleration (g)	0.3
- Maximum vertical acceleration (g)	0.36
- Maximum stress in beam (psi)	317
- Maximum stress in plate (psi)	116

   - Indicating and alarm instruments - testing.

- Input frequency	Single
- Input acceleration (g)	See tables 3.10-18 through 3.10-20.
- Input motion	Sine beat 10 to 15 cycles/beat 2 beats/frequency 96 beats/axis
- Single-axis tests	Three axes independently
- TRS versus RRS	See figures 3.10-12 and 3.10-13.

3. Functional and structural verifications

   The calculated stresses in all structural elements for the panel were very low. These results indicated that the panels are capable of withstanding the prescribed seismic environment. All the indicating and alarm instruments were monitored at their normal operating mode, and no malfunctions were indicated during the tests.

Recirculation Pump Trip (RPT) Breakers

1. Method of qualification - testing in accordance with IEEE-344-1975

2. Summary of results

   - Input motion - multifrequency sine beats spaced at one-third-octave intervals over the seismic range of 1 to 33 Hz.

   - Axis of test - front to back and vertical, left to right and vertical (simultaneous horizontal and vertical).

   - Damping - 5%.

   - TRS versus RRS. (See figure 3.10-14.)

3. Functional verifications

   The equipment was subjected to an excessive number of tests. The switchgear maintained its structural integrity, and there was no physical equipment failure. This equipment performed its intended Class 1E functions during and after the specified seismic events.

3.10.2.1.1 Seismic Design Adequacy of Supports

Analyses or tests are performed for all supports of electrical equipment and instrumentation to ensure their structural capability to withstand seismic excitation. The following bases are used in the seismic design and analysis of cable tray supports and instrument tubing supports:

A. All cable tray supports and instrument tubing supports are designed by the response spectrum method.

B. Analysis and seismic restraint measures for tray supports and tubing supports are based on combined limiting values for static load, span length, and computed seismic response.

C. All Class 1E cable tray supports are designed to meet the requirement by dynamic analysis using the appropriate seismic response spectra.

D. Maximum stress is limited to 90% of minimum yield.

E. The Seismic Category I instrument tubing systems are such that the allowable stresses permitted by Section III of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code are not exceeded when the tubing is subjected to the loads specified in Section 3.9 for Classes 2 and 3 piping.

For field-mounted instruments, the following is applicable:

A. The mounting structures for Seismic Category I instruments have a fundamental frequency ≥ 20 Hz.

B. The stress level in the mounting structure does not exceed the material allowable stress when subjected to the maximum acceleration level of the mounting location.

Supports are tested with equipment installed. If the equipment is inoperative during the support test, the response at the equipment-mounting location is monitored. In such a case, equipment is tested separately, and the actual input to the equipment is more conservative in amplitude and frequency content than the monitored response.

3.10.2.2 NSSS Equipment

3.10.2.2.1 Seismic Analysis

Very few of the GE-supplied Class 1E devices were completely qualified by analysis alone (table 3.10-1). Sometimes, however, besides being used for passive mechanical devices, analysis was used in combination with testing for larger assemblies containing Class 1E devices. For instance, a test might have been run to determine whether there were natural frequencies in the equipment within the critical seismic frequency range. (See IEEE 344-1971, Paragraph 3.2.2.3.1.) If the equipment was determined to be free of natural frequencies in this range, it was assumed to be rigid, and a static analysis was performed as shown in Appendix C of NEDO-10678. (See IEEE 344-1971, Paragraph 3.2.3.4.) If the equipment had natural frequencies in the critical frequency range, calculations of transmissibility and responses to varying input accelerations were performed to determine whether Class 1E devices mounted in the assembly would operate without malfunctioning.

In addition, analyses or tests were performed for all supports of electrical and mechanical equipment and instrumentation. The requirements of the applicable paragraphs of IEEE 344-1971 are applied when conducting tests on equipment supports. In all cases, the combined stresses of the support structures are within the limits of the ASME Code Section III, Appendix XVII-2000.

The analog transmitter trip system (ATTS) instrumentation is discussed in paragraph 3.10.2.2.3.

3.10.2.2.2 Testing Procedures

Since the GE-supplied Class 1E equipment was and is used in numerous systems in many different plants under widely varying seismic requirements, the seismic qualifications tests were performed using an expected worst-case envelope of 1.5-g horizontal and 0.5-g vertical at all frequencies from 5 to 33 Hz. (The actual qualification range was 0.25 to 33 Hz, but since test-facility capability usually limited the lower frequency test to 5 Hz, a combination of test and analysis was used to assure that there were no untested resonances. A sample analysis is shown in Appendix B of NEDO-10678.) Based upon experience obtained from seismic tests conducted on devices of various designs, sizes, and types of construction, none of these devices has a resonant frequency in the 1- to 5-Hz region, and very few have any resonances < 33 Hz. Two examples of devices that have been tested only in the 5- to 33-Hz frequency range are Static-O-Ring pressure switch 145C3011 and Robertshaw level switch 145C3047. Based on the rigidity of these and similar devices in the 5- to 33-Hz frequency range, the physical size, mass, type of construction, and design, it is conservative to assume that no resonances are in the 1- to 5-Hz region. These assumptions are borne out by a multitude of seismic tests conducted on devices in the 1- to 5-Hz region with no resonances observed. All control panels and racks tested at the GE test facility in San Jose were tested over the full 1- to 33-Hz frequency range. No panels or racks have exhibited a resonant frequency in the 1- to 5-Hz range.

In general, the Class 1E equipment was tested by using the following procedures:

The test procedure for devices required that the devices be mounted on the vibration machine table in a manner similar to the way it was to be installed. The device was tested in the operating states to be used while performing its Class 1E functions. These states were monitored before, during, and after the test to assure proper function and absence of any spurious function. In the case of a relay, both energized and deenergized states and normally open and normally closed contact configurations were tested if the relay was to be used in those configurations in its Class 1E functions.

The seismic excitation was a single-frequency, continuous test in which the applied vibration was a sinusoidal table motion at a fixed-peak acceleration and a discrete frequency at any given time. Each frequency and acceleration combination was maintained for about 30 s, except when a resonance search was made (IEEE 344-1971, Paragraph 3.2.2.4.1). The vibratory excitation was individually applied in three orthogonal axes with the axes chosen as those coincident with the most probable mounting configuration.

The first step was to search for resonances in each device. This was done since resonances cause amplification of the input vibration and are the most likely cause of malfunction. The resonance search was usually run at low-acceleration levels (0.2 g) to avoid destroying the test sample in case a severe resonance was encountered. The resonance search was run from 5 to 33 Hz, in accordance with IEEE 344, in not < 7 min. If the device was large enough, the vibrations were monitored by accelerometers placed at critical locations from which resonances were determined by comparing the acceleration level with that at the vibration machine table. Usually, the devices were either too small for an accelerometer; their critical parts were in an inaccessible location;

or they had critical parts that would be adversely affected by the mounting of an accelerometer. In these cases, the resonances were detected by strobe light (visually), by audible observation, or by performance.

Following the frequency scan and resonance determination, the devices were tested to investigate their malfunction limits. This test was a necessary adjunct to the assembly test, as will be shown later. The malfunction-limit test was run at each resonant frequency as determined by the frequency scan. In this test, the acceleration level was gradually increased until either the device malfunctioned or the limit of the vibration machine was reached. If no resonances were detected (as was usually the case), the device was considered to be rigid,[a] and the malfunction limit was, therefore, independent of frequency. To achieve maximum acceleration from the vibration machine, rigid devices were malfunction tested at the upper test frequency (33 Hz) since that allowed the maximum acceleration to be obtained from deflection-limited machines.

The summary of the results of tests on the devices used in Class 1E applications given in table 3.10-1 includes the seismic qualification level and resonant frequencies for each device tested.

The above procedures were required of purchased devices, as well as those made by GE. Vendor test results were reviewed, and, if unacceptable, the tests were repeated either by GE or the vendor. If the vendor tests were adequate, the device was considered qualified to the limits of the test.

Assemblies, i.e., control panels, containing devices that have had seismic malfunction limits established were tested by mounting each assembly on the vibration machine table in the manner that it was to be mounted when in use and by vibration testing it by running a low-level resonance search. As with the devices, the assemblies were tested in the three major orthogonal axes. The resonance search was run in the same manner as that described for devices. If resonances were present, the transmissibility between the input and the location of each Class 1E device was determined by measuring the accelerations at each device location and calculating the magnification between it and the input. Once known, the transmissibilities could be used analytically to determine the response at any Class 1E device location for any given input. As long as the device input accelerations were determined to be below their malfunction limits, the assembly was assumed to be qualified. If no resonances existed, the assembly was considered to be a rigid body with a transmissibility equal to 1 so that a device mounted on the assembly would be limited directly by the assembly input acceleration.

Since control panels and racks constitute the majority of Class 1E electric assemblies supplied by GE, seismic qualification testing of these is discussed in more detail. There are four generic types, as shown in table 3.10-2. Using the above procedures, one or more of each type was tested.

a. All parts move in unison.

Figures 3-1 through 3-4 of NEDO-10678 illustrate the panel types referenced in table 3.10-2 and show typical accelerometer locations. The results of seismic tests performed on the type of Class 1E panels supplied by GE for HNP-2 are summarized in table 3.10-3.

The full-acceleration level tests previously described disclosed that most of the panel types had more than adequate mechanical strength and that a given panel design acceptability was just a function of its amplification factor and the malfunction levels of the devices mounted in it. Subsequent panels were, therefore, tested at lower acceleration levels and the transmissibilities measured to the various devices. By dividing the device's malfunction levels with the panel transmissibility between the device and the panel input, the panel seismic qualification level could be determined. Several high-level tests have been run on selected generic panel designs to assure conservatism in using the transmissibility analysis described.

The seismic qualification of equipment supplied to GE by others was required to follow the same procedures as those used by GE. The qualification data were supplied to and reviewed by GE for conformance to the required procedures.

The following information regarding the Category I equipment required for safe shutdown is included as a part of the GE generic program on seismic qualification:

- Natural frequency in each direction.
- Functional description and method of functional verification.
- Seismic input employed in the test or analysis in each direction.
- Graphs showing TRS enveloping the RRS.
- Test or analysis results summary.
- For equipment qualified by analysis, identification of the critical structural element(s) demonstrating its structural integrity and functional capability under the DBE.

<u>Equipment Qualified by GE</u>

- Inverters.
- Relays.
- Switches.
- Pressure transmitters.
- Panels (reactor protection boards).
- Neutron monitoring systems.

- Control rod drive.
- Jet pumps.
- Recirculation pump.
- RHR pump motors.
- MSIVs.
- Main steam relief valves (see table 3.9-34).

3.10.2.2.3 ATTS Seismic Qualification

The ATTS qualification program was designed to meet or exceed the requirements of IEEE 344-1975. A summary of the program is contained in NEDE-22154-1, with details being presented in NEDC-30039-1. Component qualification was accomplished either by type testing, which simulated triaxial motion, or by similarity analysis. The individual devices covered by this program are listed in table 7.8-1, and information relating to the seismic qualification of these devices is contained in table 3.10-21. Table 3.10-22 identifies the control panels and local instrument racks covered by this program.

TABLE 3.10-1 (SHEET 1 OF 5)

NSSS CLASS 1E EQUIPMENT REQUIRING QUALIFICATION[a]

Component	Manufacturer	Primary Class 1E Function	Qualification Environment[b]	Seismic Qualification Level (g)[c] X	Y	Z	Resonant Frequency (Hz)[d]
Temperature element	California Alloy	Pressure integrity	Containment		(a)		---
Temperature element	Pyco	Temperature measurement	Reactor building	5	5	5	---
Temperature switch	Transmation	Contact transfer at temperature setpoint	Containment	1.5	1.5	0.5	---
Alarm unit	Bailey	Provide alarm at setpoint	Control room	7.5	8.5	20	---
Controller	GE	Provide control signal output	Control room	9	9	13	---
Square root converter	GE	Convert pressure signal to flow output signal	Control room	4.2	7	1.8	---
Differential pressure transmitter	Barton	Pressure integrity	Containment		(a)		---
Level-indicating switch	Barton	Contact transfer at level trip point	Containment	17	13	1.8	---
Inverter filter	Topaz	Filter-inverter input	Control room	10	10	10	---
dc power supply	GE	Convert 115 V ac to 24 V dc for safety circuits	Control room	2.5	2.5	2.5	---
Pressure switch	Barksdale	Provide contact transfer at pressure trip point (reactor protection system) (RPS)	Turbine building	2	2	2	---
Pressure switch	Barksdale	Provide contact transfer at pressure trip point	Reactor building	15	15	15	---
Power range detector	GE	Neutron flux measurement	Reactor vessel		(f)		---
Differential pressure switch	Barton	Provide contact transfer at differential pressure trip point	Reactor building	10	5	10	---
Level switch (sump)	Magnetrol	Provide contact transfer at level trip point	Condensate storage tank	1.2	6	9.5	---

TABLE 3.10-1 (SHEET 2 OF 5)

Component	Manufacturer	Primary Class 1E Function	Qualification Environment[b]	Seismic Qualification Level (g)[c] X	Y	Z	Resonant Frequency (Hz)[d]
Inverter	Topaz	Control dc input to regulated ac output	Control room	15	10	7	---
Power supply	GE	Input voltage 120-V-ac ±10%/output - 24-V-dc ±1%	Service building	2.5	2.5	2.5	---
Relay, time delay	Agastat	Maintain state or transfer	Control room	17	4.6	17	---
Relay	Agastat	Maintain state or transfer	Control room	17	6.7	17	---
Switch	GE-SBM	Supply ac power to MCC	Control room	25	25	25	---
Relay	GE-HFA	Multipurpose control, logic; annunciator functions	Control room	5	7.5	7.5[e]	30
Relay, time delay	GE (CR2820)	Multipurpose control, logic; annunciator functions	Control room	25	25	25	---
Relay	GE-HGA	Multipurpose control, logic; annunciator functions	Control room	12	12	12[e]	---
Switch	GE (CR2940)	Apply ac/dc power for manual initiation of safety systems	Control room	20	20	20	---
Timer, motor-driven	Eagle signal	Timeout signals; apply or interrupt power to load	Control room	10	10	5.5	---
Pressure transmitter	Bailey Meter	Provide current output in response to pressure input	Reactor building	5.5	5.5	3.7	---
Differential pressure indicator	Barton	Pressure integrity	Reactor building		(a)		---
Contactor	GE (CR305)[g]	Deenergize; interrupt power to system solenoids	Control room	12	12	12	27
Switch	Cutler Hammer	Manual initiation of safety systems	Control room	10	--	--	---
Level switch	Magnetrol	Apply or interrupt power to load upon setpoint trip	Reactor building	5.0	4.1	9.5	---

TABLE 3.10-1 (SHEET 3 OF 5)

Component	Manufacturer	Primary Class 1E Function	Qualification Environment[b]	Seismic Qualification Level (g)[c] X	Y	Z	Resonant Frequency (Hz)[d]
Switch, bank mode	GE	Mode selection	Control room	10	10	10	---
Range switch	GE	Range selection	Control room	8.5	8.5	8.5	---
Pressure switch	Robertshaw	Pressure integrity	Reactor building		(a)		---
Power supply	GE-MAC (7000)	Provide power for control circuitry	Control room	2.5	2.5	2.5	---
Relay	Agastat	Multipurpose control; low-annunciator functions	Control room	3.5	--	--	20
Indicator/trip unit	GE	Provide trip output for safety system	Control room	15	15	15	31
Log radiation monitor	GE	Provide high-radiation signal to RPS	Control room	3	3	3	---
Trip auxiliary unit	GE	Provide high-radiation signal to RPS	Control room	17	17	17	---
Pressure switch	Barton	Provide contact transfer at pressure trip point	Reactor building	5	10	10	---
Temperature switch	Riley/scam	Provide contact transfer at thermocouple input trip point	Control room	8	8.5	9.5	---
Pressure switch	Static-O-Ring	Provide contact transfer at pressure trip point	Reactor building	15	15	15	---
Pressure switch	Barksdale (D2T)	Provide contact transfer at pressure trip point	Reactor building	13	13	10	---
Pressure transmitter	Rosemount (11510)	Provide current output response to pressure input	Reactor building	2	2	2	7
Pressure switch	Static-O-Ring (12N)	Provide contact transfer at pressure trip point	Reactor building	15	15	15	---
Temperature switch	Fenwall	Provide contact transfer at temperature trip point	Reactor building	20	20	20	---

TABLE 3.10-1 (SHEET 4 OF 5)

Component	Manufacturer	Primary Class 1E Function	Qualification Environment[b]	Seismic Qualification Level (g)[c] X	Y	Z	Resonant Frequency (Hz)[d]
Intermediate range monitor (IRM) preamplifier	GE	Amplify IRM voltage signal	Reactor building	8.5	8.5	8.5	---
Rod worth monitor	GE	Rod worth monitor in RC&IS system-rod control and information system	Control room	8.5	8.5	8.5	33
IRM detector	GE	Convert IRM signals to voltage output	Reactor vessel		(f)		---
Detector	GE	Radiation monitor	Reactor building	3	3	2	---
Summer	GE	Measure flow difference signal	Control room	4.2	7	1.8	---
Pressure transmitter	GE-MAC	Provides current output signal proportional to pressure input	Reactor building	10	10	10	---
Level-indicating transmitter switch	Barton	Provide contact transfer at level trip point	Reactor building	5	2	5	8
Alarm unit	Bailey Meter	Provides contact transfer at trip point	Control room	9	9.5	13	---
Pressure switch	Barksdale	Pressure integrity	Reactor building		(a)		---
Pressure switch	Static-O-Ring	Pressure integrity	Reactor building		(a)		---
Pressure switch	Barton	Pressure integrity	Reactor building		(a)		---
Pressure indicator	Robertshaw	Pressure integrity	Reactor building		(a)		---
Differential pressure transmitter	Rosemount	Pressure integrity	Reactor building		(a)		---
Level transmitter	Yarway	Shroud water level; analog signal	Reactor building	10	8	1	31
Level transmitter	Bailey	Analog output to RHR	Reactor building	10	10	10	---
Level-indicator switch	Yarway	Contact output for remote shutdown	Reactor building	10	8	1	31
Level-indicator switch	Yarway	Reactor water level contact output for scram; isolation	Reactor building	10	8	1	31
Temperature switch	Fenwal	Pressure integrity	Reactor building		(a)		---

TABLE 3.10-1 (SHEET 5 OF 5)

Component	Manufacturer	Primary Class 1E Function	Qualification Environment[b]	Seismic Qualification Level (g)[c] X	Y	Z	Seismic Resonant Frequency (Hz)[d]
MV/I	Bailey Meter	Convert voltage signal to current signal	Control room	8	9	8	---
Sensor and converter	GE	Analog electrical input; contact output	Control room	15	15	15	---
Transducer	Fisher	Pressure integrity	Reactor building		(a)		---
Conductivity element	Beckman	Pressure integrity	Reactor building		(a)		---
Level transmitter	Rosemount	Pressure integrity	Reactor building		(a)		---
Flow transmitter	Rosemount	Provide current output in response to flow input	Reactor building	3	3	3	---
IRM	GE	Provide contact transfer response to current input	Control room	5		4	1 2
M/A station	GE	Provide control signal output	Control room	5	5	5	---
Indicator	Moeller	Pressure integrity	Reactor building	5	5	5	---
Flow element	Vickery Sims	Analog output for leak detection	Reactor building		(a)		
Power range instrument	GE	Provide contact transfer and signal conditioning on current input	Control room	1.8	1.8	1.2	8, 19, 26

a. Class 1E equipment listed as "pressure integrity" is not seismically tested. Integrity is verified by analysis.
b. Qualification environments (normal and accident) are given in tables 3.11-1 through 3.11-4.
c. Seismic qualification data represent the device fragility level or the maximum level at which the device could be tested because of shaker capability or testing restraints.
d. Most devices have resonant frequencies beyond the normal earthquake qualification range of 0.25 to 33 Hz. Data are given only for devices with resonant frequencies within the 0.25- to 33-Hz range.
e. The malfunction limit for relay is worst case with the relay deenergized and normally closed contacts monitored for a 10-ms discontinuity.
f. Qualified by analysis to withstand seismic and vibrational loads in the reactor vessel.
g. Power relays on RPS system which interrupt the scram pilot solenoids have been replaced with GE series CR305 relay.

TABLE 3.10-2

NSSS PANEL TYPES

Panel Type	Use	Number Used
Vertical board, benchboard	Operating information and controls	13
Instrument racks, cabinets	NSSS monitoring instrumentation	4
Local racks	Process instruments	26
NEMA-type 12 enclosures[a]	Miscellaneous	1

a. NEMA-National Electrical Manufacturers Association.

TABLE 3.10-3 (SHEET 1 OF 3)

SEISMIC QUALIFICATION TEST SUMMARY - NSSS CONTROL PANELS AND LOCAL PANELS AND RACKS

Control Panel	Description	Type	Class 1E Equipment Description	Comments
H11-P601	Reactor cooling and isolation	Benchboard	SBM and CR2940 switches, GE-MAC instruments	Too long for test table-not tested
H11-P602	Reactor water cleanup (RWC) and recirculation	Benchboard	SBM and CR2940 switches, GE-MAC instruments	Seismic test in similar-type panel
H11-P603	Reactor control	Benchboard	Mode switch, IRM range switches	Seismic test completed
H11-P606	Startup neutron monitor	4-bay instrument rack	Trip auxiliary unit, IRM, SRM, main steam line radiation monitor, refueling floor vent exhaust radiation monitor, reactor building potential contaminate area vent exhaust radiation monitor	Seismic test completed
H11-P608	Power range neutron monitor	5-bay instrument rack	Average power range monitor, fiber optic bypass switch, quad low-voltage power supply	Seismic test completed
H11-P609	Reactor protection system (RPS)	Vertical board	HFA and HMA relays, CR305 contactor	Panel identical to H11-P611 panel below
H11-P611	RPS	Vertical board	HFA and HMA relays, CR305 contactor	Seismic test completed
H11-P612	Process instrumentation rack	2-bay instrument rack	GE-MAC instruments (GE-MAC 7000)	Seismic test completed
H11-P613	Process instrumentation rack	2-bay instrument rack	GE-MAC instruments (GE-MAC 7000)	Seismic test completed
H11-P614	Steam temperature recorders	Vertical board	CR2940 switches, HMA, relays, timers, temperature monitor, inverter	Seismic test completed
H11-P617	RHR relays	Vertical board	HFA and HMA relays	Seismic test on similar-type panel
H11-P618	RHR relays	Vertical board	HFA and HMA relays	Seismic test on similar-type panel
H11-P620	High-pressure coolant injection (HPCI) relays	Vertical board	HFA and HMA relays	Seismic test on similar-type panel

TABLE 3.10-3 (SHEET 2 OF 3)

Control Panel	Description	Type	Class 1E Equipment Description	Comments
H11-P621	Reactor core isolation cooling (RCIC) relays	Vertical board	HFA and HMA relays	Seismic test on similar-type panel
H11-P622	Inboard isolation valve relays	Vertical board	HFA and HMA relays	Seismic test on similar-type panel
H11-P623	Outboard isolation valve relays	Vertical board	HFA and HMA relays	Seismic test on similar-type panel
H11-P628	Automatic depressurization relays	Vertical board	HFA and HMA relays	Seismic test on similar-type panel
H21-P001	Core spray (CS) system A	Local rack	Barton 288, 289, and Barksdale P214 pressure switch	Seismic test completed
H21-P002	RWC	Local rack	Rosemont 1151D pressure transmitter	Seismic test on similar-type panel
P21-P004	Reactor pressure vessel (RPV) level and pressure - A	Local rack	Pressure switches, level indicator/transmitter	Seismic test on similar-type panel
P21-P005	RPV level and pressure - C	Local rack	Pressure switches, level indicator/transmitter	Seismic test on similar-type panel
H21-P006	Recirculation pump A	Local rack	Pressure transmitter	Seismic test on similar-type panel
H21-P009	Jet pump	Local rack	Pressure transmitter	Seismic test completed
H21-P010	Jet pump	Local rack	Pressure transmitter, pressure switch	Seismic test on similar-type panel
H21-P011	Standby liquid control	Local rack	Pressure transmitter	Seismic test on similar-type panel
H21-P013	Source range monitor (SRM) - IRM preamplifiers	NEMA-12 enclosures	SRM-IRM preamplifiers	Seismic test on similar-type panel
H21-P014	HPCI instruments	Local rack	Pressure transmitter, pressure switch	Seismic test on similar-type panel

TABLE 3.10-3 (SHEET 3 OF 3)

Control Panel	Description	Type	Class 1E Equipment Description	Comments
H21-P015	Main steam flow	Local rack	Pressure switch	Seismic test on similar-type panel
H21-P017	RCIC panel A	Local rack	Pressure transmitter, pressure switches	Seismic test on similar-type panel
H21-P018	RHR-CHA	Local rack	Pressure switches	Seismic test on similar-type panel
H21-P019	CS-CHB rack	Local rack	Pressure transmitter, pressure switch	Seismic test on similar-type panel
H21-P021	RHR-CHB	Local rack	Pressure switches	Seismic test on similar-type panel
H21-P025	Main steam flow	Local rack	Pressure switch	Seismic test completed
H21-P022	Recirculation pump B	Local rack	Pressure transmitter, pressure switch	Seismic test on similar-type panel
H21-P160	Recirculation flow instruments	Local rack	Pressure transmitter	Seismic test on similar-type panel
H21-P161	Recirculation flow instruments	Local rack	Pressure transmitter	Seismic test on similar-type panel
H21-P161	Recirculation flow instruments	Local rack	Pressure transmitter	Seismic test on similar-type panel
H21-P162	Recirculation flow instruments	Local rack	Pressure transmitter	Seismic test on similar-type panel
H21-P163	Recirculation flow instruments	Local rack	Pressure transmitter	Seismic test on similar-type panel

a. "Seismic test on similar-type panel" means that the required seismic tests were made on panels that are sufficiently close to the HNP-2 design to provide adequate representation. These panels are used on other plants.
b. "Seismic test completed" means that a panel identical to the HNP-2 design was tested in another plant.

TABLE 3.10-4 (SHEET 1 OF 2)

SEISMIC QUALIFICATION OF MAJOR BOP ELECTRICAL AND MECHANICAL EQUIPMENT

	Item	Analysis	Testing	Justification of Method Selected
1.	600-V station service switchgear		IEEE-344-71	B1
2.	600-V-ac MCC		IEEE-344-71	B1
3.	New battery chargers		IEEE-344-75	B1
4.	Large induction motors	Dynamic		A2, A3
5.	Power transformers	Dynamic		A2, A3
6.	Inverters		IEEE-344-75	B1
7.	Diesel generators and auxiliary equipment	Dynamic	IEEE-344-71	A2, A3
8.	MCREC system dampers	Dynamic		A1
9.	Fisher air-operated nuclear control valves	Dynamic	IEEE-344-71	A1
10.	WKM air-operated nuclear control valves	Dynamic	IEEE-344-71	A1
11.	Excess-flow check valves		IEEE-344-71	B1
12.	SGTS current-to-current converter		IEEE-344-71	B1
13.	Q panels in diesel generator building	Dynamic		A1
14.	Nuclear service power-operated valves 2 1/2 in. and larger	Dynamic	IEEE-344-71 IEEE-344-75	A1
15.	Nuclear service air-operated valves 2 1/2 in. and larger	Dynamic		A1
16.	Nuclear service power-operated butterfly valves	Dynamic	IEEE-344-71 IEEE-344-75	A1
17.	Nuclear service air-operated butterfly valves	Dynamic (valves)	IEEE-344-71 (Solenoid)	A1/B1

TABLE 3.10-4 (SHEET 2 OF 2)

	Item	Analysis	Testing	Justification of Method Selected
18.	Instrument and service air system accumulators for outboard MSIVs	Dynamic		A1
19.	PSW system pumps	Dynamic		A2, A3
20.	H2 recombiners		IEEE-344-75	B1
21.	Remote shutdown panels and associated components	Dynamic	IEEE-344-71	A1
22.	Post-accident monitoring indicators and recorders	Dynamic	IEEE-344-71	A1
23.	RPT breakers		IEEE-344-75	B1

NOTES:

A. Qualification by analysis is selected for the following reasons:

1. The equipment can be physically idealized by a mathematical model. Because of the advance in mathematical technology and availability of high-speed computers, complex equipment can be described mathematically, and its dynamic behavior can be predicted with high level of accuracy.[a]

2. It is impractical to test the equipment because of its size.

3. The equipment is subjected to environments that cannot be simulated by test, e.g., pressure and thermal transient loads.[b]

B. Qualification by test is selected for the following reason:

1. The equipment is so complex in nature that conclusions derived from analysis may not be reliable.

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a. Stafford, J. R., "Finite Element Predictions of the Dynamic Response of Power Plant Control Cabinets," Second ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities, Vol. 1-A, p 266, 1975.
b. Meligi, A. E., "Unreliability of Qualifying Active Mechanical Equipment by Testing Only," Second ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities, Vol. II, p II-11, 1975.

TABLE 3.10-5

TEST RESULTS FOR 600-V-ac MOTOR CONTROL CENTER

Required Response Spectra					Test Results					
					Minimum Input (g)			Computed TRS (g)		
Location	Direction	Frequency (Hz)	RRS (g)	RRS for f = 33 Hz (g)	Front to Back	Vertical	Side to Side	Front to Back	Vertical	Side to Side
Reactor (el 111 ft)	Horizontal	4.0	2.7	0.24	1.0	-	0.43	25.0	-	10.8
		10.0	1.7	0.24	0.60	-	0.90	15.0	-	45.0
	Vertical	6.5	6.9	0.29	-	0.55	-	-	27.5	-
(el 130 ft)	Horizontal	4.0	4.9	0.28	1.0	-	0.43	50.0	-	10.8
		10.0	2.0	0.28	0.6	-	0.90	30.0	-	45.0
	Vertical	6.5	12.1	0.43	-	0.55	-	-	55.0	-
		18.0	1.7	0.43	-	1.00	-	-	100.0	-
Intake structure (el 111 ft)	North-South	10.0	1.75	0.37	0.60	-	0.90	6.0	-	9.0
	East-West	7.0	4.34	0.43	0.80	-	0.85	8.0	-	8.5
	Vertical	16.0	0.70	0.17	-	1.0	-	-	10.5	-
Diesel generator (el 130 ft)	Horizontal	4.0	3.65	0.25	1.0	-	0.43	10.0	-	4.3
	Vertical	5.0	2.40	0.25	-	0.30	-	-	3.0	-

TABLE 3.10-6

BATTERY CHARGER
TEST RUN DESCRIPTIONS AND INPUT ACCELERATIONS

		Input Acceleration (g)		
Number	Axes	HZPA[c]	VZPA[d]	Test Level
1	S-S/V[a]	0.48	0.25	< OBE
2	S-S/V	0.60	0.42	OBE
3	S-S/V	0.70	0.40	OBE
4	S-S/V	0.64	0.40	OBE
5	S-S/V	0.63	0.40	OBE
6	S-S/V	0.86	0.52	OBE
7	S-S/V	1.35	0.60	DBE
8	F-B/V[b]	0.64	0.44	OBE
9	F-B/V	0.67	0.44	OBE
10	F-B/V	0.60	0.46	OBE
11	F-B/V	0.66	0.41	OBE
12	F-B/V	0.93	0.61	OBE
13	F-B/V	1.31	0.65	DBE

a. S-S/V = side to side and vertical.
b. F-B/V = front to back and vertical.
c. HZPA = horizontal zero-period acceleration.
d. VZPA = vertical zero-period acceleration.

TABLE 3.10-7

100-kW INVERTER TEST RUN DESCRIPTIONS

		Input Acceleration (g)		
Number	Axes	HZPA	VZPA	Test Level
1	S-S/V	0.16	0.09	< OBE
2	S-S/V	0.24	0.14	OBE
3	S-S/V	0.27	0.15	OBE
4	S-S/V	0.27	0.16	OBE
5	S-S/V	0.25	0.17	OBE
6	S-S/V	0.3	0.19	OBE
7	S-S/V	0.5	0.26	DBE
8	F-B/V	0.29	0.21	OBE
9	F-B/V	0.28	0.21	OBE
10	F-B/V	0.28	0.2	OBE
11	F-B/V	0.28	0.19	OBE
12	F-B/V	0.29	0.19	OBE
13	F-B/V	0.49	0.28	DBE

TABLE 3.10-8

BASLER RELAYS INPUT ACCELERATION

Hz	Input (g)
50 - 12	5.0
12	4.7
11	3.4
10	2.7
9	2.3
8	1.5
7	1.0
6	0.8
5	0.8

TABLE 3.10-9 (SHEET 1 OF 2)

EXCESS-FLOW CHECK VALVES INPUT ACCELERATION

Frequency (Hz)	Inputs (g -peak)		Outputs (g -peak)	
	1	2	3	4
1	0.3	0.3	0.3	0.3
2	1.0	1.0	1.0	1.0
3	1.0	1.0	1.0	1.0
4	1.0	1.0	1.0	1.0
5	1.1	1.1	1.1	1.0
6	1.1	1.1	1.1	1.1
7	1.1	1.1	1.1	1.1
8	1.0	1.0	1.0	1.0
9	1.0	1.0	1.0	1.0
10	1.0	1.0	1.1	1.0
11	1.1	1.0	1.1	1.0
12	1.1	1.1	1.1	1.1
13	1.1	1.0	1.1	1.0
14	1.0	1.0	1.0	1.0
15	1.0	1.0	1.0	1.0
16	1.0	1.0	1.0	1.0
17	1.0	1.1	1.0	1.1
18	1.0	1.0	1.0	1.0
19	1.0	1.0	1.0	1.0
20	1.0	1.0	1.0	1.0
21	1.1	1.1	1.1	1.1
22	1.1	1.1	1.1	1.1
23	1.0	1.0	1.0	1.0
24	1.0	1.0	1.0	1.0
25	1.0	1.0	1.0	1.0
26	1.0	1.0	1.0	1.0
27	1.0	1.1	1.0	1.1
28	1.0	1.0	1.0	1.1
29	1.0	1.0	1.0	1.0
30	1.1	1.1	1.1	1.1
31	1.0	1.0	1.0	1.0
32	1.0	1.0	1.0	1.0
33	1.0	1.1	1.0	1.1
34	1.0	1.0	1.0	1.0
35	1.0	1.0	1.0	1.0

TABLE 3.10-9 (SHEET 2 OF 2)

<u>NOTES</u>:

1. Each frequency was maintained for 30 s.

2. No apparent indication of malfunction, degradation of performance, shell leakage, physical damage, or circuit interruptions (NC or NO) > 10 µs was observed.

3. Inputs: 1 - Horizontal control on steel bedplate
   2 - Vertical control on steel bedplate

   Outputs: 3 - On valve case, horizontal (across the flow)
   4 - On valve case, vertical

TABLE 3.10-10

SGTS CURRENT-TO-CURRENT CONVERTER INPUT ACCELERATION

Frequency	g Level	Duration at Each Frequency	Total Vibration Time
1 to 2 Hz	1 g	15 s	2.5 min
2 to 4 Hz	2 g	15 s	
4 to 8 Hz	2.5 g	15 s	
8 to 10 Hz	2.5 to 4 g	15 s	
10 to 20 Hz	4 g	10 s	1.5 min
20 to 30 Hz	2 g	Sweep 4 min per octave (up only, no down sweep)	~ 8 min
30 to 50 Hz	1.5 g		
50 to 100 Hz	1.0 g		

TABLE 3.10-11

SEISMIC ANALYSIS OF 18-in., 900-lb OSY GATE VALVE (MOTOR OPERATOR)[a]

Component (Critical)	Material	Actual Stress (psi) Seismic (3 g)	Thrust	Pressure	Total (psi)	Allowable S_m (psi)
Body neck	SA-352 grade LCB	423	--	5183	5606	18,900
Bonnet flange bolt	A-193 grade B7	1920	10,726	--	12,646	35,000
Yoke arm	A-216 grade WCB	738	3445	--	4183	23,300
Operator fasten bolt	A-193 grade B7	1218	32,621	--	33,839	35,000

a. Frequency = 36 Hz.

TABLE 3.10-12 (SHEET 1 OF 2)

SEISMIC TEST REPORT INDEX

Unit Size	Test Facility	Report No.	Report Date	Test Base	g Level Each Axis
1. Electric Operator					
SMB-0-25	Lockheed Electronics	2768-4768A	10/21/71	Uniaxial	5
SMB-0-25 + brake	Lockheed Electronics	2768-4768	10/21/71	Uniaxial	5.3
SMC-000-5	Ogden	7K112-11	11/27/72	Uniaxial	5.5 nom.
SMB-0-25	Ogden	7K112-11	11/27/72	Uniaxial	5.5 nom.
SMB-0-40	Lockheed Electronics	3521-4811	6/17/74	Uniaxial IEEE-344-71	6
SMB-0-25	Aero Nav	5720	1/6/75	IEEE-344-75 modified	5 at 3 g 1 at 6 g
SMB-000-5	Aero Nav	5721	1/7/75	IEEE-344-75 modified	5 at 3 g 1 at 6 g
SMB-1-40	Aero Nav	5722	1/7/75	IEEE-344-75 modified	5 at 3 g 1 at 6 g
SB-3-100	Aero Nav	5770	10/20/75	IEEE-344-75	5 at 3 g 1 at 6 g
SMB-000-5	Aero Nav	5771	10/17/75	IEEE-344-75	5 at 3 g 1 at 6 g
SMB-3-100	Aero Nav	5773	10/16/75	IEEE-344-75	2 at 5 g 1 at 6 g
SB-0-25	Aero Nav	5774	10/22/75	IEEE-344-75	2 at 5 g 1 at 6 g
SMB-0-25DC	Aero Nav	5772	10/21/75	IEEE-344-75	2 at 5 g 1 at 6 g
SMB-1-100 E-line motor	Aero Nav	5775	10/22/75	IEEE-344-75	2 at 5.3 g 1 at 6.3 g

TABLE 3.10-12 (SHEET 2 OF 2)

	Unit Size	Test Facility	Report No.	Report Date	Test Base	g Level Each Axis
	SMB-5T-250DC	Wyle Labs	43059-1	10/6/75	Spec. biaxial	1 g
	SMB-0-25DC	AEL	75-149ET	10/29/75	Single axis	4 g
	SMB-5T-250AC	Wyle Labs	43059-02	10/30/75	Single axis	6 g
2.	Electric/Manual Operator[a]					
	SMB-000-2/HOBC	Lockheed Electronics	2773C-4773	5/3/72	Single axis	4.4 g
	SMB-0-25/H3BC	Lockheed Electronics	2786-4786 Issue 2	9/5/72	Single axis	3 g
	SMB-3-100-H5BC	Lockheed Electronics	2786-4-4786	2/1/73	Single axis	4 g
	SMB-0-H3BC	Lockheed Electronics	2786-3-4786	2/6/73	Single axis	3.7 g
	SMB-1-25/H4BC standard adapter	Aero Nav	5-6167-5	12/17/75	IEEE-344-75 fragility test	8.0 g capacity of machine
	SMB-00-15/H3BC special steel adapter	Aero Nav	5-6167-4	12/16/75	IEEE-344-75	2 at 5.3 g 1 at 6.3 g
3.	Manual Operator					
	H1BC	Lockheed Electronics	2553-4737	12/28/70	Single axis	5.3 g
	H1BC	Lockheed Electronics	2786-5-4786	1/30/73	Single axis	4.6 g
	H4BC	Lockheed Electronics	2786-6-4786	1/30/73	Single axis	4.6 g
	H6BC	Lockheed Electronics	2786-7-4786	1/30/73	Single axis	3.6 g

a. The g levels tested for the unit sizes are not to be construed as applicable to all sizes and combinations of SMB/H-BC units. The maximum g level allowed for all sizes is limited to 3 g in any axis. In the future, further testing will qualify other sizes for high g levels.

TABLE 3.10-13 (SHEET 1 OF 2)

STRESS LEVELS FOR VALVE COMPONENTS

Component	Name	Symbol	Material	Stress Level (psi)	Allowable Stress Level (psi)
Body	Primary membrane stress in crotch region	P_m	ASME SA-516 grade 55	891	S_m = 13,700
	Primary plus secondary stress due to internal pressure	Q_p	ASME SA-516 grade 55	2674	S_m = 13,700
	Pipe reaction stress		ASME SA-516 grade 55		1.5 S_m = 20,550
	Axial load	Ped		950	
	Bending load	Peb		1731	
	Torsional load	Pet		1731	
	Thermal secondary stress	Q_t	ASME SA-516 grade 55	2096	S_m = 13,700
	Primary plus secondary stress	S_n	ASME SA-516 grade 55	5817	3 S_m = 41,100
	Normal-duty fatigue stress (NA ≥ 2000)	S_p	ASME SA-516 grade 55	4052	S_m = 13,700
Operator mounting	Shear tearout of trunnion bolts through tapped hole in trunnion	S(1)	ASME SA-516 grade 55	320	0.5 S_m = 6850
	Bearing stress of trunnion bolt on tapped hole in trunnion	S(2)	ASME SA-516 grade 55	3384	S_m = 13,700
	Combined stress in trunnion bolt	S(5)	SAE grade 2	9374	18,500
	Combined stress in trunnion body	S(45)	ASME SA-516 grade 55	444	S_m = 13,700

TABLE 3.10-13 (SHEET 2 OF 2)

Component	Name	Symbol	Material	Stress Level (psi)	Allowable Stress Level (psi)
Banjo assembly	Maximum combined stress in disc	S(50)	ASME SA-351 grade CF8M	12,753	S_m = 16,500
	Maximum combined stress in shaft	S(53)	ASME SA-564 type 630 condition H-1150	18,566	S_m = 33,700
	Squeeze pin shear stress	S(58)	ASME SA-479 type 316	5215	0.5 S_m = 9200
	Shaft-bearing compressive stress	S(61)	Nylatron GS	2404	S_m = 3000
Thrust-bearing assembly	Thrust collar bearing stress	S(64)	SAE 660	120	S_m = 8800
	Clamp ring load	S(65)	-	519	2610 lb
	Shear stress across thrust collar	S(66)	SAE 660	98	0.5 S_m = 4400
	Tensile stress in thrust-bearing bolt	S(67)	ASME SA-193 grade B-7	915	S_m = 25,000
	Shear stress in thrust-bearing bolts	S(68)	ASME SA-193 grade B-7	367	0.5 S_m = 12,500

NOTES:

1. Valve size is 18 in.
2. Operator is MDT-4 (handwheel).

TABLE 3.10-14

NATURAL FREQUENCIES OF VALVE COMPONENTS
18-in. VALVE

Component	Natural Frequency Symbol	Material	Natural Frequency (Hz)
Body	F_{N^1}	ASME SA-516	56,423
Banjo	F_{N^2}	ASME SA-564 type 630 condition H-1150	7762
Cover cap	F_{N^3}	ASME SA-515 grade 70	1033

TABLE 3.10-15 (SHEET 1 OF 2)

STRESS LEVELS FOR VALVE COMPONENTS

Component	Name	Symbol	Material	Stress Level (psi)	Allowable Stress Level (psi)
Body	Primary membrane stress in crotch region	P_m	ASME SA-516 grade 55	575	S_m = 13,700
	Primary plus secondary stress due to internal pressure	Q_p	ASME SA-516 grade 55	1725	S_m = 13,700
	Pipe reaction stress		ASME SA-516 grade 55		1.5 S_m = 20,550
	Axial load	Ped		1647	
	Bending load	Peb		2516	
	Torsional load	Pet		2516	
	Thermal secondary stress	Q_t	ASME SA-516 grade 55	1097	S_m = 13,700
	Primary plus secondary stress	S_n	ASME SA-516 grade 55	3566	3 S_m = 41,100
	Normal-duty fatigue stress (NA ≥ 2000)	S_p	ASME SA-516 grade 55	3418	S_m = 65,500
Operator mounting	Shear tearout of trunnion bolts through tapped hole in trunnion	S(1)	ASME SA-516 grade 55	197	0.5 S_m = 6850
	Bearing stress of trunnion bolt on tapped hole in trunnion	S(2)	ASME SA-516 grade 55	1124	S_m = 13,700
	Combined stress in trunnion bolt	S(5)	SAE grade 2	3909	S_m = 18,500
	Combined stress in trunnion body	S(45)	ASME SA-516 grade 55	201	S_m = 13,700

TABLE 3.10-15 (SHEET 2 OF 2)

Component	Name	Symbol	Material	Stress Level (psi)	Allowable Stress Level (psi)
Banjo assembly	Maximum combined stress in disc	S(50)	ASME SA-351 grade CF8M	9254	S_m = 16,500
	Maximum combined stress in shaft	S(53)	ASME SA-564 type 630 condition H-1150	21,703	S_m = 33,700
	Squeeze pin shear stress	S(58)	SA-479 type 316	6014	0.5 S_m = 9200
	Shaft-bearing compressive stress	S(61)	Nylatron GS	1270	S_m = 3000
Thrust-bearing assembly	Thrust collar bearing stress	S(64)	SAE 660	40	S_m = 8800
	Clamp ring load	S(65)	-	64.8 lb	2180 lb
	Shear stress across thrust collar	S(66)	SAE 660	27.5	0.5 S_m = 4400
	Tensile stress in thrust-bearing bolt	S(67)	ASME SA-193 grade B-7	209	S_m = 25,000
	Shear stress in thrust-bearing bolts	S(68)	ASME SA-193 grade B-7	122	0.5 S_m = 12,500

NOTES:

1. Valve size is 6 in.
2. Operator is MDT-2 (handwheel).

TABLE 3.10-16

NATURAL FREQUENCIES OF VALVE COMPONENTS
6-in. VALVE

Component	Natural Frequency Symbol	Material	Natural Frequency (Hz)
Body	F_{N^1}	ASME SA-516 grade 55	53,386
Banjo	F_{N^2}	ASME SA-564 type 630 condition H-1150	5499
Cover cap	F_{N^3}	ASME SA-515 grade 70	4501

TABLE 3.10-17

TABULATION OF STRESSES - PSW PUMPS[(a)(b)]

Item	Material	Allowable Stress OBE	Allowable Stress DBE	Calculated Stress OBE	Calculated Stress DBE
Discharge head - shell	SA283 grade D	12,600	29,700	-	580
Discharge head - base	SA285 grade D	13,700	27,000	10,986	16,637
Subbase	SA283 grade D	12,600	29,700	11,823	18,164
Column pipe	SA106 grade B	15,000	31,500	4562	6549
Pipe - hub stress	SA106 grade B	22,500[(c)]	31,500	10,792	12,997
Column flange	SA285 grade C	20,550[(d)]	27,000	16,592	23,959
Lineshaft	A276-410A	12,000[(e)]	36,000	-	11,069
Bolting					
Motor-to-head	SA193-B7	35,000	94,500	-	3477
Head-to-sub base	SA193-B7	35,000	94,500	17,155	26,255
Column flange	SA193-B8	20,000[(f)]	27,000	15,577	22,398
Anchor bolts	NF	-	-	8147	17,812

a. Allowable stresses at OBE are taken from Tables I-7.3 and I-8.3, ASME Code Section III, except where noted.
b. Allowable stresses at DBE are 90% of YS.
c. In accordance with NB 3647.1 Section III. Allowable $S_H = 1.5\ S_m$.
d. In accordance with NB 3647.1 Section III. Allowable $S_R = 1.5\ S_m$.
e. Combined shear allowable = 30% of YS in accordance with ANSI B58.1.
f. In accordance with NB 3232.1. Allowable = two times the value listed in I-1.3.

TABLE 3.10-18

LATERAL PLANE TESTING

The test fixture was subjected to two beats at each of the following:

Beat Frequency (Hz)	Peak Acceleration (g)
0.5	0.6
1.0	0.8
2.0	1.0
3.0	1.5
4.0	4.0
5.0	6.4
6.0	6.4
7.0	6.4
8.0	6.4
9.0	6.4
10.0	6.4
11.0	6.4
12.0	6.4
13.0	6.4
14.0	6.4
15.0	6.4
16.0	6.4
18.0	3.6
20.0	2.0
22.0	1.0
24.0	1.0
26.0	1.0
28.0	1.0
30.0	1.0

TABLE 3.10-19

LONGITUDINAL PLANE TESTING

The test fixture was subjected to two beats at each of the following:

Beat Frequency (Hz)	Peak Acceleration (g)
0.5	0.6
1.0	0.8
2.0	1.0
3.0	1.5
4.0	4.0
5.0	6.4
6.0	6.4
7.0	6.4
8.0	6.4
9.0	6.4
10.0	6.4
11.0	6.4
12.0	6.4
13.0	6.4
14.0	6.4
15.0	6.4
16.0	6.4
18.0	3.6
20.0	2.0
22.0	1.0
24.0	1.0
26.0	1.0
28.0	1.0
30.0	1.0

TABLE 3.10-20

VERTICAL PLANE TESTING

The test fixture was subjected to two beats at each of the following:

Beat Frequency (Hz)	Peak Acceleration (g)
0.5	0.4
1.0	0.6
2.0	0.8
3.0	1.2
4.0	2.8
5.0	4.0
6.0	4.0
7.0	4.0
8.0	4.0
9.0	4.0
10.0	4.0
11.0	4.0
12.0	4.0
13.0	4.0
14.0	4.0
15.0	4.0
16.0	4.0
18.0	2.8
20.0	1.3
22.0	0.7
24.0	0.7
26.0	0.7
28.0	0.7
30.0	0.7

TABLE 3.10-21

CLASS 1E EQUIPMENT COMPRISING THE ATTS

Component	Manufacturer	Primary Class 1E Function	Qualification Environment[b]	Seismic Qualification Level	Resonant Frequency[c] (Hz)
Pressure transmitter	Barton	Provide current output response to pressure input	Reactor building	(d)	----
Differential pressure transmitter	Barton	Provide current output response to differential pressure input	Reactor building	(d)	----
Pressure transmitter	Rosemont[a]	Provide current output response to pressure input	Reactor building	(e)	----
Differential pressure transmitter	Rosemont[a]	Provide current output response to differential pressure input	Reactor building	(e)	----
Resistance temperature detector (RTD)	Weed	Provide current output response to temperature input	Reactor building	(f)	----
Pressure switch	PCI	Provide contact transfer at pressure trip point	Drywell	(f)	----
Trip units (master, slave, RTD, differential voltage)	GE	Provide trip function at the process variable trip point	Control room	(g)	----
Relay	Agastat	Contact transfer in response to trip unit trip	Control room	(g)	----
Voltage converter	Datametrics	Provide power to the ATTS cabinets and instrument loops	Control room	(g)	----

a. The Rosemont transmitters are also used in other applications.
b. For service environments, see tables 4-1 through 4-3 of NEDE-22154-1.
c. No resonant frequencies ≤ to 33 Hz were identified for any of the devices.
d. See figure 4-12 of NEDE-22154-1. The horizontal qualification levels for these devices are equal to half the acceleration levels defined in figure 4-12. This reduction is employed to account for the simulation of triaxial testing.
e. The Rosemont transmitters, which were not qualified as a part of the original ATTS qualification program, are qualified to seismic levels that exceed the seismic requirements at the transmitter location.
f. See figure 4-11 of NEDE-22154-1. The horizontal qualification levels for these devices are equal to half the acceleration levels defined in figure 4-11. This reduction is employed to account for the simulation of triaxial testing.
g. See figures 4-5 and 4-6 of NEDE-22154-1 for the seismic qualification levels for the cabinets in which these devices are mounted.

TABLE 3.10-22 (SHEET 1 OF 2)

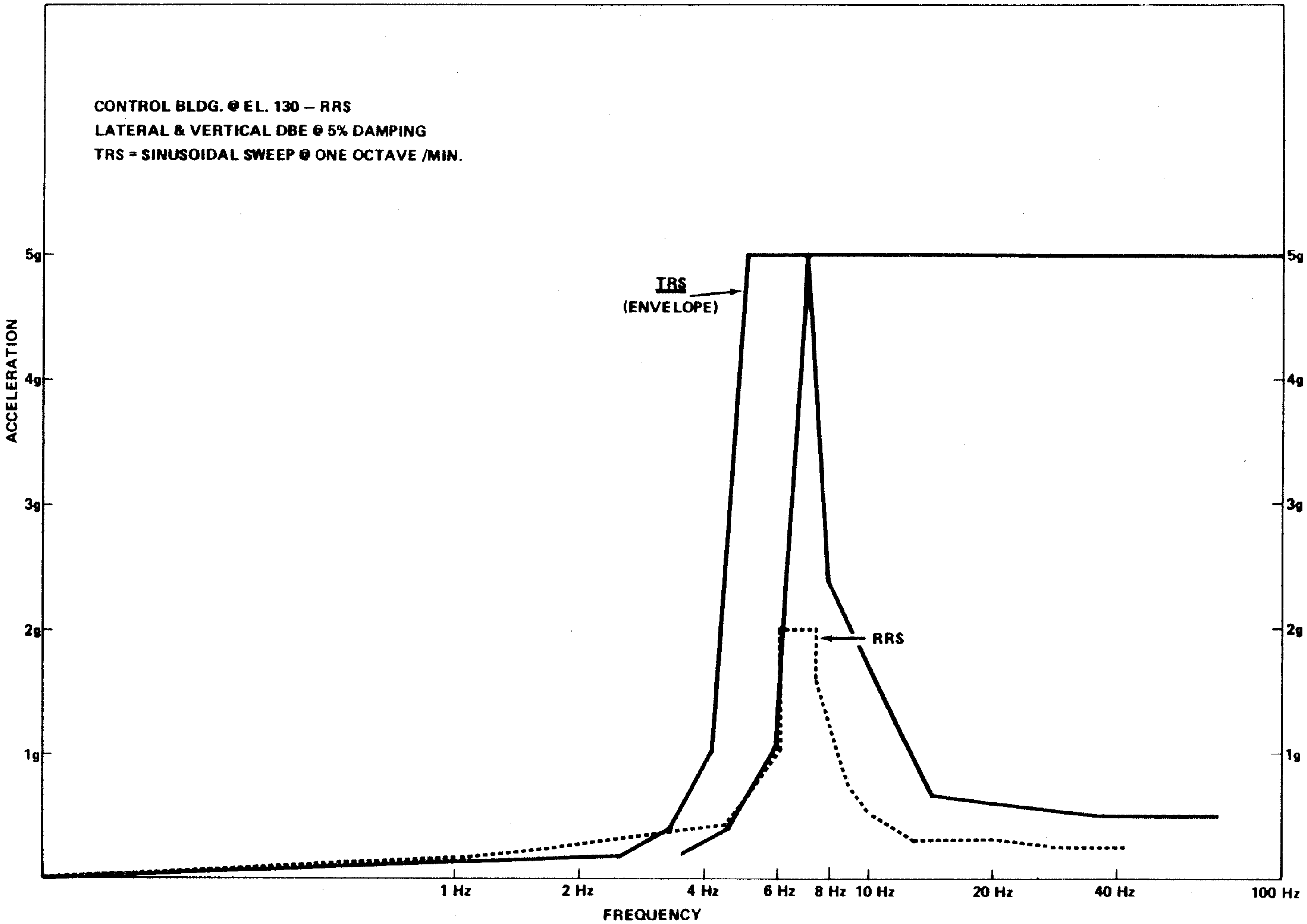
SEISMIC QUALIFICATION TEST SUMMARY FOR ATTS CONTROL PANELS AND LOCAL RACKS

Control Panel No.	Description	Type	Class 1E Equipment Description	Comments
H11-P921	RPS cabinet	Control panel	Agastat relays, GE trip units, Datametrics voltage converters	Seismic test completed
H11-P922	RPS cabinet	Control panel	Agastat relays, GE trip units, Datametrics voltage converters	Seismic test completed
H11-P923	RPS cabinet	Control panel	Agastat relays, GE trip units, Datametrics voltage converters	Seismic test completed
H11-P924	RPS cabinet	Control panel	Agastat relays, GE trip units, Datametrics voltage converters	Seismic test completed
H11-P925	ECCS cabinet	Control panel	Agastat relays, GE trip units, Datametrics voltage converters	Seismic test completed
H11-P926	ECCS cabinet	Control panel	Agastat relays, GE trip units, Datametrics voltage converters	Seismic test completed
H11-P927	ECCS cabinet	Control panel	Agastat relays, GE trip units, Datametrics voltage converters	Seismic test completed
H11-P928	ECCS cabinet	Control panel	Agastat relays, GE trip units, Datametrics voltage converters	Seismic test completed
H21-P016	CS/HPCI leak detector	Local rack	Process transmitter	Seismic test completed
H21-P036	HPCI leak detector	Local rack	Process transmitter	Seismic test completed
H21-P038	RCIC leak detector	Local rack	Process transmitter	Seismic test completed
H21-P401	CS system	Local rack	Process transmitter	Seismic test completed
H21-P402	RWC system	Local rack	Process transmitter	Seismic test completed
H21-P404A	Reactor pressure/level	Local rack	Process transmitter	Seismic test completed
H21-P404B	Reactor pressure/level	Local rack	Process transmitter	Seismic test completed
H21-P404C	Reactor pressure/level	Local rack	Process transmitter	Seismic test completed
H21-P404D	Reactor pressure/level	Local rack	Process transmitter	Seismic test completed

TABLE 3.10-22 (SHEET 2 OF 2)

Control Panel No.	Description	Type	Class 1E Equipment Description	Comments
H21-P405A	Reactor pressure/level	Local rack	Process transmitter	Seismic test completed
H21-P405B	Reactor pressure/level	Local rack	Process transmitter	Seismic test completed
H21-P405C	Reactor pressure/level	Local rack	Process transmitter	Seismic test completed
H21-P405D	Reactor pressure/level	Local rack	Process transmitter	Seismic test completed
H21-P409	Jet pump	Local rack	Process transmitter	Seismic test completed
H21-P410	Jet pump	Local rack	Process transmitter	Seismic test completed
H21-P414A	HPCI system	Local rack	Process transmitter	Seismic test completed
H21-P414B	HPCI system	Local rack	Process transmitter	Seismic test completed
H21-P415A	Main steam line flow	Local rack	Process transmitter	Seismic test completed
H21-P415B	Main steam line flow	Local rack	Process transmitter	Seismic test completed
H21-P417A	RCIC system	Local rack	Process transmitter	Seismic test completed
H21-P417B	RCIC system	Local rack	Process transmitter	Seismic test completed
H21-P418A	RHR system	Local rack	Process transmitter	Seismic test completed
H21-P418B	RHR system	Local rack	Process transmitter	Seismic test completed
H21-P419	CS system	Local rack	Process transmitter	Seismic test completed
H21-P425A	Main steam line flow	Local rack	Process transmitter	Seismic test completed
H21-P425B	Main steam line flow	Local rack	Process transmitter	Seismic test completed
H21-P434	HPCI system	Local rack	Process transmitter	Seismic test completed
H21-P435	RCIC leak detection	Local rack	Process transmitter	Seismic test completed
H21-P437	RCIC leak detection	Local rack	Process transmitter	Seismic test completed

a. Emergency core cooling system.
b. Reactor core isolation cooling.



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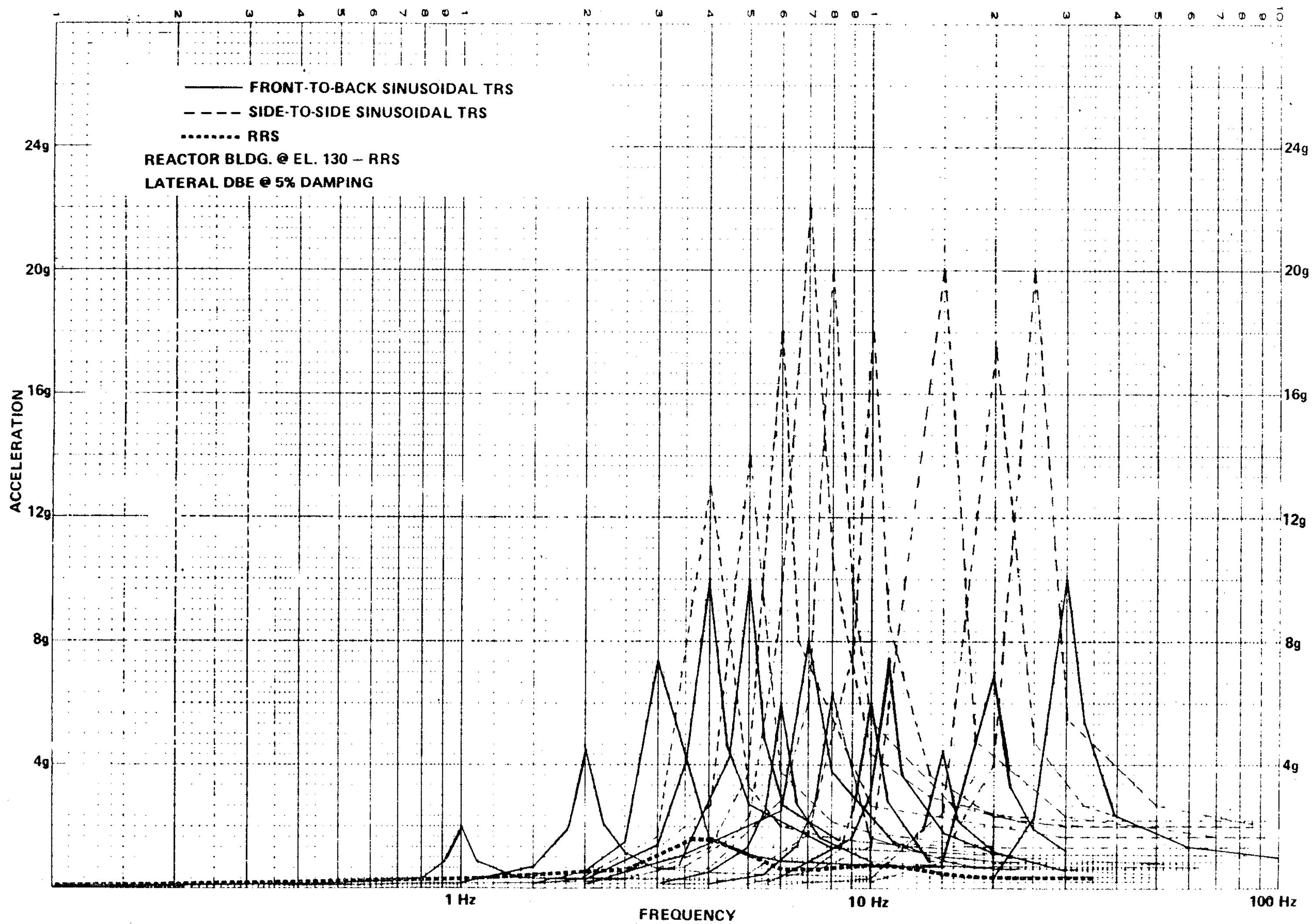
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

TRS VERSUS RRS – CONTROL BUILDING
el 130 (LATERAL AND VERTICAL)

FIGURE 3.10-1



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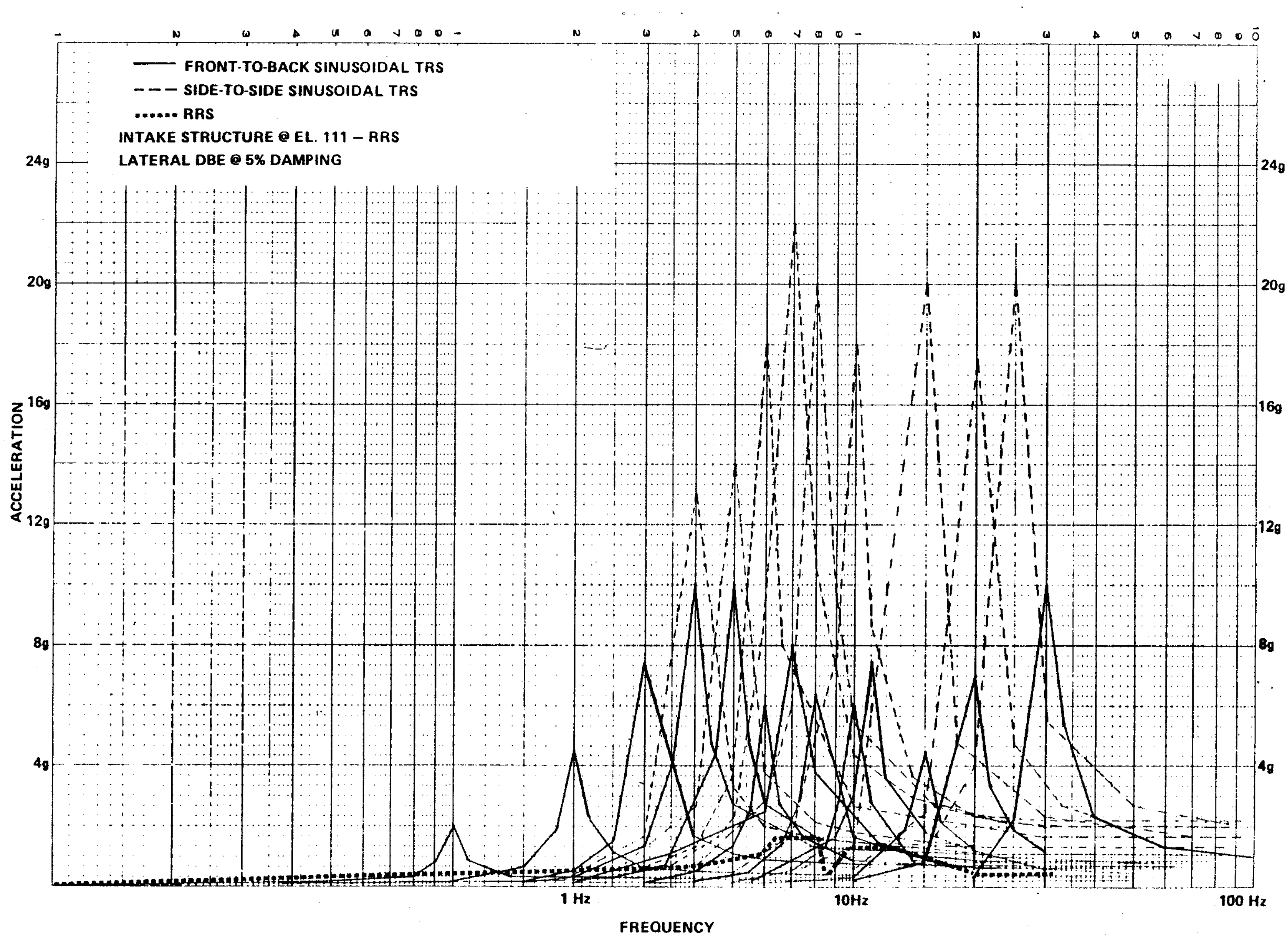
TRS VERSUS RRS – REACTOR BUILDING
el 130 (LATERAL)

FIGURE 3.10-2



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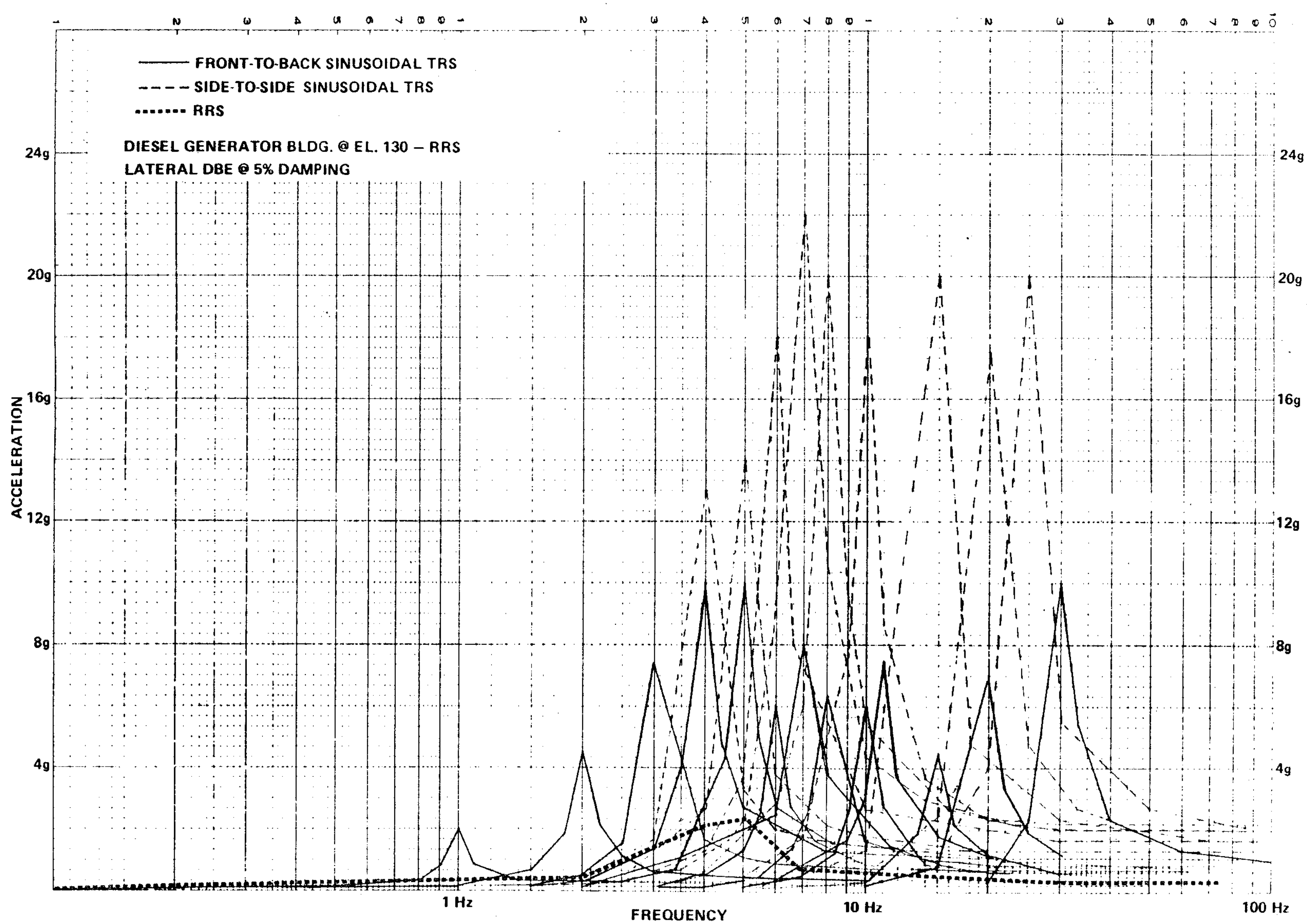
TRS VERSUS RRS – INTAKE STRUCTURE
el 111 (LATERAL)

FIGURE 3.10-3



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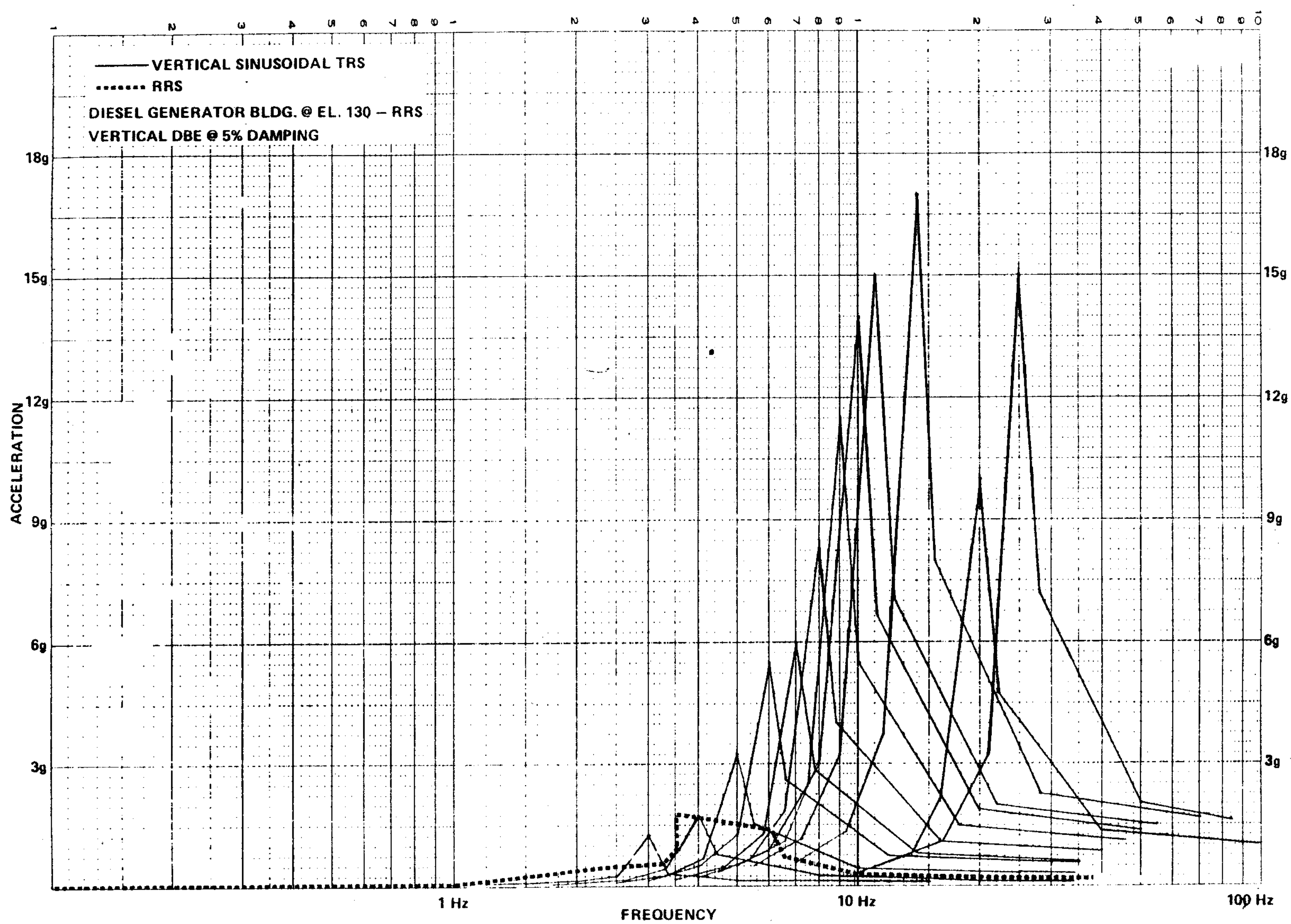
TRS VS RRS – DIESEL GENERATOR BUILDING
el 130 (LATERAL)

FIGURE 3.10-4



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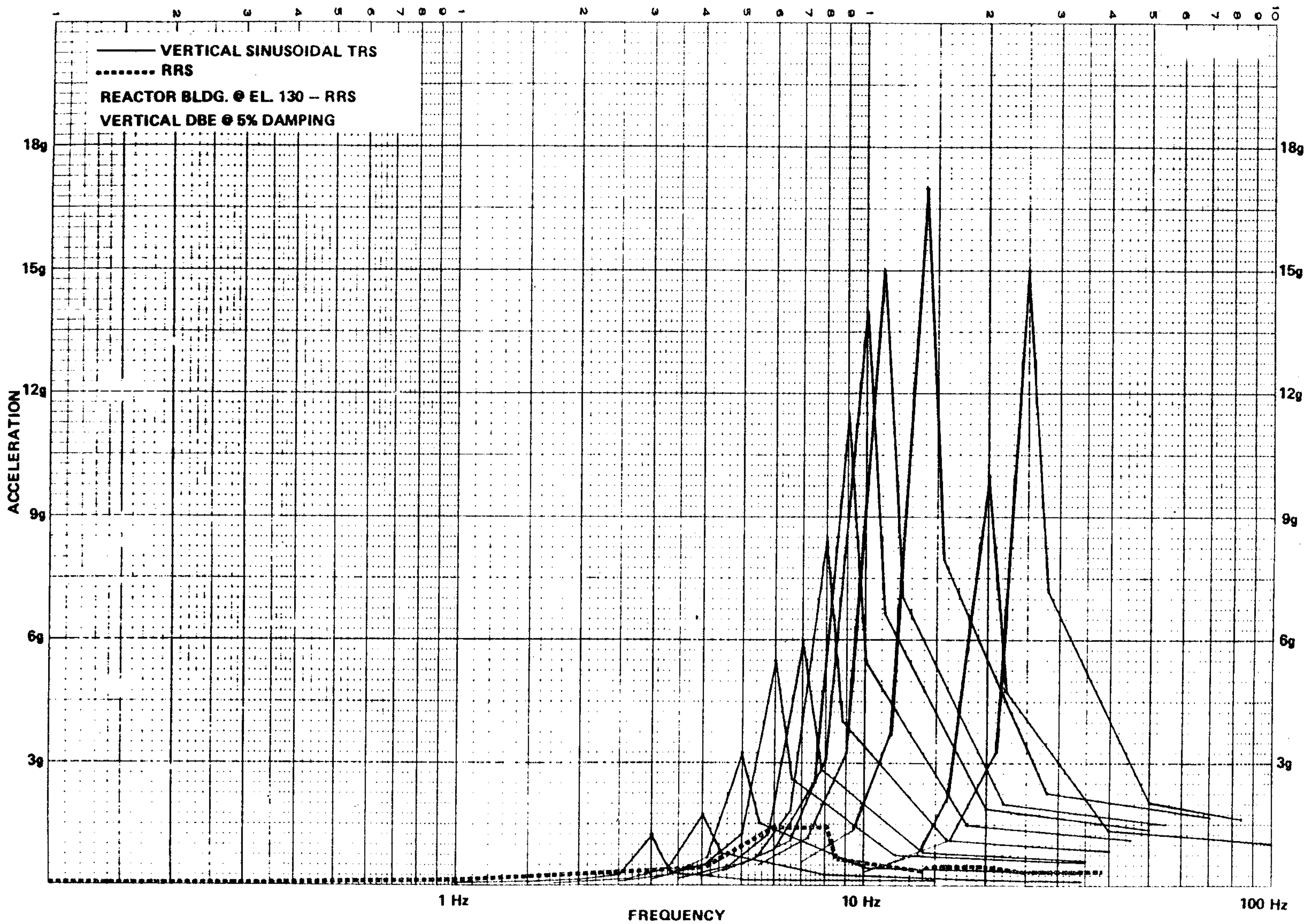
TRS VS RRS - DIESEL GENERATOR BUILDING
el 130 (LATERAL)

FIGURE 3.10-5



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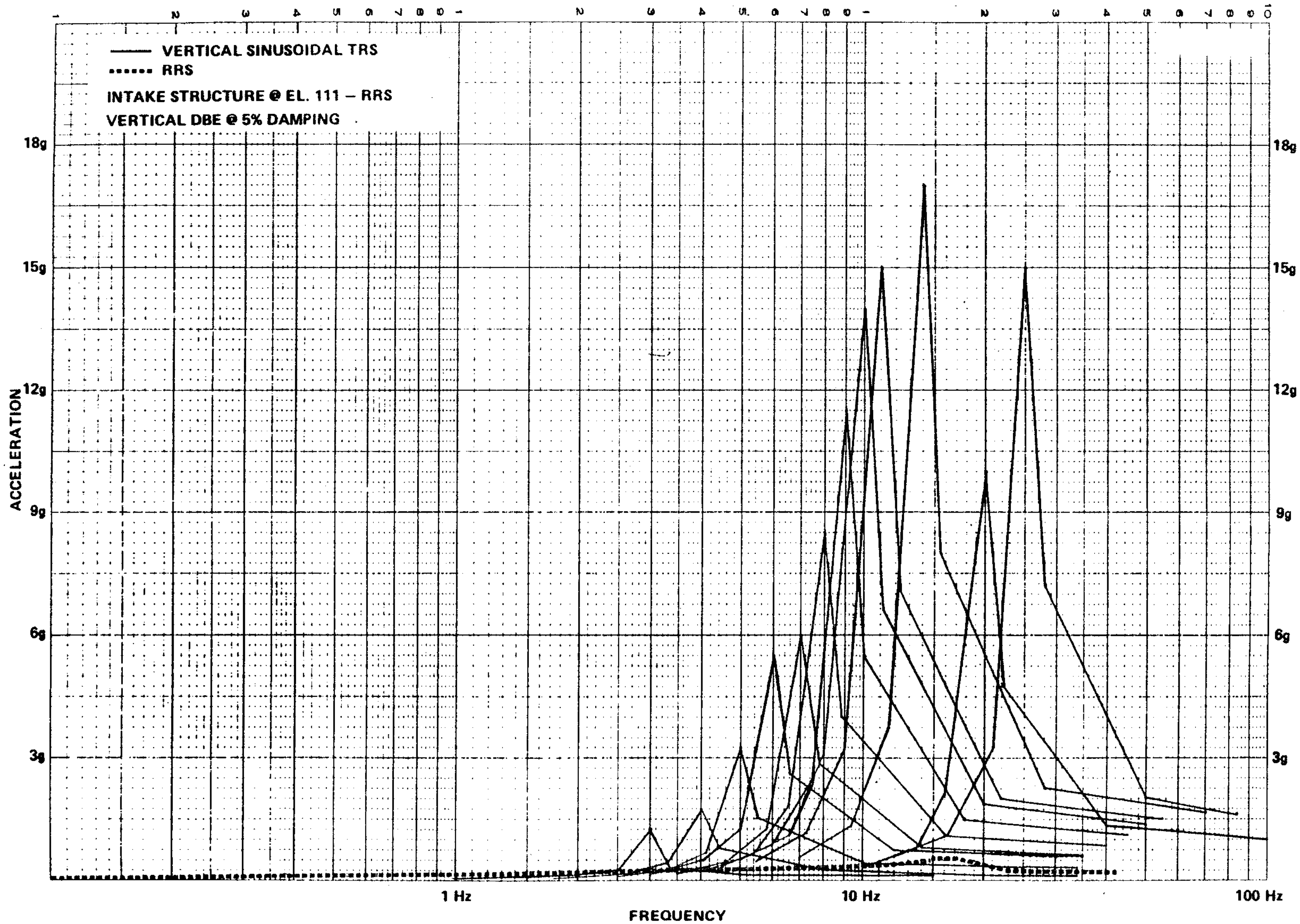
TRS VS RRS – REACTOR BUILDING
el 130 (VERTICAL)

FIGURE 3.10-6



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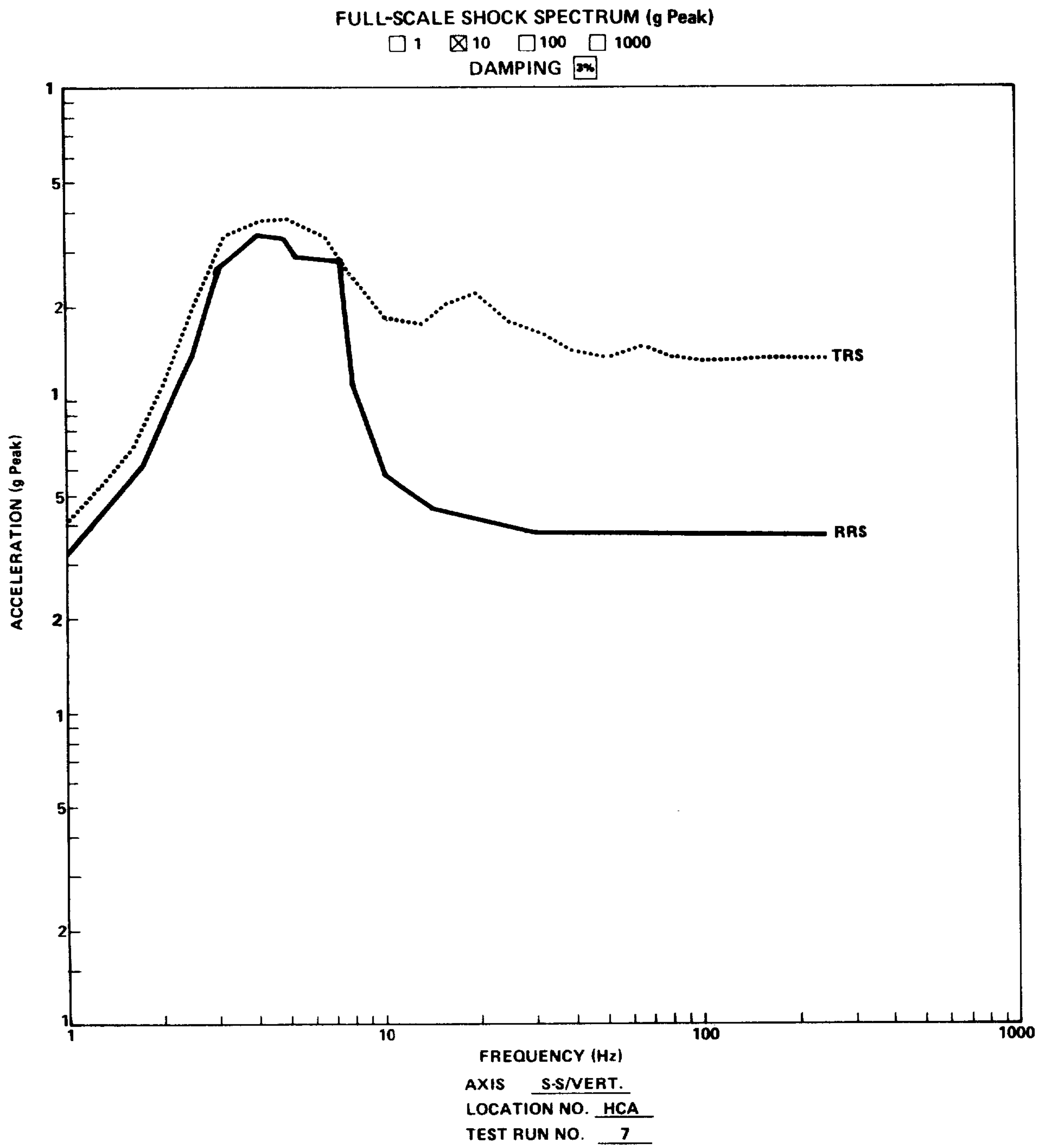
es 111 (VERTICAL)

FIGURE 3.10-7



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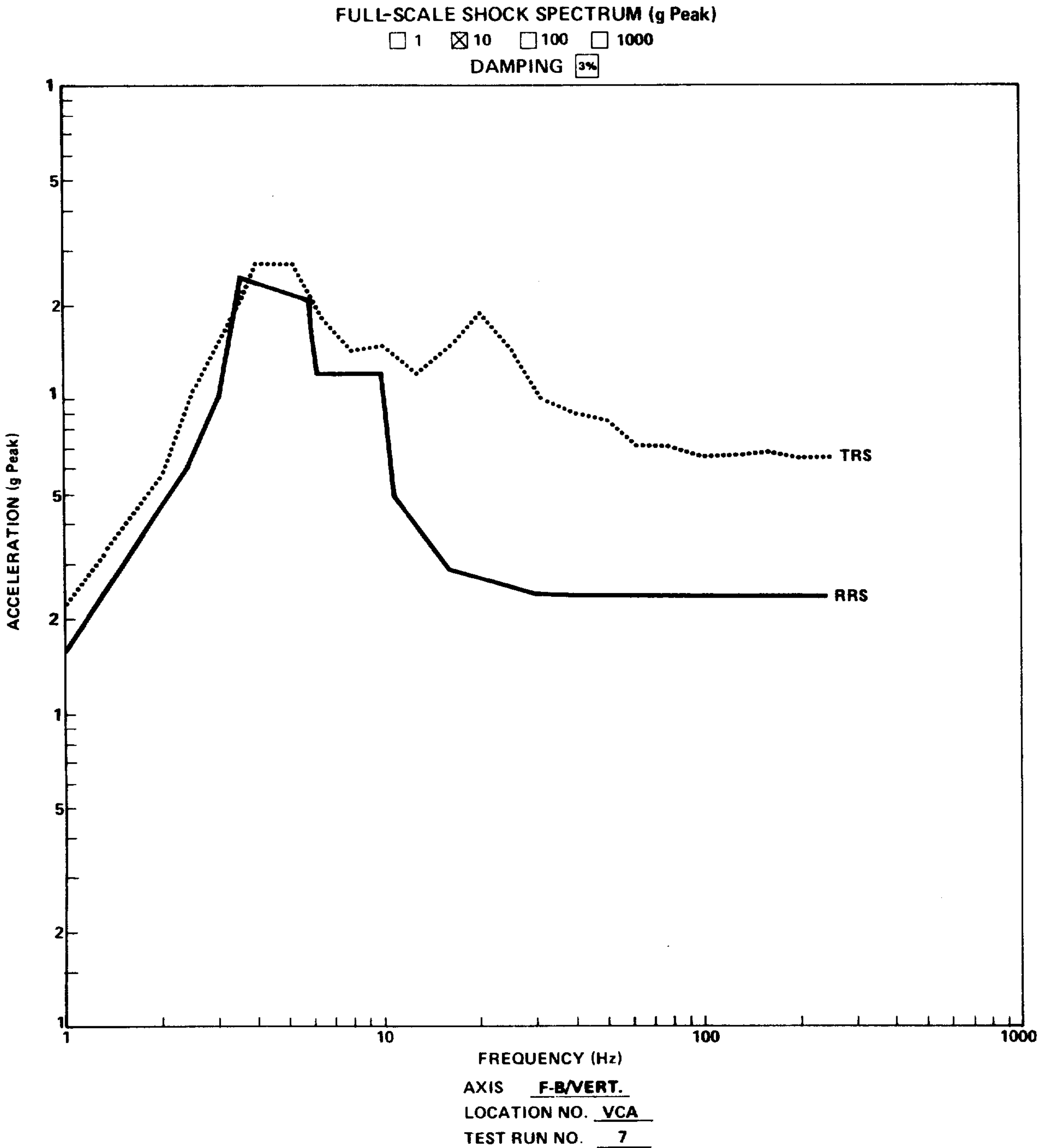
UNIT 2

TRS VS RRS – BATTERY CHARGER
FULL-SCALE SHOCK SPECTRUM

FIGURE 3.10-8 (SHEET 1 OF 4)



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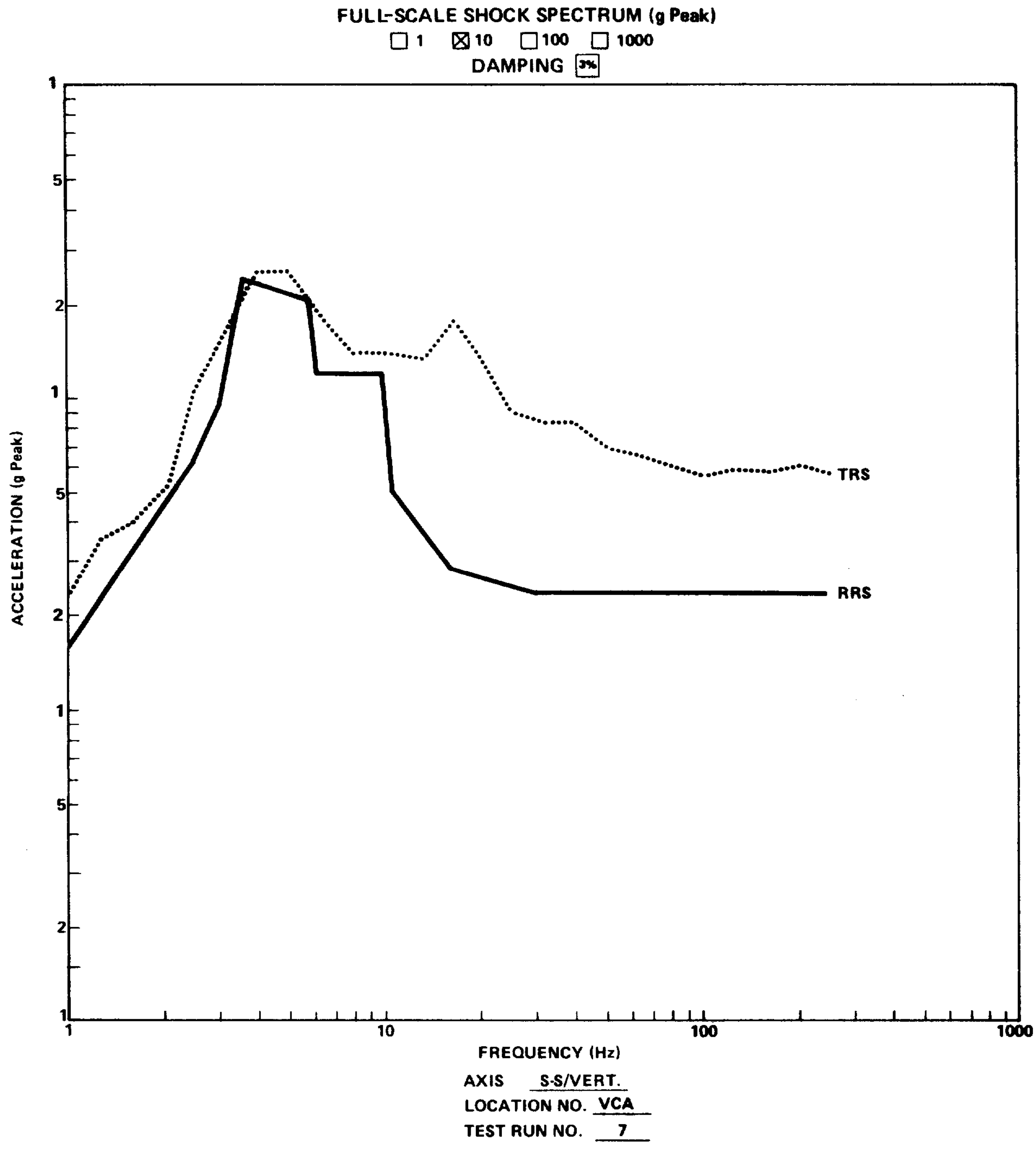
TRS VS RRS – BATTERY CHARGER
FULL-SCALE SHOCK SPECTRUM

FIGURE 3.10-8 (SHEET 2 OF 4)



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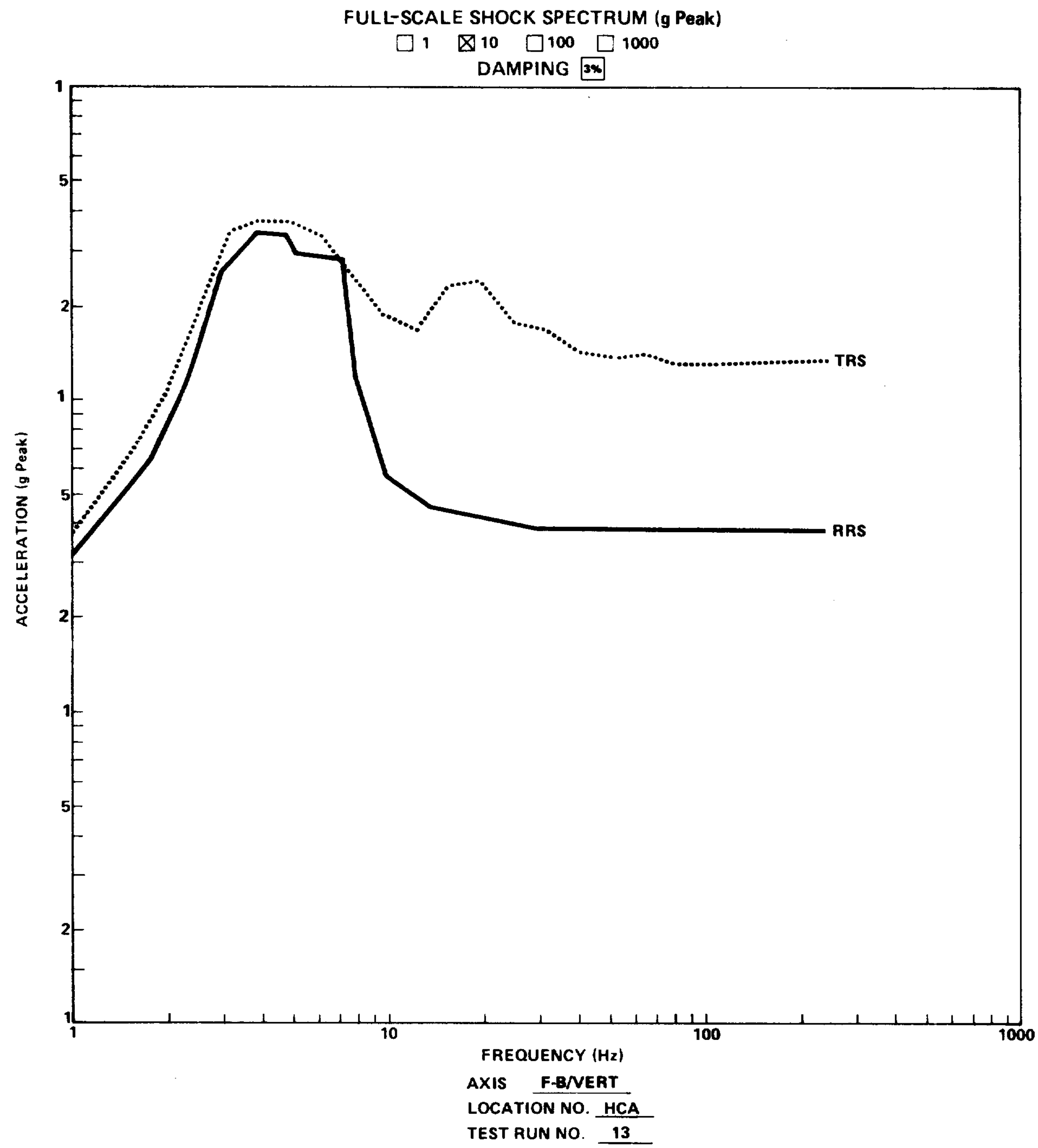
FULL-SCALE SHOCK SPECTRUM

FIGURE 3.10-8 (SHEET 3 OF 4)



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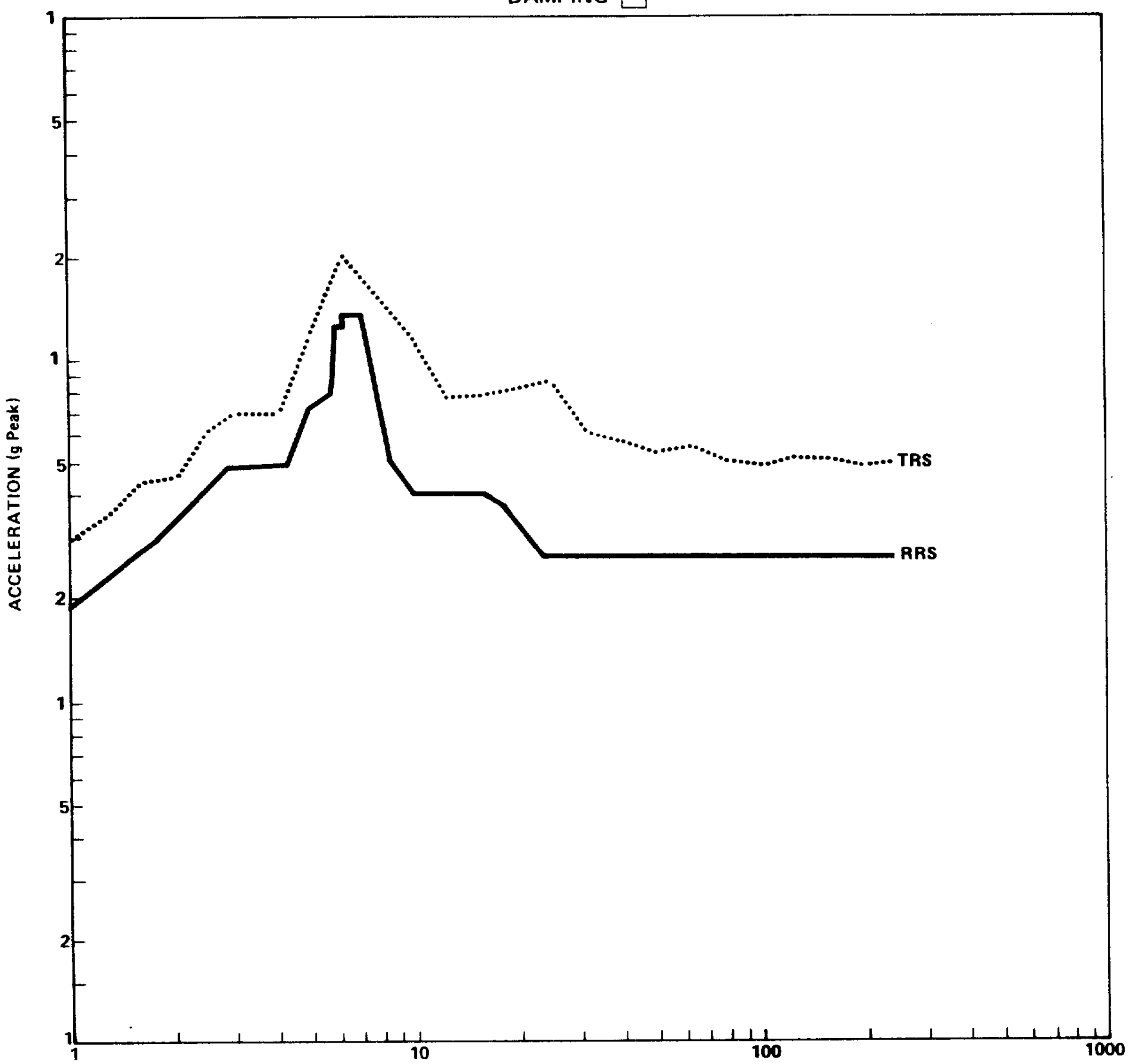
TRS VS RRS – BATTERY CHARGER
FULL-SCALE SHOCK SPECTRUM

FIGURE 3.10-8 (SHEET 4 OF 4)

FULL-SCALE SHOCK SPECTRUM (g Peak)

☐ 1 ☒ 10 ☐ 100 ☐ 1000

DAMPING 5%



AXIS S-S/VERT.

LOCATION NO. HCA

TEST RUN NO. 7

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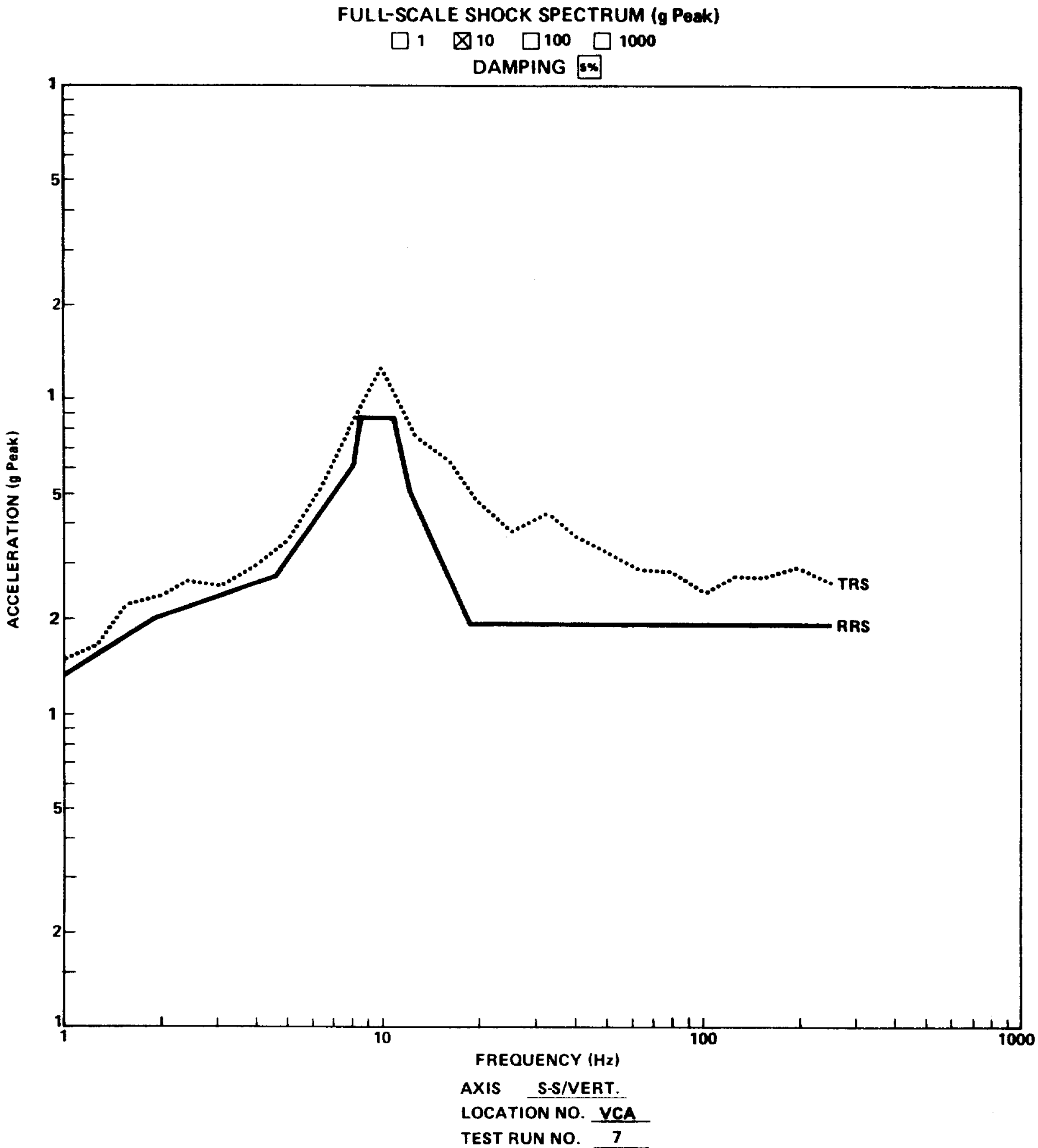
TRS VS RRS – 100-kW INVERTERS
FULL-SCALE SHOCK SPECTRUM

FIGURE 3.10-9 (SHEET 1 OF 4)



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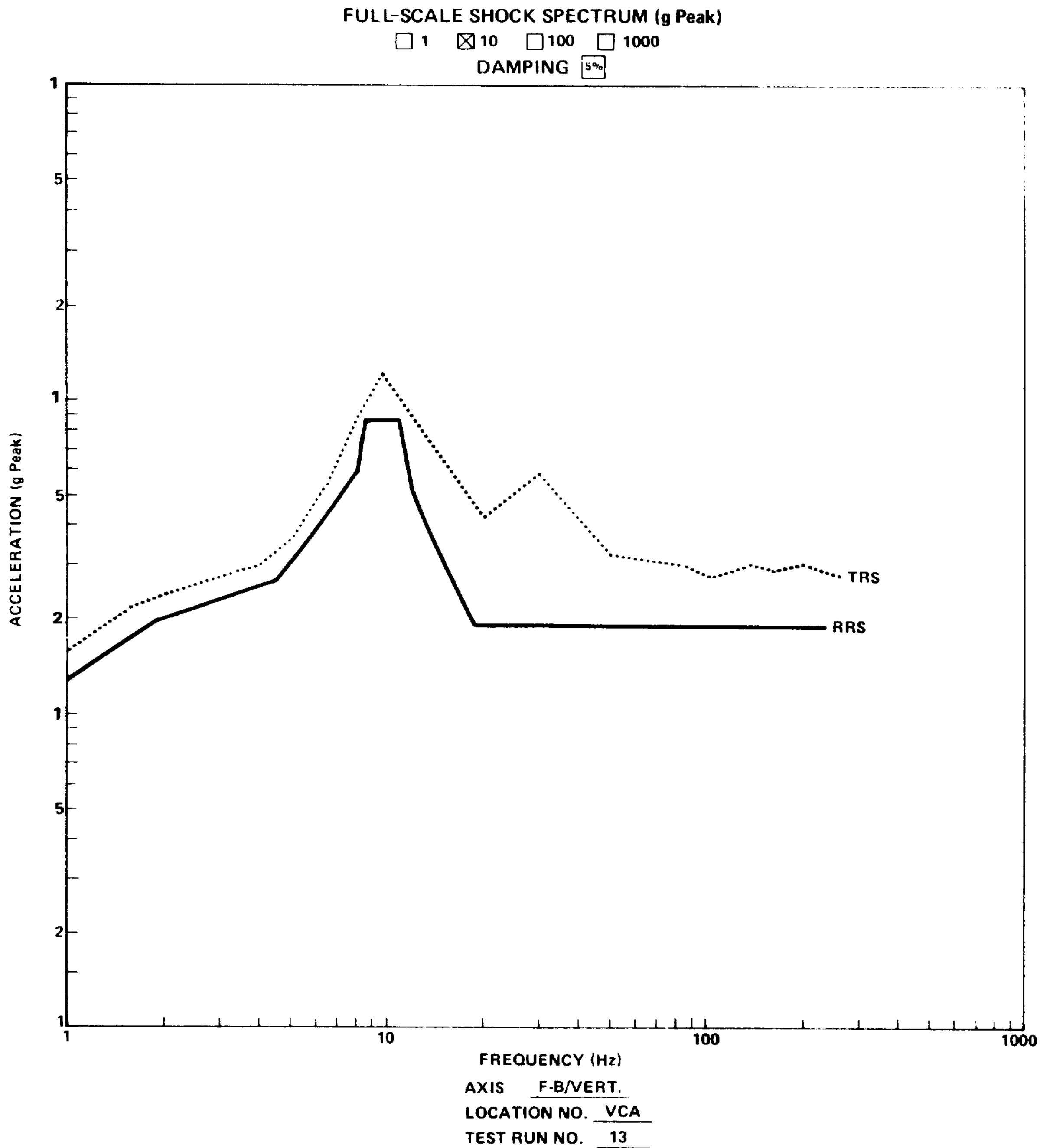
UNIT 2

TRS VS RRS – 100-kW INVERTERS
FULL-SCALE SHOCK SPECTRUM

FIGURE 3.10-9 (SHEET 2 OF 4)



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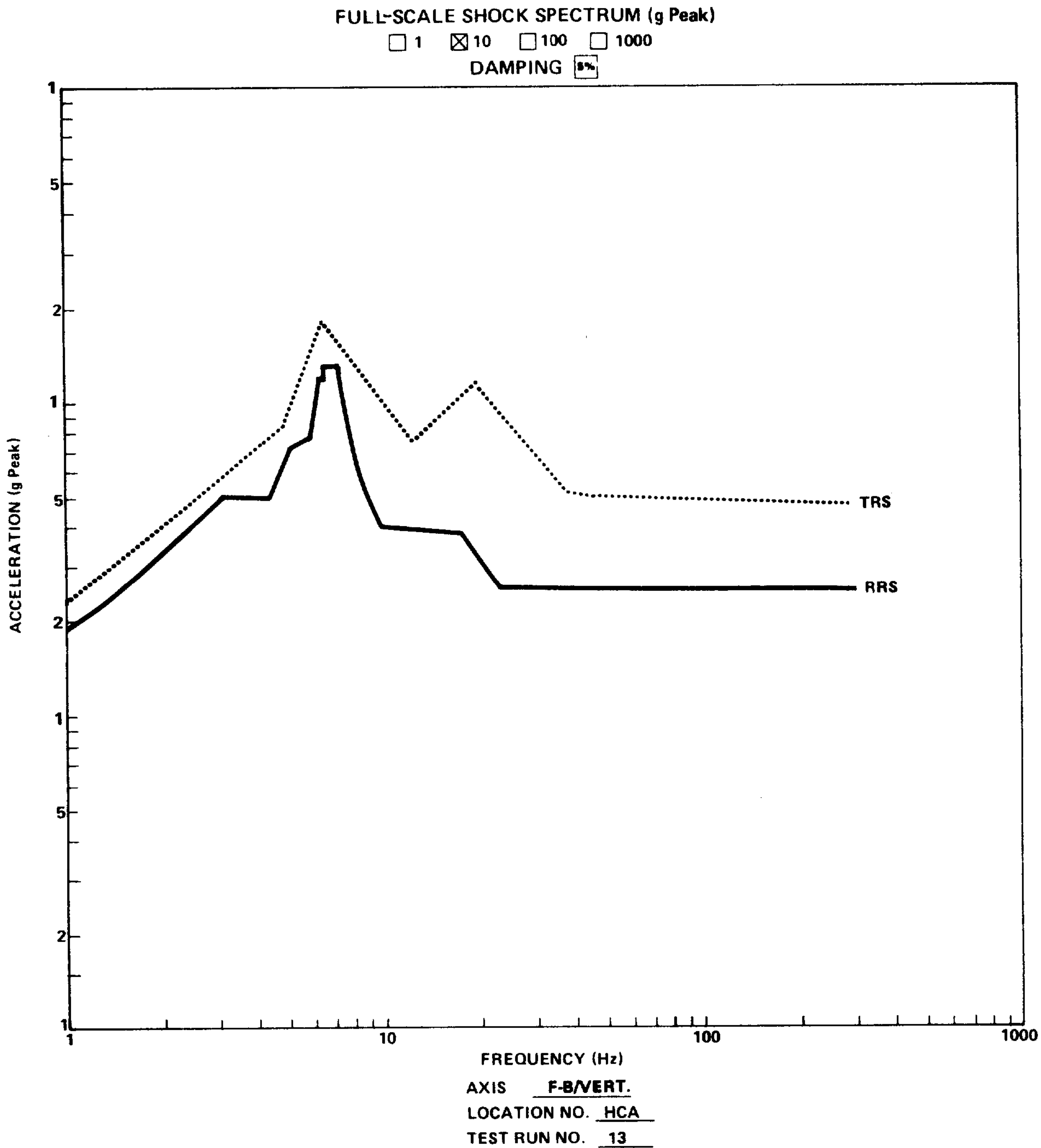
UNIT 2

TRS VS RRS – 100-kW INVERTERS
FULL-SCALE SHOCK SPECTRUM

FIGURE 3.10-9 (SHEET 3 OF 4)



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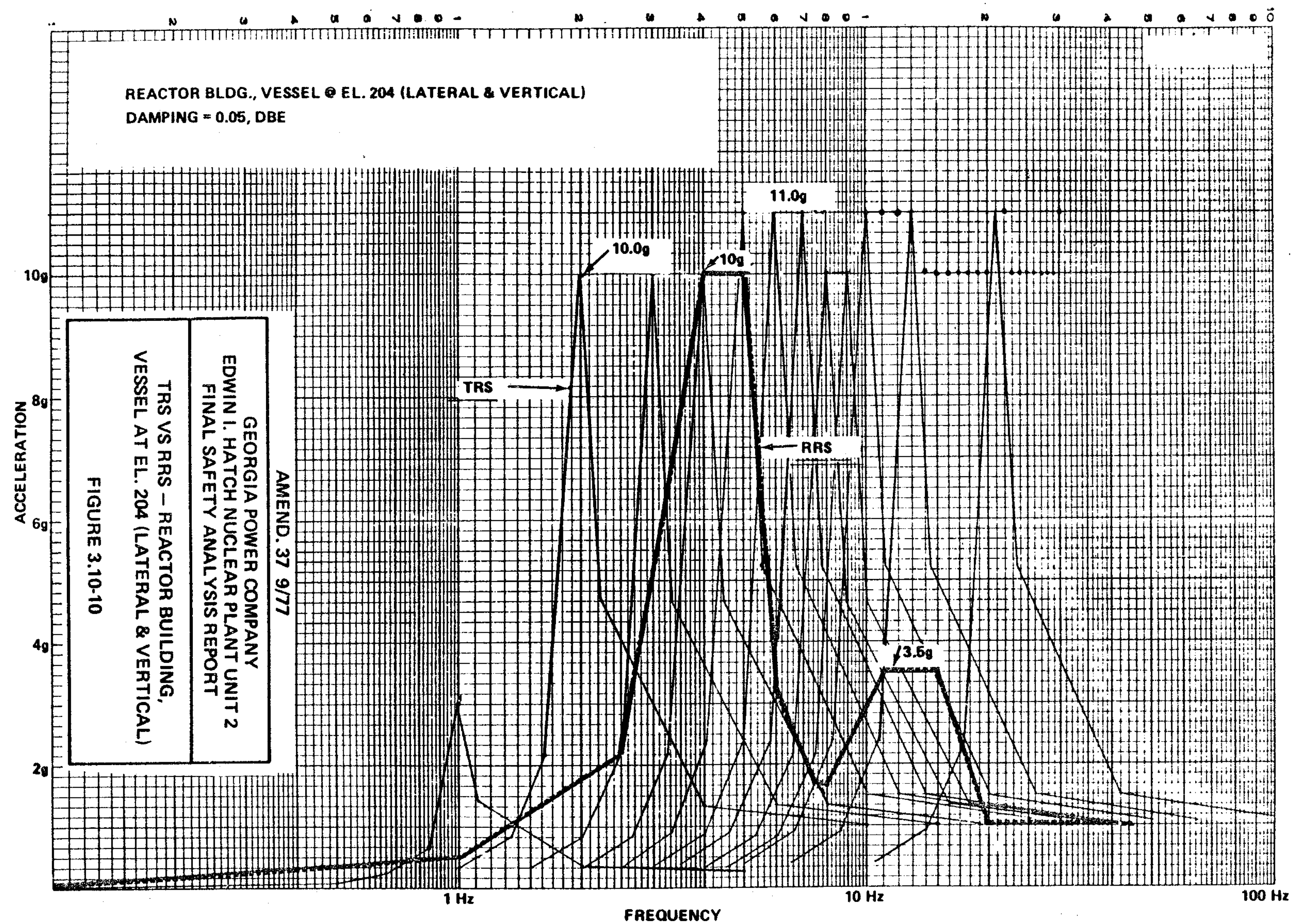
UNIT 2

TRS VS RRS – 100-kW INVERTERS
FULL-SCALE SHOCK SPECTRUM

FIGURE 3.10-9 (SHEET 4 OF 4)



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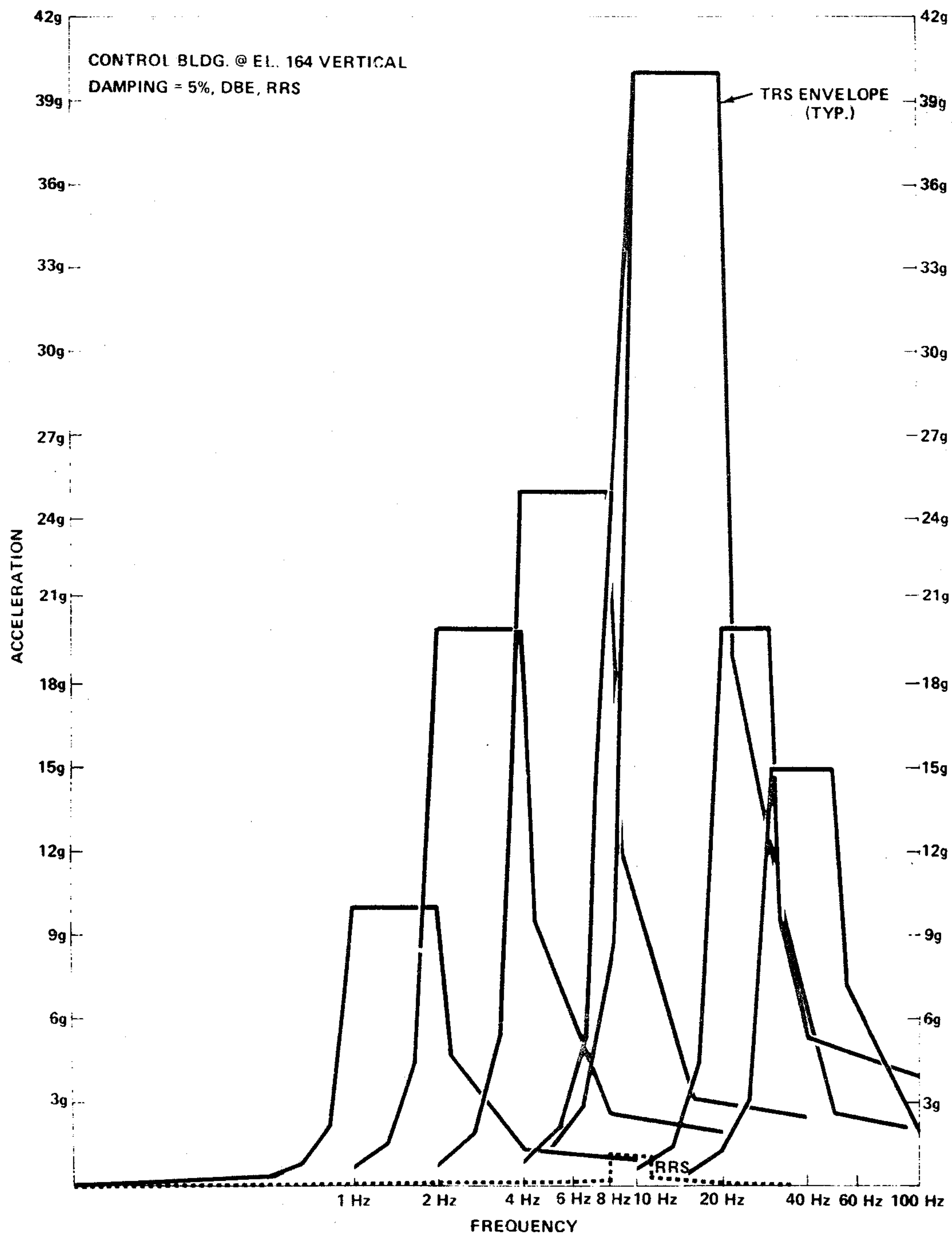
UNIT 2

TRS VS RRS – REACTOR BUILDING VESSEL AT el 204
(LATERAL AND VERTICAL)

FIGURE 3.10-10



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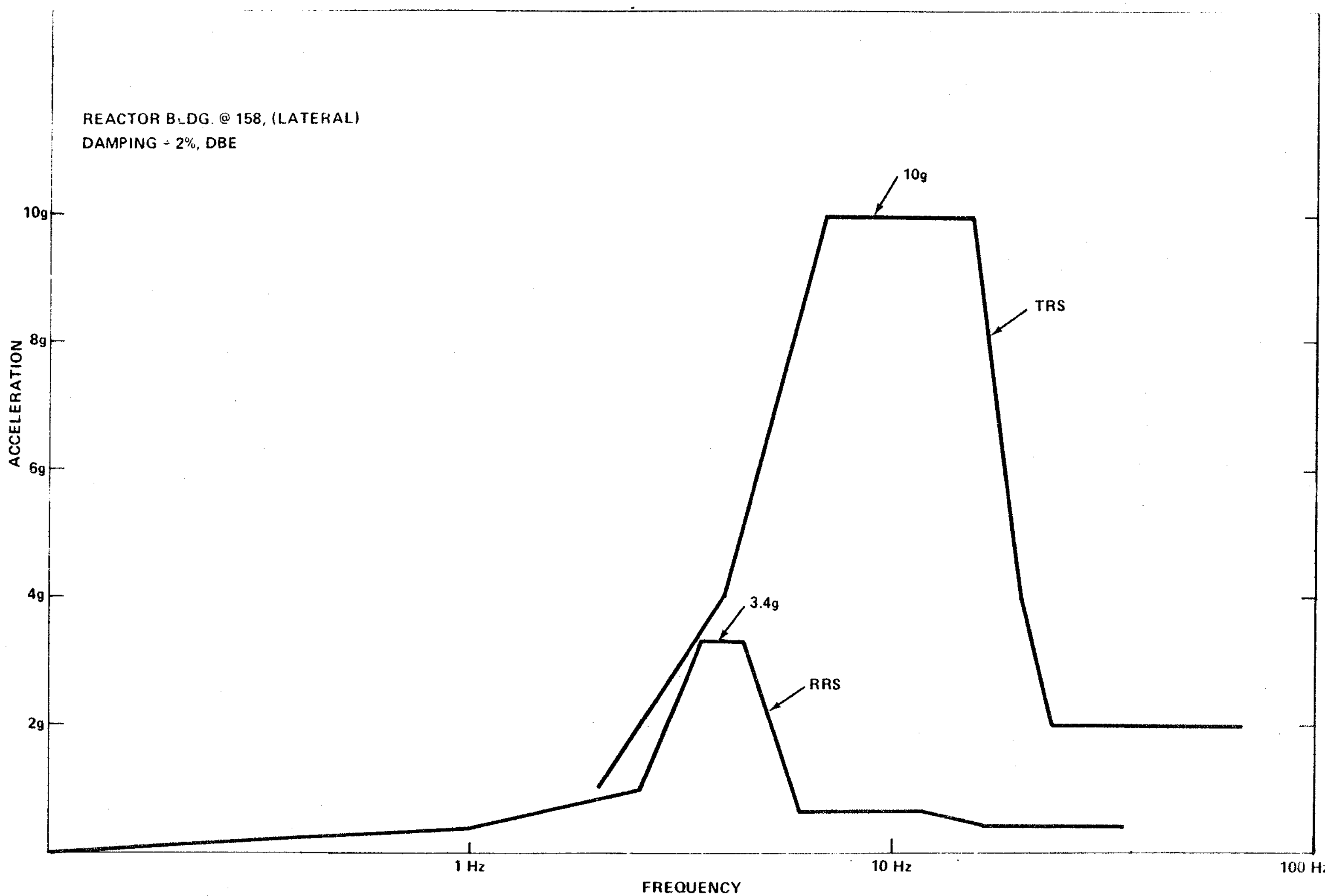
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

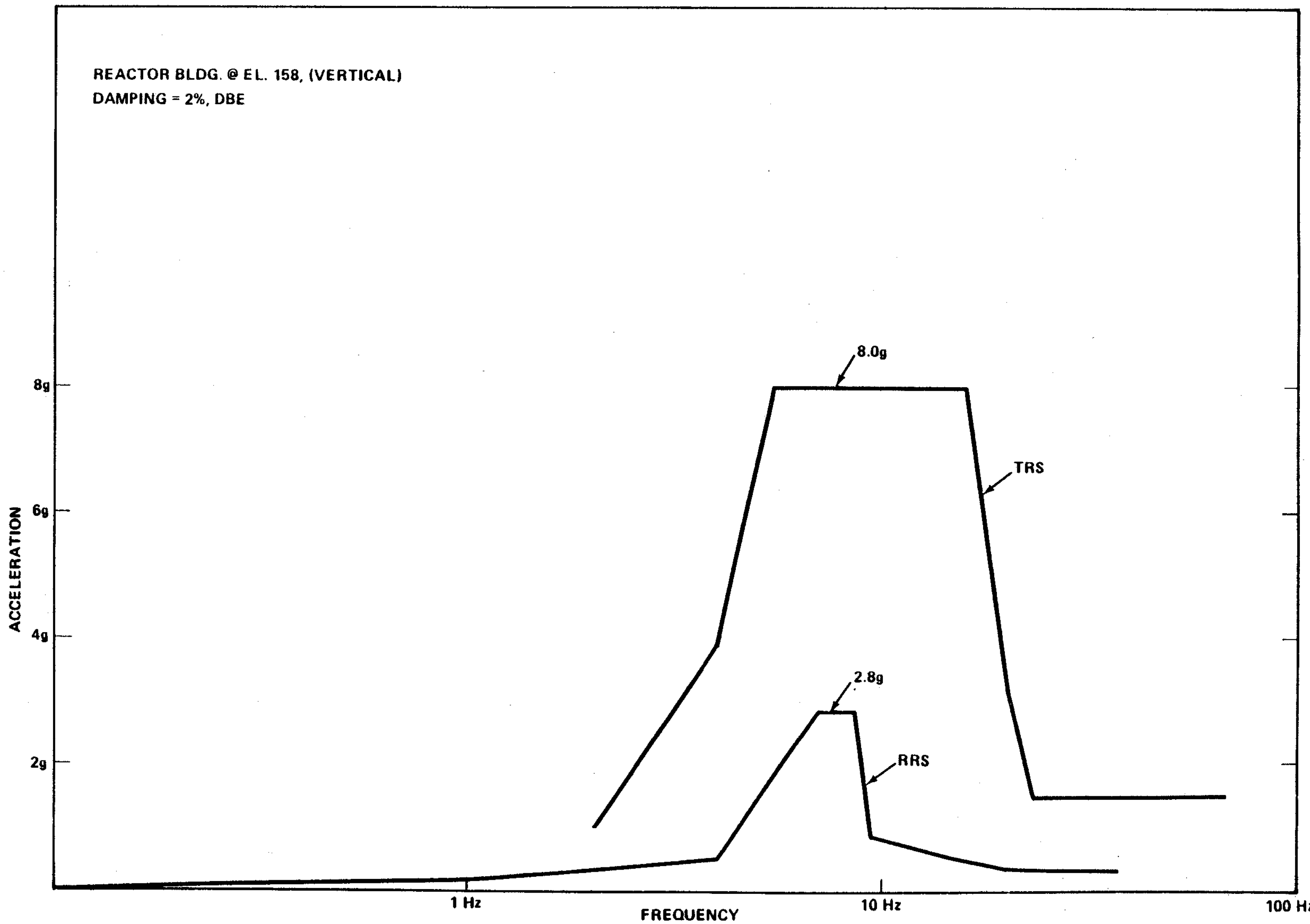
TRS VERSUS RRS – CONTROL BUILDING
el 164 (VERTICAL) (5-PERCENT DAMPING)

FIGURE 3.10-11



REV 19 7/01

SOUTHERN COMPANY *Energy to Serve Your World®*	SOUTHERN NUCLEAR OPERATING COMPANY EDWIN I. HATCH NUCLEAR PLANT UNIT 2	TRS VS RRS – REACTOR BUILDING el 158 (LATERAL) FIGURE 3.10-12



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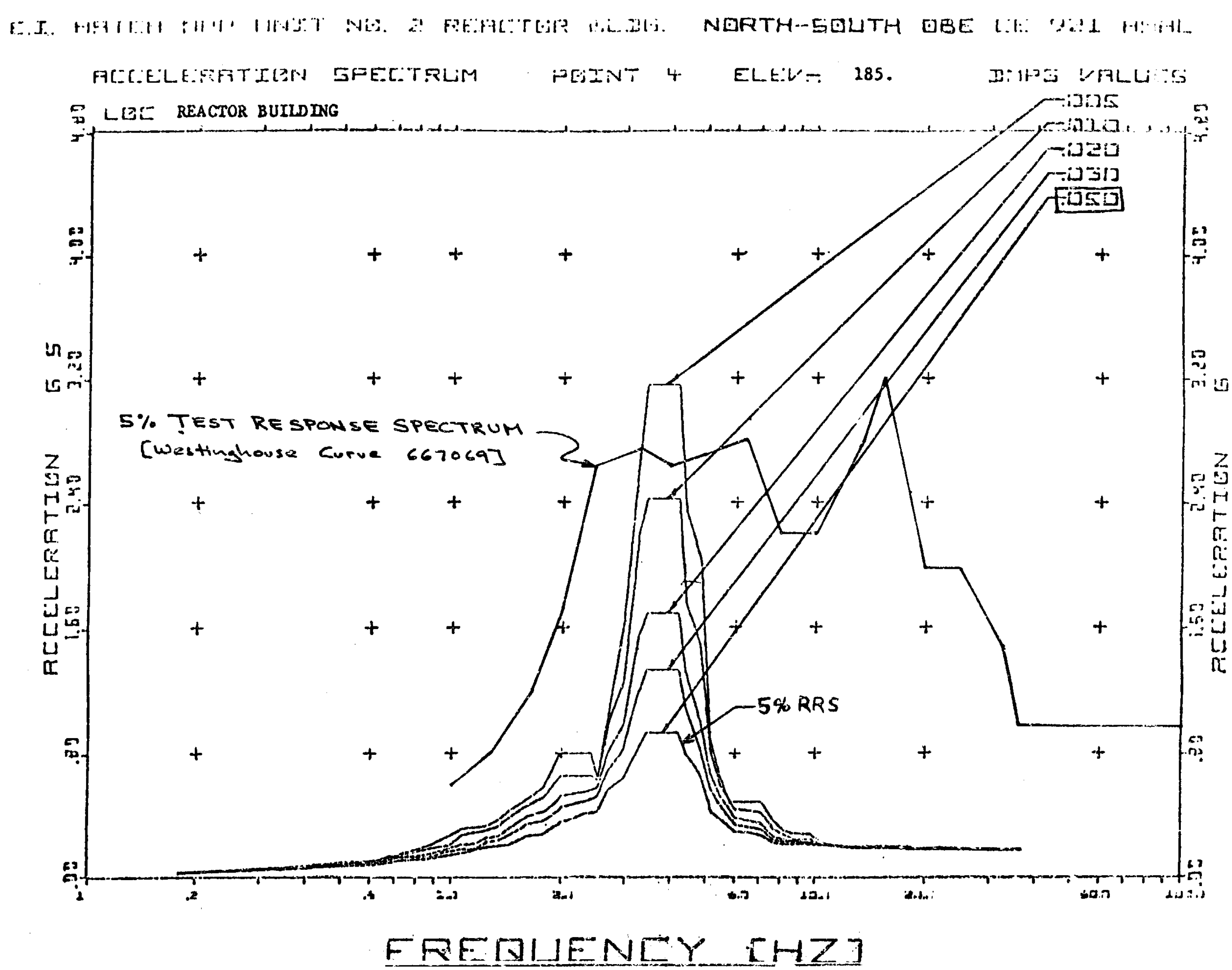
UNIT 2

TRS VS RRS – REACTOR BUILDING
el 158 (VERTICAL)

FIGURE 3.10-13



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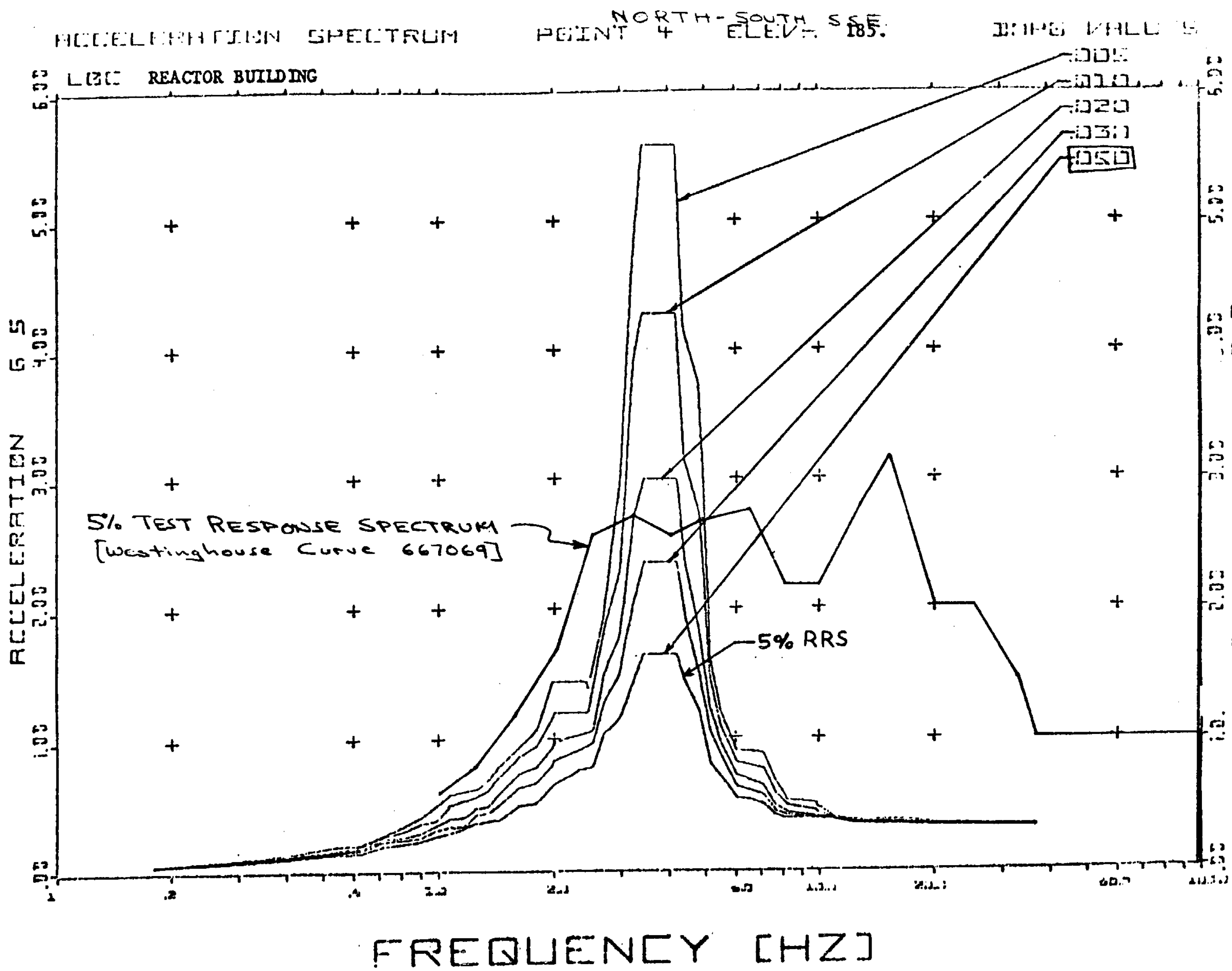
TRS VS RRS
RPT BREAKERS

FIGURE 3.10-14 (SHEET 1 OF 6)



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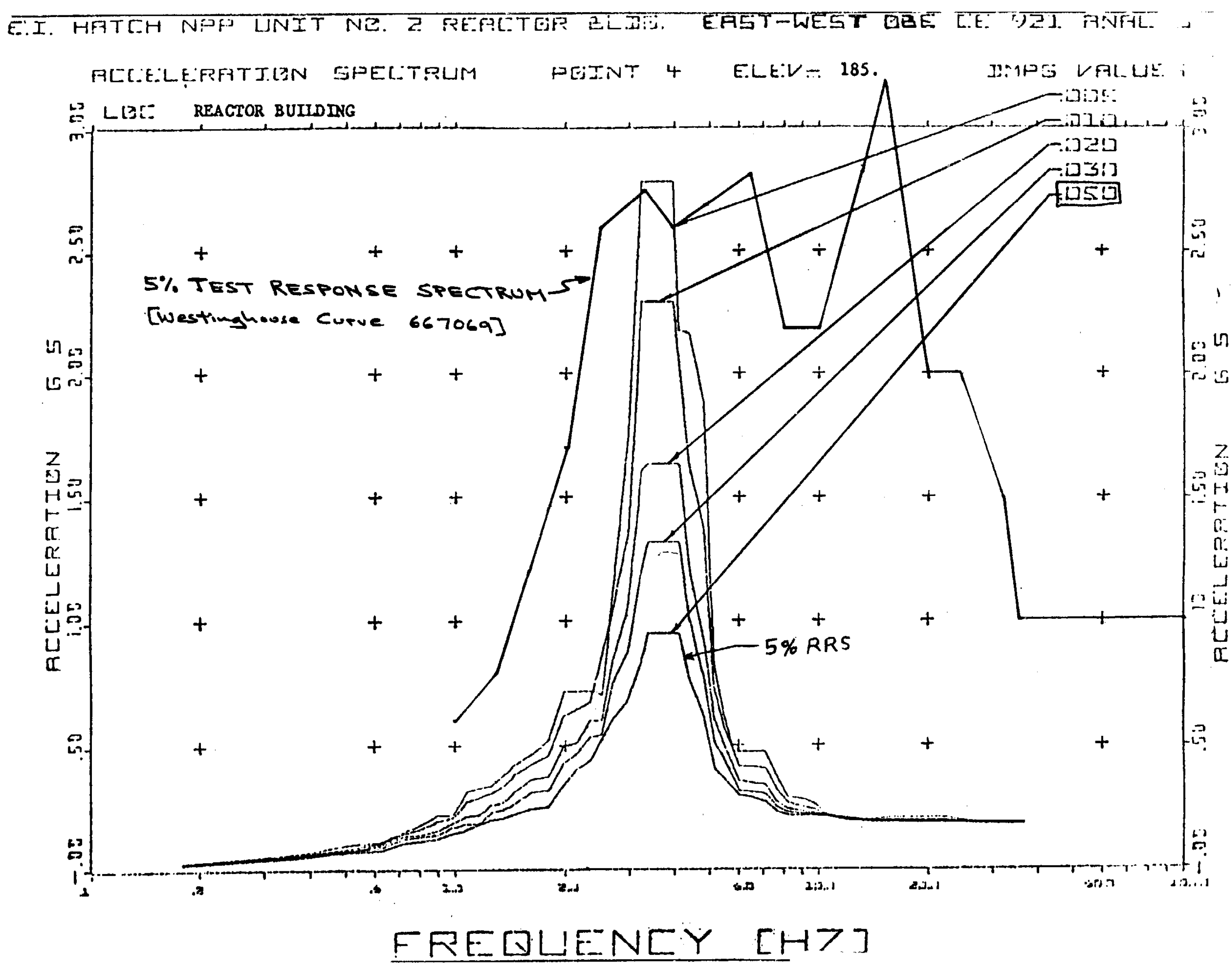
TRS VS RRS
RPT BREAKERS

FIGURE 3.10-14 (SHEET 2 OF 6)



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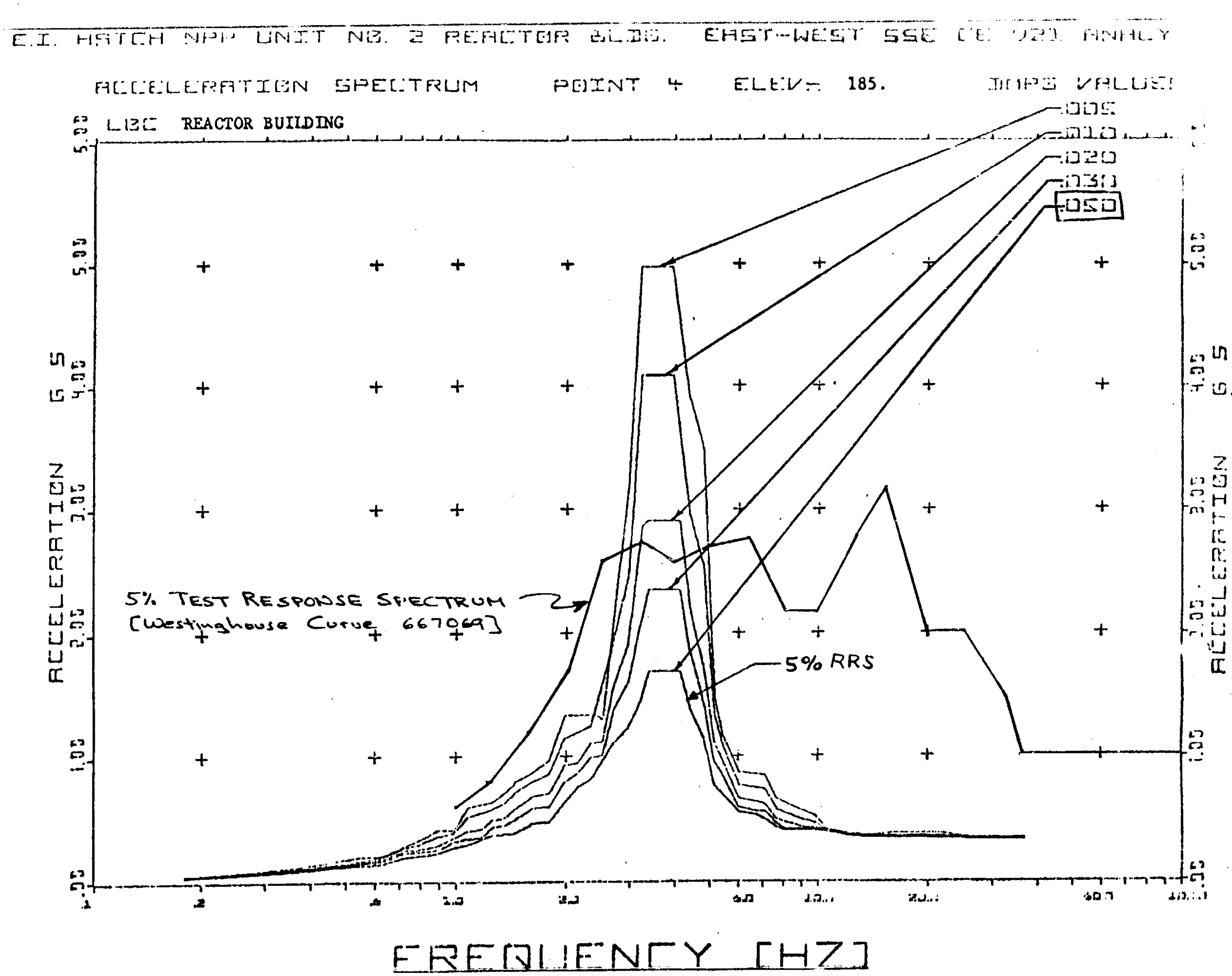
UNIT 2

TRS VS RRS
RPT BREAKERS

FIGURE 3.10-14 (SHEET 3 OF 6)



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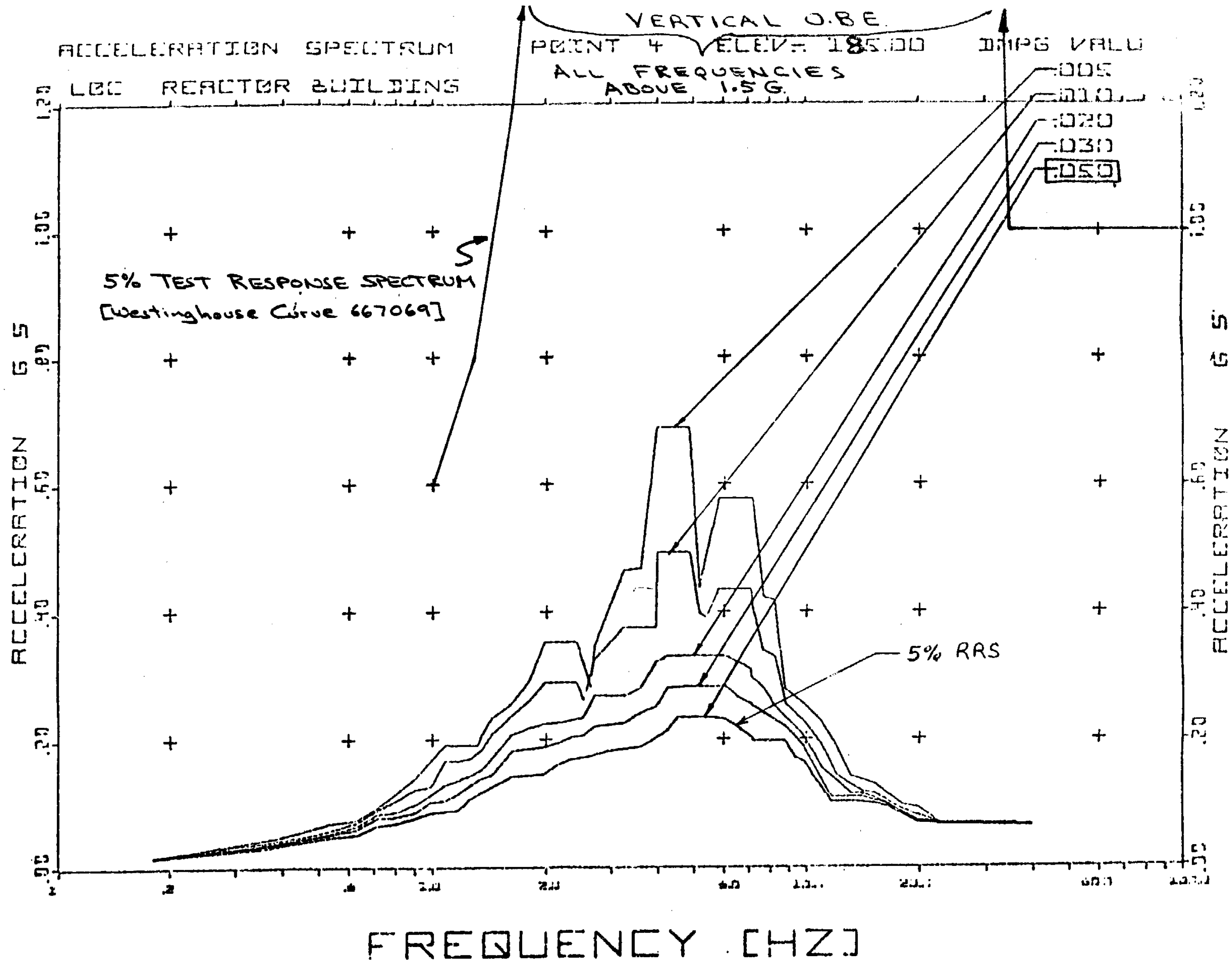
UNIT 2

TRS VS RRS
RPT BREAKERS

FIGURE 3.10-14 (SHEET 4 OF 6)



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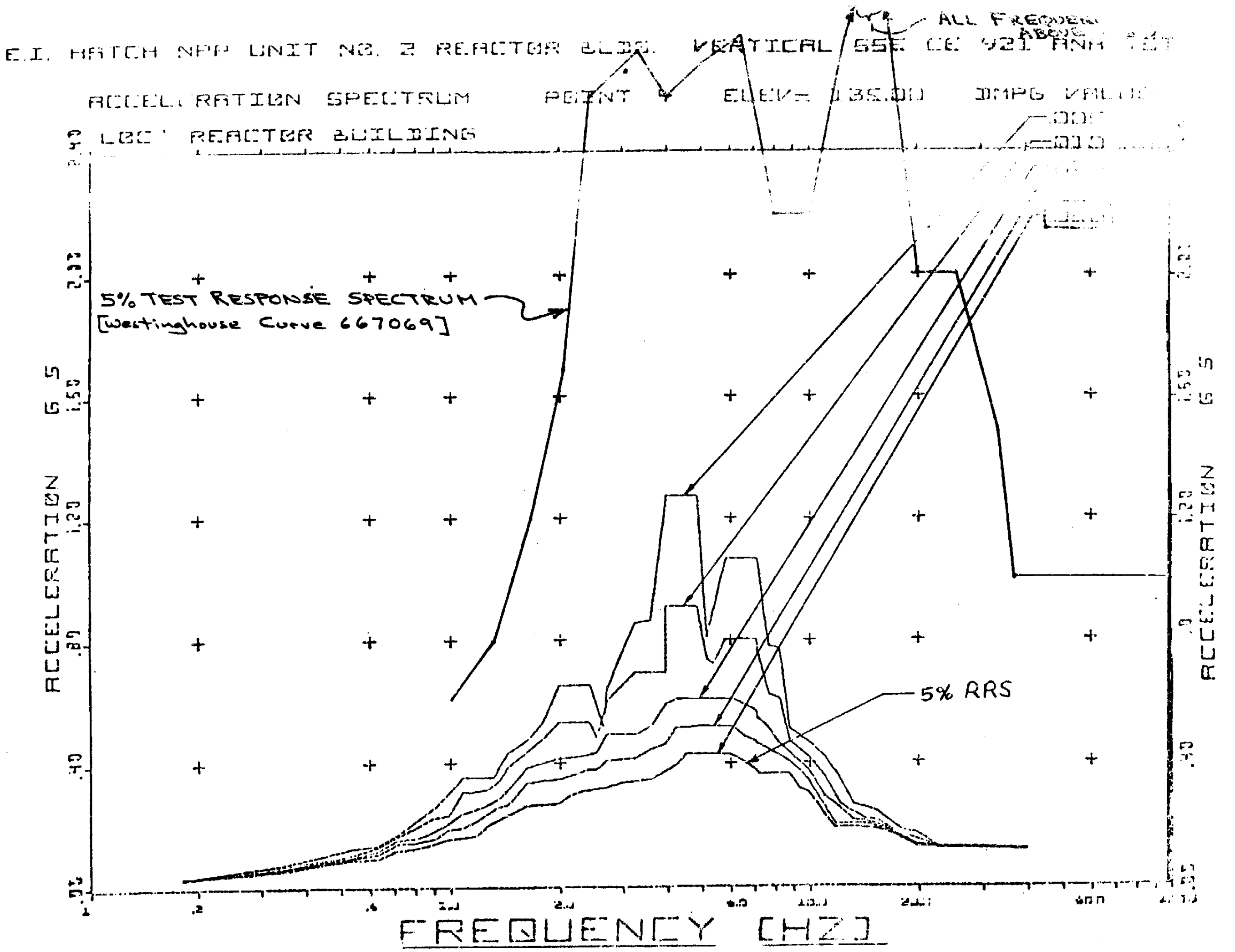
UNIT 2

TRS VS RRS
RPT BREAKERS

FIGURE 3.10-14 (SHEET 5 OF 6)



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UNIT 2

TRS VS RRS
RPT BREAKERS

FIGURE 3.10-14 (SHEET 6 OF 6)

3.11 ENVIRONMENTAL DESIGN OF MECHANICAL AND ELECTRICAL EQUIPMENT

The mechanical and electrical portions of the engineered safety feature and reactor protection systems are designed, tested, and/or analyzed for the worst-case environmental service conditions.

Many of the analyses confirming the environmental qualification of safety-related equipment meet the definition of a time-limited aging analysis pursuant to 10 CFR 54.3. (See subsection 18.1.3 and section 18.5 for additional information.)

3.11.1 EQUIPMENT IDENTIFICATION

All equipment that is required to function during and subsequent to any of the design basis accidents (DBAs) is identified in the Final Safety Analysis Report as listed below:

- Equipment supplied by General Electric (GE).
  - Mechanical equipment - tables 3.2-1 and 3.9-34.
  - Electrical equipment (including instrumentation) - paragraphs 3.2.1.4 and 3.2.1.5.
- Equipment not supplied by GE.
  - Mechanical equipment - tables 3.2-1 and 3.9-33.
  - Electrical equipment (including instrumentation) - paragraphs 3.2.1.4 and 3.2.1.5.

3.11.2 QUALIFICATION TESTS AND ANALYSIS

3.11.2.1 Qualification Tests and Analyses for Equipment Supplied by General Electric

A. Mechanical Equipment

Mechanical equipment required to operate during and subsequent to DBAs is designed to the applicable codes and standards specified in table 3.2-1 and the purchase specification.

Where applicable, mechanical equipment is designed to the environments specified in table 3.9-34.

B. Electrical Equipment (Including Instrumentation)

1. Electrical Class 1E Equipment Located in a Harsh Environment and Required to Mitigate a Loss-of-Coolant Accident (LOCA) or High-Energy Line Break.

On May 23, 1980, the Nuclear Regulatory Commission (NRC) issued a Commission Memorandum And Order CLI-80-21 which required the licensee to ensure that all Class 1E equipment meet the requirements of the NRC Division of Operating Reactors (DOR) Guidelines if the equipment was installed before May 23, 1980. The order required that Class 1E equipment installed after May 23, 1980, shall meet the requirements of NUREG-0588, Category I.

Subsequently, the NRC issued Rulemaking 10 CFR 50.49 on February 22, 1983, concerning environmental qualification of electric equipment important to safety. The rule superseded the May 23, 1980 order and required that all equipment installed after February 22, 1983, be upgraded from the DOR guidelines unless there are "sound reasons to the contrary." The acceptable "sound reasons to the contrary" can be found in Regulatory Guide 1.89, Revision 1.

Current information regarding equipment qualification is maintained in the Central File for the Environmental Qualification of Safety Related Equipment.

2. Electrical Class 1E Equipment Not Required to Mitigate a LOCA or High-Energy Line Break but Located in a Harsh Environment, or Class 1E Equipment Located in a Non-Harsh Environment.

All Class 1E equipment supplied by GE was qualified in accordance with Institute of Electrical and Electronics Engineers (IEEE) 323-1971. Since many of these items are used in several systems and in different plant locations, they were tested and analyzed for the worst-case situation.

The Class 1E equipment supplied by GE, which was qualified by testing, was first described by equipment specification which included or enveloped the intended application environment. The equipment design specification requirements are included on the purchased-part drawing for each essential device or are part of a design and performance specification drawing for GE-manufactured devices. Type tests were performed on pilot units to show conformance to the requirements of the equipment specifications. The test results were documented in a qualification test report or vendor-supplied certification.

The test plan, setup, procedure, and acceptability goals and requirements are all part of a qualification file maintained for each essential device. This information is filed according to product drawing number. Two types of documents are written as a condensed version of the detailed information

contained in the actual test reports. These documents are an environmental qualification summary and a seismic qualification summary.

3.11.2.2 Qualification Tests and Analysis for Equipment Not Supplied by General Electric

A. Mechanical Equipment

Mechanical equipment required to operate during and subsequent to DBAs is designed to the applicable codes and standards specified in table 3.2-1 and the purchase specification.

Table 3.9-33 lists the active valves in Seismic Category I systems and the environmental conditions to which they are designed.

B. Electrical Equipment (Including Instrumentation)

1. Electrical Class 1E Equipment Located in a Harsh Environment and Required To Mitigate a LOCA or High-Energy Line Break. (See paragraph 3.11.2.1.B.1.)

   All Class 1E equipment was qualified as a minimum in accordance with the provisions of 10 CFR 50.49. Since this equipment is used in different systems and plant locations, it was tested or analyzed for the worst-case environmental conditions as specified in the equipment specifications and as listed in the Central File. Table 3.11-1 provides information relative to area design conditions.

2. Electrical Class 1E Equipment Not Required to Mitigate a LOCA or High-Energy Line Break but Located in a Harsh Environment or Class 1E Equipment Located in a Non-Harsh Environment.

   For equipment with standardized and proven design, the representative equipment was tested up to and beyond the nominal ratings. In addition, production inspection and testing were performed on the equipment purchased in accordance with National Electrical Manufacturers Association (NEMA) publications and American National Standards Institute (ANSI) standards as indicated below:

Equipment	Standards
Control panels and racks	NEMA ICS
5-kV switchgears	ANSI C-37.20
600-V load centers	NEMA SG-3
Motors	NEMA MG-1

Equipment	Standards
250-V-dc switchgears	NEMA SG-3
Diesel generator control equipment	NEMA ICS

3.11.3 QUALIFICATION TEST RESULTS

3.11.3.1 Qualification Test Results for Equipment Supplied by General Electric

A. Mechanical Equipment

For equipment of standardized and proven design, the production inspection and testing ensure the proper functioning of the equipment under service environmental conditions.

B. Electrical Equipment (Including Instrumentation)

1. Electrical Class 1E Equipment Located in a Harsh Environment and Required to Mitigate a LOCA or High-Energy Line Break. (See paragraph 3.11.2.1.B.1.)

   The Plant Hatch Central File provides the normal and accident qualification environments to which Class 1E equipment supplied by GE is exposed during qualification testing. Detailed test results are on file at GE-San Jose.

   The analog transmitter trip system (ATTS) instrumentation is qualified to the requirements of IEEE 323-74 and Rulemaking 10 CFR 50.49. Table 7.8-1 provides the instrumentation included in this system. NEDE-22154-1 and NEDC-30039-1 provide a detailed discussion of the environmental qualification program and its applicability to Plant Hatch.

2. Electrical Class 1E Equipment Not Required to Mitigate a LOCA or High-Energy Line Break but Located in a Harsh Environment or Class 1E Equipment Located in a Non-Harsh Environment. (See paragraph 3.11.2.1.B.2.)

   For equipment of standardized and proven design, the production inspection and testing ensure the proper functioning of the equipment under service environmental conditions.

3.11.3.2 Qualification Test Results for Equipment Not Supplied by General Electric

A. Mechanical Equipment

For equipment of standardized and proven design, the production inspection and testing ensure the proper functioning of the equipment under service environmental conditions.

B. Electrical Equipment (Including Instrumentation)

1. Electrical Class 1E Equipment Located in a Harsh Environment and Required to Mitigate a LOCA or High-Energy Line Break. (See paragraph 3.11.2.1.B.1.)

2. Electrical Class 1E Equipment Not Required to Mitigate a LOCA or High-Energy Line Break but Located in a Harsh Environment, or Class 1E Equipment Located in a Non-Harsh Environment.

   For equipment of standardized and proven design, the production inspection and testing ensure the proper functioning of the equipment under service environmental conditions.

3.11.4 PROCUREMENT OF NEW EQUIPMENT

All new equipment, which falls under the scope of 10 CFR 50.49, is purchased to meet that requirement. In general, as part of that requirement, new equipment is evaluated against the worst-case environmental profiles through which the equipment must function. These profiles are provided in the central documentation file.

For the applicable equipment inside containment, the evaluation is performed against the composite profile provided in figure 3.11-1. This composite profile was developed using the worst-case guillotine break inside containment and the plant-specific main steam line break analysis developed by General Electric in NSEO-52-0583, dated June 1983. Operator action to initiate drywell spray is assumed to occur after 30 min.

This analysis was developed using the guidelines of NUREG-0588. For equipment inside containment that cannot meet the composite profile, an evaluation against the individual profiles may be performed.

3.11.5 LOSS OF VENTILATION

A. The operability of all Seismic Category I control and electric equipment located in the control room and other areas is ensured by providing each room with redundant Seismic Category I heating, ventilation, and/or air-conditioning systems as described in section 9.4.

B. The equipment qualification tests and analyses are described in subsection 3.11.2 and table 3.9-24. The summaries of qualification test results the tests and analyses are provided in subsection 3.11.3.

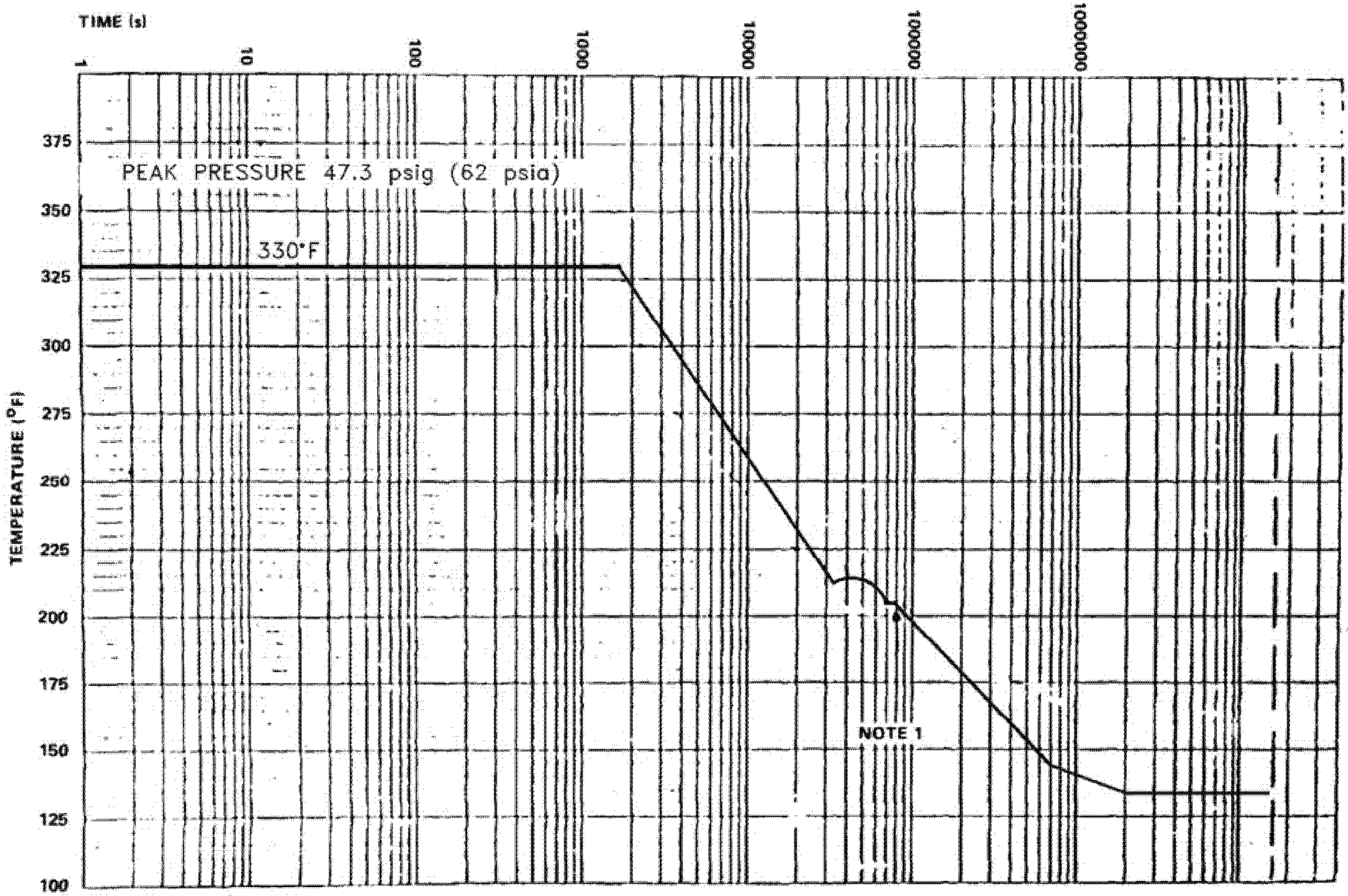
TABLE 3.11-1 (SHEET 1 OF 2)

AREA ENVIRONMENTAL CONDITIONS FOR EQUIPMENT QUALIFICATION[a]

Location	Temperature (°F) Normal	Temperature (°F) DBE (max)	Pressure (psia) Normal	Pressure (psia) DBE (max)	Humidity (%) Range	Humidity (%) DBE	Radiation (rads)[b]
Containment (drywell)	(e)	330	16.7	62.0	40-90	100	1.22×10^8
Reactor bldg el 203 ft	90	200	14.7	15.0	50-80	100	1.90×10^5
Reactor bldg el 185 ft	90	205	14.7	15.04	50-80	100	1.90×10^5
Reactor bldg el 158 ft	90	210	14.7	15.06	50-80	100	2.51×10^6
Reactor bldg el 130 ft	90	213	14.7	16.7	50-80	100	2.37×10^6
RWC heat exchanger room	90	217	14.7	16.36	50-80	100	9.93×10^6
RWC pump room	90	218	14.7	16.59	50-80	100	2.19×10^5
Pipe penetration room	105	215	14.7	16.50	50-80	100	1.55×10^7
Pipe chase	105	300	14.7	17.75	50-80	100	1.27×10^7
Torus room	105	216	14.7	16.74	50-90	100	1.40×10^7
RCIC corner room (NW)	105	295	14.7	15.8	50-90	100	9.85×10^4
SW corner room	105	295	14.7	15.8	50-90	100	9.85×10^4
HPCI room	105	148 for 12 h[c]	14.7	14.7[d]	50-90	100	9.85×10^4
NE corner room (RHR)	104	148	14.7	14.7	50-90	100	6.15×10^6
SE corner room (RHR)	104	215	14.7	16.0	50-90	100	6.15×10^6

TABLE 3.11-1 (SHEET 2 OF 2)

a. Individual component equipment qualification is based on environmental conditions specified in the Plant Hatch Central File of Environmental Qualification of Safety-Related Equipment. The information in this table should be verified before use.
b. Total integrated dose for the area specified (DBA + 60 years, normal dose).
c. The temperature is based on a high-energy line break outside the high pressure coolant injection room; an analysis indicates the reactor core decay heat will not produce sufficient steam to drive the HPCI turbine after 12 h.
d. 27.4 psia for isolation equipment only.
e. Temperature varies depending on drywell location.



NOTES: 1. The profile after this time is to simulate post-accident operation and may be modified by the vendor to accelerate the aging time based on material aging characteristics.

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WORST-CASE ACCIDENT PROFILE FOR EQUIPMENT LOCATED IN CONTAINMENT

FIGURE 3.11-1

4.0 REACTOR

4.1 REACTOR SUMMARY DESCRIPTION (HNP-1 AND HNP-2)

The information provided in this section is applicable to both HNP-1 and HNP-2, unless specified otherwise.

The reactor assembly of the Edwin I. Hatch Nuclear Plant Unit 1 (HNP-1) and Unit 2 (HNP-2) consists of the reactor pressure vessel (RPV) and its internal components of the core, shroud, steam separator and dryer assemblies, and jet pumps. Also included in the reactor assembly are the control rods, control rod drive (CRD) housings, and the CRD. The HNP-1 and the HNP-2 RPV assembly cutaway (figure 4.1-1) shows the arrangement of reactor assembly components. Loading conditions for reactor assembly components are specified in HNP-1-FSAR appendix C and HNP-2-FSAR table 3.9-4. Summary tables of the pertinent reactor data are presented at the end of sections 4.2, 4.3, and 4.4.

4.1.1 REACTOR PRESSURE VESSEL

The RPV design and description are covered in HNP-1-FSAR section 4.2 and appendix C, and HNP-2-FSAR section 5.4.

4.1.2 REACTOR INTERNAL COMPONENTS

The major reactor internal components are as follows:

- Core (fuel, channels, control blades, and instrumentation).
- Core support structure, including core shroud, top guide, and core support.
- Shroud head and separators.
- Steam dryer assembly.
- Jet pumps.

Except for the Zircaloy in the reactor core, the reactor internals are either stainless steel or other corrosion-resistant alloys. With the exception of the jet pump diffusers, core shroud, core spray (CS) spargers, and jet pump inlet piping, all the major internal components can be removed. The removal of the steam dryers, shroud head and separators, fuel assemblies, incore instruments, control rods, CRDs, and control rod guide tubes can be accomplished on a routine basis.

4.1.2.1 Reactor Core

The reactor core consists of 560 channeled boiling water reactor (BWR) fuel assemblies; 137 cruciform-shaped, bottom-entry control blades; and 124 fission chamber detectors that continuously monitor core thermal power during power operation. Each core component is described in detail in other sections.

4.1.2.1.1 Core Configuration

The reactor core is arranged as an upright circular cylinder containing a large number of fuel cells and is located within the RPV. The coolant flows upward through the core. The core arrangement (plan view) is shown in figure 4.1-2. The BWR core is essentially composed of only two components -- fuel assemblies and control rods. Core configurations for each reload core are described in the corresponding supplemental reload licensing report. (Table 15.1-1 provides a list of the cycle-specific supplemental reload licensing reports for HNP-1 and HNP-2.)

4.1.2.1.2 Fuel Assembly Description

Fuel assemblies are described in subsection 4.2.1.

4.1.2.2 Shroud

The shroud is a cylindrical, stainless-steel structure that surrounds the core and provides a barrier to separate the upward flow through the core from the downward flow in the annulus. The shroud also provides a floodable volume in the unlikely event of an incident that tends to drain the RPV. A flange at the top of the shroud cylinder mates with a flange on the shroud head to form the core discharge plenum. The jet pump discharge diffusers penetrate the shroud support below the core elevation to introduce the coolant to the lower inlet plenum. The shroud is designed, constructed, and installed to prevent a direct flow path between the inlet and outlet of each recirculation system loop. The shroud support is designed to support and locate the jet pumps and the core support structure, and provides lateral support for the fuel assemblies.

Mounted inside the shroud top cylinder in the space between the top of the core and the flange at the top of the shroud are the two CS spargers with spray nozzles for injection of cooling water. The CS spargers and nozzles do not interfere with the installation or removal of fuel from the core. A pipe for the injection of neutron absorber (sodium pentaborate) solution is mounted below the core to ensure mixing with the cooling water rising through the core.

4.1.2.3 Shroud Head and Separators

The shroud head and separators consist of a flange and dome onto which is welded an array of standpipes with a steam separator located at the top of each standpipe. The shroud head mounts on the flange at the top of the shroud top cylinder and forms the cover (shroud head) of

the core discharge plenum region. The joint between the shroud head and shroud top cylinder does not require a gasket or other replacement sealing techniques. The fixed axial flow-type steam separators have no moving parts and are made of stainless steel.

In each separator, the steam water mixture rises from the standpipe and impinges on vanes which give the mixture a spin to establish a vortex wherein the centrifugal forces separate the steam from the water. Steam leaves the separator at the top and passes into the wet steam plenum below the dryer. The separated water exits from the lower end of the separator and enters the pool that surrounds the standpipes to enter the downcomer annulus. An internal steam separator schematic is shown on figure 4.1-3.

For ease of removal, the shroud head and separators are bolted to the top cylinder by long shroud head bolts that extend above the separators for easy access during refueling. The shroud head and separators are guided into position on the shroud and flange with guide rods and locating pins. The objective of the long-bolt design is to provide direct access to the bolts during reactor refueling operations with minimum depth underwater tool manipulation during the removal and installation of the assemblies.

4.1.2.4 Steam Dryer Assembly

The steam dryer assembly is mounted in the RPV above the shroud head and separators, and forms the top and sides of the wet steam plenum. Vertical guide rods on the inside of the RPV provide alignment for the dryer assembly during installation. The dryer assembly is supported by brackets extending from the RPV wall. The RPV top head prevents significant upward movement. Steam from the separators flows upward into the dryer assembly. The steam leaving the top of the dryer assembly flows into four RPV steam outlet nozzles which are located alongside the steam dryer assembly. Moisture is removed by the dryer vanes and flows first through a system of troughs and pipes to the pool surrounding the separators and then into the downcomer annulus between the shroud and RPV wall. A partial schematic of a typical steam dryer is shown on figure 4.1-4.

4.1.3 REACTIVITY CONTROL SYSTEMS

4.1.3.1 Operation

The 137 control rods perform dual functions of power distribution shaping and reactivity control. Power distribution in the core is controlled during operation of the reactor by manipulation of selected patterns of rods. The rods, which enter from the bottom of the near-cylindrical reactor core, are positioned in such a manner to counterbalance steam voids in the top of the core and effect significant power flattening. These groups of control elements, used for power flattening, experience a somewhat higher duty cycle and neutron exposure than the other rods in the control system.

The reactivity control function requires that all rods be available for either reactor scram or reactivity regulation. Because of this, the control elements are mechanically designed to

withstand the dynamic forces resulting from a scram. They are connected to bottom-mounted, hydraulically actuated drive mechanisms which allow either axial positioning for reactivity regulation or rapid scram insertion. The design of the rod-to-drive connection permits each blade to be attached or detached from its CRD without disturbing the remainder of the control system. The bottom-mounted CRDs permit the entire control system to be left intact and operable for tests with the RPV open.

4.1.3.2 Description of Control Rods

The control rods are described in paragraph 4.2.3.1.

4.1.3.3 Supplementary Reactivity Control

Control requirements of the core are met by use of the combined effects of the movable control rods and a supplemental burnable poison. The supplementary burnable poison found in several fuel rods in each bundle is gadolinia (Gd_2O_3) mixed with UO_2.

4.1.4 ANALYSIS TECHNIQUES

4.1.4.1 Reactor Internal Components

The analysis techniques for HNP-1 are found in HNP-1-FSAR appendix C.

Computer codes used for the HNP-2 initial analysis of the internal components are as follows:

- MASS (Mechanical Analysis of Space Structure).
- SNAP (MULTISHELL).
- GASP.
- NOHEAT.
- FINITE.
- SAMIS (Structural Analysis and Matrix Interpretive System).
- SHELL 5 and SHELL 9.
- HEATER.
- FAP-7I (Fatigue Analysis Program).

For HNP-1 and HNP-2, the reactor internal components and retrofit repairs were evaluated for the increase in rated thermal power to 2804 MWt and reactor operating pressure increase from 1050 psia to 1060 psia.[1] [2] All structural criteria for all accident cases required by the safety analysis are met.

4.1.4.2 Fuel Rod Thermal Analysis

Reference section 4.3.1 of ***NEDE-24011-P-A, "GESTAR II - General Electric Standard Application for Reactor Fuel" (incorporated by reference into the FSAR)***. Fuel rod thermal analysis and documentation of methods for the four Westinghouse SVEA-96 Optima2 lead use assemblies loaded into HNP-1 are contained in reference 3.

4.1.4.3 Reactor Systems Dynamics and Nuclear Analysis

Reference section 3.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.1.4.4 Neutron Fluence Calculations

Historically, HNP-1 and HNP-2 neutron fluence calculations were performed as described in A and B below. This process forms the basis for the currently approved P/T limit curves in the Technical Specifications. The results from employing the methodology described in C[4][5] were used to generate P/T limit curves which were determined to be bounded by the approved P/T limit curves. The most recent fluence updates[6][7] resulted in even lower beltline fluences with resulting lower adjusted reference temperatures. All future P/T limit curve submittals when desired or necessitated will employ fluence inputs based on the methodology in C.

A. HNP-1

Neutron vessel fluence calculations for HNP-1 are described in HNP-1-FSAR appendix R, section R.1.

B. HNP-2

Neutron vessel fluence calculations were carried out using a two-dimensional, discrete ordinates S_n transport code with general anisotropic scattering. This code is a widely used discrete ordinates code that will solve a wide variety of radiation transport problems. Slab, cylinder, and spherical geometries are allowed with various boundary conditions. The fluence calculations incorporate, as an initial starting point, a distributed fission neutron source distribution prepared from core physics data. Anisotropic scattering is considered for all regions. The cross-sections are represented by third-order Legendre polynomial expansions.

Fast neutron fluxes at locations other than the core midplane were calculated using a one-dimensional, discrete ordinates code which is similar to the two-dimensional

code. One-dimensional calculations were performed for several elevations to determine the relative variation of fast flux with elevation.

The fast neutron flux calculations are used to establish the ratio of flux between the surveillance capsule locations and the location of peak vessel inside surface flux, known as the lead factor. Use of the lead factor is discussed in paragraph 4.3.2.8.

C. HNP-1 and HNP-2 neutron fluence calculations are performed using the Radiation Analysis Modeling Application (RAMA) a methodology sponsored by the Boiling Water Reactor Vessel and Internals Project (BWRVIP). RAMA was developed to be compliant with NRC Regulatory Guide 1.190, submitted to the NRC staff, and approved in 2005. The RAMA fluence methodology is a system of computer codes, a data library, and an uncertainty methodology that determines best-estimate fluence in light water reactor pressure vessels. The primary codes include model builder codes, particle transport code, and a fluence calculator code. Plant Hatch developed specific RAMA models for both reactors based on this methodology. The mechanical design inputs were obtained from plant design drawings with as-built measurements when available. For earlier fuel cycles (cycles 1-13 for HNP-1 and 1-9 for HNP-2) the reactor operating units were developed from using three sources for approximation: 1) power and exposure distributions derived from process computer listings and cycle arrangement reports; 2) detailed operating data from cycles of comparable core design and energy productions; and 3) traversing in-core probe (TIP) data. Ensuring fuel cycles used core simulator data which provided a historical accounting of how the reactor operated up until the most recent cycle of calculation “run”. Since operating data for future fuel cycles is not available the most current fuel cycle is assumed to represent the best available operating data for projecting plant end of license (EOL). This EOL projection repeats an equilibrium fuel cycle incorporating the best available information based on the expected cycle length, fuel bundle loading, and operating strategies out to the end of calendar license. Providing no appreciable changes to the operating strategy that could invalidate the equilibrium assumptions, the projection need not be updated at any set interval. Changes that could affect the equilibrium cycle validity include new fuel designs, power uprates, fuel cycle length, core loading strategy changes, core/reactor flow changes, and extended outages. The calculated fluences for HNP-1 and HNP-2 were initially reported in references 4 and 5, respectively, and included an evaluation of results against surveillance capsules to establish a predicted versus measure benchmark. Updated model calculations were completed more recently with results reported in references 6 and 7 for HNP-1 and HNP-2, respectively.

4.1.4.5 <u>Thermal-Hydraulic Calculations</u>

Thermal-hydraulic calculations are discussed in section 4.3.1 ***of NEDE-24011-P-A (GESTAR II)***. Fuel rod thermal-hydraulic analysis and documentation of methods for the four Westinghouse SVEA-96 Optima2 lead use assemblies loaded into HNP-1 are contained in reference 3.

<u>DOCUMENTS INCORPORATED BY REFERENCE INTO THE FSAR</u>

"GESTAR II - General Electric Standard Application for Reactor Fuel," <u>NEDE-24011-P-A</u>.

REFERENCES

1. "Safety Analysis Report for Edwin I. Hatch Units 1 and 2 Thermal Power Optimization," <u>NEDC-33085P</u>, GE Nuclear Energy, December 2002.

2. "10-PSI Dome Pressure Increase Project Report for Edwin I. Hatch Units 1 and 2," <u>GE-NE-0000-0003-0634-01</u>, Revision 1, GE Nuclear Energy, July 2003.

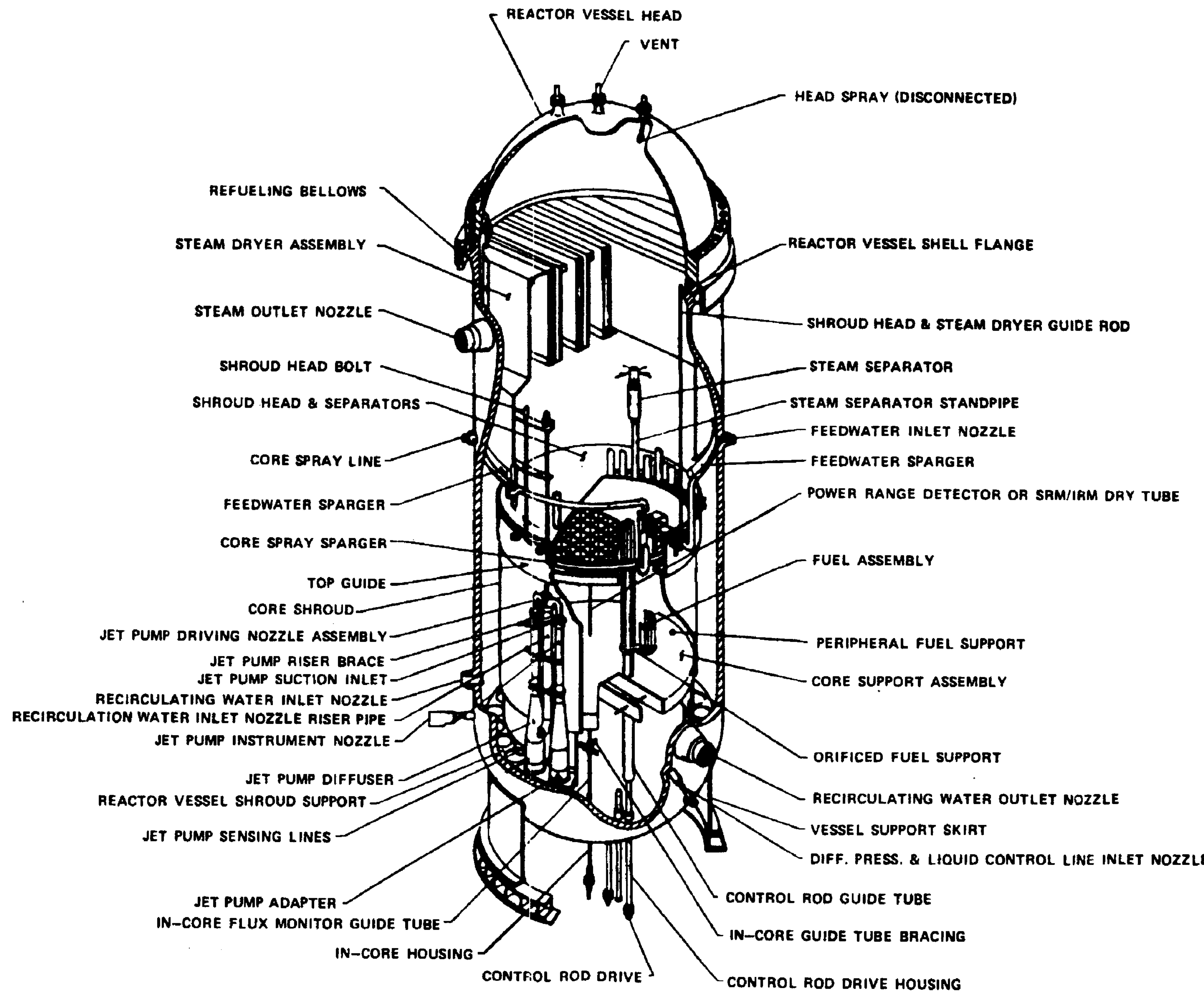
3. Westinghouse Report NF-BSN-10-10, "Supplemental Licensing Report, SVEA-96 Optima2 Lead Use Fuel Assemblies for Edwin I. Hatch Nuclear Plant, Unit 1," Revision 0, February 2010.

4. SNC-FLU-001-R-001 Revision 1, Edwin I. Hatch Unit 1 Reactor Pressure Vessel Fluence Evaluation at End of Cycle 21 and 49.3 EFPY, dated June 16, 2006.

5. SNC-FLU-002-R-001 Revision 0, Edwin I. Hatch Unit 2 Reactor Pressure Vessel Fluence Evaluation at End of Cycle 18 and 50.1 EFPY, dated April 23, 2007.

6. SNC-HA1-002-R-001 Revision 0, Edwin I. Hatch Unit 1 Fluence Evaluation at End of Cycle 25 and 49.3 EFPY, dated October 5, 2012.

7. SNC-HA2-001-R-001 Revision 0, Edwin I. Hatch Unit 2 Fluence Evaluation at End of Cycle 22 and 50.1 EFPY, dated April 16, 2014.



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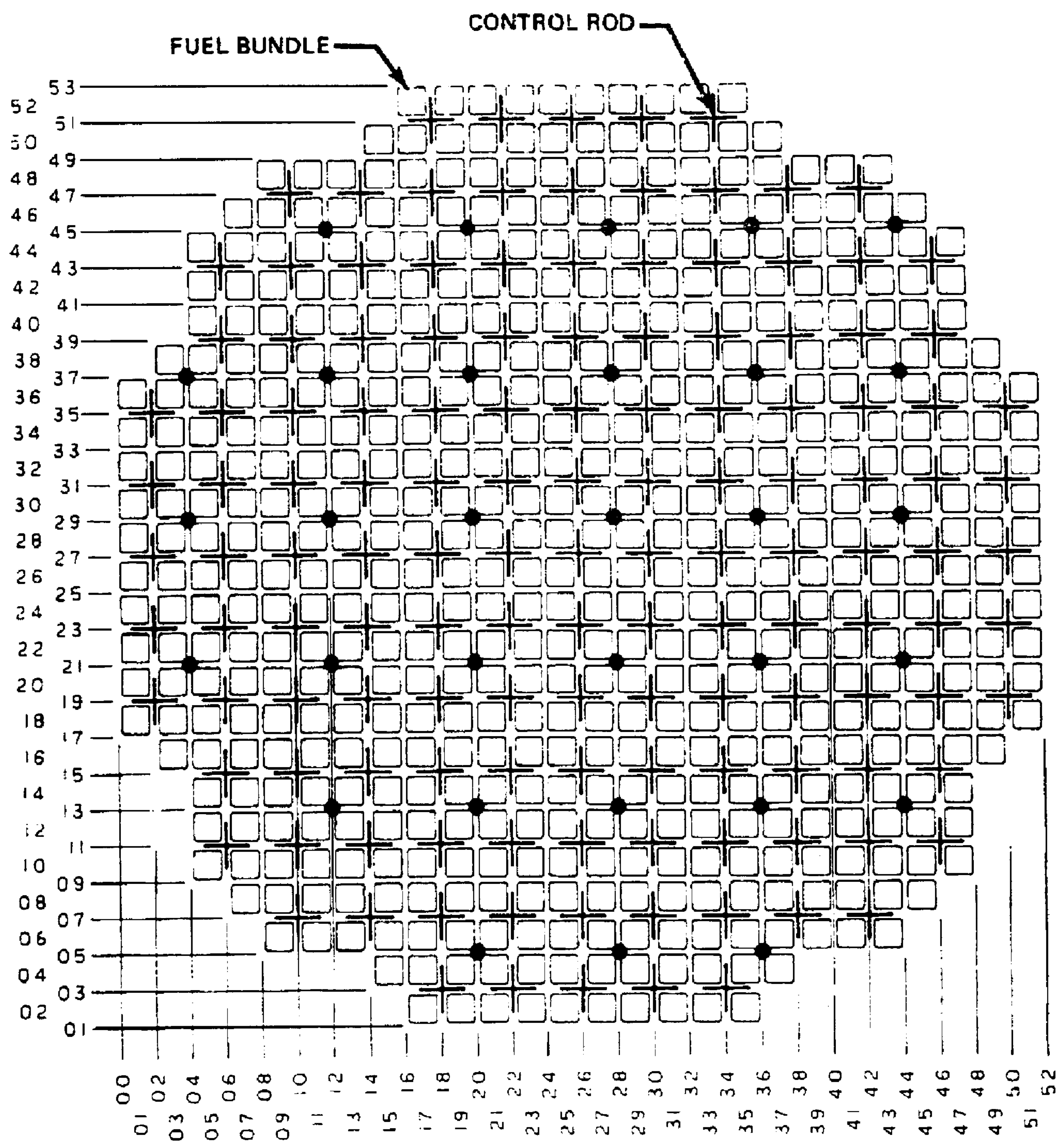
REACTOR VESSEL CUTAWAY

FIGURE 4.1-1



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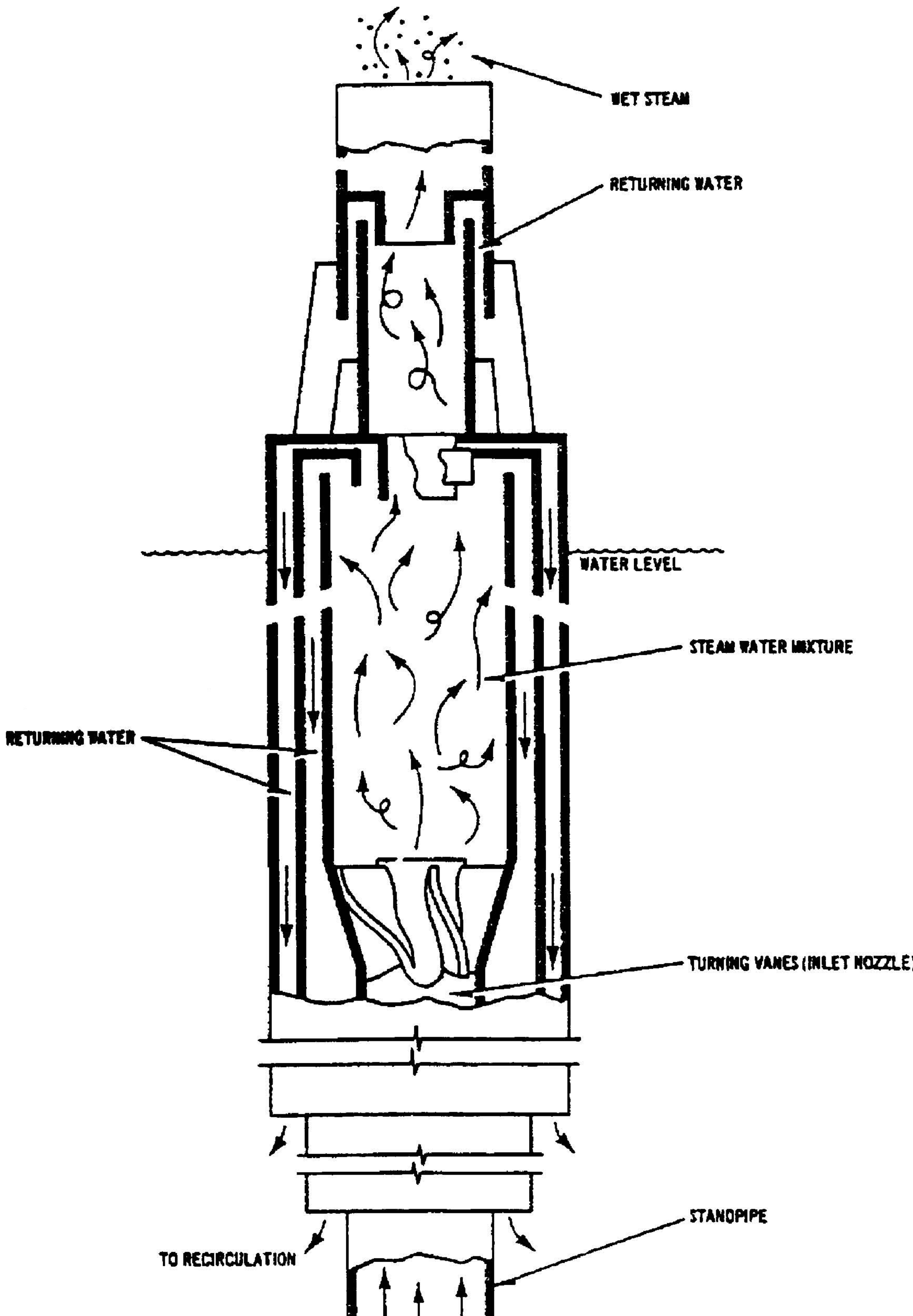
CORE ARRANGEMENT AND LATTICE CONFIGURATION

FIGURE 4.1-2



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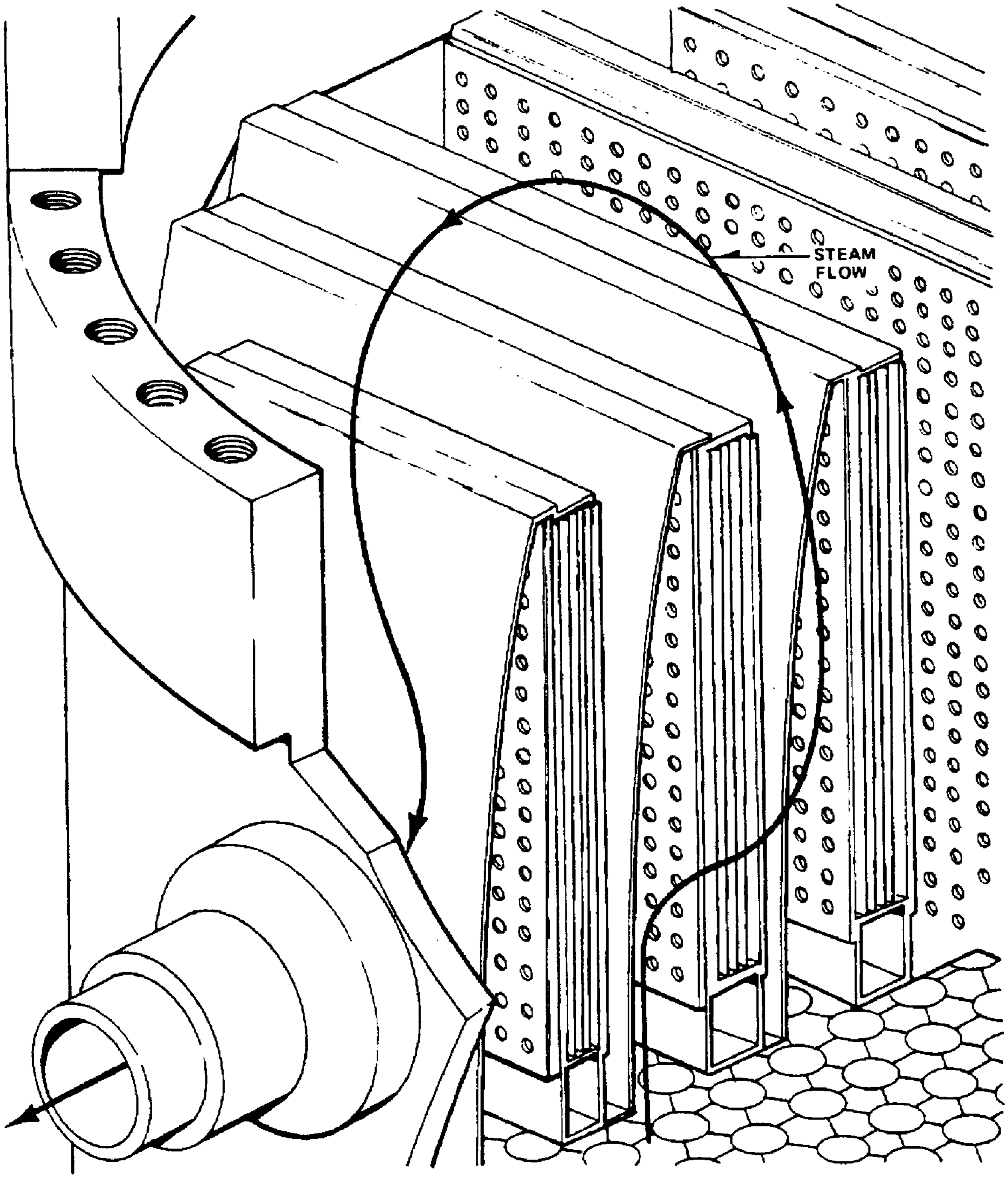
STEAM SEPARATOR

FIGURE 4.1-3



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STEAM DRYER

FIGURE 4.1-4

4.2 MECHANICAL DESIGN (HNP-1 AND HNP-2)

The design of the fuel system, the reactor core support structures and internals, and the reactivity control and standby liquid control systems is applicable to both HNP-1 and HNP-2, unless specified otherwise.

4.2.1 FUEL SYSTEM DESIGN

The description of fuel assemblies in the following sections pertains to fuel bundles supplied by Global Nuclear Fuel (GNF) and described in GESTAR-II. In addition, four SVEA-96 Optima2 lead use assemblies (LUAs) supplied by Westinghouse Electric Company have been installed in the HNP Unit 1 core. The mechanical design description and documentation of methods for these LUAs are contained in references 30, 31, and 32.

4.2.1.1 General Design Description

The design bases for fuel bundles are contained in section 2.2 of ***NEDE-24011-P-A, "GESTAR II - General Electric Standard Application for Reactor Fuel" (incorporated by reference into the FSAR)***.

4.2.1.2 Design Bases

4.2.1.2.1 General Design Bases

Reference section 2.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.2.1.2.2 Basis for Fuel Safety Evaluation

Reference section 2.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.2.1.2.3 Design Ratios

Reference section 2.2.1.1.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.2.1.2.4 Maximum Allowable Stresses and Cycling and Fatigue Limits

Reference section 2.2.1.1.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.2.1.3 Results of Thermal Mechanical Evaluations

Reference chapters 2 and 4 of ***NEDE-24011-P-A (GESTAR II)***.

4.2.1.4 Operating and Developmental Experience

Reference section 2.3.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.2.1.5 Inspection, Testing, and Surveillance

Reference section 2.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.2.2 REACTOR CORE SUPPORT STRUCTURES AND INTERNALS MECHANICAL DESIGN

4.2.2.1 Design Bases

4.2.2.1.1 General Design Bases

4.2.2.1.1.1 Safety Design Bases. The reactor core support structures and internals meet the following safety design bases:

A. Internals are arranged to provide a floodable volume in which the core can be adequately cooled in the event of a breach in the reactor coolant pressure boundary, external to the reactor pressure vessel (RPV). The floodable inner volume is inside the core shroud up to the level of the jet pump suction inlet. The boundary of the inner volume consists of the following:

- The jet pumps from the jet pump's suction inlet down to the shroud support ring.
- The shroud support ring, which forms a barrier between the outside of the shroud and the inside of the reactor vessel.
- The reactor vessel wall below the shroud support ring.
- The core shroud up to the level of the jet pump suction inlet.

B. Deformation is limited to ensure the control rods and the emergency core cooling system can perform their safety functions.

C. The mechanical design of applicable structures ensure safety design bases A and B are satisfied so the safe shutdown of the plant and removal of decay heat are not impaired.

4.2.2.1.1.2 **Power Generation Design Bases**. The reactor core support structures and internals are designed to the following power generation design bases:

A. They provide the proper coolant distribution during all anticipated normal operating conditions to allow power operation of the core without fuel damage.

B. They are arranged to facilitate refueling operations.

C. They are designed to facilitate inspection.

4.2.2.1.2 Specific Design Characteristics

4.2.2.1.2.1 **Design Loading Combinations**. The design of the RPV internals specified in this subsection is in accordance with the intent of American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section III. The design condition categories (comparable to normal, upset, emergency, and faulted specified in the ASME Code) and the associated loading combinations may be noted by reference to HNP-1-FSAR section C.2 and table C.2-1, and HNP-2-FSAR paragraph 4.2.2.3.1.1.

4.2.2.1.2.2 **Stress, Deformation, and Fatigue Limits for RPV Internals (Except Core Support Structures).** For HNP-1 and HNP-2, the criteria used for deformation, primary stress, buckling, and fatigue limits are provided in HNP-2-FSAR tables 4.2-1, 4.2-2, 4.2-3, and 4.2-4, respectively. For HNP-1, a more detailed stress summary on a component basis is provided in HNP-1-FSAR table C.3-1. For HNP-2, criteria based upon applicable codes and standards, manfacturer's standards, or empirical methods based upon field experience and testing can also be used.

The following minimum safety factor values (quantity SF_{min}) are used for both HNP-1 and HNP-2:

Design Condition	SF_{min}
Normal	2.25
Upset	2.25
Emergency	1.5
Faulted	1.125

4.2.2.1.2.3 **<u>Stress, Deformation, and Fatigue Limits for Core Support Structures</u>.** For HNP-1, the stress, deformation, and fatigue limits for core support structures are provided in HNP-1-FSAR table C.3-1. For HNP-2, the stress deformation and fatigue criteria presented in tables 4.2-5, 4.2-6, and 4.2-7 are used. Where applicable, these criteria are supplemented by the criteria for the reactor internals in the previous paragraph, but in no case are the criteria for core support structures presented in these tables exceeded.

4.2.2.1.2.4 **<u>Fuel Assembly Restraints</u>**. The fuel assembly structural design demonstrates sufficient dimensional stability and sufficient fuel rod support to maintain core geometry, thus avoiding fuel damage for both planned operation and anticipated operational occurrences (AOOs).

4.2.2.1.2.5 **<u>Material Selection</u>.** The material used for fabricating most of the reactor core support and reactor internal structures is solution-heat-treated, Type 3O4 austenitic stainless steel conforming to American Society of Testing Materials (ASTM) specifications. Weld procedures and welders are qualified in accordance with the intent of Section IX of the ASME Boiler and Pressure Vessel Code. Further controls for stainless-steel welding are covered in HNP-1-FSAR subsection 4.2.4 and HNP-2-FSAR subsection 5.2.5.

All materials of construction exposed to the reactor coolant are resistant to stress corrosion in the BWR. Conservative corrosion allowances are to be provided for all exposed surfaces of carbon or low-alloy steels.

Contaminants in the reactor coolant are controlled to very low limits by the reactor water quality specifications. No detrimental effects occur on any of the materials from allowable contaminant levels in the high-purity reactor coolant. Radiolytic products in a BWR have no adverse effect on the construction materials.

4.2.2.1.2.6 **<u>Radiation Effects</u>**. Where feasible, the design is such that irradiation effects on the material properties are minimized. Where irradiation effects cannot be minimized, the design of the RPV internals either has provision for replaceable components, or the design is shown to satisfy a set of stress and fatigue design limits. The fatigue design limits have been determined considering the effect of irradiation damage on the fracture toughness, ductility, and tensile properties of the materials.

4.2.2.1.2.7 **<u>Shock Loads</u>.** The components are designed to accommodate the loadings discussed in HNP-1-FSAR appendix C and HNP-2-FSAR section 3.9.

4.2.2.1.2.8 Vibration of Reactor Internal Components.

A. HNP-1

The vibration of reactor internal components is discussed in HNP-1-FSAR appendix C.

B. HNP-2

The core plate bypass leakage holes, that were the source of excessive tube vibration and channel wear, were not drilled in the HNP-2 core plate. Bypass leakage flow is provided by small holes in the fuel assembly lower tie plates, which have been shown by test to produce greatly reduced levels of tube vibration.

The adequacy of these design characteristics, with respect to reduction of vibration and wear of instrument tubes and fuel channels, has been demonstrated by vibration monitoring in a plant similar to HNP-2, prior to HNP-2 initial startup testing.

The HNP-2 design characteristics do not have a significant effect on the vibratory excitation and response of core support or other reactor internals structures. Flow parameters, which could affect the response of these structures, such as the core plate pressure differential and the core coolant flowrate, are not significantly altered by the design modification. Dynamic response characteristics of the structures themselves were not altered by the modification.

4.2.2.2 Description

The core support structures and RPV internals (exclusive of fuel, control rods, and incore nuclear instrumentation) include the following components:

A. Core Support Structures

- Shroud.
- Shroud support.
- Core support and hold-down bolts.
- Top guide (including wedges, bolts, and keepers).
- Fuel support pieces.
- Control rod guide tubes.

B. Reactor Internals

- Jet pump assemblies and instrumentation.
- Shroud head and steam separator assembly (including shroud head bolts).
- Steam dryers.
- Feedwater spargers.
- RPV head cooling-spray nozzle (capped).
- Differential pressure and liquid control line.
- Incore flux monitor guide tubes and stabilizers.
- Neutron sources.
- Surveillance sample holders.
- Core spray (CS) lines.

For HNP-1 and HNP-2, a list of the materials and their specifications for major components of the reactor internals and core support structures is given in table 4.2-8.

The overall arrangement of the structures within the RPV is shown in figure 4.1-1. A general assembly drawing of the important reactor components is shown in HNP-1 drawing nos. SX-16121, SX-16122, and SX-16123, and HNP-2 drawing nos. S-28220, S-28221, S-28222, S-28223, S-28224, and S-28225.

The floodable inner volume of the RPV, which is the volume inside the core shroud up to the level of the jet pump suction inlet, is provided in figure 4.2-1.

The core support structure is used to:

- Form partitions within the RPV.
- Sustain pressure differentials across the partitions.
- Locate laterally and support the fuel assemblies, control rod guide tubes, and steam separators.
- Direct the flow of the coolant water.

Figure 4.2-1 shows the RPV internal flow paths.

4.2.2.2.1 Shroud

The shroud is a stainless-steel cylindrical assembly that provides a partition to separate the upward flow of coolant through the core from the downward recirculation flow. This partition separates the core region from the downcomer annulus, thus providing a floodable region following a recirculation line break.

The volume enclosed by the shroud is characterized by three regions. The upper shroud surrounds the core discharge plenum, which is bounded by the shroud head on top and the top guide below. The central portion of the shroud surrounds the active fuel and forms the longest section of the shroud. This section is bounded at the bottom by the core support. The lower shroud, surrounding part of the lower plenum, is welded to the RPV shroud support (HNP-1-FSAR section 4.2 and HNP-2-FSAR section 5.4).

A set of four radial acting stabilizer assemblies is used to maintain the alignment of the core shroud to the RPV during seismic loading and a design basis accident (DBA). The set of stabilizers replaces the structural functions of the shroud horizontal girth welds. Each stabilizer attaches to the shroud flange at the top of the shroud, and for HNP-1, to a shroud support gusset at the bottom and for HNP-2, to a collet connector installed in the shroud support.

4.2.2.2.2 Shroud Head and Steam Separator Assembly

The shroud head and steam separator assembly is bolted to the top of the upper shroud to form the top of the core discharge plenum, which provides a mixing chamber for the steam-water mixture before it enters the steam separators. Individual stainless-steel axial flow steam separators, shown in figure 4.1-3, are attached to the top of standpipes that are welded into the shroud head. The steam separators have no moving parts. In each separator, the steam-water mixture rising through the standpipe passes vanes that impart a spin to establish a vortex separating the water from the steam. The separated water flows from the lower portion of the steam separator into the downcomer annulus.

4.2.2.2.3 Core Support

The core support consists of a circular stainless-steel plate with bored holes stiffened with a rim and beam structure. The plate provides lateral support and guidance for the control rod guide tubes, incore flux monitor guide tubes, peripheral fuel supports, and startup neutron sources. The last two items are also supported vertically by the core support plate.

The entire assembly is bolted to a support ledge between the central and lower portions of the core shroud. Alignment pins that engage slots and bear against the shroud are used to position the assembly correctly before it is secured. Plant modifications to eliminate significant incore vibrations in HNP-1 are described in references 20, 21, and 22.

4.2.2.2.4 Top Guide

The top guide is formed by a series of stainless-steel beams joined at right angles to form square openings, with the beams fastened to a peripheral rim. Each large opening provides lateral support and guidance for four fuel assemblies or, in the case of peripheral fuel, one or two fuel assemblies. Hanger slots are provided at the top of the beams to receive the top hooks of the temporary control curtains. Notches are provided in the bottom of the beam intersections to anchor the incore flux monitors and startup neutron sources. The top fuel guide is positioned with alignment pins which bare against the shroud.

4.2.2.2.5 Fuel Supports

The two basic types of fuel supports shown in figures 4.2-2 (HNP-1) and 4.2-3 (HNP-2) are:

1. Peripheral Supports

   The peripheral fuel support is located at the outer edge of the active core and is not adjacent to control rods. Each peripheral fuel support supports one fuel assembly and contains a single orifice assembly designed to ensure proper coolant flow to the fuel peripheral assembly.

2. Four-Lobed Orificed Fuel Supports

   Each four-lobed orificed fuel support supports four fuel assemblies and is provided with orifice plates to ensure proper coolant flow distribution to each rod-controlled fuel assembly. The four-lobed orificed fuel supports rest in the top of the control rod guide tubes, which are supported laterally by the core support. The control rods pass through slots in the center of the four-lobed orificed fuel support. A control rod and the four adjacent fuel assemblies represent a core cell (subsection 4.2.1).

4.2.2.2.6 Control Rod Guide Tubes

The control rod guide tubes, located inside the RPV, extend from the top of the control rod drive (CRD) housing up through holes in the core support plate. Each tube is designed as the guide for a control rod and as the vertical support for a four-lobed orificed fuel support piece and the four fuel assemblies surrounding the control rod. The bottom of the guide tube is supported by the CRD housing (HNP-1-FSAR section 4.2 and HNP-2-FSAR section 5.4), which in turn transmits the weight of the guide tube, fuel support, and fuel assemblies to the RPV bottom head. A thermal sleeve is inserted into the CRD housing from below and is rotated to lock the control rod guide tube in place. A key is inserted into a locking slot in the bottom of the CRD housing to hold the thermal sleeve in position.

4.2.2.2.7 Jet Pump Assemblies

The jet pump assemblies are located in two semicircular groups in the downcomer annulus between the core shroud and the RPV wall. The design and performance of the jet pump is covered in detail in references 1 and 2. Each stainless-steel jet pump consists of driving nozzles, suction inlet, throat, or mixing section, and diffuser (figure 4.2-4). The driving nozzle, suction inlet, and throat are joined together as a removable unit, and the diffuser is permanently installed. High-pressure water from the recirculation pumps (HNP-1-FSAR subsection 4.3.4 and HNP-2-FSAR subsection 5.5.1) is supplied to each pair of jet pumps through a riser pipe welded to the recirculation inlet nozzle thermal sleeve. A riser brace consists of cantilever beams extending from pads on the RPV wall. The nozzle entry section is connected to the riser by a metal-to-metal, spherical-to-conical seal joint. Firm contact is maintained by a holddown clamp. The throat section is supported laterally by a bracket attached to the riser. A slip-fit joint is located between the throat and diffuser. The diffuser is a gradual conical section changing to a straight cylindrical section at the lower end.

4.2.2.2.8 Steam Dryers

Steam dryers remove moisture from the wet steam leaving the steam separators. The extracted moisture flows down the dryer vanes to the collecting troughs and flows through tubes into the downcomer annulus (figure 4.1-4). A skirt extends from the bottom of the dryer vane housing to the steam separator standpipe, below the water level. This skirt forms a seal between the wet steam plenum and the dry steam flowing from the top of the dryers to the steam outlet nozzles.

Vertical guide rods facilitate positioning the dryer and shroud head in the vessel. The dryers rest on steam dryer support brackets attached to the reactor vessel wall. The dryers are restricted from lifting by steam dryer holddown brackets attached to the RPV top head over the top of the steam dryer lifting lugs when the head is in place.

4.2.2.2.9 Feedwater Spargers

The feedwater spargers are perforated stainless-steel headers located in the mixing plenum above the downcomer annulus (figure 4.1-1).

During an accident condition, feedwater piping spargers deliver water from the HPCI and RCIC systems to maintain RPV inventory.

A. HNP-1

HNP-1 feedwater spargers 284x402G001 have top-mounted elbows, each with a converging discharge nozzle to assure the sparger/thermal sleeve remains full of cold feedwater during low-flow conditions. This design assures that, during low-flow conditions, low-flow stratification is eliminated. The converging discharge orifices eliminate flow separation, which could cause flow hole cracking.

A separate sparger is fitted to each feedwater nozzle thermal sleeve via a forged tee connected to the sparger arms and is shaped to conform to the curvature of the RPV wall. The thermal sleeve configuration is drastically different from previous designs. The inner thermal sleeve is the feed pipe for the sparger and is sealed against the safe-end with a piston ring. The inner thermal sleeve is welded to the forged tee. A secondary seal is attached to an intermediate thermal sleeve that is open to the reactor at its downstream end. The annulus between the intermediate and inner thermal sleeves has a low hydraulic resistance and serves to channel leakage to the reactor without impinging on the feedwater nozzle. As a further impediment to leakage and to provide damping against vibration an interference fit is provided between the ring, which contains the secondary seal, and the nozzle safe-end. Sparger end brackets are attached to vessel brackets to support the weight of the spargers. Each feedwater sparger assembly has 28 flow nozzles. The header is fabricated from Type 304 austenitic stainless-steel 6.0-in. schedule 80 pipe.

B. HNP-2

The design of HNP-2 feedwater sparger 283x688G6 is based on the full-scale flow tests conducted on a Millstone feedwater sparger in the feedwater sparger test facility and is adequately designed to withstand the vibratory loads induced by flow transients (paragraph 4.2.2.3). It is also based on the successful operation of the Millstone "Design IV" feedwater sparger. The feedwater sparger is a top-mounted flow-nozzle design with a welded-in thermal sleeve to the feedwater nozzle safe end. Test data in the full-scale test facility show no differences between the welded-in and the interference fit designs of feedwater spargers in their ability to minimize or eliminate flow-induced vibration.

Feedwater flow enters the center of the spargers and is discharged radially inward to mix the cooler feedwater with the downcomer flow from the steam separators before it contacts the RPV wall. The feedwater also condenses the steam in the region above the downcomer annulus and subcools the water flowing to the jet pumps and recirculation pumps.

Each feedwater sparger assembly has 30 flow nozzles. The header is fabricated from Type 304 austenitic stainless-steel 6.0-in. schedule 80 pipe.

4.2.2.2.10 Core Spray Lines

The CS lines are the means for directing flow to the CS nozzles, which distribute coolant so that peak fuel cladding temperatures of 2200°F are not exceeded during accident conditions.

Two CS lines enter the RPV through the two CS nozzles (figure 4.1-1, and HNP-1-FSAR subsection 6.4.3 and HNP-2-FSAR section 6.3). The lines divide immediately inside the RPV. The two halves are routed to opposite sides of the RPV and are supported by clamps attached to the RPV wall. The lines are then routed downward into the downcomer annulus and pass through the upper shroud immediately below the flange. The flow divides again as it enters the

center of the semicircular sparger, which is routed halfway around the inside of the upper shroud. The ends of the two spargers are supported by brackets designed to accommodate thermal expansion. The line routing and supports are designed to accommodate differential movement between the shroud and RPV. The other CS line is identical, except that it enters the opposite side of the RPV. The spargers are at a slightly different elevation inside the shroud. The correct spray distribution pattern is provided by a combination of distribution nozzles pointed radially inward and downward from the spargers.

4.2.2.2.11 Differential Pressure and Liquid Control Line

The differential pressure and liquid control line (figure 4.1-1) serves a dual function within the RPV:

- Provide a path for the injection of the liquid control solution into the coolant stream (HNP-1-FSAR subsection 3.8.4 and HNP-2-FSAR paragraph 4.2.3.4).

- Sense the differential pressure across the core support plate (HNP-1-FSAR section 4.2 and HNP-2-FSAR section 5.4).

The differential pressure and liquid control line enters the RPV at a point below the core shroud as two concentric pipes. In the lower plenum, the two pipes separate. The inner pipe terminates near the lower shroud with a perforated length below the core support plate. The inner pipe is used to sense the pressure below the core support plate during normal operation and to inject liquid control solution if required. This location facilitates good mixing and dispersion. The inner pipe also reduces thermal shock to the RPV nozzle should the standby liquid control system (SLCS) be actuated. The outer pipe terminates immediately above the core support plate and senses the pressure in the region outside the fuel assemblies.

4.2.2.2.12 Incore Flux Monitor Guide Tubes

The incore flux monitor guide tubes provide a means of positioning fixed detectors in the core and a path for calibration monitors [traversing incore probe (TIP) system] and extend from the top of the incore flux monitor housing (HNP-1-FSAR section 4.2 and HNP-2-FSAR section 5.4) in the lower plenum to the top of the core support plate. The power range detectors for the power range monitoring units and the dry tubes for the source range (SRM) and intermediate range monitor (IRM) detectors are inserted through the guide tubes and are held in place below the top guide by spring tension. A latticework of clamps, tie bars, and spacers gives lateral support and rigidity to the guide tubes. The bolts and clamps are welded in place, after assembly, to prevent loosening during reactor operation.

4.2.2.2.13 Surveillance Sample Holders

The surveillance sample holders are welded baskets containing impact and tensile specimen capsules (HNP-1-FSAR appendix R and HNP-2-FSAR paragraph 5.2.4.4). The baskets hang from brackets that are attached to the inside wall of the RPV and extend to mid-height of the active core. The radial positions are chosen to expose the specimens to the same environment

and maximum neutron fluxes experienced by the RPV itself while avoiding jet pump removal interference or damage.

4.2.2.3 Safety Evaluation

4.2.2.3.1 Evaluation Methods

To determine that the safety design bases are satisfied, responses of the RPV internals to loads imposed during normal, upset, emergency, and faulted conditions are examined. The effects on the ability to insert control rods, cool the core, and flood the inner volume of the RPV are determined.

4.2.2.3.1.1 Input for Safety Evaluation. The operating conditions that provide the basis for the design of the reactor internals to sustain normal, upset, emergency, and faulted conditions, as well as combinations of design loadings accounted for in design of the core support structure, are covered in HNP-1-FSAR table C.3-1 and HNP-2-FSAR table 4.2-9.

In addition, each combination of operating loads is categorized with respect either to normal, upset, emergency, or faulted conditions, as well as the associated design stress intensity or deformation limits.

The bases for the proposed design stress and deformation criteria are specified in HNP-1-FSAR appendix C and HNP-2-FSAR chapter 3.

4.2.2.3.1.2 Events To Be Evaluated. Examination of the spectrum of conditions for which the safety design basis must be satisfied reveals three significant upset events:

A. Recirculation Line Break [Loss of Coolant Accident (LOCA)]: the accident results in pressure differentials within the RPV.

B. Main Steam Line Break Accident (MSLBA): a break in one main steam line between the RPV and the flow restrictor. The accident results in significant pressure differentials across some of the structures within the reactor.

C. Earthquake: it subjects the core support structures and reactor internals to significant forces as a result of ground motion.

For other conditions existing during normal operation, AOOs, and accidents, the loads affecting the core support structures and reactor internals are less severe than these three postulated events.

4.2.2.3.1.3 Pressure Differential During Rapid Depressurization. A digital computer code is used to analyze the transient conditions within the RPV following the recirculation line

break accident (LOCA) and the MSLBA.[3] The analytical model of the RPV consists of nine nodes that are connected to the necessary adjoining nodes by flow paths having the required resistance and inertial characteristics. The program solves the energy and mass conservation equations for each node to give the depressurization rates and pressure in the various regions of the reactor. Figure 4.2-5 shows the nine reactor nodes.

4.2.2.3.2 Recirculation Line Break Accident and MSLBA

4.2.2.3.2.1 **Accident Definition**. Both a recirculation line break accident (the largest liquid break) and an inside MSLBA (the largest steam break) are considered in determining the design basis accident (DBA) for the reactor internals. The recirculation line break is the same as the design basis LOCA, as described in HNP-2-FSAR subsection 6.3.3 for both HNP-1 and HNP-2. A sudden, complete circumferential break is assumed to occur in one recirculation loop. The analysis of the MSLBA assumes a sudden, complete circumferential break of one main steam line between the RPV vessel and the main steam line restrictor. This is not the same accident described in subsection 15.3.5, which has greater potential radiological effects. A steam line break upstream of the flow restrictors produces a larger blowdown area and, thus, a faster depressurization rate than a break downstream of the restrictors. The larger blowdown area results in greater pressure differentials across the reactor assembly internal structures.

The MSLBA produces higher pressure differentials across the reactor internal structures than does the recirculation line break. This results from the higher reactor depressurization rate associated with the MSLB. The depressurization rate is proportional to the mass flowrate and the excess of fluid escape enthalpy above saturated water enthalpy, h_f. Mass flowrate is inversely proportional to escape enthalpy, h_e; therefore, the depressurization rate is approximately proportional to $1-h_f/h_e$. Consequently, depressurization rate decreases as h_e decreases; that is, the depressurization rate is less for mixture flow than for steam flow. Therefore, the MSLBA is the DBA for internal pressure differentials.

4.2.2.3.2.2 **Effects of Reactor Power and Core Flow**. For illustration, the maximum internal pressure loads can be considered to be composed of two parts: steady-state and transient pressure differentials. For a given plant, core flow and power are the two major factors that influence the reactor internal pressure differentials. The core flow essentially affects only the steady-state part. For a fixed power, the greater the core flow, the larger the steady-state pressure differential. The core power affects both the steady-state and the transient parts. As the power is decreased, there is less voiding in the core, and consequently, the steady-state core pressure differential is less. However, less voiding in the core also means that less steam is generated in the RPV and, thus, the depressurization rate and the transient part of the maximum pressure load are increased.

It is necessary to determine the combination of core power and flow, which results in the maximum internal differential pressure loads. This condition could occur at high power and flow (the upper right corner of the power-to-flow map), or low power and high flow (the lower right corner of the power-to-flow map). The power-to-flow map is provided in figure 15.1-3.

The initial safety analysis was performed at 2537 MWt and 100% core flow for the high power and high flow condition. These faulted pressure differentials were compared to a low power, high core flow case in which the core flow reached ~ 110% of rated. This analysis showed that the low power, high core flow case is more limiting with regard to maximum pressure differentials following a steam line break inside containment. As explained above, this is because the decrease in core flow and power reduces the steady-state part of the maximum pressure load more than the corresponding increase in the transient part. Hence, the maximum pressure loads (steady state plus transient) are less if core flow is reduced from its maximum value.

The safety analysis was performed again for the increase in licensed power level to 2804 MWt and reactor operating pressure increase from 1050 psia to 1060 psia.[24] [25] The limiting condition for the MSLBA (faulted) condition, as well as normal operation and AOO (upset) conditions, is reported for HNP-1 and HNP-2 in tables 4.2-10 and 4.2-11, respectively.

Reference 15 contains additional information on reactor internal pressure differences.

4.2.2.3.2.3 Response of Structures Within the Reactor Pressure Vessel to Pressure Differences. Maximum differential pressures are used in combination with other structural loads to determine the total loading on the various structures within the reactor. The structures are then evaluated to assess the extent of deformation and buckling instability, if any. Of particular interest are the responses of the guide tubes and the metal channels around the fuel bundles and the potential leakage around the jet pump joints.

A. Guide Tube

The guide tube is evaluated for buckling instability caused by externally applied pressure. The two primary modes of failure analyzed are described in paragraph 4.2.3.1.3. For a guide tube with minimum wall thickness and maximum allowed ovality, the pressure that causes yield stress is 105 psi compared to the design pressure of 37.5 psi. The design pressure is greater than the 30-psi maximum pressure differential the guide tube experiences, including accident conditions. The stress the guide tube could experience is ~ 5400 psi due to external pressure (37.5 psi), a 1.2-g earthquake (include deadweight loading), and lateral loading due to coolant flow, while yield stress at 575°F is l7,500 psi. It is concluded that the guide tube does not fail under the assumed conditions.

B. Fuel Channel

The fuel channel load resulting from an internally applied pressure is evaluated, using a fixed-beam analytical model under a uniform load. Tests to verify the applicability of the analytical model indicate that the model is conservative. A roller at the top of the control rod guides the blade as it is inserted. If the gap between channels is less than the diameter of the roller, the roller deflects the channel walls as it makes its way into the core. The friction force is a small percentage of the total force available to the CRDs for overcoming such friction, and it is concluded that the MSLBA does not impede the insertability of the control rod.

C. Jet Pump Joints

Jet pump joints were analyzed to evaluate the potential leakage from within the floodable inner volume of the RPV during the recirculation line break and subsequent low-pressure coolant injection (LPCI) reflooding. Because the jet pump diffuser is welded to the shroud support, the only remaining source of leakage from the lower plenum to the downcomer annulus is the jet pump throat-to-diffuser joint at 225 gal/min.

LPCI capacity is sized to accommodate on HNP-1 3000 gal/min and on HNP-2 500-gal/min leakage at these locations. It is concluded that the RPV structures retain sufficient integrity during the recirculation line break accident to allow reflooding of the inner volume of the RPV and in sufficient time to prevent significant increases in cladding temperature.

4.2.2.3.3 Earthquake

Seismic loads acting on the structures within the RPV are based upon a dynamic analysis of a model as described in HNP-1-FSAR appendix C and HNP-2-FSAR section 3.7.

4.2.2.3.4 Conclusions

Response analyses of the reactor structures show that deformations are sufficiently limited to allow both adequate control rod insertion and proper operation of the ECCS. Sufficient integrity of the structures is retained during accident conditions to allow successful reflooding of the RPV inner volume. The analyses considered various loading combinations, including loads imposed by external forces. Thus, the safety design bases listed in paragraph 4.2.2.1.1.1 are satisfied.

4.2.2.4 Design Bases Criteria

The reactor core support structures and internals meet the safety design bases and power generation design bases specified in paragraph 4.2.2.1.1. This is accomplished without exceeding the design basis conditions for normal, upset, emergency, and faulted conditions described in HNP-1-FSAR appendix C and HNP-2-FSAR table 4.2-9. The internals and core support structures design stress and deformation criteria are specified in HNP-1-FSAR appendix C and HNP-2-FSAR chapter 3.

4.2.3 REACTIVITY CONTROL SYSTEM

The reactivity control system consists of the control rods, the CRDs, supplementary reactivity control, and the SLCS.

4.2.3.1 Control Rods

4.2.3.1.1 Design Bases

4.2.3.1.1.1 General Design Bases. The general design bases for the control rods are as follows:

A. Safety Design Bases

1. Control rods have sufficient mechanical strength to prevent displacement of their reactivity control material.

2. Control rods have sufficient strength and are designed to prevent deformation that could inhibit their motion.

3. Each control rod has a device to limit its free-fall velocity sufficiently to avoid damage to the nuclear system process barrier by the rapid reactivity increase resulting from a free fall of one control rod from its fully inserted position to the position where the drive was withdrawn.

B. Power Generation Design Bases

The reactivity control mechanical design includes reactivity control devices (control rods and gadolinia burnable poison) that contain and position the material controlling the excess reactivity in the core. Control rods have the capability of being either removed or replaced as required.

4.2.3.1.1.2 Specific Design Characteristics. The specific design characteristics of the control rods are as follows:

A. Control Rod Clearances

The basis of the mechanical design of the control rod blade clearances is that there is no interference, which restricts passage of the control rod blade.

B. Mechanical Insertion Requirements

Mechanical insertion requirements during normal operation are selected to provide adequate operability and the capability to control the reactivity addition resulting from burnout of peak shutdown xenon at 100% power.

Scram insertion requirements are chosen to provide sufficient shutdown margin to meet all safety criteria for AOOs described in section 15.2.

C. Material Selection

The selection of materials for use in the control rod design is based upon their in-reactor properties. Type 304 high purity stabilized stainless steel is used for absorber tubing for both the GE Duralife models (figures 4.2-6 and 4.2-7) and the GE Marathon model (Figure 4.2-17) control blade designs, comprising a major portion of the control rod assembly for both design types. Type 316L stainless steel is used for the Westinghouse CR 99 control rods.

The absorber tubing In the Duralife control rod designs is thinner than in the Marathon design, since the absorber tubes for the Duralife model are not intended to provide the main structure of the control rod assembly. Type 304 commercial grade stainless steel is typically used for the Duralife sheath and frame structure, with Type 316 stainless steel as an alternate material used later for the tie rod material. The absorber tubing for the GE Marathon design is welded together to form the absorber section and provide the main structure for the control rod assembly. Therefore, the Marathon absorber tubing is thicker with added surface features to allow the tubes to be welded together.

Boron carbide (B_4C) powder and Inconel-X are used in the Duralife and Marathon control rod designs. The B_4C for the Marathon design is first loaded into thin vented capsules of Type 304 commercial grade stainless steel, before being loaded and sealed into the absorber rods. Solid boron carbide (B_4C) pins are used in the Westinghouse CR 99 control rod designs.

The primary materials used in the Duralife, Marathon, and Westinghouse designs are well known and are taken into account in establishing the mechanical design of the control rod components. The basic cruciform control rod design and materials have been operating in all GE reactors since the 1980s and before.

Hafnium absorber parts are used in the GE Duralife and Marathon control rod designs. The performance of Hafnium as a reactivity control in a BWR environment is documented in NEDE-22290A.(8) The hafnium absorber material used for some of the absorber rods In the Marathon blade design are contained and sealed within the absorber rod tubes and are not exposed to reactor coolant, as described in NEDE-31758-P-A.(24)

GE Duralife and Marathon model control rod designs have used 13-8-MO PH stainless steel for the roller pin material since the early 1980s. The roller material is Inconel-X, also used since the early 1980s.

D. Radiation Effects

The radiation effects on B_4C powder and solid B_4C pins include the release of gaseous products, and the B_4C cladding is designed to sustain the resulting internal pressure buildup. The corrosion rate and the physical properties (density, modulus of elasticity, dimensional aspects) of Type 304 commercial grade and high-purity stabilized stainless steels, Type 316 stainless steel, and Inconel-X are

essentially unaffected by the irradiation experienced in the BWR reactor core. The effects upon the mechanical properties, such as yield strength, ultimate tensile strength, percentage of elongation, and ductility on the Type 304 and Type 316 stainless-steel cladding also are well known and are considered in mechanical design.

E. Positioning Requirements

Rod-positioning increments (notch lengths) are selected to provide adequate power-shaping capability. The combination of rod speed and notch length must also meet the limiting reactivity addition rate criteria.

4.2.3.1.2 Description

Plant Hatch uses control rods designed by GE (Duralife and Marathon) and Westinghouse (CR 99).

4.2.3.1.2.1 **GE Control Rods.** The control rods (figures 4.2-6 and 4.2-7 for the GE Duralife model and figure 4.2-17 for the GE Marathon) perform the dual function of power shaping and reactivity control. Power distribution in the core is controlled during operation of the reactor by manipulating selected patterns of control rods. Control rod displacement tends to counterbalance steam void effects at the top of the core and results in significant power flattening.

The control rods are 9.75 In. in total span and are separated uniformly throughout the core on a 12-in. pitch. Each control rod is surrounded by four fuel assemblies.

For the GE Duralife model design, the control rod consists of a sheathed cruciform array of stainless-steel tubes filled with B_4C powder. For neutron absorption, the GE Duralife design (D-190 and D-230 models used at Hatch) uses B_4C -filled tubes and solid-Hafnium strips, along with solid-Hafnium plates in the upper 6 in. of the B_4C -filled tubes.[11] [12] The main structural member of the GE Duralife control rod Is made of Type 304 and/or Type 316 stainless steel and consists of a top handle, bottom casting or assembly with a velocity limiter and CRD coupling, vertical cruciform center post, and four U-shaped absorber tube sheaths. The top handle, bottom velocity limiter assembly, and center post are welded into a single skeletal structure. The U-shaped sheaths are resistance welded to the center post, handle, and castings to form a rigid housing to contain the absorbing rodlets.

For the GE Marathon model design, the absorber rods are welded together to form the absorber section without the sheathing strip used in the Duralife design. The wings are welded to spacers which make up the tie rod structure to form the cruciform-shaped member of the control rod. Depending on the desired-nuclear design application the absorber rods are loaded with B_4C-filled capsules, empty capsules (for extra plenum), solid hafnium rods (typically along the outside length of the absorber sectlons) or left empty (typically next to the tie rod structure). The absorber rods are sealed, then the top handle and bottom velocity limiter assembly with CRD coupler, of similar design to that used on the Duralife model control rod, are attached.[24]

Operating experience has shown that control rods, constructed as described above, are not susceptible to dimensional distortions. The B_4C powder in the absorber tubes is compacted to ~ 70% of its theoretical density. The B_4C contains a minimum of 76.5% by weight natural boron. The boron-10 (B-10) minimum content of the boron is 18% by weight.

Absorber tubes for Duralife and Marathon control rods are made of Type 304 high-purity stabilized stainless steel. Each absorber tube in the Duralife D-190 control rod is 0.188 in. in outside diameter (OD) (0.138-in. inside diameter (ID)). In the GE D-230 control rod, the absorber tube is 0.220 in. in OD (0.180-in. ID). The Hafnium strip thickness is 0.188 in. for both the D-190 and D-230 designs. The Hafnium plates in the D-190 control rods are 0.188 in. thick. The plates in the GE D-230 control rods are 0.220 in. thick.(8)

The OD of the absorber tube in the GE Marathon design is 0.298 in. and the OD of the absorber rod capsule in the GE Marathon design is 0.241 in. (~ 0.236-in. ID). The Hafnium rods are ~ 0.215 in. in diameter.(24) (25)

For the GE Duralife control rod design, the absorber tubes are sealed by a plug welded into each end. The boron carbide is longitudinally separated into individual compartments by stainless-steel balls at ~ 16-in. intervals. The steel balls are held in place by a slight crimp of the tube. Should boron carbide tend to compact further during service, the steel balls distribute the resulting voids over the length of the absorber tube.

For the GE Marathon control rod design, the absorber rod capsules of ~ 11 to 36 in. in length control the compaction of the B_4C.(24) (25)

The Duralife D-190 and D-230, as well as Marathon control rod designs, are designed such that their control strength (i.e., negative reactivity) nearly matches that of the original all-B_4C control rod design. The reduction in control strength for these hybrid B_4C and Hafnium control rod designs is described in terms of B-10 depletion.

The operating lifetime of the control blades is governed by either nuclear reactivity or mechanical stress considerations, whichever proves most limiting.

A. The nuclear lifetime limit is reached when the peak boron depletion results in a 10% loss in relative control worth of any 3-ft axial section of the blade.

B. The mechanical lifetime limit is reached when the internal helium pressure from the B-10 (neutron, alpha) reaction results in stresses in any absorber tube of the control rod reaching the most restrictive design limit.

The actual replacement of control rods by these criteria depends on the service history of individual control blades.

If the control rod blades are subjected to sufficient exposure to cause ~ 50% local depletion of the poison tube B-10, the potential for tube cracking and boron-leaching exists. (4, 5, 6, 7)

The cracking is due to stress corrosion induced by solidification of B_4C particles and swelling of the compacted B_4C as helium and lithium concentrations grow. Once primary coolant

penetrates the cladding, i.e., the cracking has progressed through the cladding wall and the helium-lithium pressures are sufficient to open the crack, boron is leached out of the tube at locations with more then 50% B-10 local depletion. (Local depletion is considered to be twice the average depletion.) The cracking and boron loss shorten the design life of the control blade.

The end-of-design life is reached when the reactivity worth of the blade is reduced by 10%, which corresponds to 62% and 58% B-10 depletion averaged over the top quarter in the D-190 and D-230 control blades, respectively, and 56% depletion in the bottom three-quarter segments of the control blade. The end-of-design life for the GE Marathon design (corresponding to the 10% reduction in the reactivity worth of the blade) is 68% B-10 depletion averaged over any of the four axial quarter segments. The average mechanical lifetime of the control rods is calculated to be ~18 years of full-power operation. The actual replacement of control rods depends on the loss of reactivity control capability and gas pressure buildup and varies among control rods. The average expected service life of control rods is ~15 years.

The control rod velocity limiter (figures 4.2-8 and 4.2-9) is an integral part of the bottom assembly of each control rod. This engineered safety feature (ESF) system protects against a high-reactivity insertion rate by limiting the control rod velocity in the event of a control rod drop accident (CRDA). It is a one-way device in that the control rod scram velocity is not significantly affected, but the control rod dropout velocity is reduced to a permissible limit.

The velocity limiter utilizes an optimized twin-reverse jet design.
The hydraulic drag forces on a control rod are proportional to approximately the square of the rod velocity and are negligible at normal rod withdrawal or rod insertion speeds. However, during the scram stroke the rod reaches high velocity, and the drag forces must be overcome by the drive mechanism.

To limit control rod velocity during dropout but not during scram, the velocity limiter is provided with a streamlined profile in the scram (upward) direction. Thus, when the control rod is scrammed, water flows over the smooth surface of the upper side of the limiter into the annulus between the guide tube and the limiter. In the dropout direction, however, water is trapped by the lower curvature of the limiter and discharged through the annulus between the two sections. Because this water is jetted in a partially reversed direction into water flowing upward in the annulus, a severe turbulence is created, thereby slowing the descent of the control rod assembly to < 3.11 ft/s at 70°F.

4.2.3.1.2.2 **Westinghouse BWR CR 99 Control Rod Design**. The basic design of the Westinghouse BWR control rods (figure 4.2-18) consists of four stainless steel plates welded together to form a cruciform shaped rod. Each sheet has horizontally drilled holes to contain the absorber material. A velocity limiter is welded to the bottom of the control rod, also including a coupling device that connects the control rod to the control rod drive.

The CR 99 control rod design for the Hatch units is outlined as follows:

Absorber

The absorber consists of four stainless steel (AISI 316L SS) blade wings discontinuously welded together in the center to a cruciform shape, providing the necessary mechanical stability. There are 15 welded shoulders in the center of the absorber cross. To minimize activation and dose, the cobalt content in the stainless steel blade wings is below 0.02 w/o.

The blade wings are 8.05 mm (0.317-in.) thick. In each blade, 454 holes are horizontally drilled to contain the absorber material, hot isostatic pressed (HIP) boron carbide pins. Each absorber hole contains two boron carbide pins separated by a spring, which presses the pins toward the bar at the edge of the blade and toward the bottom gable of the hole, respectively (see figure 4.2-19).

The boron carbide pins are tapered to provide more space for diametrical swelling in the high peaking factor area close to the outer edge of the blade wing.

In addition to the tapering, the upper 10 and the lower 113 holes are filled with boron carbide pins with reduced dimensions to provide more space for diametrical and axial boron swelling in the upper holes, with high axial peaking factor, and for helium gas expansion in the lower holes.

All absorber holes are covered with a steel bar that fits in a slot along the outer blade wing edge. A leaktight closure is then obtained by welding together the shanks that are rolled over the bar. The holes are still connected to each other through a communication channel in that pressure equalization of the helium gas generated in the boron carbide pins during irradiation can take place. Each blade wing forms a separate enclosure which is tested for leaks after the welding.

Handle

The design of the lifting double handle is compatible to the control rods of GE design. The handle is integrated with the blade wing, welded together in the center to form the lifting handle.

Velocity Limiter and Coupling Device

The bottom part of the control rod includes a velocity limiter with a coupling device. The velocity limiter is welded to the blade wings, and the coupling device is mounted by a thread and finally lock-welded to the velocity limiter.

4.2.3.1.3 Safety Evaluation

4.2.3.1.3.1 Materials Adequate Throughout Design Lifetime. The adequacy of the materials throughout the design life was evaluated in the mechanical design of the control rods. The primary materials, B_4C powder, solid B_4C pins, commercial grade and high-purity stabilized Type 304 stainless steel, Type 316 stainless steel, Inconel-X, and Hafnium were found suitable in meeting the demands of the BWR environment.

4.2.3.1.3.2 Dimensional and Tolerance Analysis. Layout studies are made to ensure that, given the worst combination of extreme detail part tolerance ranges at assembly, no interference exists which restricts the passage of control rods. In addition, preoperational verification is made on each control blade system to show that acceptable levels of operational performance are met.

4.2.3.1.3.3 Thermal Analysis of the Tendency to Warp. The various parts of the control rod assembly remain at approximately the same temperature during reactor operation, negating the problem of distortion or warpage. What little differential thermal growth could exist is allowed for in the mechanical design. A minimum axial gap is maintained between absorber rod tubes and the control rod frame assembly for this purpose. In addition, dissimilar metals are avoided to further this end.

4.2.3.1.3.4 Forces for Expulsion. An analysis of the maximum pressure forces that could tend to eject a control rod from the core was performed. The results of this analysis are given in paragraph 4.2.3.2.3.1. In summary, if the collet were to remain open, which is unlikely, calculations indicate that the steady-state control rod withdrawal velocity will be 2 ft/s for a pressure-under line break, the limiting case for rod withdrawal. (A complete description of the collet is provided in paragraph 4.2.3.2.2.2.)

4.2.3.1.3.5 Functional Failure of Critical Components. The consequences of a functional failure of critical components were evaluated, and the results are covered in paragraph 4.2.3.2.3.2.

4.2.3.1.3.6 Precluding Excessive Rates of Reactivity Addition. To preclude excessive rates of reactivity addition, the design is based upon analyses that were performed both on the velocity limiter device and the effect of probable control rod failures (paragraph 4.2.3.2.3.1).

4.2.3.1.3.7 Effect of Fuel Rod Failure on Control Rod Channel Clearances. The CRD mechanical design ensures a sufficiently rapid insertion of control rods to preclude the occurrence of fuel rod failures which could hinder reactor shutdown by causing significant distortions in channel clearances.

4.2.3.1.3.8 Effects of Blowdown Loads on Control Rod Channel Clearances. The fuel channel load resulting from an internally applied pressure is evaluated, using a fixed-beam analytical model under a uniform load. Tests to verify the applicability of the analytical model indicate that the model is conservative. A roller at the top of the control rod guides the blade as it is inserted. If the gap between channels is less than the diameter of the roller, the roller deflects the channel walls as it makes its way into the core. The friction force is a small percentage of the total force available to the CRDs for overcoming such friction, and it is concluded that the MSLBA does not impede the insertability of the control rod.

4.2.3.1.3.9 **Mechanical Damage.** Analyses performed for all areas of the control system showed that system mechanical damage does not affect the capability to provide reactivity control continuously.

In addition to the analysis performed on the CRD (paragraphs 4.2.3.2.3.1 and 4.2.3.2.3.2), the following discussion summarizes the analysis performed on the control rod guide tube.

The guide tube can be subjected to any or all of the following loads:

- Inward load due to pressure differential.
- Lateral loads due to flow across the guide tube.
- Deadweight.
- Seismic.

In all cases, an analysis was performed considering both the LOCA and the MSLBA, events that result in the largest hydraulic loadings on a control rod guide tube.

The two primary modes of failure considered in the guide tube analysis are exceeding allowable stress and excessive elastic deformation. It was found that the allowable stress limit is not exceeded and that the elastic deformations of the guide tube are never great enough to cause free movement of the control rod to be jeopardized.

The first mode of failure is evaluated by the addition of all stresses resulting from maximum loads for the faulted condition. This results in the maximum theoretical stress value for that condition. Making a linear supposition of all calculated stresses and comparing this value to the allowable limit defined by the ASME Boiler and Pressure Vessel Code yields a factor of safety of ~ 3. For faulted conditions, the factor of safety is ~ 4.4.

Evaluation of the second mode of failure is based upon clearance reduction between the guide tube and the control rod. The minimum allowable clearance is ~ 0.1 in. This assumes maximum ovality and minimum diameter of the guide tube and the maximum control rod dimension. The analysis showed that if the approximate 6000 psi for the faulted condition were entirely the result of differential pressure, the clearance between the control rod and the guide tube will reduce by a value of ~ 0.01 in. This gives a design margin of 10 between the theoretically calculated maximum displacement and the minimum allowable clearance.

The two types of instability considered in the analysis of guide tube design are:

- The classic instability associated with vertically loaded columns.
- The diametral collapse when a circular tube experiences external-to-internal differential pressure.

The limiting axially applied load is ~ 77,500 lb, resulting in a material compressive stress of 17,450 psi (Code allowable stress). Comparing the actual load to the yield stress level gives a design margin > 20 to 1. From these values, it is concluded that the guide tube is not an unstable column.

When a circular tube experiences external-to-internal differential pressure, two modes of failure are possible, depending on whether the tube is long or short. In the analysis here, the guide tube is taken to be an infinitely long tube with the maximum allowable ovality and minimum wall thickness. The conditions result in the lowest critical pressure calculation for the guide tube. That is, if the tube is short, the critical pressure calculation gives a higher number. The critical pressure is ~ 140 psi. However, if the maximum allowable stress is reached at a pressure lower than the critical pressure, then that pressure is limiting. This is the case for a BWR guide tube. The allowable stress of 17,450 psi is reached at ~ 93 psi. Comparing the maximum possible pressure differential for a steam line break to the limiting pressure of 93 psi gives a design margin > 3 to 1. Therefore, the guide tube is not unstable with respect to differential pressure. References 17 and 18 provide a detailed discussion of analyses and design margins for the control rod guide tube.

4.2.3.1.3.10 Evaluation of Control Rod Velocity Limiter. The control rod velocity limiter limits the free-fall velocity of the control rod to a value that cannot result in nuclear system process barrier damage. This velocity is evaluated by the CRDA analysis in section 15.3.

4.2.3.1.4 Tests and Inspections

The control rod absorber tube tests are examples of the quality control tests performed on the control rods. The absorber tube tests include the following:

- Material integrity of the tubing and end plug verified by ultrasonic inspection.
- The B-10 fraction of the boron content of each lot of boron carbide verified.
- Weld integrity of the finished absorber tubes verified by helium leak testing.

4.2.3.1.5 Instrumentation

The instrumentation for both the control rods and the CRDs is defined by that given for the reactor manual control system (RMCS). The objective of the RMCS is to provide the operator with the means to make changes in nuclear reactivity so that reactor power level and power distribution can be controlled. The system allows the operator to manipulate control rods.

The design bases and further discussion are presented in HNP-1-FSAR section 7.7 and HNP-2-FSAR subsection 7.7.1.

4.2.3.2 <u>CRD System</u>

4.2.3.2.1 Design Bases

4.2.3.2.1.1 <u>General Design Bases</u>. The general design bases for the CRD system are as follows:

A. Safety Design Bases

The CRD mechanical system meets the following safety design bases:

- Design provides for a sufficiently rapid control rod insertion so that no calculated fuel damage results from any AOO.
- Design includes positioning devices, each of which individually supports and positions a control rod.
- Each positioning device:
  - Prevents its control rod from withdrawing as a result of a single malfunction.
  - Be individually operated so that a failure in one positioning device does not affect the operation of any other positioning device.
  - Be individually energized when rapid control rod insertion (scram) is signaled so that failure of power sources external to the positioning device does not prevent the positioning devices of other control rods from being inserted.
  - Be locked to its control rod to prevent undesirable separation.
- The CRD mechanisms and that part of the CRD hydraulic system (CRDHS) necessary for scram shall be designed to Seismic Category 1 requirements.

B. Power Generation Design Bases

The CRD system design provides for positioning the control rods to control power generation in the core.

4.2.3.2.2 Description

The CRD system controls gross changes in core reactivity by incrementally positioning neutron-absorbing control rods within the reactor core in response to manual control signals. It

is also required to scram the reactor in emergency situations by rapidly inserting withdrawn control rods into the core in response to a manual or automatic signal. The CRD system consists of locking piston, CRD mechanisms, alternate rod insertion (ARI) system, and the CRDHS (including hydraulic control units, interconnecting piping, instrumentation, and electrical controls).

4.2.3.2.2.1 **Control Rod Drive Mechanism.** The CRD mechanism (drive) used for positioning the control rod in the reactor core is a double-acting, mechanically latched, hydraulic cylinder using water as its operating fluid (figures 4.2-10 through 4.2-13). The individual drives are mounted on the bottom head of the RPV. The drives do not interfere with refueling and are operative even when the RPV head is removed. The drives are also readily accessible for inspection and servicing. The bottom location makes maximum use of the water in the reactor as a neutron shield and gives the least possible neutron exposure to the drive components. Using water from either the condensate and feedwater system taken downstream of the condensate polishing system or the condensate storage tank (CST) as the operating fluid eliminates the need for special hydraulic fluid. Simple piston seals are utilized in drives since leakage does not contaminate the RPV water and allows cooling of the drive mechanisms and their seals.

The drives are capable of inserting or withdrawing a control rod at a slow, controlled rate as well as providing rapid insertion when required. A mechanism on the drive locks the control rod in 6-in. increments of stroke over the length of the core.

A coupling spud at the top end of the drive index tube (piston rod) engages and locks into a mating socket at the base of the control rod. The weight of the control rod is sufficient to engage and lock this coupling. Once locked, the drive and rod form an integral unit that must be manually unlocked by specific procedures before the components can be separated.

The drive holds its control rod in distinct latch positions until the CRDHS actuates movement to a new position. Withdrawal of each rod is limited by the seating of the rod in its guide tube. Withdrawal beyond the overtravel limit can be accomplished only if the rod and drive are uncoupled. Withdrawal past the overtravel limit is annunciated by an alarm.

The individual rod indicators, grouped in one control panel display, correspond to relative rod locations in the core. A separate, smaller display is located just below the large display on the vertical part of the benchboard. This display presents the positions of the control rod selected for movement and the other rods in the affected rod group.

For display purposes, the control rods are considered in groups of four adjacent rods centered around a common core volume. Each group is monitored by four local power range monitor (LPRM) strings (HNP-1-FSAR subsection 7.5.6, and HNP-2-FSAR subsections 4.4.6 and 7.6.1). Rod groups at the periphery of the core may have less than four rods. The small rod display shows the positions, in digital form, of the rods in the group to which the selected rod belongs. A white light indicates which of the four rods is the one selected for movement.

4.2.3.2.2.2 **Control Rod Drive Components**. Figure 4.2-11 illustrates the operating principle of a CRD. Figures 4.2-12 and 4.2-13 illustrate the CRD in more detail. The main components of the CRD and their functions are described as follows.

The CRD piston is mounted at the lower end of the index tube. This tube functions as a piston rod. The CRD piston and index tube make up the main moving assembly in the CRD. The CRD piston operates between positive end stops, with a hydraulic cushion provided at the upper end only. The piston has both inside and outside seal rings and operates in an annular space between an inner cylinder (fixed piston tube) and an outer cylinder (drive cylinder). Because the type of inner seal used is effective in only one direction, the lower sets of seal rings are mounted with one set sealing in each direction.

A pair of nonmetallic bushings prevents metal-to-metal contact between the piston assembly and the inner cylinder surface. The outer piston rings are segmented step-cut seals with expander springs holding the segments against the cylinder wall. A pair of split bushings on the outside of the piston prevents piston contact with the cylinder wall. The effective piston area for downtravel, or withdrawal, is ~ 1.2 in.^{2} vs 4.1 in.^{2} for uptravel, or insertion. This difference in driving area tends to balance the control rod weight and ensures a higher force for insertion than for withdrawal.

The index tube is a long, hollow shaft made of nitrided Type 304 stainless steel. Circumferential locking grooves spaced every 6 in. along the outer surface transmit the weight of the control rod to the collet assembly.

The collet assembly serves as the index tube locking mechanism. It is located in the upper part of the drive unit. This assembly prevents the index tube from accidentally moving downward. The assembly consists of the collet fingers, a return spring, guide cap, collet housing (part of the cylinder, tube, and flange) and the collet piston.

Locking is accomplished by fingers mounted on the collet piston at the top of the drive cylinder. In the locked or latched position, the fingers engage a locking groove in the index tube. The collet piston is normally held in the latched position by a force of ~ 150 lb supplied by a spring. Metal piston rings are used to seal the collet piston from RPV pressure. The collet assembly does not unlatch until the collet fingers are unloaded by a short, automatically sequenced drive-in signal. A pressure ~ 180 psi above RPV pressure must then be applied to the collet piston to overcome spring force, slide the collet up against the conical surface in the guide cap, and spread the fingers so they do not engage a locking groove.

A guide cap is fixed in the upper end of the CRD assembly. This member provides the unlocking cam surface for the collet fingers and serves as the upper bushing for the index tube.

If RPV water is used during a scram to supplement accumulator pressure, it is drawn through a filter on the guide cap.

The piston tube is an inner cylinder, or column, extending upward inside the CRD piston and index tube. The piston tube is fixed to the bottom flange of the CRD and remains stationary. Water is brought to the upper side of the CRD piston through this tube. A series of orifices at

the top of the tube provides progressive water shutoff to cushion the CRD piston at the end of its scram stroke.

A stationary piston, called the stop piston, is mounted on the upper end of the piston tube. This piston provides the seal between RPV pressure and the space above the drive piston. It also functions as a positive end stop at the upper limit of control rod travel. A stack of spring washers just below the stop piston helps absorb the final mechanical shock at the end of control rod travel. The piston rings are similar to the outer drive piston outer rings. A bleedoff passage to the center of the piston tube is located between the two pairs of rings. This arrangement allows seal leakage from the RPV (during a scram) to be bled directly to the discharge line. The lower pair of seals is used only during the cushioning of the CRD piston at the upper end of the stroke.

The center tube of the CRD mechanism forms a well to contain the position-indicator probe. This probe is an aluminum extrusion attached to a cast aluminum housing. Mounted on the extrusion are hermetically sealed, magnetically operated position-indicator switches. Each switch is sheathed in a braided glass sleeve, and the entire probe assembly is protected by a thin-walled stainless-steel tube. The switches are actuated by a ring magnet located at the bottom of the drive piston.

The drive piston, piston tube, and indicator tube are all of nonmagnetic stainless steel, allowing the individual switches to be operated by the magnet as the piston passes. One switch is located at each position corresponding to an index tube groove, thus allowing indication at each latching point. An additional switch is located at each midpoint between latching points to indicate the intermediate positions during drive motion. Thus, indication is provided for each 3 in. of travel. Duplicate switches are provided for the full-in and full-out positions. One additional switch (an overtravel switch) is located at a position below the normal full-out position. Because the limit of down travel is normally provided by the control rod itself as it reaches the backseat position, the CRD can pass this position and actuate the overtravel switch only if it is uncoupled from its control rod. A convenient means is, thus, provided to verify that the drive and control rod are coupled after installation of a drive or at any time during plant operation.

A flange-and-cylinder assembly is made up of a heavy flange welded to the CRD cylinder. A sealing surface on the upper face of this flange forms the seal to the drive housing flange. The seals contain RPV pressure and hydraulic control pressure. Teflon-coated, stainless-steel rings are used for these seals. The CRD flange contains the integral ball, or two-way, check (ball-shuttle) valve. This valve directs either the RPV pressure or the driving pressure, whichever is higher, to the underside of the CRD piston. The RPV pressure is admitted to this valve from the annular space between the drive and drive housing through passages in the flange.

Water used to operate the collet piston passes between the outer tube and the cylinder tube. The inside of the cylinder tube is honed to provide the surface required for the drive piston seals. Both the cylinder tube and outer tube are welded to the CRD flange. The upper ends of these tubes have a sliding fit to allow for differential expansion.

The upper end of the index tube is threaded to receive a coupling spud. The coupling (figure 4.2-10) accommodates a small amount of angular misalignment between the CRD and

the control rod. Six spring fingers allow the coupling spud to enter the mating socket on the control rod. A plug then enters the spud and prevents uncoupling.

Two means of uncoupling are provided. With the RPV head removed, the lock plug can be raised against the spring force of ~ 50 lb by a rod extending up through the center of the control rod to an unlocking handle located above the control rod velocity limiter. The control rod, with the lock plug raised, can then be lifted from the CRD.

The lock plug can also be pushed up from below if it is desired to uncouple a drive without removing the RPV head for access. In this case, the central portion of the drive mechanism is pushed up against the uncoupling rod assembly, which raises the lock plug and allows the coupling spud to disengage the socket as the drive piston and index tube are driven down.

The control rod is heavy enough to force the spud fingers to enter the socket and push the lock plug up, allowing the spud to enter the socket completely and the plug to snap back into place. Therefore, the CRD can be coupled to the control rod by using only the weight of the control rod. However, with the lock plug in place, a force in excess of 50,000 lb is required to pull the coupling apart.

Materials of Construction

Factors that determine the choice of construction materials are discussed in the following paragraphs.

The index tube must withstand the locking and unlocking action of the collet fingers. A compatible bearing combination must be provided that is able to withstand moderate misalignment forces. The reactor environment limits the choice of materials suitable for corrosion resistance. The column and tensile loads can be satisfied by an annealed 300-series stainless steel. The wear and bearing requirements are provided by Malcomizing the completed tube. To obtain suitable corrosion resistance, a carefully controlled process of surface preparation is employed.

The coupling spud is made of Inconel 750 that is aged for maximum physical strength and the required corrosion resistance. Because misalignment tends to cause chafing in the semispherical contact area, the part is protected by a thin chromium plating (electrolyzed). This plating also prevents galling of the threads attaching the coupling spud to the index tube.

Inconel 750 is used for the collet fingers, which must function as leaf springs when cammed open to the unlocked position. Colmonoy 6 hard facing provides a long-wearing surface, adequate for design life, for the area contacting the index tube and unlocking cam surface of the guide cap.

Graphitar 14 is used for seals and bushings on the CRD piston and stop piston. The material is inert and has a low friction coefficient when water lubricated. Because some loss of strength is experienced at higher temperatures, the drive is supplied with cooling water to hold temperatures below 250°F. The Graphitar is relatively soft, which is advantageous when an occasional particle of foreign matter reaches a seal. The resulting scratches in the seal reduce

sealing efficiency until worn smooth, but the CRD design can tolerate considerable water leakage past the seals into the RPV.

All CRD components exposed to RPV water are made of American Iron and Steel Institute (AISI) 300-series stainless steel except the following:

1. Seals and bushings on the CRD piston and stop piston are Graphitar 14.

2. All springs and members requiring spring action (collet fingers, coupling spud, and spring washers) are made of Inconel 750.

3. The ball check valve is a Haynes Stellite cobalt-base alloy.

4. Elastomeric O-ring seals are ethylene propylene.

5. Collet piston rings are Haynes 25 alloy.

6. Certain wear surfaces are hard faced with Colmonoy 6.

7. Nitriding by a proprietary new Malcomizing process and chromium plating is used in certain areas where resistance to abrasion is necessary.

8. The CRD piston head is made of Armco 17-4Ph.

Pressure-containing portions of the CRDs are designed and fabricated in accordance with requirements of Section III of the ASME Boiler and Pressure Vessel Code.

4.2.3.2.2.3 **CRDHS.** The CRDHS (HNP-1 drawing nos. H-16064, H-16065, and S-15059 and HNP-2 drawing nos. H-26006, H-26007, S-25311, and S-25312) supplies and controls the pressure and flow to and from the drives through a hydraulic control unit (HCU). The water discharged from the CRDs during a scram flows through the HCUs to the scram discharge volume (SDV). The water discharged from a CRD during a normal control rod positioning operation flows through the HCU, exhaust header, return line, and back into the CRD system. There are as many HCUs as there are CRDs.

The hydraulic requirements, identified by the function they perform, are as follows:

A. An accumulator hydraulic charging pressure of ~ 1400 to 1500 psig is required. Flow to the accumulators is required only during scram reset or system startup.

B. Drive pressure of ~ 250 psi above RPV pressure is required. Flowrates of ~ 4 gal/min to insert a control rod and 2 gal/min to withdraw a control rod are required.

C. Cooling water to the CRDs is required at ~ 20 psi above RPV pressure and at a flowrate of 0.20 to 0.34 gal/min/drive unit. (Cooling water can be interrupted for short periods without damaging the drive.)

D. The SDV is sized to receive and contain all water discharged by the CRDs during a scram. A minimum volume of 3.34 gal/CRD is required.

The CRDHS provides the required functions with the pumps, filter, valves, instrumentation, and piping shown in HNP-1 drawing nos. H-16064, H-16065, and S-15059, and HNP-2 drawing nos. H-26006, S-25311, and S-25312, and described in the following paragraphs.

Duplicate components are included, where necessary, to ensure continuous system operation if an inservice component requires maintenance.

One supply pump pressurizes the system with water from either the CST or the condensate and feedwater system. One spare pump is provided for standby. A discharge check valve prevents backflow through the nonoperating pump. A portion of the pump discharge flow is diverted through a minimum-flow line to the CST. This flow is controlled by an orifice and is sufficient to prevent immediate pump damage if the pump discharge is inadvertently closed.

The drive water filter downstream of the pump is a cleanable-element type with a 50-μ absolute rating. A differential pressure indicator and a main control room (MCR) alarm monitor the filter element as it collects foreign material.

Accumulator charging pressure is established by the discharge pressure of the system supply pump. During scram, the scram inlet (and outlet) valves open and permit the stored energy in the accumulators to discharge into the CRDs. The resulting pressure decrease in the water header allows the CRD supply pump to run out (flowrate to increase substantially) resulting in high flow, ~ 200 gal/min, into the CRDs via the charging water header. The flow-sensing system upstream of the accumulator charging header detects high flow and closes the flow-control valve. This action maintains increased flow through the charging water header.

Pressure downstream of the drive water filters is monitored in the MCR with a pressure indicator and charging water high-pressure alarm.

During normal operation, the flow-control valve maintains a constant system flowrate. This flow is used for drive flow, drive cooling, and system stability.

The CRD water pressure required in the drive header is maintained by the drive pressure-control valve, which is manually adjusted from the main control room. A flowrate of ~ 6 gal/min (the sum of the flowrate required to insert and withdraw a control rod) normally passes from the CRD water pressure stage through two solenoid-operated stabilizing valves (arranged in parallel) and then goes into the return line downstream from the cooling pressure-control valve. The flow through one stabilizing valve equals the drive insert flow; that of the other stabilizing valve equals the CRD withdrawal flow. When operating a CRD, the required flow is diverted to that CRD by closing the appropriate stabilizing valve. Thus, flow through the CRD pressure-control valve is always constant.

Flow indicators in the CRD water header and in the line downstream from the stabilizing valves allow the flowrate through the stabilizing valves to be adjusted when necessary. Differential pressure between the RPV and the CRD pressure stage is indicated in the MCR.

The cooling-water header is located upstream from the cooling-pressure control valve that can be manually adjusted from the MCR to produce the required cooling-water pressure or provide water to the RPV as needed. Water not required for CRD cooling can be passed through the cooling-pressure control valve to the RPV via the reactor water cleanup (RWC) system. However, due to intergranular stress corrosion cracking (IGSCC) concerns, the cooling-pressure control valve to the RPV valve is normally closed. Thus, the CRD pump discharge valves may need to be throttled to help the cooling-pressure control valve maintain required cooling-water pressure.

To eliminate excess water and high pressure from the CRDHS, a backpressure regulated control valve tied to each CRD pump's minimum-flow line (upstream of the minimum-flow orifices via a crosstie and common bypass) bypasses a variable flow up to 30 gal/min around the minimum-flow orifices to the CST. Stop check valves provide isolation between the CRD pumps in the crosstie line. The backpressure regulated control valve's motive force is interruptible instrument air with a fail-closed operator.

The flow through the flow-control valve is virtually constant. Therefore, once adjusted, the CRD pressure-control valve and the cooling pressure-control valve can maintain their required pressures independent of RPV pressure. Changes in setting of the pressure-control valves are required only to adjust for changes in the cooling requirements of the CRDs, as their seal characteristics change with time. A flow indicator in the MCR monitors cooling-water flow. A differential pressure indicator in the MCR indicates the difference between RPV pressure and CRD cooling-water pressure. Although the CRDs can function without cooling water, seal life is shortened by long-term exposure to RPV temperatures. The temperature of each CRD is recorded in a local panel, and excessive temperatures are annunciated in the MCR.

The SDV consists of header piping that connects to each HCU and drains into an instrument volume. The header piping is sized to receive and contain all the water discharged by the CRDs during a scram, independent of the instrument volume.

During normal plant operation, the SDV is empty and vented to atmosphere through its open vent and drain valves. When a scram occurs, upon a signal from the safety circuit these vent and drain valves are closed to conserve RPV water. Lights in the MCR indicate the positions of these valves.

Redundant vent and drain valves and pilot valves are provided to ensure single-failure-proof isolation of the scram discharge header. The pilot valves are redundant-coil solenoid-operated quick exhaust valves which enable vent and drain valve closure within limits set forth in the Technical Specifications. Unit 1 has needle valves installed in the air supply lines to allow sequencing of the inboard and outboard sets of vent and drain valves to preclude possible hydrodynamic forces which might otherwise be present during the opening and closing of these valves. Unit 2 air supply pilot valves to the inboard and outboard sets of vent and drain valves are sequenced to preclude possible hydrodynamic forces via time delay relays installed in the RPS logic and located in MCR panels.

During a scram, the SDV partly fills with water discharged from above the drive pistons. While scrammed, the CRD seal leakage from the reactor continues to flow into the SDV until the

discharge volume pressure equals the RPV pressure. A check valve in each HCU prevents reverse flow from the scram discharge header volume to the CRD.

When the initial scram signal is cleared from the reactor protection system (RPS), the SDV signal is overridden with a keylock override switch, and the SDV is drained and returned to atmospheric pressure.

Remote manual switches in the pilot valve solenoid circuits allow the discharge volume vent and drain valves to be tested without disturbing the RPS. Closing the SDV valves allows the outlet scram valve seats to be leak tested by timing the accumulation of leakage inside the SDV.

Ten liquid-level switches (six float level switches and four thermal probes) connected to the instrument volume monitor the volume for abnormal water level. It is set at three levels. At the lowest level, a level switch actuates to indicate that the volume is not completely empty during post scram draining or to indicate that the volume starts to fill through leakage accumulation at other times during reactor operation. At the second level, one level switch produces a rod-withdrawal block to prevent further withdrawal of any control rod when leakage accumulates to half the capacity of the instrument volume. The remaining eight switches (four float level switches and four thermal probes) are interconnected with the trip channels of the RPS and initiate a reactor scram should water accumulation fill the instrument volume.

<u>Hydraulic Control Units</u>

Each HCU furnishes pressurized water, on signal, to a CRD unit. The CRD then positions its control rod as required. Operation of the electrical system that supplies scram and normal control rod positioning signals to the HCU is described in HNP-1-FSAR and HNP-2-FSAR sections 7.2 and 7.7.

The basic components in each HCU (figure 4.2-14) are manual, pneumatic, and electrical valves; an accumulator; related piping; electrical connections; filters; and instrumentation (HNP-1-FSAR drawing nos. H-16064, H-16065, and S-15059, and HNP-2 drawing nos. H-26006 and H-26007). The components and their functions are described in the following paragraphs.

A. The insert CRD valve is solenoid operated, opens on an insert signal, and supplies drive water to the bottom side of the main CRD position.

B. The insert exhaust valve opens by solenoid on an insert signal and discharges water from above the CRD piston to the exhaust water header.

C. The withdraw CRD valve is solenoid, operated, opens on a withdraw signal, and supplies drive water to the top of the drive piston.

D. The solenoid-operated withdraw exhaust valve opens on a withdraw signal, discharges water from below the main CRD piston to the exhaust header, and serves as the settle valve. During the settle mode, the valve opens following drive insert and remains open following withdrawal to allow the CRD to settle back into the nearest latch position.

E. The speed-control valves regulate the control rod insertion and withdrawal rates during normal operation. They are manually adjustable flow-control valves used to regulate the water flow to and from the volume beneath the main drive piston. A correctly adjusted valve does not require readjustment, except to compensate for changes in CRD seal leakage.

F. The scram pilot valves are operated from the RPS trip system. Two scram pilot valves control both the scram inlet valve and the scram exhaust valve. The scram pilot valves are identical three-way, solenoid-operated, normally energized valves. On loss of electrical signal to the pilot valves, such as the loss of external ac power, the inlet ports close and the exhaust ports open on both pilot valves. The pilot valves (HNP-1 drawing nos. H-16064, H-16065, and S-15059, and HNP-2 drawing nos. H-26006 and H-26007) are arranged so that the trip system signal must be removed from both valves before air pressure can be discharged from the scram valve operators. This prevents the inadvertent scram of a single CRD in the event of failure of one of the solenoid pilot valves.

G. The scram inlet valve opens to supply pressurized water to the bottom of the CRD piston. This quick-opening globe valve is operated by an internal spring and system pressure, and is closed by air pressure applied to the top of its diaphragm operator.

H. The scram exhaust valve opens slightly before the scram inlet valve, exhausting water from above the CRD piston. The exhaust valve opens faster than the inlet valve because of a larger spring in the valve operator; otherwise, the valves are similar. A position-indication switch on the scram exhaust valve in series with a position-indication switch on the scram inlet valve energizes a light in the MCR as soon as both valves start to open.

I. The scram accumulator stores sufficient energy to fully insert a control rod at lower RPV pressures. At higher RPV pressures, the accumulator pressure is assisted or supplanted by RPV pressure. The accumulator is a hydraulic cylinder with a free-floating piston that separates the water on top from the nitrogen below. A check valve in the accumulator charging line prevents loss of water pressure in the event supply pressure is lost.

   During normal plant operation, the accumulator piston is seated at the bottom of its cylinder.

   To ensure the accumulator is always able to produce a scram, it is continuously monitored for water leakage. A float-type level switch actuates an alarm if water leaks past the piston barrier and collects in the accumulator instrumentation block.

4.2.3.2.2.4 **Alternate Rod Insertion System**. The ARI installation fulfills requirement C.3 of 10 CFR 50.62 pertaining to the reduction of risk from anticipated transients without scram (ATWS) events.[10]

ATWS events are the anticipated occurrences, as defined in Appendix A of 10 CFR 50, followed by a failure of the reactor trip portion of the reactor protection system (RPS). The ARI provides the necessary signals in response to an ATWS event or manual initiation to depressurize the CRD scram pilot valve air header through valves that are different from the RPS scram valves (HNP-1 drawing nos. H-16064 and H-16065, and HNP-2 drawing no. H-26007), providing a parallel path for initiating control rod insertion.

The ARI valves are operated from logic independent from the RPS trip system. They are solenoid-operated and are normally deenergized. There are four valves. One is a three-way valve. On an actuation signal, the inlet will close to block the air supply, and the exhaust will open to vent the scram valve pilot air header. Three of the valves are two-way. On an actuation signal, they open to speed depressurization of the header.

The signals to initiate the ARI function will come from high RPV dome pressure, low RPV water level 2, or manual initiation by pushbuttons on the 1H11-P603 and 2H11-P603 panels in the MCR. The high RPV dome pressure setpoint is set higher, and the low RPV water level setpoint is set lower than the normal scram setpoints such that a normal scram should have already been initiated at the time an ARI setpoint is reached. The signals which initiate the ARI will also initiate an ATWS recirculation pump trip (RPT), which is described in HNP-1-FSAR subsection 7.2.3 and HNP-2-FSAR paragraph 7.6.10.7.

The actuation signals to the ARI system will seal in for 30 to 35 s to assure all control rods have time to fully insert.

4.2.3.2.2.5 CRD System Operation. The CRD system performs rod insertion, rod withdrawal, and scram. These operational functions are described in the following paragraphs.

Rod insertion is initiated by a signal from the operator to the insert valve solenoids. This signal causes both insert valves to open. The insert CRD valve applies RPV pressure, plus ~ 90 psi to the bottom of the CRD piston. The insert exhaust valve allows water from above the CRD piston to discharge to the exhaust header.

As is illustrated in figure 4.2-10, the locking mechanism is a ratchet-type device and does not interfere with rod insertion. The speed at which the CRD moves is determined by the flow through the insert speed-control valve, which is set for ~ 4 gal/min for a shim speed (nonscram operation) of 3 in./s. During normal insertion, the pressure on the downstream side of the speed-control valve is 90 to 100 psi above RPV pressure. However, if the CRD slows for any reason, the flow through and pressure drop across the insert speed-control valve decreases and full drive header pressure, up to RPV pressure plus 260 psi, is then available to cause continued insertion. With 260-psi differential pressure acting on the CRD piston, the piston exerts an upward force of 1040 lb.

By design, rod withdrawal is more involved than insertion. The collet finger (latch) must be raised to reach the unlocked position (figure 4.2-11). The index tube notches and the collet fingers are shaped so that the downward force on the index tube holds the collet fingers in place. The index tube must be lifted before the collet fingers can be released. This is done by opening the CRD insert valves (in the manner described in the preceding paragraph) for

approximately one second. The withdraw valves are then opened, applying driving pressure above the drive piston and opening the area below the piston to the exhaust header. As the piston rises, the collet fingers are cammed outward, away from the index tube, by the guide cap.

The pressure required to release the latch is set and maintained at a level high enough to overcome the force of the latch return spring plus the force of RPV pressure opposing movement of the collet piston. When this occurs, the index tube is unlatched and free to move in the withdraw direction. Water displaced by the CRD piston flows out through the withdraw speed-control valve, which is set to give the control rod a shim speed of 3 in./s. The entire valving sequence is automatically controlled and is initiated by a single operation of the rod withdraw switch.

During a scram, the scram pilot valves and scram valves are operated as previously described. With the scram valves open, accumulator pressure is admitted under the CRD piston, and the area over the drive piston is vented to the SDV.

The large differential pressure (always several hundred psi, depending on RPV pressure) produces a large upward force on the drive piston. This force gives the rod high initial acceleration and provides a large margin of force to overcome friction. After the initial acceleration is achieved, the CRD continues at a nearly constant velocity. This characteristic provides a high initial rod-insertion rate. As the CRD piston nears the top of its stroke, the piston seals close off the large passage (buffer orifices) in the stop piston tube, and the CRD slows.

Prior to a scram signal, the accumulator in the HCU has ~ 1400 psig on the water side and 1100 psig on the nitrogen side. As the inlet scram valve opens, the full water-side pressure is available at the CRD, acting on a 4.1-in.^{2} area. As CRD motion begins, this pressure drops to the gas-side pressure, less the pressure loss between the accumulator and the CRD.

The CRD accumulators are required to scram the control rod when the RPV pressure is low. When the RPV pressure is low, the accumulator retains sufficient stored energy to ensure the complete insertion of the control rod in the required time. The accumulator is not required to scram the control rod in time when the reactor is close to or at full operating pressure. In this instance, the RPV pressure alone scrams the control rod in the required time. However, the accumulator does provide an additional energy boost to the RPV pressure in providing scram action at RPV pressures less than accumulator pressures.

The CRD system, with accumulators, provides the following scram performances at any RPV pressure in terms of average elapsed time after the opening of the RPS trip actuator (scram signal) for the CRDs to attain the scram strokes listed:

Percentage of Full Control Rod Stroke	Stroke (in.)	Average Time(s)
5.0	7.2	0.49
20.0	28.8	0.90
50.0	72.0	2.0
90.0	129.6	3.5

The Technical Specifications specify the insertion time required to ensure the scram reactivity assumed in the DBA and AOO analyses is met. Accident and AOO analyses use a specific scram reactivity insertion rate that assumes all control rods scram at the same speed. In general, as long as the insertion time of all control rods equals the time used in the analyses, the scram reactivity insertion rate will be maintained.

However, this assumes that the insertion times are normally distributed, with adequate separation of slower CRDs. Therefore, if the average insertion time is maintained, some control must be placed on the local scram reactivity. For simplicity, the Technical Specifications incorporate a format that measures individual CRD insertion time to determine whether the analytical bases (scram reactivity) is met and also checks for signs of degraded scram performance. This ensures that core-wide and local reactivity requirements are met.

The times used for the Technical Specifications are based on actual performance test data and are accurate representations of the expected CRD insertion time. These insertion times are actually faster than the values used in the accident and AOO analyses to allow for a certain number of slow control rods and account for single failures. The Technical Specifications impose limitations on the number and location of slow control rods. The Technical Specifications 5% insertion time is increased above the design average scram time. Based on the evaluation of actual insertion time data, the increased 5% insertion time has been shown to have a negligible impact on plant accident and AOO performance.

4.2.3.2.3 Evaluation of Scram Time

The rod scram function of the CRD system provides the negative reactivity insertion required by the safety design bases (see first entry under the third bullet) in paragraph 4.2.3.2.1.1. The scram time shown in the description is adequate as shown by the safety analysis discussed in HNP-2-FSAR chapter 15.

Sufficient driving force is available to overcome the retarding force during a scram. The control rod weighs 158 lb in water and 186 lb in air. The index tube weighs 62 lb in water and 71 lb in air. Other moving parts weigh ~ 5 lb; thus, the wet drive line weight is ~ 225 lb.

At the start of motion, assuming the accumulator is normally charged, the CRD pump supplies ~ 1500 psi at the inlet scram valve. This supplies a 500-psi differential to assist opening of the valve and exists until drive motion starts. Pressure at the CRD immediately drops to RPV pressure due to losses in the piping and valves, and RPV pressure is applied through the ball check valve in the CRD. This pressure, actually slightly less than RPV pressure due to flow

losses as the water comes down the annulus between the CRD and thermal sleeve, is applied to the 4.1-in.^{2} under-piston area. The area above the piston, 1.25 in.^{2}, is vented to the SDV and initially drops to atmospheric pressure. As soon as drive motion starts, line losses in the discharge line raise the pressure over the piston to ~ 180 psi. The balance of the over-piston area (4.1 minus 1.25 in.^{2}) is exposed to RPV pressure. Available force, assuming a stuck rod, reduces simply to 1.25 x 1000 or 1250 lb throughout the stroke after accumulator energy is expended. Since the available force is constant at 1250 lb from the beginning of motion to the end of the strokes, no plot of the force developed by the CRD mechanism versus stroke for a scram with the accumulator and RPV at nominal pressure is necessary.

4.2.3.2.3.1 Analysis of Malfunction Relating to Rod Withdrawal. There are no known single malfunctions that cause the unplanned withdrawal of even a single control rod. However, if multiple malfunctions are postulated, studies show that an unplanned rod withdrawal can occur at withdrawal speeds that vary with the combination of malfunctions postulated. In all cases the subsequent withdrawal speeds are less than those assumed in the CRDA analysis as discussed in HNP-2-FSAR subsection 15.3.2. Therefore, the physical and radiological consequences of such rod withdrawals are less than those analyzed in the CRDA.

A. Drive Housing Fails at Attachment Weld

The bottom head of the RPV has a penetration for each CRD location. A CRD housing is raised into position inside each penetration and fastened by welding. The CRD is raised into the CRD housing and bolted to a flange at the bottom of the housing. The housing material is seamless, Type 304 stainless-steel pipe with a minimum tensile strength of 75,000 psi. The basic failure considered here is a complete circumferential crack through the housing wall at an elevation just below the J-weld.

Static loads on the housing wall include the weight of the CRD and the control rod, the weight of the housing below the J-weld, and the RPV pressure acting on the 6-in.-diameter cross-sectional area of the housing and the CRD. Dynamic loading results from the reaction force during CRD operation.

If the housing were to fail as described, the following sequence of events is foreseen:

1. The housing separates from the RPV.

2. The control rod, the CRD, and the housing are blown downward against the support structure by RPV pressure acting on the cross-sectional area of the housing and the CRD.

3. The downward motion of the CRD and associated parts is determined by the gap between the bottom of the CRD and the support structure and by the deflection of the support structure under load. In the current design, maximum deflection is ~ 3 in. If the collet remains latched, no further control

rod ejection will occur,[19] and the housing will not drop far enough to clear the RPV penetration.

4. RPV water leaks at a rate of ~ 220 gal/min through the 0.03-in. diametrical clearance between the housing and the RPV penetration.

If the basic housing failure occurs while the control rod is being withdrawn (this is a small fraction of the total CRD operating time), and if the collet stays unlatched, the following sequence of events is foreseen:

1. The housing separates from the RPV.

2. The drive and housing are blown downward against the CRD housing support. Calculations indicate the steady-state rod-withdrawal velocity is 0.3 ft/s. During withdrawal, pressure under the collet piston is ~ 250 psi greater than the pressure over it. Therefore, the collet is held in the unlatched position until driving pressure is removed from the pressure-over port.

B. Rupture of Hydraulic Line(s) to CRD Housing Flange

The three types of possible rupture of hydraulic lines to the CRD housing flange are:

- Pressure-under line break.
- Pressure-over line break.
- Coincident breakage of both of these lines.

For the case of a pressure-under line break, a partial or complete circumferential opening is postulated at or near the point where the line enters the housing flange. Failure is more likely to occur after another basic failure wherein the CRD housing or housing flange separates from the RPV. Failure of the housing, however, does not necessarily lead directly to failure of the hydraulic lines.

If the pressure-under line fails and the collet is latched, no control rod withdrawal will occur. There is no pressure differential across the collet piston and, therefore, no tendency to unlatch the collet. Consequently, the associated control rod cannot be inserted or withdrawn.

The ball check valve is designed to seal off a broken pressure-under line by using RPV pressure to shift the check ball to its upper seat. If the ball check valve is prevented from seating, RPV water will leak to the atmosphere. Because of the broken line, cooling water cannot be supplied to the CRD involved. Loss of cooling water will cause no immediate damage to the CRD. However, prolonged exposure of the CRD to temperatures at or near RPV temperature can lead to deterioration of material in the seals. High temperature is indicated to the operator by the

thermocouple in the position indicator probe. A second indication is high cooling water flow.

If the basic line failure occurs while the control rod is being withdrawn, the hydraulic force will not be sufficient to hold the collet open, and the spring force normally causes the collet to latch and stop rod withdrawal. However, if the collet remains open, calculations indicate the steady-state control rod withdrawal velocity will be 2 ft/s.

The case of the pressure-over line breakage considers the complete breakage of the line at or near the point where it enters the housing flange. If the line breaks, pressure over the CRD piston will drop from RPV pressure to atmospheric pressure. Any significant RPV pressure (~ 500 psig or greater) acts on the bottom of the CRD piston and fully insert the CRD. Insertion occurs regardless of the operational mode at the time of the failure. After full insertion, RPV water leaks past the stop piston seals, the contracting seals on the drive piston, and the collet piston seals. This leakage exhausts to the atmosphere through the broken pressure-over line. The leakage rate at 1000-psi RPV pressure is estimated to be 4 gal/min nominal but not more than 10 gal/min, based upon experimental measurements. If the RPV is hot, drive temperature will increase. This situation is indicated to the reactor operator by the drift alarm, by the fully inserted drive, by a high drive temperature (indicated and printed out on a recorder in the control room), and by operation of the drywell sump pump.

For the simultaneous breakage of the pressure-over and pressure-under lines, pressures above and below the CRD piston will drop to zero, and the ball check valve will close the broken pressure-under line. RPV water flows from the annulus outside the CRD, through the RPV ports, and to the space below the drive piston. As in the case of pressure-over line breakage, the CRD inserts at a speed dependent on RPV pressure. Full insertion occurs regardless of the operational mode at the time of failure. RPV water leaks past the CRD seals and out the broken pressure-over line to the atmosphere, as described previously. CRD temperature increases. Indication in the MCR includes the drift alarm, the fully inserted CRD, an HCU high temperature alarm which comes from the CRD temperature recorder located in a local panel, and operation of the drywell sump pump.

C. All CRD Flange Bolts Fail in Tension

Each CRD is bolted to a flange at the bottom of a CRD housing. The flange is welded to the CRD housing. Bolts are made of AISI-4140 steel, with a maximum tensile strength of 125,000 psi. Each bolt has an allowable load capacity of 15,200 lb. Capacity of the 8 bolts is 121,600 lb. As a result of the RPV design pressure of 1250 psig, the major load on all 8 bolts is 30,400 lb.

If a progressive or simultaneous failure of all bolts occurs, the CRD will separate from the housing. The control rod and the CRD are blown downward against the support structure. Impact velocity and support structure loading are slightly less

than that for CRD housing failure, because RPV pressure acts on the CRD cross-sectional area only and the housing remains attached to the RPV. The CRD is isolated from the cooling-water supply. RPV water flows downward past the velocity limiter piston, through the large CRD filter, and into the annular space between the thermal sleeve and the CRD. For worst-case leakage calculations, the large filter is assumed to be deformed or swept out of the way so it will offer no significant flow restriction. At a point near the top of the annulus, where pressure drops to 350 psi, the water flashes to steam and cause choke-flow conditions. Steam flows down the annulus and out the space between the housing and the CRD flanges to the atmosphere. Steam formation limits the leakage rate to ~ 840 gal/min.

If the collet is latched, control rod ejection is limited to the distance the CRD can drop before coming to rest on the support structure. There will be no tendency for the collet to unlatch, because pressure below the collet piston drops to zero. Pressure forces, in fact, exert 1435 lb to hold the collet in the latched position.

If the bolts fail during control rod withdrawal, pressure below the collet piston will drop to zero. The collet, with 1650-lb return force, latches and stops rod withdrawal.

D. Weld Joining Flange to Housing Fails in Tension

The failure considered is a crack in or near the weld that joins the flange to the housing. This crack will extend through the wall and completely around the housing. The flange material is forged, Type 304 stainless steel, with a minimum tensile strength of 75,000 psi. The housing material is seamless, Type 304 stainless-steel pipe, with a minimum tensile strength of 75,000 psi. The conventional full-penetration weld of Type 308 stainless steel has a minimum tensile strength approximately the same as that for the parent metal. The design pressure and temperature are 1250 psig and 575°F, respectively. RPV pressure acting on the cross-sectional area of the CRD; the weight of the control rod, CRD, and flange; and the dynamic reaction force during CRD operation result in a maximum tensile stress at the weld of ~ 6000 psi.

If the basic flange-to-housing joint failure occurs, the flange and the attached CRD are blown downward against the support structure. The support structure loading is slightly less than that for CRD housing failure because RPV pressure acts only on the CRD cross-sectional area. Lack of differential pressure across the collet piston causes the collet to remain latched and limit control rod motion to ~ 3 in. Downward CRD movement is small; therefore, most of the CRD remains inside the housing. The pressure-under and pressure-over lines are flexible enough to withstand the small displacement and remain attached to the flange. RPV water follows the same leakage path described above for the flange-bolt failure, except that exit to the atmosphere will be through the gap between the lower end of the housing and the top of the flange. Water flashes to steam in the annulus surrounding the CRD. The leakage rate is ~ 840 gal/min.

If the basic failure occurs during control rod withdrawal (a small fraction of the total operating time) and the collet is held unlatched, the flange will separate from the housing. The CRD and flange are blown downward against the support structure. The calculated steady-state rod-withdrawal velocity is 0.13 ft/s. Because pressure-under and pressure-over lines remain intact, driving water pressure to the CRD will continue, and the normal exhaust line restriction will exist. The pressure below the velocity limiter piston drops below normal as a result of leakage from the gap between the housing and the flange. This differential pressure across the velocity-limiter piston will result in a net downward force of ~ 70 lb. Leakage out of the housing greatly reduces the pressure in the annulus surrounding the CRD. Thus, the net downward force on the CRD piston is less than normal. The overall effect of these events reduces rod withdrawal to approximately one-half of normal speed. With a 560-psi differential across the collet piston, the collet remains unlatched; however, it should relatch as soon as the CRD signal is removed.

E. Housing Wall Ruptures

This failure is a vertical split in the CRD housing wall just below the bottom head of the RPV. The flow area of the hole is considered equivalent to the annular area between the CRD and the thermal sleeve. Thus, flow through this annular area, rather than flow through the hole in the housing, governs leakage flow. The housing is made of Type 304 stainless-steel seamless pipe with a minimum tensile strength of 75,000 psi. The maximum hoop stress of 11,900 psi results primarily from the RPV design pressure (1250 psig) acting on the inside of the housing.

If such a rupture occurs, RPV water flashes to steam and leak through the hole in the housing to the atmosphere at ~ 1030 gal/min. Choke-flow conditions exist as described above for the flange-bolt failure. However, leakage flow is greater because flow resistance is less; that is, the leaking water and steam will not have to flow down the length of the housing to reach the atmosphere. A critical pressure of 350 psi causes the water to flash to steam.

No pressure differential across the collet piston tends to unlatch the collet, but the CRD inserts as a result of loss of pressure in the CRD housing, causing a pressure drop in the space above the CRD piston.

If the failure occurs during control rod withdrawal, CRD withdrawal is stopped by a reduction of the net downward force acting on the CRD line; however, the collet remains unlatched. The net force reduction occurs when the leakage flow of 1030 gal/min reduces the pressure in the annulus outside the CRD to ~ 540 psig, thereby reducing the pressure acting on top of the CRD piston to the same value. A pressure differential of ~ 710 psi will exist across the collet piston and hold the collet unlatched as long as the operator held the withdraw signal.

F. Flange Plug Blows Out

To connect the RPV ports with the bottom of the ball check valve, a 3/4-in.-diameter hole is drilled in the CRD flange. The outer end of this hole is

sealed with a plug of 0.812-in. diameter, 0.250-in. thick. A full-penetration, Type 308 stainless-steel weld holds the plug in place. The postulated failure is a full circumferential crack in this weld and subsequent blowout of the plug.

If the weld fails, the plug blows out, and the collet remains latched. There is no control rod motion. There is no pressure differential across the collet piston acting to unlatch the collet. RPV water will leak past the velocity-limiter piston, down the annulus between the CRD and the thermal sleeve, through the RPV ports and drilled passage, and out the open plug hole to the atmosphere at ~ 320 gal/min. Leakage calculations assume only liquid flows from the flange. Actually, hot RPV water flashes to steam, and choke-flow conditions exist. Thus, the expected leakage rate is lower than the calculated value. Control rod temperature increases and initiates an alarm in the MCR.

If this failure occurs during control rod withdrawal and the collet stays unlatched, calculations indicate that control rod withdrawal speed will be ~ 0.24 ft/s. Leakage from the open plug hole in the flange will cause RPV water to flow downward past the velocity limiter piston. A small differential pressure across the piston will result in an insignificant driving force of ~ 10 lb, tending to increase withdraw velocity.

A pressure differential of 295 psi across the collet piston holds the collet unlatched as long as the driving signal is maintained.

Flow resistance of the exhaust path from the CRD is normal because the ball check valve is seated at the lower end of its travel by pressure under the CRD piston.

G. CRD Pressure-Control Valve Closure (RPV Pressure, 0 psig)

The pressure to move a CRD is generated by the pressure drop of practically the full-system flow through the CRD pressure-control valve. This valve is motor operated and adjusted to a fixed opening. The normal pressure drop across this valve develops a pressure 250 psi in excess of RPV pressure.

If the flow through the CRD pressure-control valve is stopped, as by a valve closure of flow blockage, the CRD pressure increases to the shutoff pressure of the supply pump. The occurrence of this condition, during withdrawal of a CRD at zero RPV pressure, results in a CRD pressure increase from 250 psig to not more than 1700 psig. Calculations indicate that the drive will accelerate from 3 to ~ 6 in./s. A pressure differential of 1670 psi across the collet piston holds the collet unlatched. Flow is upward past the velocity limiter piston, but retarding force would be negligible. Rod movement stops as soon as the driving signal is removed.

H. Ball Check Valve Fails to Close Passage to RPV Ports

Should the ball check valve sealing the passage to the RPV ports become dislodged and prevented from reseating following the insert portion of a CRD withdrawal sequence, water below the CRD piston returns to the RPV through the

RPV ports and the annulus between the CRD and the housing rather than through the speed-control valve. Because the flow resistance of this return path is lower than normal, the calculated withdrawal speed is 2 ft/s. During withdrawal, differential pressure across the collet piston is ~ 40 psi. Therefore, the collet tends to latch and has to stick open before continuous withdrawal at 2 ft/s could occur. Water flows upward past the velocity limiter piston, generating a small retarding force of ~ 120 lb.

I. HCU Valve Failures

Various failures of the valves in the HCU can be postulated, but none can produce differential pressures approaching those described in the preceding paragraphs, and none alone can produce a high-velocity withdrawal. Leakage through either one or both of the scram valves produces a pressure that tends to insert the control rod rather than to withdraw it. If the pressure in the SDV should exceed RPV pressure following a scram, a check valve in the line to the scram discharge header prevents this pressure from operating the CRD mechanism.

J. Collet Fingers Fail to Latch

When the CRD withdraw signal is removed, the drive continues to withdraw at a fraction of normal speed. Without some initiating signal, there is no known means for the collet fingers to become unlocked. If the withdrawal CRD valve fails to close following a rod withdrawal, it would have the same effect as failure of the collet fingers to latch in the index tube. Because the collet fingers remain locked until they are unloaded, accidental opening of the withdrawal CRD valve does not unlock them.

K. Withdrawal Speed Control-Valve Failure Normal withdrawal speed is determined by differential pressures in the CRD and is set for a nominal value of 3 in/s. Withdrawal speed is maintained by the pressure-regulating system and is independent of RPV pressure. Tests have shown that accidental opening of the speed control valve to the full-open position produces a velocity of ~ 6 in./s.

The CRD system prevents rod withdrawal, and it has been shown above that only multiple failures in a CRD unit and in its control unit could cause an unplanned rod withdrawal.

4.2.3.2.3.2 **Scram Reliability.** High scram reliability is the result of a number of features of the CRD system. For example:

A. Two sources of scram energy are used to insert each control rod when the reactor is operating, accumulator pressure and RPV pressure.

B. Each drive mechanism has its own scram and pilot valves so only one drive can be affected if a scram valve fails to open. Two pilot valves are provided for each drive. Both pilot valves must be deenergized to initiate a scram.

C. The RPS and the HCUs are designed so that the scram signal and mode of operation override all others.

D. The collet assembly and index tube are designed so they do not restrain or prevent control rod insertion during scram.

E. The SDV is monitored for accumulated water and scrams the reactor before the volume is reduced to a point that could interfere with a scram.

F. An ARI system reduces the probability of occurrence of an ATWS event and provides the necessary signals in response to an ATWS event or manual initiation to depressurize the CRD scram pilot valve air header through valves that are different from the RPS scram valves, providing a parallel path for initiating control rod insertion.

4.2.3.2.3.3 **<u>Analysis of BWR Scram System Pipe Breaks</u>.** A generic evaluation of the applicability of postulated breaks in the BWR scram system, which includes an estimate of the probability of occurrence of the postulated sequences of events and the bases for the conclusions, was performed. A generic evaluation of the applicability of the AEOD report relative to BWR plant construction, design, and operation, and a generic evaluation of the AEOD recommendations were performed. The results of the evaluation are in Topical Report NEDE-24342, "GE Evaluation in Response to NRC Request Regarding BWR Scram System Pipe Breaks".(23) This conclusion is based upon the following:

A. The postulated accident is very unlikely due to the GE design and installation specifications, as well as the specified QA requirements. Specifications require the piping system be designed for high pressure and temperature, even though the systems are only exposed to this environment 1% of the time. No scram discharge piping system at any reactor has ruptured in over 20 years of reactor operation.

B. GE analysis of the probability of this sequence of events resulted in a postulated pipe break of < 10^{-7} per reactor year. This places the frequency beyond the range of events that is taken into account in the design of nuclear facilities.

C. Even if a break occurs, alarms and resulting normal inspection by the operating staff will clearly indicate leakage of water, detectable radiation level in the reactor building, and a measurable water level in the reactor building sump. Current procedures provide the operator sufficient guidance to depressurize the reactor; thus, leakage of coolant would be under control to preclude any damage.

D. Additional pumps that are not part of the ECCS are also available to provide more than sufficient water supply, even if all the ECCS pumps fail to operate.

In summary, the consequences of the small-break LOCA, as a result of a postulated SDV pipe rupture, are well within the mitigation capabilities of the normal and emergency cooling systems. Also, the potential for a SDV pipe rupture and the potential for

unacceptable consequences resulting from a postulated SDV pipe rupture are < 1 x 10^{-7} events per year.

4.2.3.2.3.4 Control Rod Support and Operation. As described previously, each control rod is independently supported and controlled as required by safety design bases.

4.2.3.2.4 Tests and Inspections

4.2.3.2.4.1 Operational Tests. After installation, all rods and CRD mechanisms can be tested through their full stroke for operability.

During normal operation, each time a control rod is withdrawn a notch, the operator can observe the incore monitor indications to verify that the control rod is following the CRD mechanism. All control rods that are partially withdrawn from the core can be tested for rod following by inserting or withdrawing the rod one notch and returning it to its original position while the operator observes the incore monitor indications.

To make a positive test of control rod-to-CRD coupling integrity, the operator can withdraw a control rod to the end of its travel and then attempt to withdraw the CRD to the overtravel position. Failure of the CRD to overtravel demonstrates rod-to-drive coupling integrity.

Hydraulic supply subsystem pressures can be observed from instrumentation in the MCR. Scram accumulator pressures can be observed on the nitrogen pressure gauges.

4.2.3.2.4.2 Surveillance Tests. The surveillance requirements for the CRD system are:

A. Sufficient control rods are withdrawn, following a refueling outage when core alterations are performed, to demonstrate that the core can be made subcritical at any time in the subsequent fuel cycle with the strongest operable control rod fully withdrawn and all other operable rods fully inserted. This can be demonstrated analytically with a shutdown margin of 0.38% $\Delta k/k$, using data derived from measurements made at the beginning of each cycle during normal in-sequence rod pulls. Alternately, this can be demonstrated by actual test, in which case, the value of the shutdown margin must be ≥ 0.28% $\Delta k/k$ throughout the cycle.

B. Each fully withdrawn control rod is exercised one notch at least once each week when above the preset power level of the rod worth minimizer (RWM). Each partially withdrawn control rod is exercised once every 31 days when above the preset power level of the RWM. In the event that one or more control rods become incapable of insertion, each operable partially and fully withdrawn control rod must be exercised within 24 h.

   These control rod exercises serve as a periodic check against deterioration of the control rod system. The tests also verify the ability of CRDs to scram because, if a

rod can be moved with CRD pressure, it will surely insert when subjected to the higher pressure applied during a scram. This further assures the reliability of the remaining control rods.

C. The coupling integrity is verified for each withdrawn control rod as follows:

Each time a control rod is withdrawn to the full-out position and prior to declaring the control rod operable after work on the control rod or any work on the CRD system that could affect the coupling, a coupling check is performed by verifying that the control rod does not go to the overtravel position.

The overtravel position feature provides a positive check on the coupling integrity, since only an uncoupled CRD can reach the overtravel position.

D. During operation, accumulator pressure and level at the normal operating value are verified.

Experience with CRD systems of the same type indicates that weekly verification of accumulator pressure and level is sufficient to ensure operability of the accumulator portion of the CRD system.

E. During each major refueling outage, each operable control rod is subjected to scram time tests from the fully withdrawn position.

Experience indicates that the scram times of the control rods do not significantly change over the time interval between refueling outages. A test of the scram times at each refueling outage is sufficient to identify any significant lengthening of the scram times.

F. Float chamber water level activation of the six liquid level switches in the SDV is performed on a frequency corresponding to the refueling frequency, as part of a channel functional test and a logic system functional test.

4.2.3.2.4.3 **Instrumentation**. The general functional requirements for the CRD are discussed in paragraph 4.2.3.1.5.

4.2.3.3 Supplementary Reactivity Control

Refer to section 3.2.4.2 of ***NEDE-24011-P-A (GESTAR II)*.**

4.2.3.4 SLCS

4.2.3.4.1 Design Bases

The SLCS meets the following safety design bases:

A. Backup capability for reactivity control is provided, independent of normal reactivity control provisions in the nuclear reactor, to be able to shut down the reactor if normal control ever becomes inoperative.

B. The backup system has the capacity for controlling the reactivity difference between the steady-state-rated operating condition of the reactor with voids and the cold shutdown condition, including shutdown margin, to ensure complete shutdown from the most reactive condition at any time in core life.

C. The time required for actuation and effectiveness of the backup control is consistent with the nuclear reactivity rate of change predicted between rated operating and cold shutdown conditions. A fast scram of the reactor or operational control of fast reactivity transients is not specified to be accomplished by this system.

D. Means are provided by which the functional performance capability of the backup control system components can be verified periodically under conditions approaching actual use requirements. Demineralized water, rather than the actual neutron absorber solution, can be injected into the reactor to test the operation of all components of the redundant control system.

E. The neutron absorber is dispersed within the reactor core in sufficient quantity to provide a reasonable margin for leakage or imperfect mixing.

F. The system is reliable to a degree consistent with its role as a special safety system; the possibility of unintentional or accidental shutdown of the reactor by this system is minimized.

G. As part of the implementation of an alternative source term (AST)(reference subsection 15.1.11), a new design function was added for SLCS to buffer the suppression pool by injection of a sufficient amount of sodium pentaborate solution to the suppression pool to prevent iodine re-evolution following a LOCA. The mass of sodium pentaborate available for injection is 1975 lbm.

4.2.3.4.2 Description

The SLCS (HNP-1 drawing nos. H-16061 and H-19926, and HNP-2 drawing no. H-26009) is manually initiated from the MCR to pump a boron neutron absorber solution into the reactor if the operator believes the reactor cannot be shut down or kept shut down with the control rods.

The SLCS is required to shut down the reactor and keep the reactor from going critical again as it cools.

The SLCS is used in the highly improbable event that not enough control rods can be inserted in the reactor core to accomplish shutdown and cooldown in the normal manner. SLCS is required for postulated ATWS events by the Emergency Operating Procedures (EOPs) when it is determined that the reactor cannot be shut down prior to suppression pool temperature reaching the boron injection initiation temperature (BIIT).

The BIIT is a limit defined by appendix B and calculated by appendix C of the Revision 4 Emergency Procedure Guidelines (references 13 and 14). BIIT is identified in the EOPs.

Inhibiting the ADS during an ATWS event can mitigate the event, because RPV water level can be lowered to enhance the effectiveness of the SLCS in shutting down the reactor. This mode of operation is described in HNP-1-FSAR paragraph 7.4.3.1 and HNP-2-FSAR paragraph 7.3.1.2.2.

The SLCS is credited for the injection of sufficient sodium pentaborate solution to prevent the re-evolution of iodine from the suppression pool for a 30-day period following a DBA LOCA. The mass of sodium pentaborate available for injection is 1975 lbm. The pH buffering effect of SLCS injection is sufficient to offset the effects of acids that are transported to the suppression pool and maintain suppression pool pH at or above 7, thus precluding the re-evolution of elemental iodine.

The boron solution tank, test water tank, the two positive displacement pumps, the two explosive valves, and associated local valves and controls are mounted in the reactor building. The solution is piped into the RPV and discharged near the bottom of the core shroud, so it mixes with the cooling water rising through the core (subsection 4.2.2).

The boron-10 isotope absorbs thermal neutrons and, thereby, terminates the nuclear fission chain reaction in the uranium fuel. The boron-10 isotope makes up ~ 19.8 atomic percent of naturally occurring boron. The boron used is enriched to at least 60-atomic percent boron-10 isotope.

The specified neutron absorber solution is sodium pentaborate ($NA_2B_{10}O_{16}10H_2O$), which is prepared by dissolving granular-enriched sodium pentaborate in demineralized water. A sparger is provided in the tank for mixing, using air. To prevent system plugging, the tank outlet is located above the bottom of the tank.

Whenever it is possible to make the reactor critical, the SLCS is able to deliver at least a 7.0% solution of 60-atomic percent boron-10 enriched sodium pentaborate into the reactor to ensure reactor shutdown. Figure 4.2-15 shows the allowable region of operation for solution concentration and volume.

The calculation methodology for post-LOCA suppression pool pH control is based on the approach outlined in NUREG-1465 and NUREG/CR-5950. Specifically, credit is taken for sodium pentaborate addition to the suppression pool water as a result of SLCS system operation. The pH of the suppression pool water is analyzed as a function of time. Calculations

are performed to verify that sufficient sodium pentaborate solution is available to maintain the suppression pool pH at or above 7 for 30 days post-accident. The design inputs are conservatively established to maximize the post-LOCA production of acids and to minimize the post-LOCA production and/or addition of bases. Other design input values such as initial suppression pool volume and pH are selected to minimize the calculated pH.

Storage tank solution temperature is maintained by adjusting the storage tank heater-indicating controller to maintain temperature between 65°F and 75°F to prevent precipitation of the sodium pentaborate from solution. Thermostat controlled heat tracing is run along the pump suction piping to maintain suction piping solution temperature. Figure 4.2-16 shows the minimum specified borate solution temperature to ensure that the boron remains in solution and does not precipitate out in the storage tank or in the pump suction piping. The temperature versus concentration curve of figure 4.2-16 ensures a 10°F margin will be maintained above the saturation temperature. A temperature switch located in the suction line will actuate a low-solution temperature alarm in the MCR. Storage tank high or low temperature, and high or low liquid level actuate MCR alarms. Tank level is also indicated in the MCR.

Each positive displacement pump is sized to inject the solution into the reactor in 30 to 90 min (~ 43 gal/min), depending on the amount of solution in the tank. The pump and system design pressure between the explosive valves and the pump discharge is 1400 psig. The two relief valves are set slightly under 1400 psig to exceed the RPV operating pressure by a sufficient margin to avoid valve leakage. The relief valves are installed with the discharge lines flooded to prevent evaporation and precipitation within the valve. To prevent bypass flow from one pump, in case of relief valve failure in the line from the other pump, a check valve is installed downstream of each relief valve line in the pump discharge pipe.

The two explosive-actuated injection valves assure opening, when needed, and ensure boron does not leak into the reactor even when the pumps are being tested. Each explosive valve is closed by a plug in the inlet chamber. The plug is circumscribed with a deep groove so the end readily shears off when pushed with the valve plunger. This opens the inlet hole through the plug. The sheared end is pushed out of the way in the chamber; it is shaped so it does not block the ports after release.

The shearing plunger is actuated by an explosive charge with dual ignition primers inserted in the side chamber of the valve. Ignition circuit continuity is monitored by a trickle current, and an alarm occurs in the MCR if either circuit opens. Indicator lights show which primer circuit opened. To service a valve after firing, a 6-in. length of pipe (spool piece) must be removed immediately upstream of the valve to gain access to the shear plug. The SLCS is actuated by a three-position keylocked switch on the MCR console. This ensures that switching from the off position is a deliberate act. Switching to either side starts an injection pump, actuates both of the explosive valves, and closes the RWC system outboard isolation valve to prevent loss or dilution of the boron.

A green light in the MCR indicates that power is available to the pump motor contactor and that the contactor is open (pump not running). A red light indicates that the contactor is closed (pump running).

If the storage tank level, pump lights, pump discharge pressure, or explosive valve lights indicate that liquid may not be flowing, the operator can immediately turn the switch to the other side, which actuates the alternate pump. Cross-piping and check valves ensure a flow path through either pump and either explosive valve. The selected pump starts even though its local switch at the pump is in the STOP position for test or maintenance. Pump discharge pressure indication is also provided in the MCR.

Equipment drains and tank overflow are piped either to a chemical waste drain or to separate containers (such as 55-gal drums) in an effort to prevent boron from reaching the RPV. Although not a safety concern, it is undesirable for boron to reach the RPV due to the effect on reactor power level. The potential for such an occurrence is eliminated if the liquid is routed to separate containers (such as 55-gal drums). If the liquid is routed to a chemical waste drain, it is either filtered (and neutralized, as required) prior to dilution and discharge from the plant or filtered (and neutralized, as required) prior to being treated by an ion exchange and returned to the CST. If the chemical waste is discharged, the potential for boron to enter the reactor vessel is eliminated. If the chemical waste is treated and returned to the CST, a small potential exists for trace amounts of boron to enter the RPV. Such an occurrence requires the use of the RWC system for mitigation.
Instrumentation consisting of solution temperature indication and control, tank level, and heater system status is provided locally at the storage tank.

4.2.3.4.3 Safety Evaluation

Reactivity Control Function

The SLCS is a reactivity control system and is maintained in a STANDBY operational status whenever it is permissible for the reactor to be critical. The system is expected never to be needed for safety reasons because of the large number of independent control rods available to shut down the reactor.

However, to ensure availability of the SLCS, two sets of the components required to actuate the system (pumps and explosive valves) are provided in parallel redundancy.

The SLCS is designed to bring the reactor from rated power to a cold shutdown at any time in core life. The reactivity compensation provided reduces reactor power from rated to zero level and permits cooling the nuclear system to room temperature, with the control rods remaining withdrawn in the rated power pattern. It includes the reactivity gains that result from complete decay of the rated power xenon inventory. It also includes the positive reactivity effects from eliminating steam voids, changing water density from hot to cold, reduced Doppler effect in uranium, reducing neutron leakage from boiling to cold, and decreasing control rod worth as the moderator cools.

The specified minimum average concentration of natural boron in the reactor to provide the specified shutdown margin, after operation of the SLCS, is 800 ppm. Calculation of the minimum quantity of sodium pentaborate to be injected into the reactor is based on the required 800-ppm average concentration in the reactor coolant, including recirculation loops, the RHR system in the shutdown cooling mode at 70°F, and RPV normal water level. The result is

increased by 25% to allow for imperfect mixing and leakage and to account for the volume in other small piping connected to the reactor.

In addition to meeting the required concentration of 800-ppm natural boron in the reactor, the SLCS also meets the injection rate requirements of 10 CFR 50.62(c)(4), which requires that the system have a control capacity equivalent to that of a system with an injection rate of 86 gal/min of 13 weight percent solution, normalized to a 251-in. diameter RPV. The control capacity of the SLCS refers to the rate at which boron-10 isotopes are injected into the reactor. The SLCS meets the requirements of 10 CFR 50.62(c)(4) by using a sodium pentaborate solution enriched with at least 60-atomic percent boron-10 isotope. Naturally occurring boron contains ~ 19.8% of the boron-10 isotope. The method used to show equivalence to 10 CFR 50.62 is set forth in reference 10.

In figure 4.2-15, the 7.0% minimum concentration limit ensures that the SLCS can meet the injection rate requirements of 10 CFR 50.62. The curve showing the minimum volume versus concentration limit ensures that the SLCS can provide a minimum boron-10 isotope concentration in the reactor equivalent to 800 ppm of natural boron, plus 200-ppm margin. The sodium pentaborate solution temperature versus concentration curve shown in figure 4.2-16 ensures a 10°F margin will be maintained above the solution saturation temperature. The 7.0% concentration limit is also identified on this curve. Additionally, Region B, which is outside 10 CFR 50.62 limits, is identified on both curves. Region B corresponds to the original licensing basis limits. If Region A limits are not met, the Technical Specifications allow brief (72 h) operation in Region B.

The 200-ppm margin of natural boron concentration is provided to compensate for any dilution that may occur due to leakage of borated solution from the reactor and replacement with water that is not borated. This margin is adequate to assure minimum boron concentration in the reactor for a period of ~ 24 h after initiation of shutdown. During this 24-h period, an additional charge of sodium pentaborate solution can be prepared in the SLC tank for injection into the reactor if the shutdown period should extend beyond 24 h.

The 24-h time period was determined based upon the following:

A. Boron injection rate into the reactor is at a conservatively low rate of 8 ppm/min (based upon natural boron concentration in the reactor).

B. Leakage flow from the reactor is continuous and at the rate of 50 gal/min at 1000-psig reactor operating pressure.

C. The system is required to shut down the reactor from full power and maintain the reactor subcritical with at least a 0.03-Δk margin.

Cooling down of the nuclear system requires a minimum of several hours to remove the thermal energy stored in the reactor, cooling water, and associated equipment and to remove most of the radioactivity decay heat. The controlled limit for the RPV cooldown is 100°F/h, and normal operating temperature is ~ 550°F. Use of the main condenser and various shutdown cooling systems requires 10 to 24 h to lower the RPV to room temperature (70°F).

The SLCS equipment essential for injection of neutron absorber solution into the reactor is designed as Seismic Category I for withstanding the specified earthquake loadings (HNP-1-FSAR subsection 2.5.7 and HNP-2-FSAR chapter 3). Nonprocess equipment such as the test tank is not Seismic Category I. The system piping and equipment are designed, installed, and tested in accordance with requirements given in HNP-1-FSAR appendix A and HNP-2-FSAR chapter 3.

The SLCS is required to be operable in the event of an offsite power failure. Therefore, the pumps, heaters, valves, and controls are powered from the standby ac power supply. The pumps and valves are powered and controlled from separate buses and circuits so that a single active failure does not prevent system operation.

The SLCS and pumps have sufficient pressure margin, up to the system relief valve setting of ~ 1400 psig, to ensure solution injection into the reactor above the normal pressure in the bottom of the reactor. The nuclear system relief and safety valves begin to relieve pressure at setpoints listed in HNP-1-FSAR table 4.11-1 and HNP-2-FSAR table 5.2-4. Therefore, the SLCS positive displacement pumps cannot overpressurize the nuclear system.

Only one of the two SLCS pumps is needed for system operation. If one pump becomes inoperable, there is no immediate threat to shutdown capability, and reactor operation can continue during repairs. The time during which one redundant component upstream of the explosive valves may be out of operation should be consistent with the probability of failure of both the control rod shutdown capability and the alternative component in the SLCS, and the fact that nuclear system cooldown takes several hours, while liquid control solution injection takes ~ 1 h. Since this probability is small, considerable time is available for repairing and restoring the SLCS to an operable condition while reactor operation continues. Assurance that the system still fulfills its function during repairs is obtained by demonstrating operation of the operable pump.

The SLCS was evaluated for the increase in rated thermal power to 2804 MWt and reactor operating pressure increase from 1050 psia to 1060 psia.[24] [25] The ability of the SLCS boron solution to achieve and maintain safe shutdown is not a direct function of core thermal power and reactor vessel pressure; therefore, it is not affected by power uprate.

The SLCS is designed for injection at a maximum RPV pressure equal to the SRV upper analytical pressure, plus system flow and head losses. The SLCS pumps are positive displacement pumps that deliver a constant flowrate regardless of discharge pressure. Therefore, the capability of the SLCS to provide its backup shutdown function is not affected by the SRV mechanical relief setpoint increase. Also, the resulting increase in system operating pressure does not reduce the SLCS pump relief valve pressure margin below recommended levels.

Post-LOCA Suppression Pool pH Control Function

The SLCS system is considered a special safety system or safe shutdown system, and not an engineered safety feature (ESF) system. Therefore, the NRC review guideline “Guidance on the Assessment of a BWR SLC System for pH Control,” dated February 12, 2004, was used to evaluate the SLC system for its ability to perform its AST function of post-LOCA suppression

pool pH control. The SLCS system meets the following key criteria from the referenced NRC review guideline as noted:

- The SLCS should be provided with standby ac power supplemented by the emergency diesel generators.

  The SLCS is required to be operable in the event of an offsite power failure. Therefore, the pumps, valve, and controls are powered from the standby ac power supply. The pumps and valves are powered and controlled from separate buses and circuits so that a single active failure does not prevent system operation.

- The SLCS should be seismically qualified in accordance with Regulatory Guide 1.29 and Appendix A to 10 CFR Part 100.

  The SLCS process equipment, instrumentation, and controls essential for post-LOCA injection of sodium pentaborate solution into the reactor are designed as Seismic Category I in accordance with Appendix A to 10 CFR 100 and Regulatory Guide 1.29 (August 1973).

- The SLCS should be incorporated into the plant's ASME Code Inservice Inspection (ISI) and Inservice Testing (IST) programs based upon the plant's code of record (10 CFR 50.55a).

  The applicable components of the SLCS are inspected and tested in accordance with the HNP-1 and HNP-2 ISI and IST programs as required by 10 CFR 50.55a.

- The SLCS should be incorporated into the plant's Maintenance Rule program consistent with 10 CFR 50.65.

  The functions of the SCLS are evaluated in the HNP Maintenance Rule program consistent with 10 CFR 50.65 to provide reasonable assurance that the system will perform reliably.

- The SLCS should meet 10 CFR 50.49 and Appendix A (GDC 4) to 10 CFR 50.

  The post-LOCA environment in which the SLCS would be required to operate has been determined to be a mild environment. Cables associated with the SLC system were evaluated and determined to be environmentally qualified for the SLCS post-LOCA mission. Therefore, the SLC system meets the requirements of 10 CFR 50.49 and Appendix A to 10 CFR Part 50.

4.2.3.4.4 Tests and Inspections

Operational testing of the SLCS is performed in at least two parts to avoid inadvertently injecting boron into the reactor. With the valves from the storage tank and to the reactor closed and the

three valves to and from the test tank opened, demineralized water in the test tank can be recirculated by locally starting either pump.

During a refueling or maintenance outage, the injection portion of the system can be functionally tested by valving the suction lines to the test tank and actuating the system from the MCR. Both injection valves open on actuation. System operation is indicated in the MCR.

After functional tests, the injection valve shear plugs and explosive charges must be replaced and all the valves returned to their normal positions as indicated on HNP-1 drawing no. H-16061 and HNP-2 drawing no. H-26009.

After closing a local locked-open valve to the reactor, leakage through the injection valves can be determined by opening valves at a test connection in the line between the containment isolation check valves. Position-indicator lights in the MCR indicate that the local valve is closed for tests or open and ready for operation. Leakage from the reactor through the first check valve can be detected by opening the same test connection when the reactor is pressurized.

The test tank contains demineralized water for ~ 3 min of pump operation. Demineralized water from the makeup system or the condensate storage system is available for refilling or flushing the system.
Should the boron solution ever be injected into the reactor (either intentionally or inadvertently), after making certain that the normal reactivity controls keep the reactor subcritical, the boron is removed from the reactor coolant system by flushing for gross dilution followed by operating the RWC system (subsection 5.5.8).

The concentration of the sodium pentaborate in the solution tank is determined periodically by chemical analysis. This analysis must be performed any time sodium pentaborate or water is added to the storage tank to verify concentration is within Region A limits of figures 4.2-15 and 4.2-16.

4.2.3.4.5 Instrumentation

The instrumentation and control system for the SLCS is designed to allow the injection of liquid poison into the reactor and the maintenance of the liquid poison solution well above the saturation temperature. Discussion of the SLCS instrumentation is included in HNP-1-FSAR paragraph 7.3.4.8 and HNP-2-FSAR subsection 7.4.2.

DOCUMENTS INCORPORATED BY REFERENCE INTO THE FSAR

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21. "BWR Channel Mechanical Design and Deflections," NEDP-21354 and Amendments 1 and 2, September 1976.

22. "Peach Bottom Atomic Power Station Units 2 and 3 Safety Analysis Report for Plant Modifications to Eliminate Significant In-Core Vibrations," NEDO-20994, September 1975.

23. "GE Evaluation in Response to NRC Request Regarding BWR Scram System Pipe Breaks," NEDE-24342.

24. NRC Safety Evaluation and GENE Technical Evaluation: “GE Marathon Control Rod Assembly,” NEDE-31758P-A, October 1991.

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TABLE 4.2-1

DEFORMATION LIMIT
(FOR REACTOR INTERNAL STRUCTURES ONLY)
(HNP-1 AND HNP-2)

Either One of (Not Both)	General Limit
$\left(\dfrac{\text{Permissible deformation (DP)}}{\text{Analyzed deformation causing loss of function}}\right)$	$\leq \dfrac{0.9}{SF_{min}}$
$\left(\dfrac{\text{Permissible deformation (DP)}}{\text{Experiment deformation causing loss of function (DE)}}\right)$[a]	$\leq \dfrac{1.0}{SF_{min}}$

where:

DP = permissible deformation under stated conditions of normal, upset, emergency, or fault.

DL = analyzed deformation which could cause a system loss of function.[b]

DE = experimentally determined deformation which could cause a system loss of function.

a. The second equation is not used unless supporting data are provided to the NRC.

b. Loss of function can be defined only quite generally until attention is focused on the component of interest. In cases of interest, where deformation limits can affect the function of equipment and components, they will be specifically delineated. From a practical viewpoint, it is convenient to interchange some deformation condition at which function is ensured with the loss-of-function condition if the required safety margins from the functioning conditions can be achieved. Therefore, it is often unnecessary to determine the actual loss-of-function condition because this interchange procedure produces conservative and safe designs. Examples where deformation limits apply are control rod drive alignment and clearances for proper insertion, core support deformation causing fuel disarrangement, or excess leakage of any component.

TABLE 4.2-2 (SHEET 1 OF 2)

PRIMARY STRESS LIMIT
(FOR REACTOR INTERNAL STRUCTURES ONLY)
(HNP-1 AND HNP-2)

Any One Of (Not More Than One Required)	General Limit
$\left(\frac{\text{Elastic evaluated primary stresses (PE)}}{\text{Permissible primary stresses (PN)}}\right)$	$\leq \frac{2.25}{SF_{min}}$
$\left(\frac{\text{Permissible load (LP}}{\text{Largest lower bound limit load (CL)}}\right)$	$\leq \frac{1.5}{SF_{min}}$
$\left(\frac{\text{Elastic evaluated primary stress (PE)}}{\text{Conventional ultimate strength at temperature (US)}}\right)$	$\leq \frac{0.75}{SF_{min}}$
$\left(\frac{\text{Elastic-plastic evaluated nominal primary stress (EP)}}{\text{Conventional ultimate strength at temperature (US)}}\right)$	$\leq \frac{0.9}{SF_{min}}$
$\left(\frac{\text{Permissible load (LP)}}{\text{Plastic instability load (PL)}}\right)^{(a)}$	$\leq \frac{0.9}{SF_{min}}$
$\left(\frac{\text{Permissible load (LP)}}{\text{Ultimate load from fracture analysis (UF)}}\right)^{(a)}$	$\leq \frac{0.9}{SF_{min}}$
$\left(\frac{\text{Permissible load (LP)}}{\text{Ultimate load or loss-of-function load from test (LE)}}\right)^{(a)}$	$\leq \frac{1.0}{SF_{min}}^{(b)}$
	$\leq \frac{0.9}{SF_{min}}^{(c)}$

where:

PE = primary stresses evaluated on an elastic basis. The effective membrane stresses are to be averaged through the load-carrying section of interest. The simplest average bending, shear, or torsion stress distribution which support the external loading are added to the membrane stresses at the section of interest.

PN = permissible primary stress levels under normal or upset conditions under ASME Boiler and Pressure Vessel Code, Section III.

LP = permissible load under stated conditions of normal, upset, emergency, or faulted.

CL = lower bound limit load with yield point equal to 1.5 S_m where: S_m = the tabulated value of allowable stress at temperature of the ASME Code, Section III, or its equivalent. The "lower bound limit load" is defined as that produced from the analysis of an

TABLE 4.2-2 (SHEET 2 OF 2)

ideally plastic (nonstrain hardening) material where deformation increases with no further increase in applied load. The lower-bound load is one in which the material everywhere satisfies equilibrium and nowhere exceeds the defined material yield strength, using either a shear theory or a strain energy-of-distortion theory to relate multiaxial yield to the uniaxial case.

US = conventional ultimate strength at temperature of loading that will cause a system malfunction, whichever is more limiting.

EP = Elastic-plastic-evaluated nominal primary stress. Strain hardening of the material may be used for the actual monotonic stress-strain curve at the temperature of loading, or any approximation to the actual stress-strain curve that everywhere has a lower stress for the same strain as the actual monotonic curve may be used. Either the shear or strain-energy-of-distortion flow rule may be used.

PL = plastic instability load. The plastic instability load is defined here as the load at which any load-bearing section begins to diminish its cross-sectional area at a faster rate than the strain hardening can accommodate the loss in area. This type of analysis requires a true stress-true strain curve or a close approximation based on monotonic loading at the temperature of loading.

UF = ultimate load from fracture analyses. For components that involve sharp discontinuities (local theoretical stress concentration < 3) the use of a fracture-mechanics analysis where applicable, using measurements of plain strain fracture toughness, may be applied to compute fracture loads. Correction for finite plastic zones and thickness effects as well as gross yielding may be necessary. The methods of linear elastic stress analysis may be used in the fracture analysis where its use is clearly conservative or supported by experimental evidence. Examples where "fracture mechanics" may be applied are for fillet welds or end-of-fatigue-life crack propagation.

LE = ultimate load or loss-of-function load as determined from experiment. In using this method, account is taken of the dimensional tolerances which may exist between the actual part and the test part or parts as well as differences which may exist in the ultimate tensile strength of the actual part and the tested parts. The guide to be used in each of these areas is that the experimentally determined load shall use adjusted values to account for material property and dimension variations, each of which has no greater probability than 0.1 of being exceeded in the actual part.

a. This equation is not used unless supporting data are provided to the NRC.
b. HNP-2 only.
c. HNP-1 only.

TABLE 4.2-3

BUCKLING STABILITY LIMIT (FOR REACTOR INTERNAL STRUCTURES ONLY) (HNP-1 AND HNP-2)

Any One Of (Not More Than One Required)	General Limit
$\left(\frac{\text{Permissible load (LP)}}{\text{Code normal event permissible load (PN)}}\right)$	$\leq \frac{2.25}{SF_{min}}$
$\left(\frac{\text{Permissible load (LP)}}{\text{Stability analysis load (SL)}}\right)$	$\leq \frac{0.9}{SF_{min}}$
$\left(\frac{\text{Permissible load (LP)}}{\text{Ultimate buckling collapse}}\right)^{(a)}$	$\leq \frac{1.0}{SF_{min}}$

where:

LP = permissible load under stated conditions of normal, upset, emergency, or faulted.

PN = applicable code normal event permissible load.

SL = stability analysis load. The ideal buckling analysis is often sensitive to otherwise minor deviations from ideal geometry and boundary conditions. These effects are accounted for in the analysis of the buckling stability loads. Examples of this are ovality in externally pressurized shells or eccentricity on column members.

SE = ultimate buckling collapse load as determined from experiment. In using this method, account is taken of the dimensional tolerances which may exist between the actual part and the tested part. The guide to be used in each of these areas is that the experimentally determined load is adjusted to account for material property and dimension variations, each of which has no greater probability than 0.1 of being exceeded in the actual part.

a. This equation is not used unless supporting data are provided to the NRC.

TABLE 4.2-4

FATIGUE LIMIT
(FOR REACTOR INTERNAL STRUCTURES ONLY)
(HNP-1 AND HNP-2)

Summation of fatigue damage usage with design and operation loads following the Miner hypotheses[(a)]

Any One Of (Not More Than One Required)	Limit for Normal and Upset Design Conditions
Mean fatigue[(b)(c)] cycle usage from analysis	≤ 0.05
Mean fatigue[(b)(c)] cycle usage from test	≤ 0.33
Design fatigue cycle usage from analysis, using the method of table 4.2-2	≤ 1.0[(d)]

a. Miner, M. A., "Cumulative Damage in Fatigue," Journal of Applied Mechanics, Vol. 12, ASME, Vol. 67, pp A159-A164, September 1945.
b. Fatigue failure is defined here as a 25% area reduction for a load-carrying member that is required to function, or excess leakage, whichever is more limiting.
c. The first two equations are not used unless supporting data are provided to the NRC.
d. HNP-2 only.

TABLE 4.2-5 (SHEET 1 OF 3)

CORE SUPPORT STRUCTURES STRESS CATEGORIES AND LIMITS OF STRESS INTENSITY FOR NORMAL AND UPSET CONDITIONS (HNP-2)

Stress Category	Primary Stresses: Membrane, P_m (a)(b)(c)	Primary Stresses: Bending, P_b (a)(b)(c)	Secondary Stresses: Secondary, Q (a)(d)(e)	Peak Stresses: Peak, F (a)(d)(e)
	P_m	$P_m + P_b$	$P_m + P_b + Q$	$P_m + P_b + Q + F$
	S_m	$1.5\ S_m$	$3\ S_m$	
	Elastic analysis(e)	Elastic analysis(e)	Elastic analysis(f)	Elastic fatigue(g)(h)
	or	or	or	
Thermal and Upset	$0.67\ L_L$	$0.67\ L_L$	S_L	S_a
	Limit analysis(i)	Limit analysis(i)	Plastic analysis	
	or	or	or	
				$P_m + P_b + Q +$ — S_a
	$0.44\ L_u$	$0.44\ L_u$		
	Test(k)	Test(k)	For cycles < 1000, use peak(l)	Elastic plastic fatigue(g)(h)(i)

TABLE 4.2-5 (SHEET 2 OF 3)

a. The symbols P_m, P_b, Q, and F do not represent single quantities, but rather sets of six quantities representing the six stress components σ_t, σ_1, σ_r, τ_{t1}, τ_{1r}, and τ_{rt}.

b. For configurations where compressive stresses occur, the stress limits are revised to take into account critical buckling stresses. (See paragraph NB-3211 (c) of ASME Code, Section III.) For external pressure, the permissible equivalent static external pressure is as specified by the rules of paragraph NB-3133 of ASME Code, Section III. Where dynamic pressures are involved, the permissible external pressure is limited to 25% of the dynamic instability pressure.

c. When loads are transiently applied, consideration is given to the use of dynamic load amplification and possible change in modulus of elasticity.

d. The stresses in category Q are those parts of the total stress which are produced by thermal gradients, structural discontinuities, etc., and do not include primary stresses which may also exist at the same point. It should be noted, however, that a detailed stress analysis frequently gives the combination of primary and secondary stresses directly, and, when appropriate, this calculated value represents the total of $P_m + P_b + Q$ and not Q alone. Similarly, if the stress in category F is produced by a stress concentration, the quantity F is the additional stress produced by the notch over and above the nominal stress. For example, if a plate has a nominal stress intensity, $P_m = S$, $P_b = 0$, $Q = 0$, and a notch with a stress concentration K is introduced, then $F = P_m(K\text{-}1)$ and the peak stress intensity equals $P_m + P_m(K^m\text{-}1) = KP_m$.

e. The triaxial stresses represent the algebraic sum of the three primary principal stresses (σ_1, + σ_2, + σ_3) for the combination of stress components. Where uniform tension loading is present, triaxial stresses are limited to 4 S_m.

f. This limitation applies to the range of stress intensity. When the secondary stress is due to a temperature excursion at the point at which the stresses are being analyzed, the value of S_m shall be taken as the average of the S_m values tabulated in tables I-1.1, I-1.2, and I-1.3 of the ASME Boiler and Pressure Vessel Code, Section III (ASME III) for the highest and lowest temperature of the metal during the transient. When part of the secondary stress is due to mechanical load, the value of S_m shall be taken as the S_m value for the highest temperature of the metal during the transient.

g. S_a is obtained from the fatigue curves, figures I-9.1 and I-9.2 of ASME Code, Section III. The allowable stress intensity for the full range of fluctuation is 2 S_a.

h. In the fatigue data curves, where the number of operating cycles is $<$ 10, use the S_a value for 10 cycles; where the number is $>$ 10, use the S_a value for 10^6 cycles.

TABLE 4.2-5 (SHEET 3 OF 3)

i. L_L is the lower bound limit load with yield point equal to 1.5 S_m (where S_m is the tabulated value of allowable stress at temperature as contained in ASME Code, Section III). The lower bound limit load is here defined as that produced from the analysis of an ideally plastic (nonstrain hardening) material where deformations increase with no further increase in applied load. The lower bound load is one in which the material everywhere satisfies equilibrium and nowhere exceeds the defined material yield strength, using either a shear theory of a strain-energy-of-distortion theory to relate multiaxial yielding to the uniaxial case.

j. S_L denotes the structural action of shakedown load as defined in paragraph NB-3213.18 of ASME Code, Section III, calculated on a plastic basis as applied to a specific location on the structure.

k. For normal and upset conditions, the limits on primary membrane plus primary bending need not be satisfied in a component if it can be shown from the test of a prototype or model that the specified loads (dynamic or static equivalent) do not exceed 44% of L_u, where L_u is the ultimate load or the maximum load or load combination used in the test. In using this method, account is taken of the size effect and dimensional tolerances which may exist between the actual part and the tested part, or parts, as well as differences which may exist in the ultimate strength or other governing material properties of the actual part and the tested part to ensure that the loads obtained from the test are a conservative representation of the load-carrying capability of the actual component under the postulated loading for normal and upset conditions.

l. The allowable value for the maximum range of this stress intensity is 3 S_m except for cyclic events which occur less than 1000 times during the design life of the plant. For this exception, in lieu of meeting the 3 S_m limit, an elastic-plastic fatigue analysis may be performed in accordance with ASME Code, Section III to demonstrate that the cumulative fatigue usage attributable to the combination of those low events, plus all other cyclic event, does not exceed a fatigue usage value of 1.0.

TABLE 4.2-6 (SHEET 1 OF 3)

CORE SUPPORT STRUCTURES STRESS CATEGORIES AND LIMITS OF STRESS INTENSITY FOR EMERGENCY CONDITIONS (HNP-2)

Stress Category	Primary Stresses: Membrane, P_m [(a)(b)(c)]		Primary Stresses: Bending, P_b [(a)(b)(c)]		Secondary Stresses: Membrane and Bending Secondary (Q)	Peak Stresses: Peak, (F)
	P_m		$P_m + P_b$			
	$1.5\ S_m$ [(d)]	Elastic analysis[(d)]	$2.25\ S_m$	Elastic analysis[(d)]		
	or		or			
Emergency	L_L	Limit analysis[(e)]	L_L	Limit analysis[(e)]	Evaluation not required	Evaluation not required
	or		or			
	$1.5\ S_m$	Elastic analysis[(f)]	$2.25\ S_m$	Plastic analysis[(f)(g)]		
	or		or			
	$0.6\ L_e$	Test[(h)]	$0.5\ S_u$ [(g]			
	or		or			
	S_E	Stress-ratio analysis[(i)]	$0.6\ L_e$	Test[(h)]		
			or			
			KS_E	Stress-ratio analysis[(j)]		

TABLE 4.2-6 (SHEET 2 OF 3)

a. The symbols P_m, P_b, Q, and F do not represent single quantities, but rather sets of six quantities representing the six stress components σ, σ_1, σ_r, τ_{t1}, τ_{1r}, and τ_{rt}.

b. For configurations where compressive stresses occur, stress limits shall be revised to take into account critical buckling stresses. For external pressure, the permissible equivalent static external pressure is taken as 150% of that permitted by the rules of paragraph HB-3133 of ASME Boiler and Pressure Vessel Code, Section III (ASME III). Where dynamic pressures are involved, the permissible external pressure shall satisfy the preceding requirements or is limited to 50% of the dynamic instability pressure.

c. When loads are transiently applied, consideration should be given to the use of dynamic load amplification and possible change in modulus of elasticity.

d. The triaxial stresses represent the algebraic sum of the three primary principal stresses ($\sigma_1 + \sigma_2 + \sigma_3$) for the combination of stress components. Where uniform tension loading is present, triaxial stresses should be limited to 6 S_m.

e. L_L is the lower bound limit load with yield point equal to 1.5 S_m (where S_m is the tabulated value of allowable stress at temperature as contained in ASME III). The lower bound limit load is here defined as that produced from the analysis of an ideally plastic (nonstrain hardening) material where deformation increase with no further increase in applied load. The lower bound load is one in which the material everywhere satisfies equilibrium and nowhere exceeds the defined material yield strength, using either a shear theory or a strain-energy-of-distortion theory to relate multiaxial yielding to the uniaxial case.

f. This plastic analysis uses an elastic-plastic-evaluation nominal primary stress. Strain hardening of the material may be used for the actual monotonic stress-strain curve at the temperature of loading; or any approximation to the actual stress-strain curve, which everywhere has a lower stress for the same strain as the actual monotonic curve, may be used. Either the shear or strain-energy-of-distortion-flow rule is used to account for multiaxial effects.

g. S_u is the ultimate strength at temperature. Multiaxial effects on ultimate strength shall be considered.

h. For emergency conditions, the stress limits need not be satisfied if it can be shown from the test of a prototype or model that the specified loads (dynamic or static equivalent) do not exceed 60% of L_e, where L_e is the ultimate load or the maximum load or load combination used in the test. In using this method, account is taken of the size effect and dimensional tolerances which may exist between the actual part and the tested part or parts as well as differences which may exist in the ultimate strength or other governing material properties of the actual part and the tested parts to assure that the

TABLE 4.2.6 (SHEET 3 OF 3)

loads obtained from the test are a conservative representation of the load-carrying capability of the actual component under postulated loading for emergency conditions.

i. Stress ratio is a method of plastic analysis which uses the stress-ratio combinations (combination of stresses that consider the ratio of the actual stress to the allowable plastic or elastic stress) to compute the maximum load a strain-hardening material can carry. K is defined as the section factor; $S_e \leq S_m$ for primary membrane loading.

j. Where deformation is of concern in a component, the deformation is limited to two-thirds the value given for emergency conditions in the design specification.

TABLE 4.2-7 (SHEET 1 OF 3)

CORE SUPPORT STRUCTURES STRESS CATEGORIES AND LIMITS OF STRESS INTENSITY FOR FAULTED CONDITIONS (HNP-2)

Stress Category	Primary Stresses: Membrane, P_m [a][b][c]		Primary Stresses: Bending, P_b [a][b][c]		Secondary Stresses: Membrane and Bending Secondary (Q)	Peak Stresses: Peak (F)
	P_m		$P_m + P_b$			
	2.4 S_m	Elastic analysis	3.0 S_m	Elastic analysis		
	or		or			
	0.75S_u[e]	Limit analysis[e]	1.33 L_L	Limit analysis[f]	Evaluation not required	Evaluation not required
	or		or			
	1.33 L_L	Limit analysis[f]	0.75 S_u	Plastic analysis[e][g]		
	or		or			
	0.67 S_u	Plastic analysis[e][f]	0.8 L_F	Tests[h]		
	or		or			
	0.8 L_F	Test[h]	KS_F	Stress-ratio analysis[i]		
	or					
	S_F	Stress-ratio analysis[i]				

TABLE 4.2-7 (SHEET 2 OF 3)

a. The symbols P_m, P_b, Q, and F do not represent quantities but rather sets of six quantities representing the six stress components, σ_t, σ_1, σ_r, τ_{t1}, τ_{1r}, and τ_{rt}.

b. When loads are transiently applied, consideration is given to the use of dynamic load amplification and possible changes in modulus of elasticity.

c. For configurations where compressive stresses occur, stress limits take into account critical buckling stresses. For external pressure, the permissible equivalent static external pressure is taken as 2.5 times that given by the rules of paragraph NB-3133 of the ASME Boiler and Pressure Vessel Code Section III (ASME III). Where dynamic pressure is involved, the permissible external pressure shall satisfy the preceding requirements or shall be limited to 75% of the dynamic instability pressure.

d. Where deformation is of concern in a component, the deformation is limited to 80% of the value given for fault conditions in the design specifications.

e. S_u is the ultimate strength at temperature. Multiaxial effects on ultimate strength are considered.

f. L_L is the lower bound limit load with yield point equal to 1.5 S_m (where S_m is the tabulated value of allowable stress at temperature as contained in ASME Code, Section III). The lower bound limit load is here defined as that produced from the analysis of an ideally plastic (nonstrain hardening) material where deformations increase with no further increase in applied load. The lower bound load is one in which the material everywhere satisfies equilibrium and nowhere exceeds the defined material yield strength, using either a shear theory or a strain-energy-of-distortion theory to relate multiaxial yielding to the uniaxial case.

g. This plastic analysis uses an elastic-plastic-evaluated nominal primary stress. Strain hardening of the material may be used for the actual monotonic stress-strain curve at the temperature of loading; or any approximation to the actual stress-strain curve, which everywhere has a lower stress for the same strain as the actual curve, may be used: either the maximum shear stress of strain-energy-of-distortion-flow rule is used to account for multiaxial effects.

h. For fault conditions, the stress limits need not be satisfied if it can be shown from the test of a prototype or model that the specified loads (dynamic or static equivalent) do not exceed 80% of L_F, where L_F is the ultimate load or load combination used in the test. In using this method, account is taken of the size, effect, and dimensional tolerances as well as differences which may exist in the ultimate strength or other governing material properties of the actual part and the tested parts to assure that the loads obtained from the test are a conservative representation of the load-carrying capability of the actual component under postulated loading for fault condition.

TABLE 4.2.7 (SHEET 3 OF 3)

i. Stress ratio is a method of plastic analysis which uses the stress-ratio combinations (combination of stresses that consider the ratio of the actual stress to the allowable plastic or elastic stress) to compute the maximum load a strain-hardening material can carry. K is defined as the section factor; S_f is the lesser of 2.4 S_m or 0.75 S_u for primary membrane loading.

TABLE 4.2-8

STEAM DRYER, CS LINES, AND CORE STRUCTURE MATERIALS (HNP-1 AND HNP-2)

Plate, sheet, strip	ASME SA240 Type 304 or 304L
Bolts	ASME SA193 Grade B8
Nuts	ASME SA194 Grade 8
Forgings	ASME SA 182 Grade F304
Bar	ASTM A276 Type 304 or 304L[a]
Bar	ASME SA479 Type 304 or 304L
Pipe	ASME SA312 Type 304 or 304L
Tube	ASTM A269 Type 304 or 304 L[b]
Pipe fittings	ASME SA403 Type WP304 or WP304L
Pipe fittings	ASME SA351 Type CF8

Material used for the shroud backing ring is ASME SB166 or SB168. Material used for the shroud seismic pin is Inconel X-750 per ASTM A 461-65 GR 688.[c]

Major components of the shroud support portion of the core support structure are fabricated from:

- SB-168 Ni-Fe-Cr plate.
- SA-533 GRB C1.1 low alloy steel (HNP-2 only).

a. ASTM A276 Condition A is equivalent to ASME SA479 Type 304.
b ASTM A269 is permitted to be used if the tensile requirements are in accordance with ASTM A249 (ASME SA249).
c. NRC Regulatory Guide 1.85 recognizes the use of ASME Code Cases 1344-5 which provide Appendix I type design information for Inconel X-750.

TABLE 4.2-9

DESIGN LOADING CONDITIONS AND COMBINATIONS (HNP-2)

Operating Condition and Stress Limits[a]	Design Loading Conditions and Combinations
Normal and upset	N and A_D or N and U.
Emergency	N and R or other conditions which have a 40-year encounter probability from 10^{-1} to 10^{-3}.
Faulted	N and A_m and $\overline{R}$ or other conditions which have a 40-year encounter probability from 10^{-3} to10^{-6}.

where:

N = normal loads.

U = upset loads, excluding earthquake.

A_D = OBE, including any associated transients.

A_m = DBE, including any associated transients.

R = any auxiliary pipe-rupture loading, including any associated transients. (Pipe-rupture loadings are not directly considered on piping itself because this is handled by a failure-mode analysis).

$\overline{R}$ = primary loadings which result from rupture of a main steam line or a recirculation line.

a. The design stress, deformation, and fatigue limits are as follows:

- For RPV and appurtenances - ASME Code, Section III.
- For core support structures - A.5.2.2 of Design Safety Standards, NEDO-10370.
- For reactor internal structures - A.5.2.1 or A.5.2.5 of Design Safety Standards, NEDO-10370.

TABLE 4.2-10

MAXIMUM DIFFERENTIAL PRESSURES ACROSS RPV ASSEMBLY INTERNALS (HNP-1)

Reactor Component	Normal Operation (psi)	Upset (AOOs) (psi)	Faulted (Accident) (psi)
Core plate & guide tube	25.59	27.99	32.00
Shroud support	33.26	35.66	53.0
Shroud	7.73	11.59	29.5
Top guide	0.65	0.72	3.6[a]
Shroud head	8.51	12.76	29.5
Dryer	0.4	0.52	<9.1[a]
Fuel channel wall (maximum power bundle)	13.25	16.15	15.5

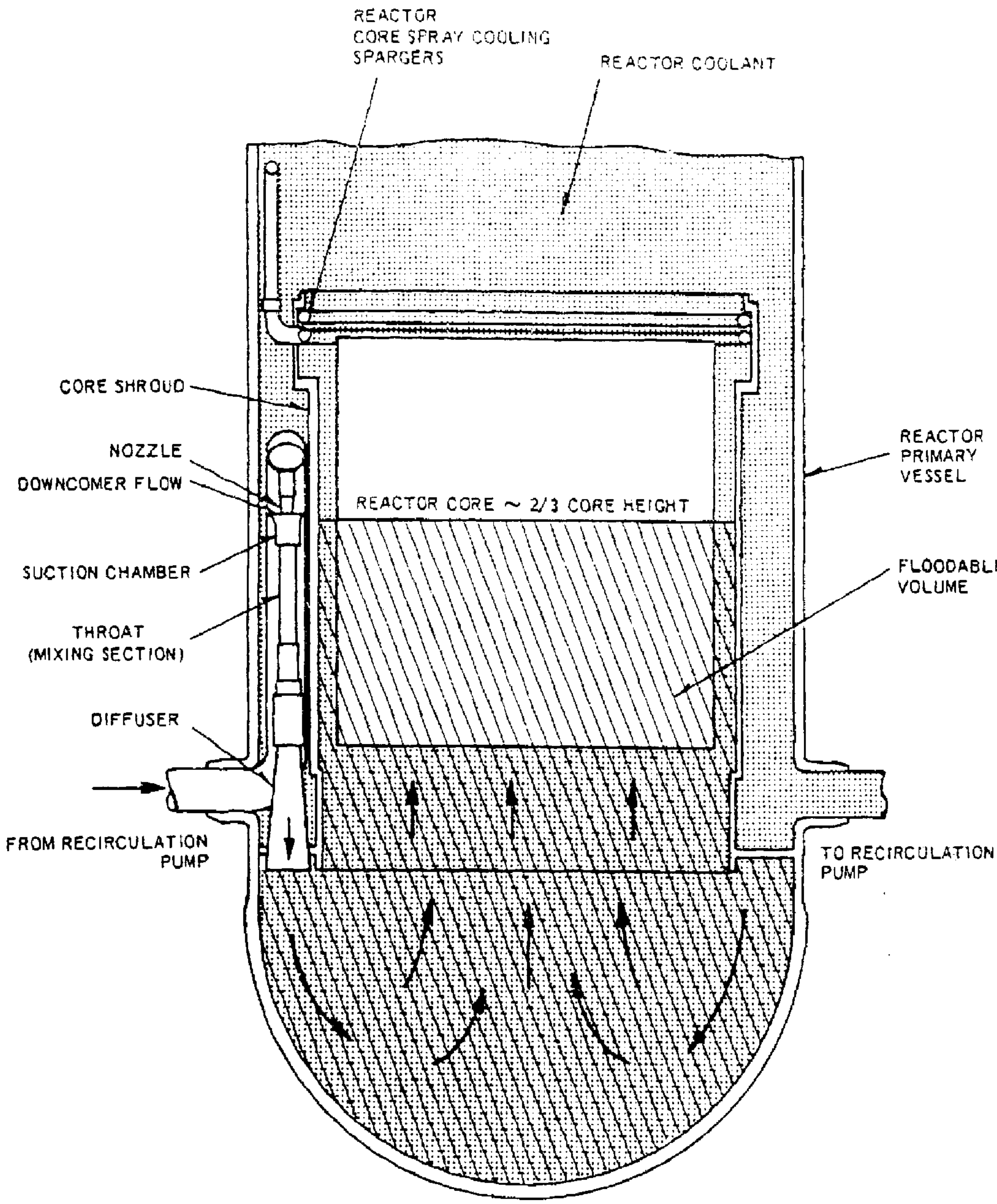
a. Limiting at low-power, high-core flow.

TABLE 4.2-11

MAXIMUM DIFFERENTIAL PRESSURES ACROSS RPV ASSEMBLY INTERNALS (HNP-2)[a]

Reactor Component	Normal Operation (psi)	Upset (AOOs) (psi)	Faulted (Accident) (psi)
Core plate and guide tube	22.17	24.57	26.5[b]
Shroud support	29.93	32.33	49.0
Shroud	7.81	11.72	29.5[b]
Top guide	0.64	0.71	3.1[b]
Shroud head	8.6	12.91	29.0
Dryer	0.43	0.56	<10[b]
Fuel channel wall (maximum power bundle)	13.47	16.37	16.0

a. Data presented is based upon the GE13 fuel design which is bounding to GE14 and GNF2 fuel.
b. Limiting at low-power, high-core flow.



ACAD 040201

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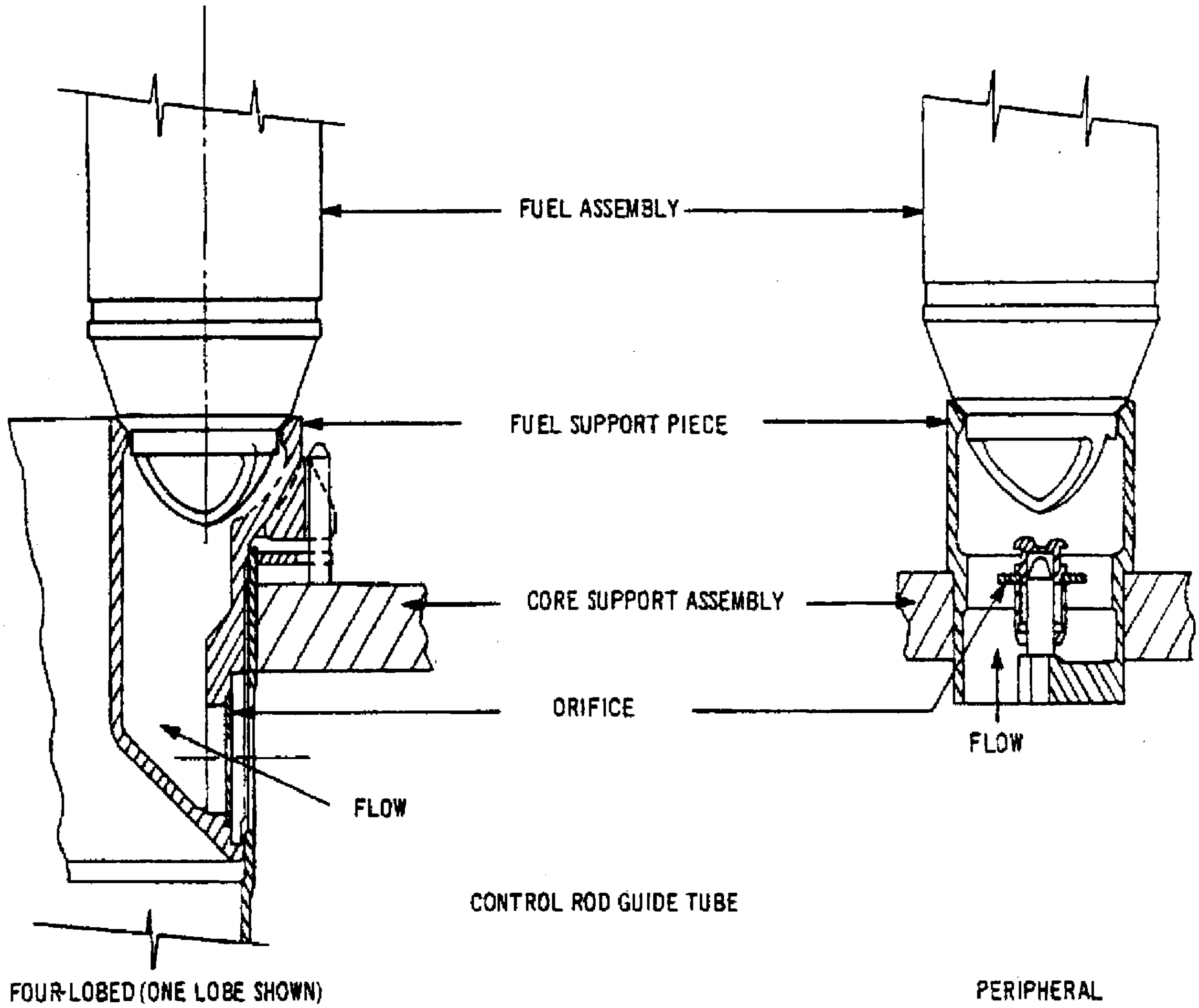
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

REACTOR INTERNALS FLOW PATHS

FIGURE 4.2-1



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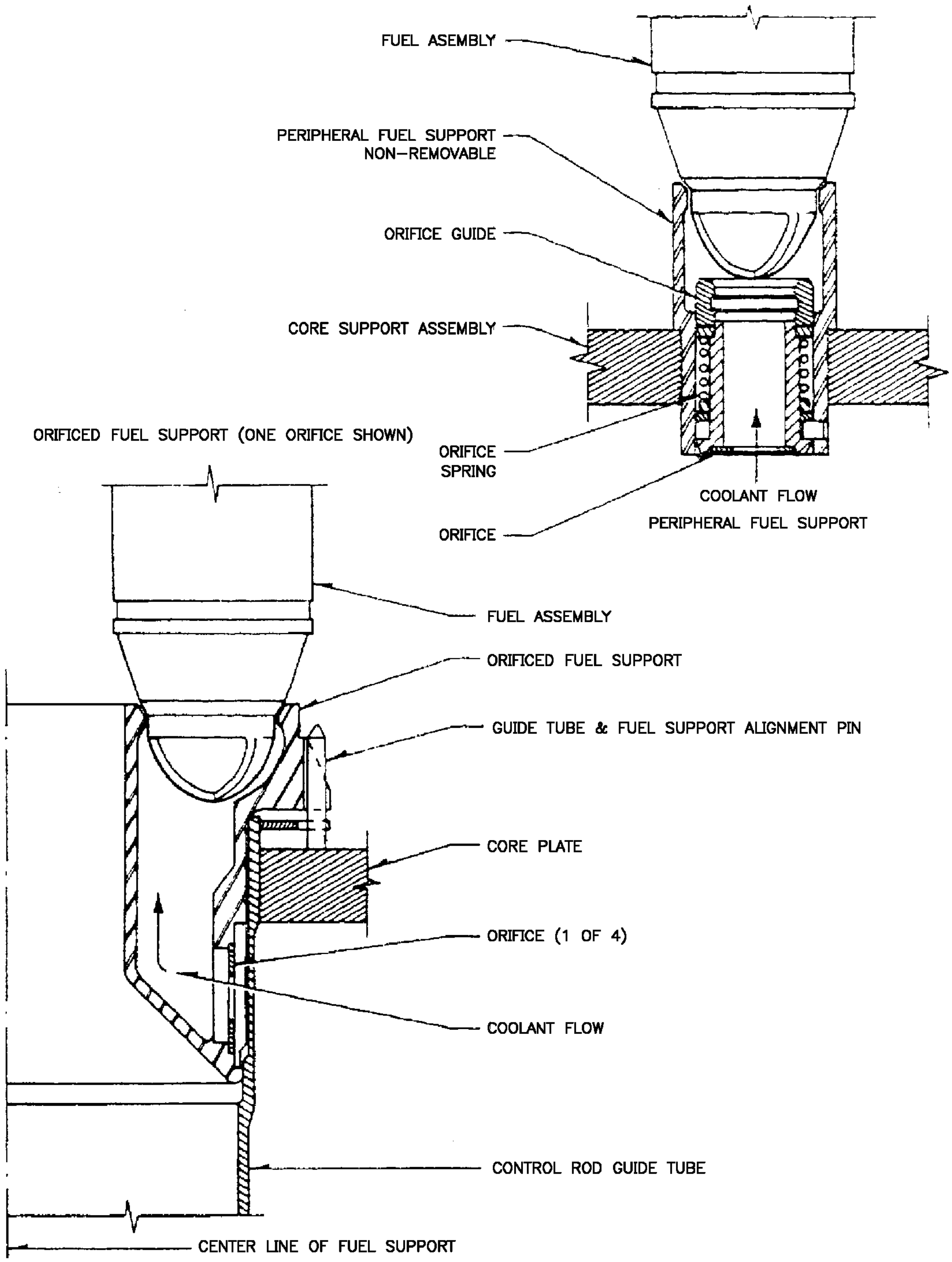
UNIT 1

HNP-1 FUEL SUPPORT PIECES

FIGURE 4.2-2



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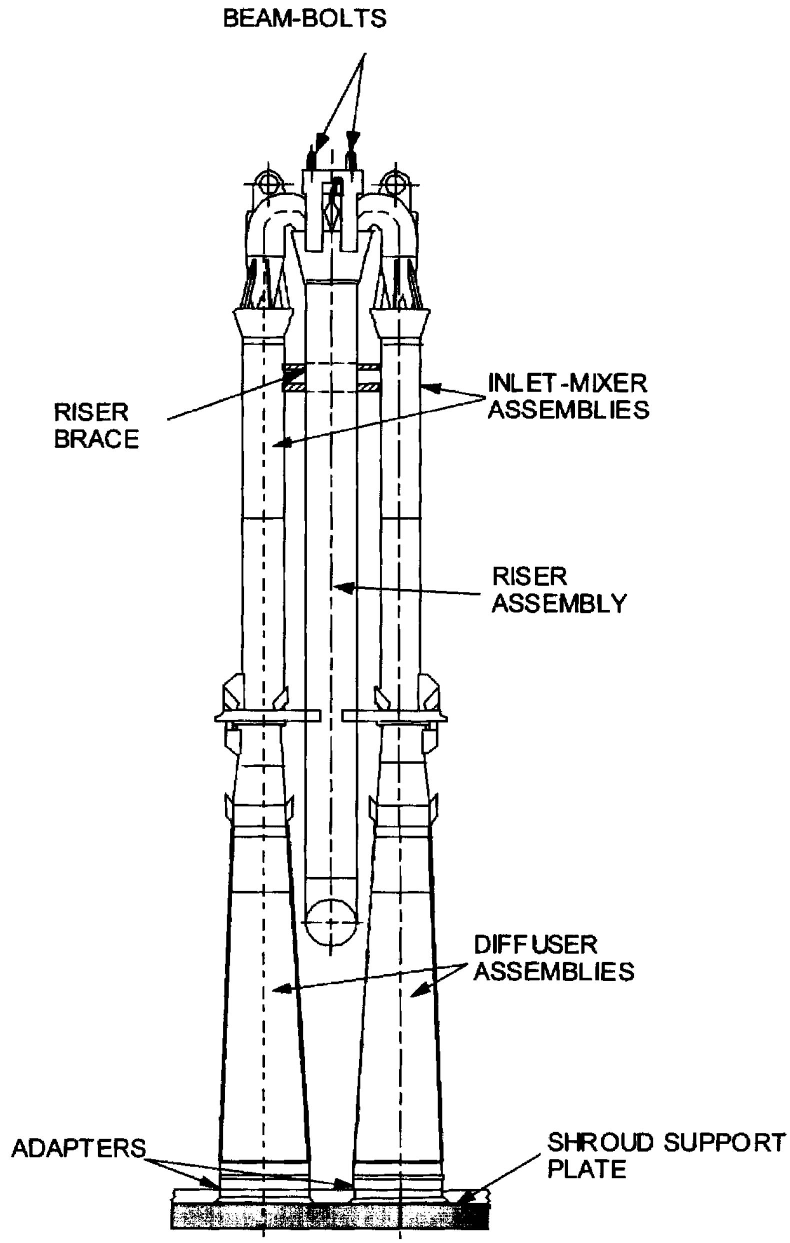
HNP-2 FUEL SUPPORT PIECES

FIGURE 4.2-3



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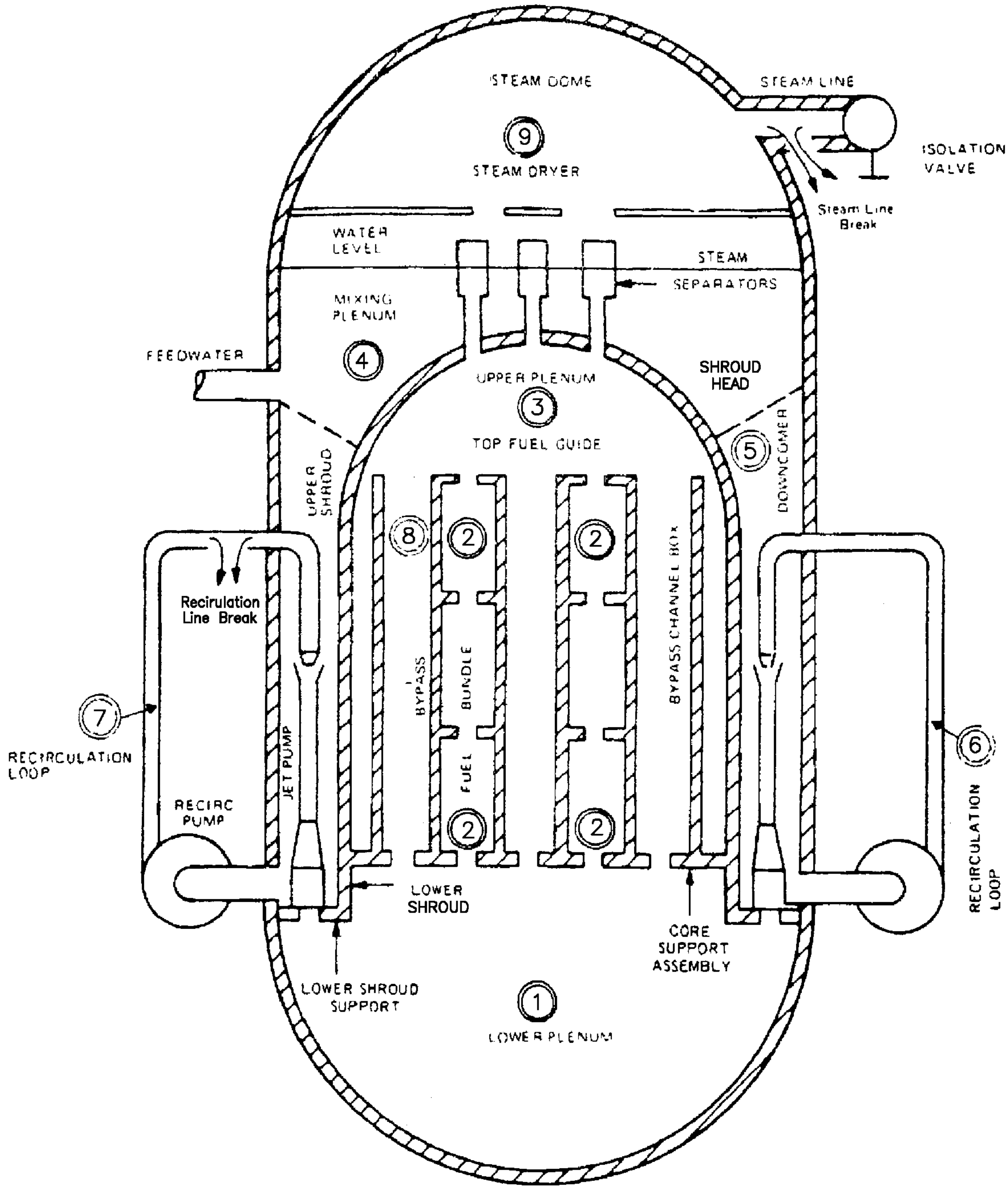
JET PUMP ISOMETRIC

FIGURE 4.2-4



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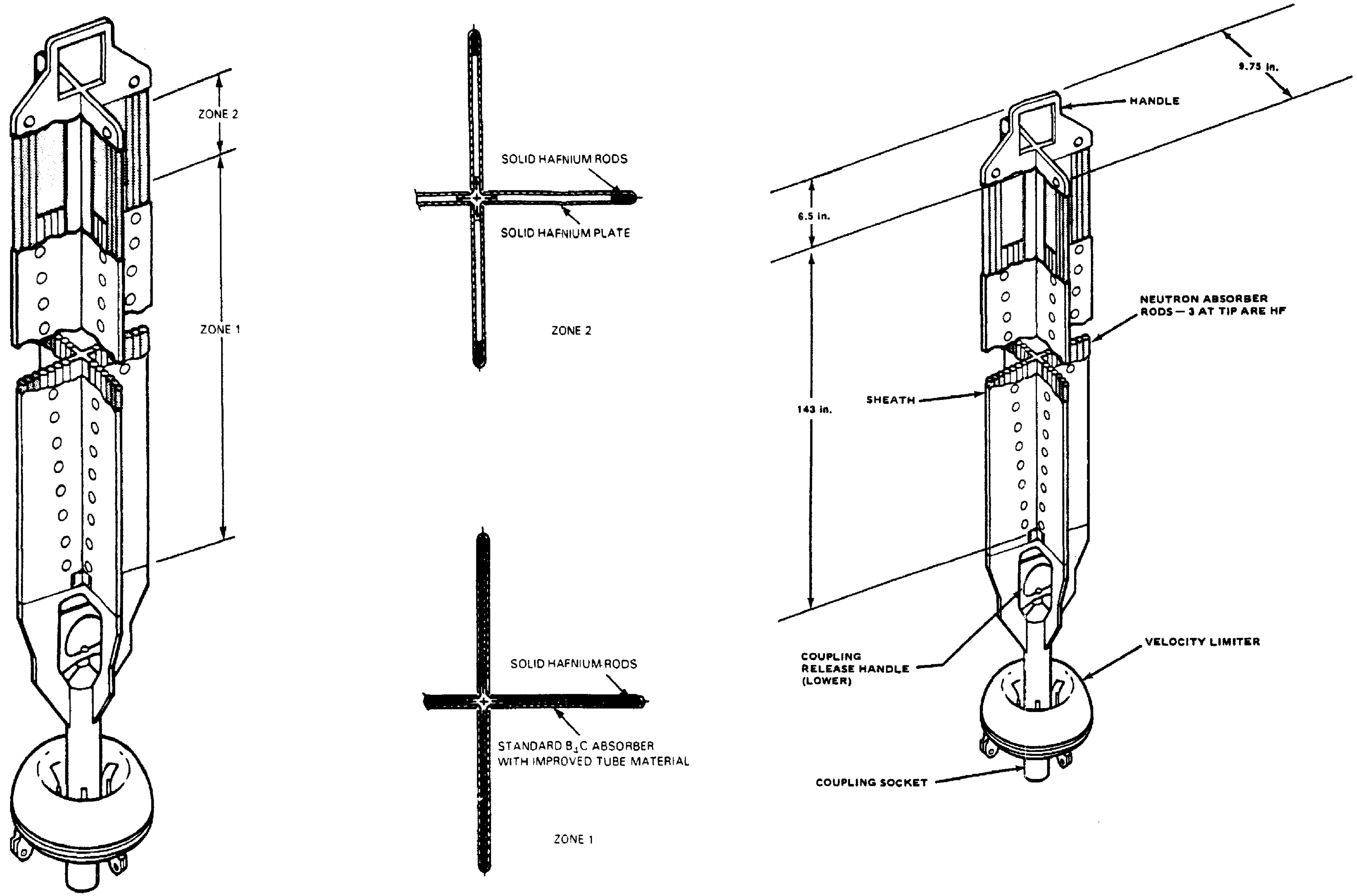
PRESSURE NODES FOR
DEPRESSURIZATION ANALYSIS

FIGURE 4.2-5



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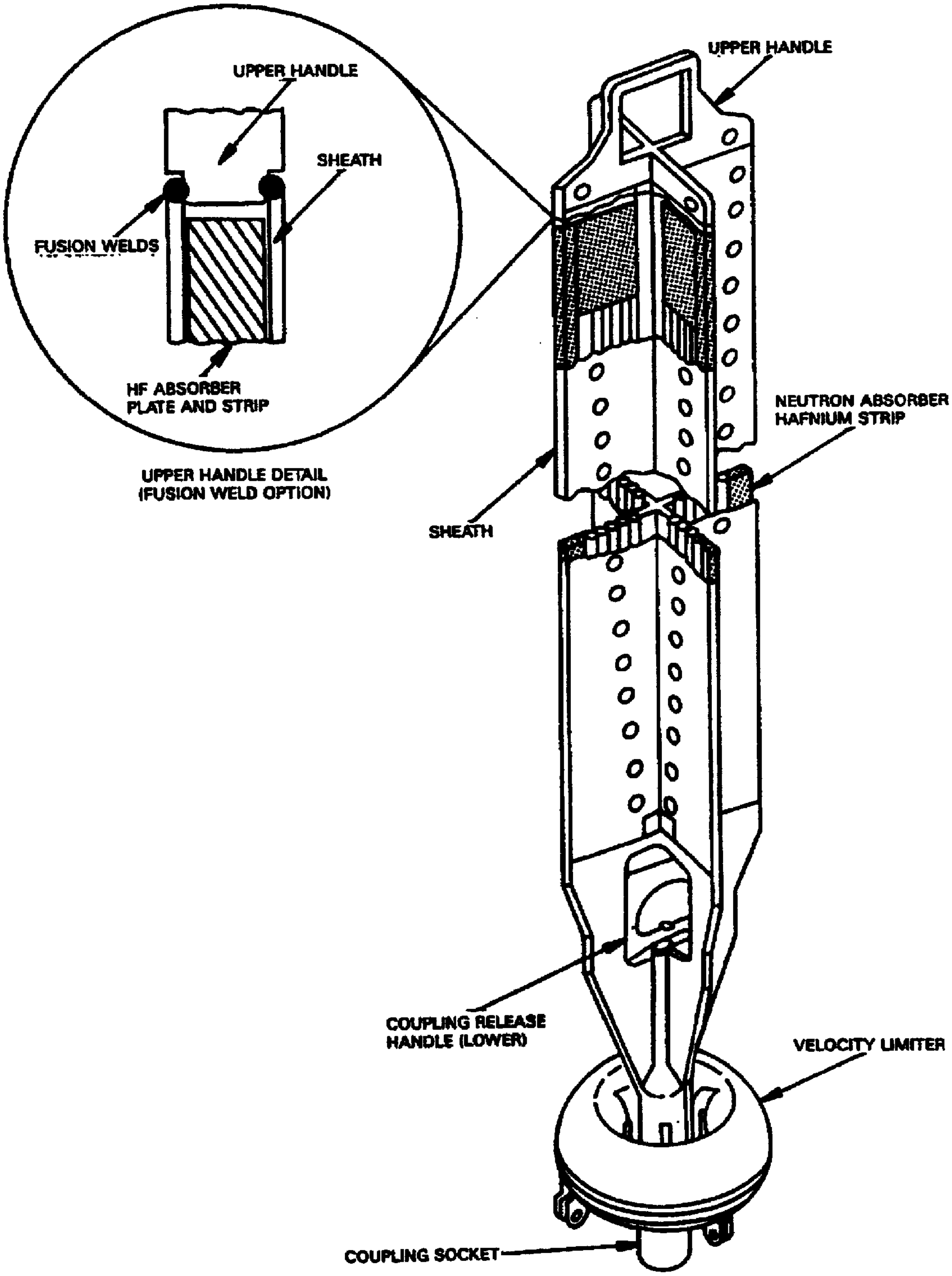
DURLIFE D-190
CONTROL ROD ASSEMBLY

FIGURE 4.2-6



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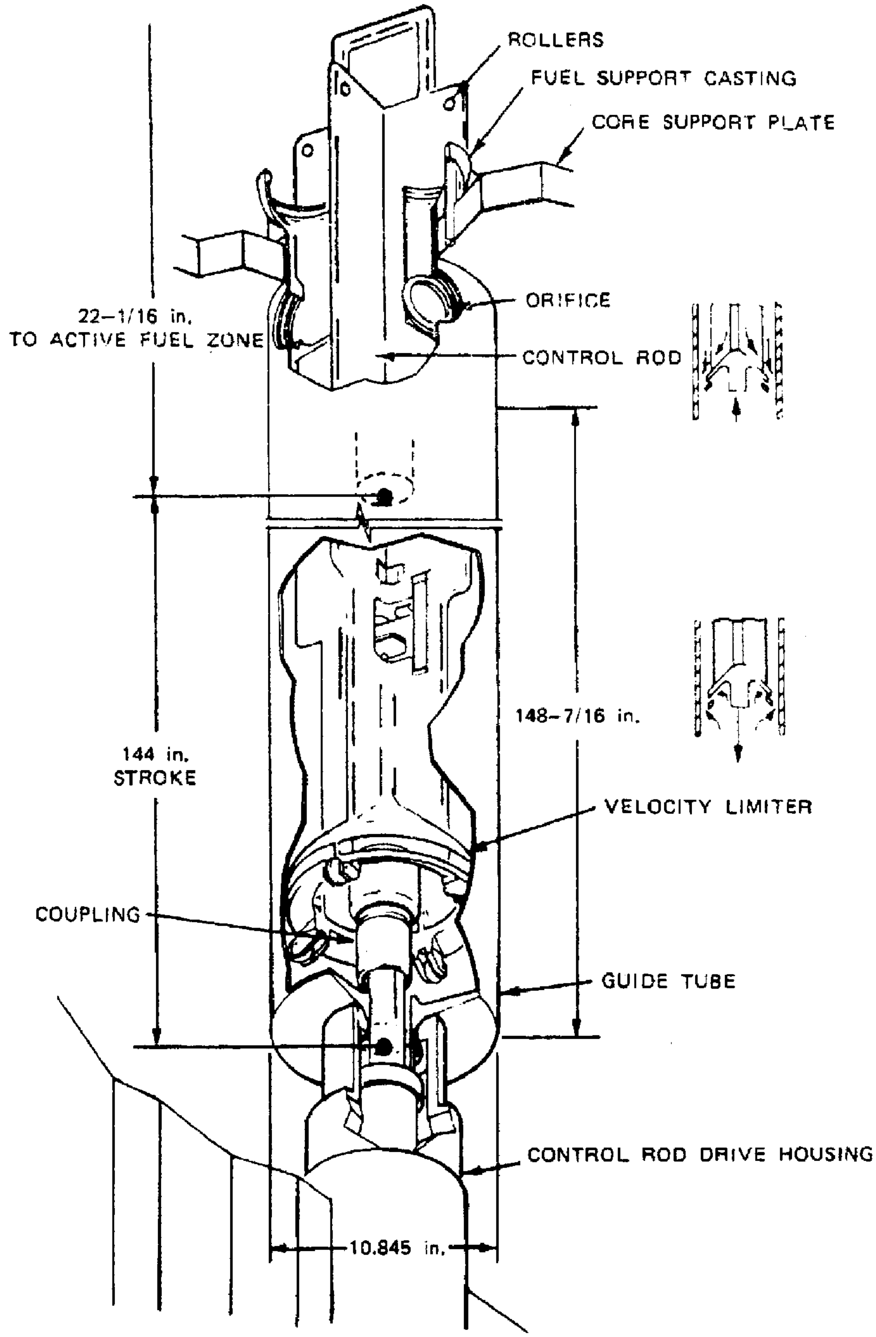
UNIT 1 AND UNIT 2

DURALIFE D-230
CONTROL ROD ASSEMBLY

FIGURE 4.2-7



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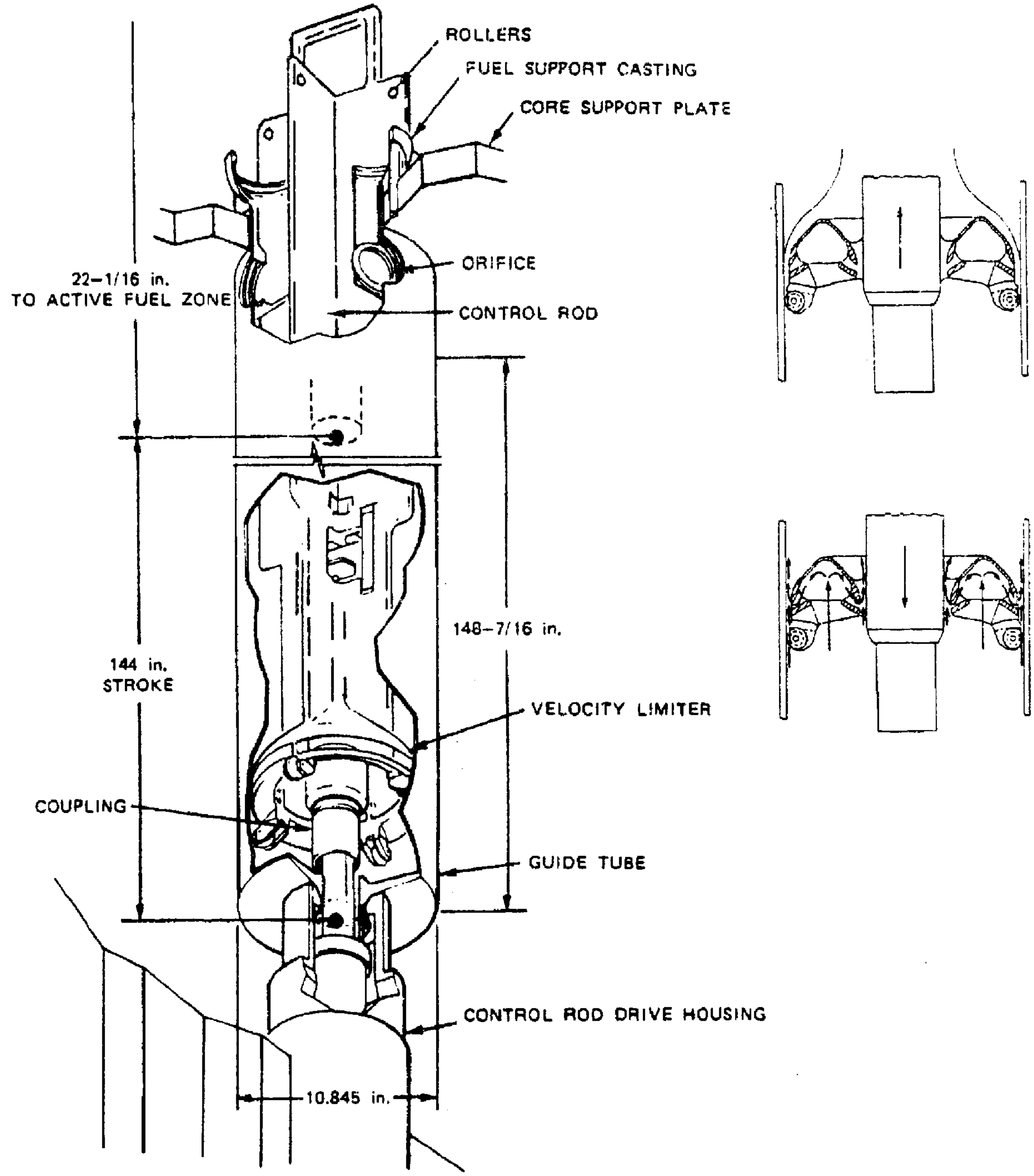
CONTROL ROD VELOCITY LIMITER

FIGURE 4.2-8



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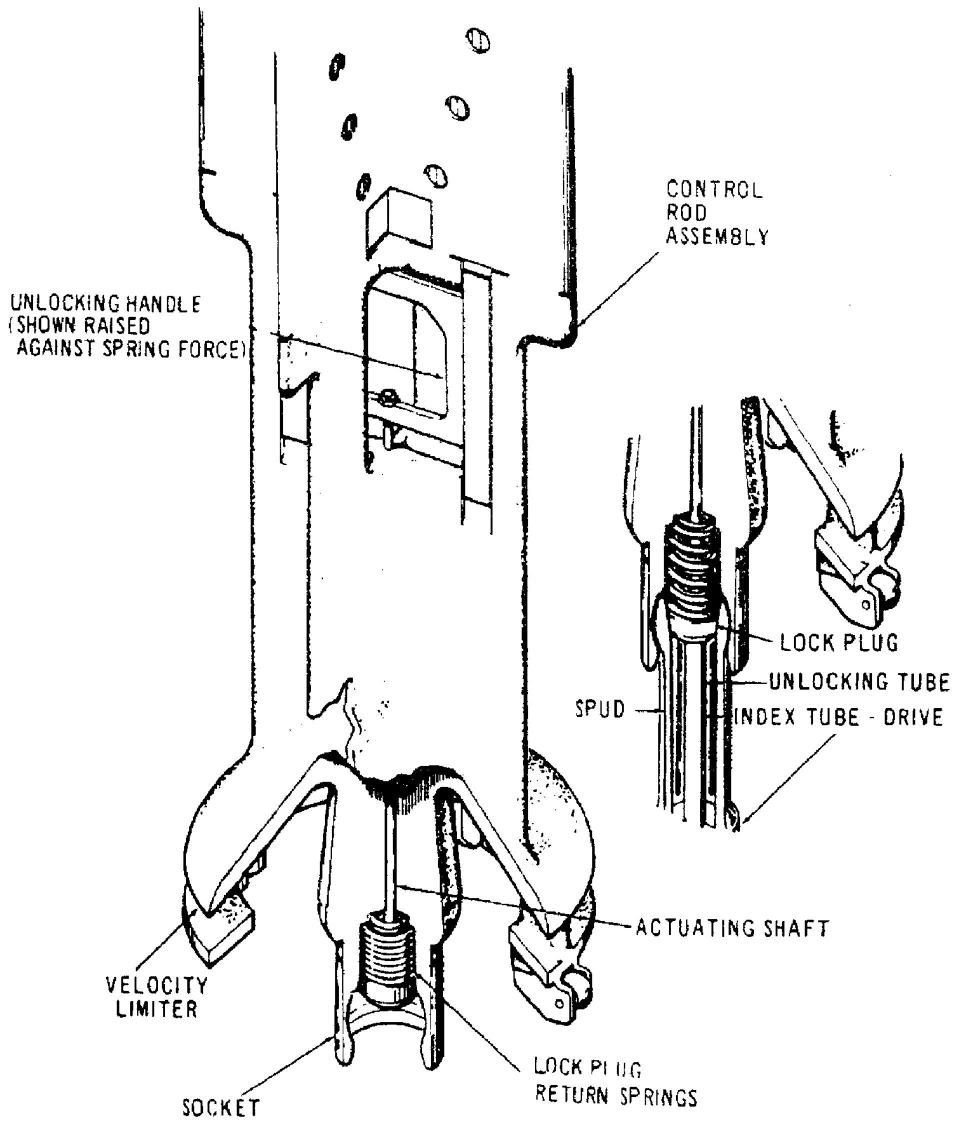
UNIT 1 AND UNIT 2

ADVANCED LONGER LIFE
CONTROL ROD VELOCITY LIMITER

FIGURE 4.2-9



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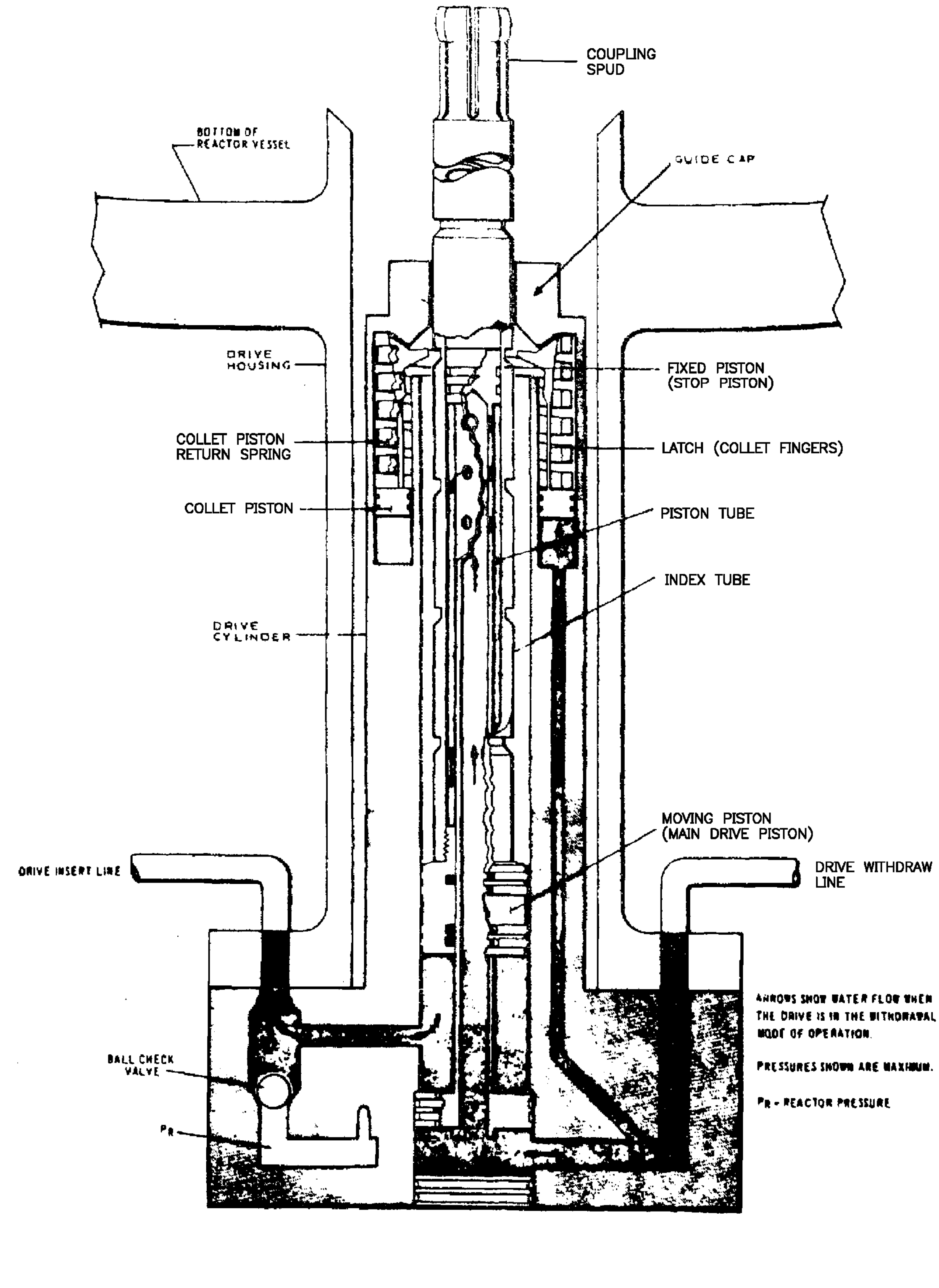
UNIT 1 AND UNIT 2

CONTROL ROD AND
CONTROL ROD DRIVE COUPLING

FIGURE 4.2-10



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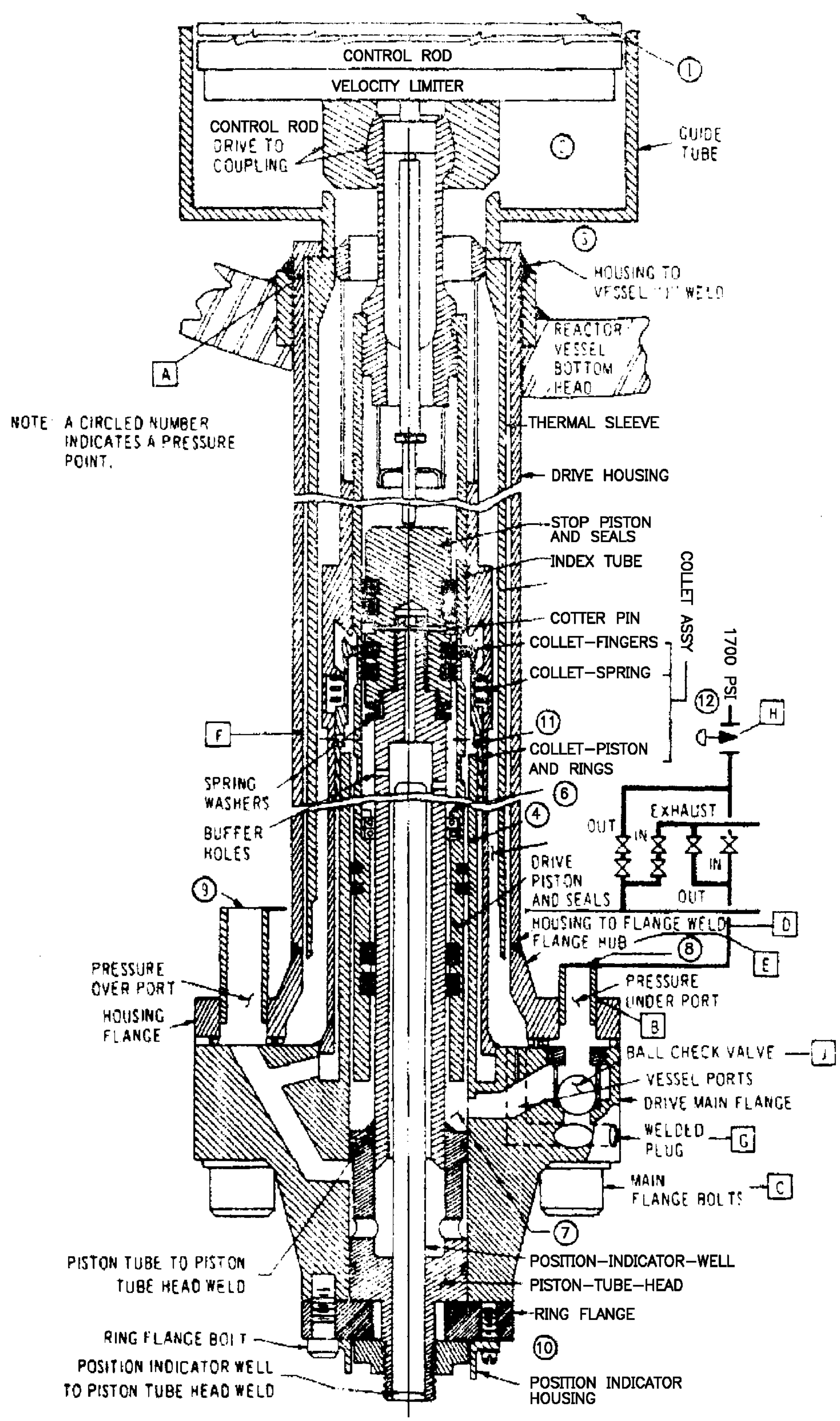
UNIT 1 AND UNIT 2

CONTROL ROD DRIVE UNIT

FIGURE 4.2-11



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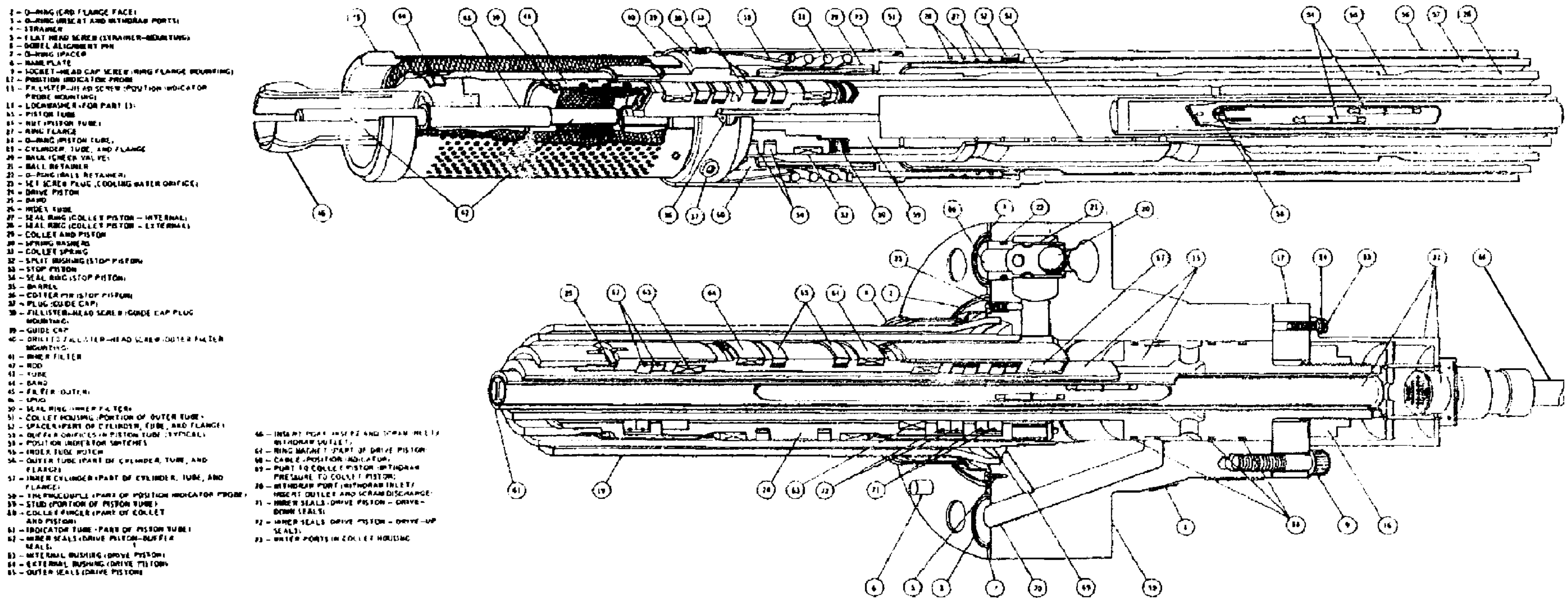
UNIT 1 AND UNIT 2

CONTROL ROD DRIVE UNIT (SCHEMATIC)

FIGURE 4.2-12



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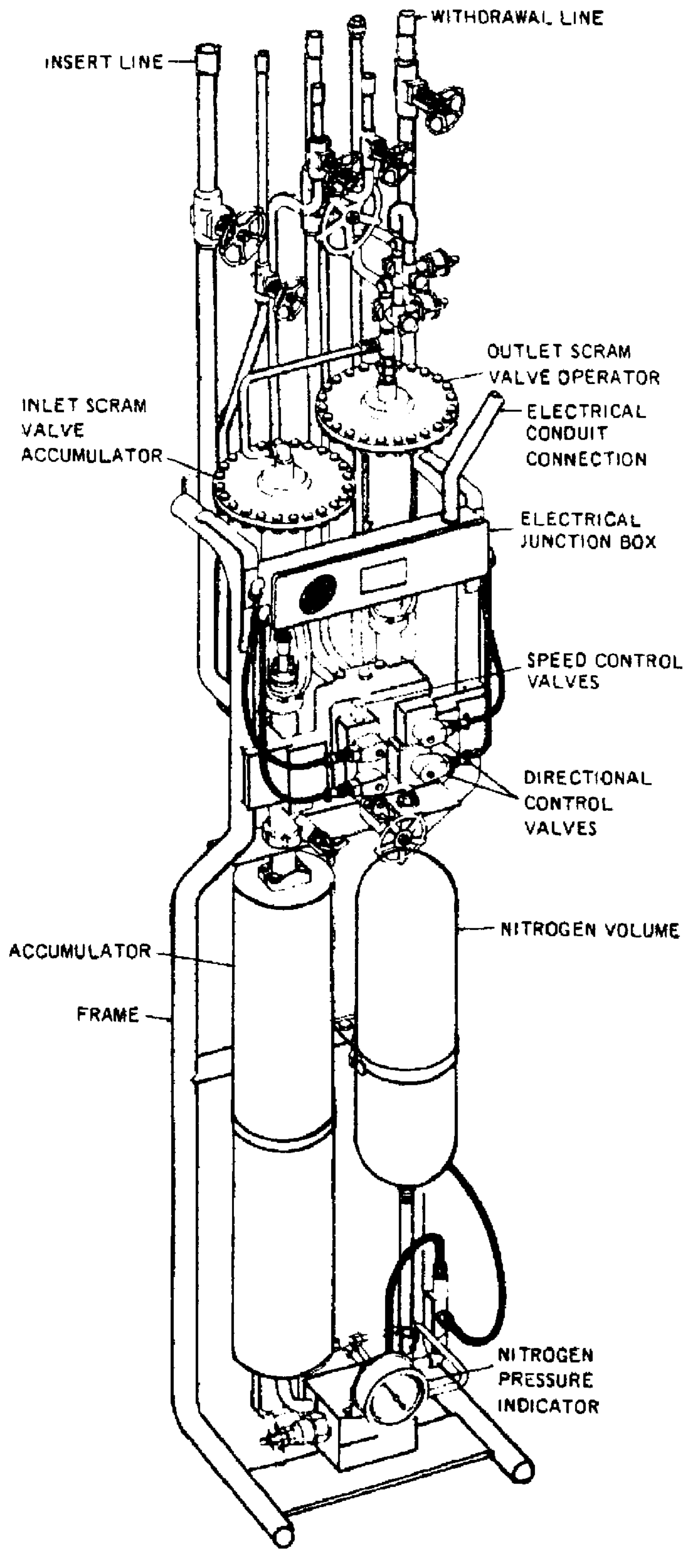
UNIT 1 AND UNIT 2

CONTROL ROD DRIVE UNIT
(CUTAWAY)

FIGURE 4.2-13



ACAD 040214

REV 19 7/01

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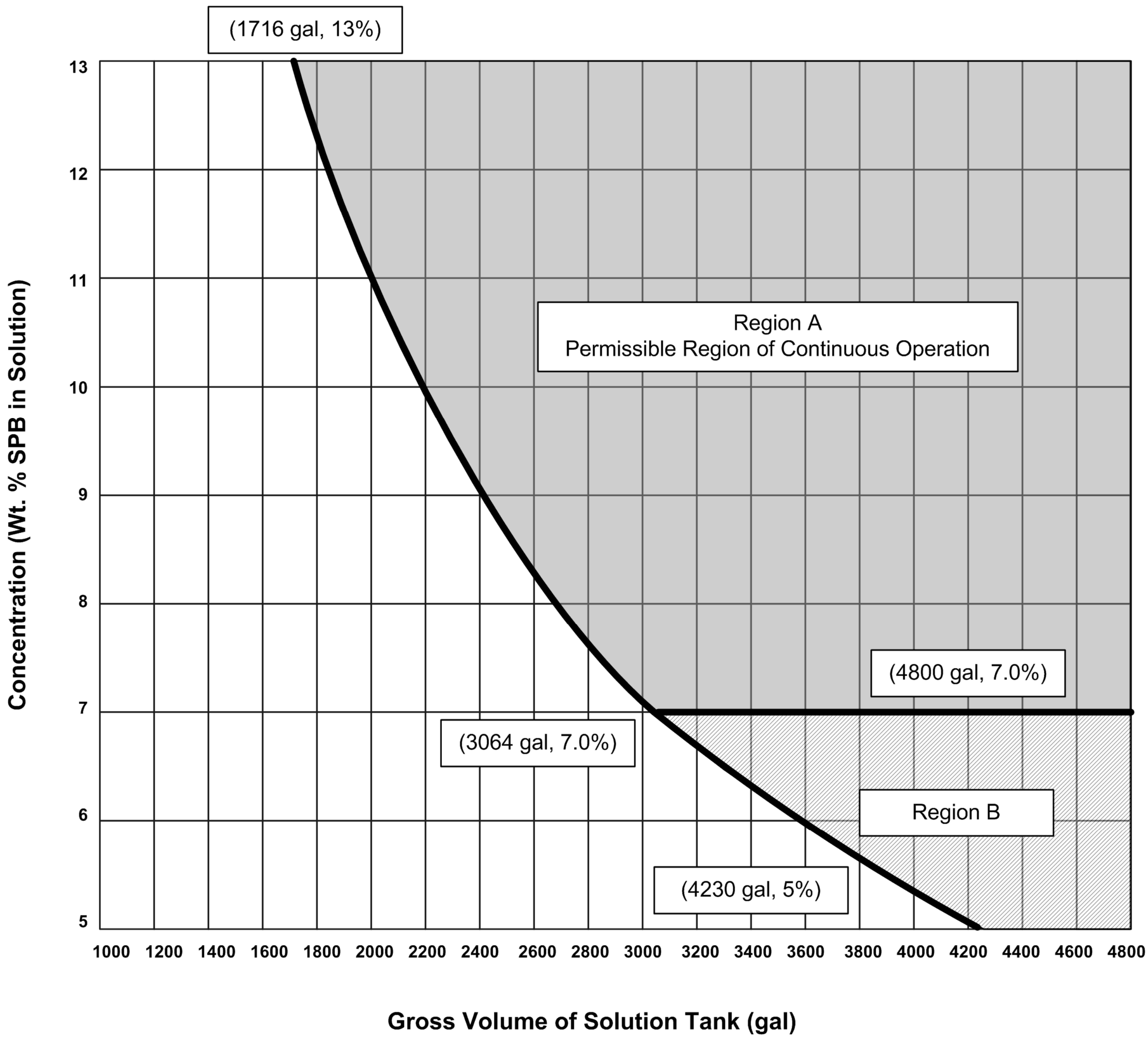
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

CONTROL ROD DRIVE
HYDRAULIC CONTROL UNIT

FIGURE 4.2-14



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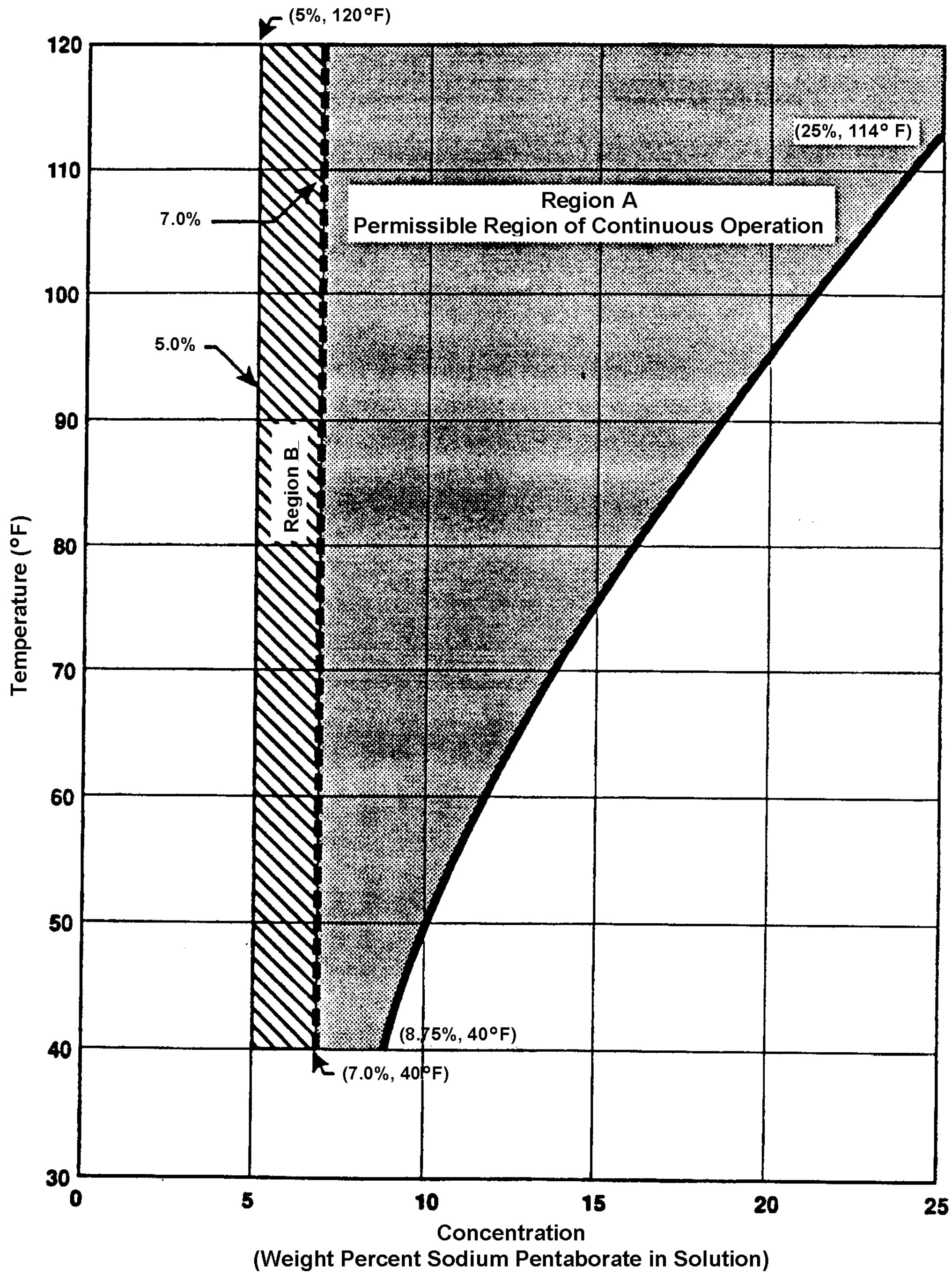
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

SODIUM PENTABORATE SOLUTION VOLUME-CONCENTRATION REQUIREMENTS

FIGURE 4.2-15



ACAD 2040216

REV 26 9/08

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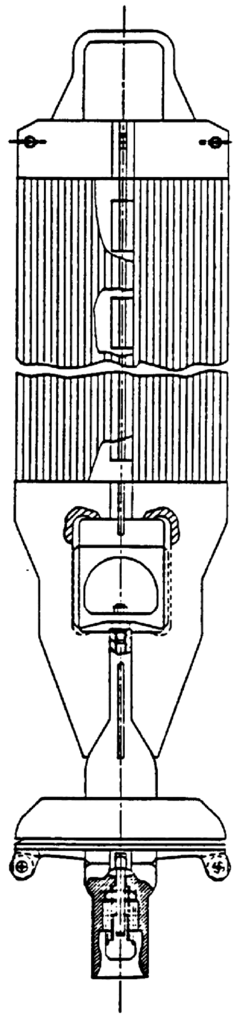
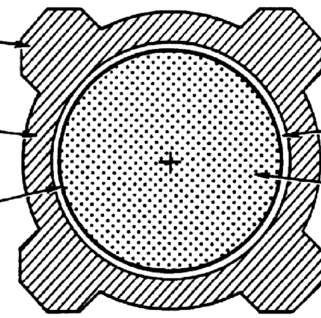
EDWIN I. HATCH NUCLEAR PLANT
UNIT 1 AND UNIT 2

SODIUM PENTABORATE SOLUTION TEMPERATURE VERSUS CONCENTRATION REQUIREMENTS

FIGURE 4.2-16





REV 24 9/06

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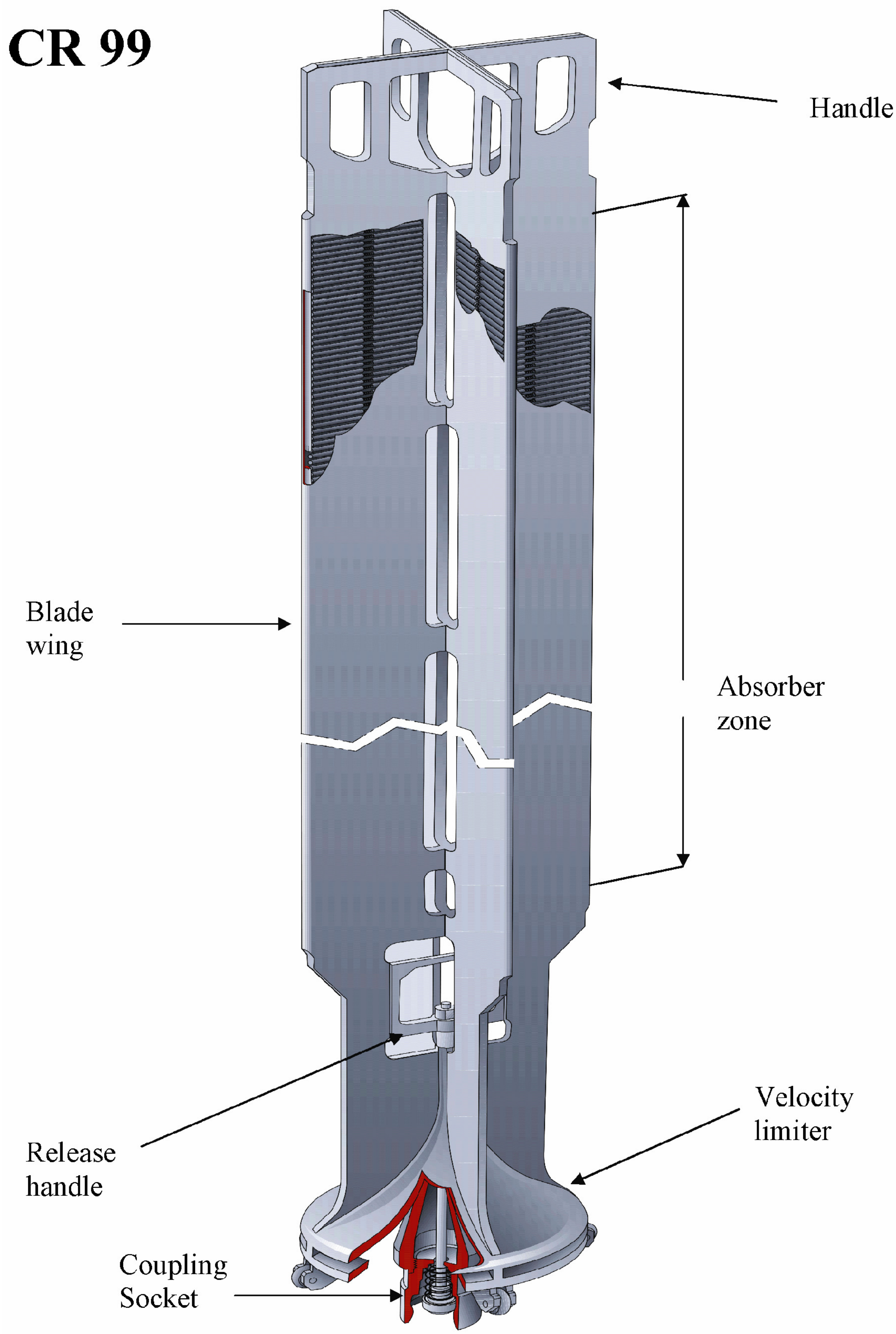
UNIT 1 AND UNIT 2

GE MARATHON CONTROL ROD ASSEMBLY

FIGURE 4.2-17



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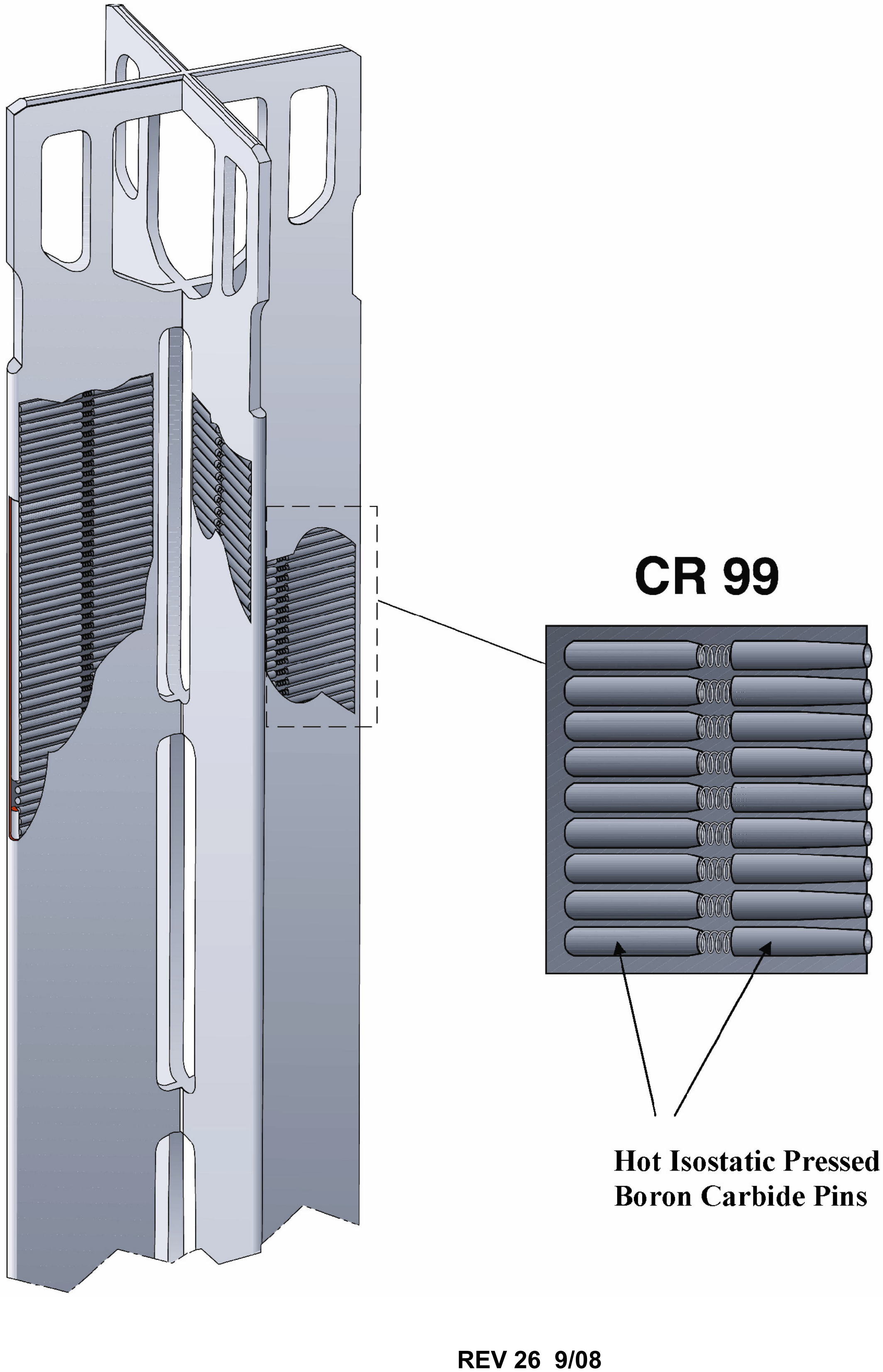
EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

WESTINGHOUSE CR 99
CONTROL ROD ASSEMBLY

FIGURE 4.2-18



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EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

WESTINGHOUSE CR 99
BORON CARBIDE PIN DETAIL

FIGURE 4.2-19

4.3 NUCLEAR DESIGN (HNP-1 AND HNP-2)

The nuclear design discussed in this section is applicable to both HNP-1 and HNP-2, unless specified otherwise.

The description of fuel assemblies in the following sections pertains to fuel bundles supplied by Global Nuclear Fuel (GNF) and described in GESTAR II. In addition, four SVEA-96 Optima2 lead use assemblies (LUAs) supplied by Westinghouse Electric Company have been installed in the HNP Unit 1 core. The thermal-hydraulic design description and documentation of methods for these LUAs are contained in references 1 and 2.

4.3.1 DESIGN BASES

Reference section 3.1 of ***NEDE-24011-P-A, "GESTAR II - General Electric Standard Application for Reactor Fuel" (incorporated by reference into the FSAR)***.

4.3.1.1 Safety Design Bases

Reference section 3.1 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.1.1.1 Reactivity Bases

Reference section 3.1.1 of ***NEDE-24011-P-A (GESTAR II).***

4.3.1.1.2 Overpower Bases

Reference section 3.1.2 of ***NEDE-24011-P-A (GESTAR II).***

4.3.2 DESCRIPTION

The general description of the reactor core is contained in section 3.2 of ***NEDE-24011-P-A (GESTAR II).***

4.3.2.1 Nuclear Design Description

The nuclear design description is contained in section 3.2.1 of ***NEDE-24011-P-A (GESTAR II)***. The cycle-specific loading patterns are contained in the appropriate supplemental reload licensing reports listed in table 15.1-1.

4.3.2.2 Power Distribution

Reference section 3.2.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.2.1 Power Distribution Measurement

Reference section 3.2.2.1 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.2.2 Power Distribution Accuracy

Reference section 3.2.2.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.2.3 Power Distribution Anomalies

Reference section 3.2.2.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.2.4 Power Distribution Calculations

Prior to the start of each operating cycle, a 3-D simulator model is used to calculate local, radial, and axial power distributions at various exposures throughout the cycle to assess the effect of different control rod patterns on the margin to power distribution limits stated in the ***Core Operating Limits Report (COLR) (incorporated by reference into the FSAR)***. Since differences between the projected operation of a cycle and what is actually achieved always exist, the same 3-D simulator model is used periodically throughout the cycle to recalculate power distributions based upon the actual operating history. The results are used to determine whether the original rod pattern recommendations require revision.

4.3.2.3 Reactivity Coefficients

Reference section 3.2.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.3.1 Doppler Reactivity Coefficient

Reference section 3.2.3.1 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.3.2 Moderator Void Coefficient

Reference section 3.2.3.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.4 Control Requirements

Reference section 3.2.4 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.4.1 Shutdown Reactivity

Shutdown reactivity for power distribution is discussed in section 3.2.4.1 of ***NEDE-24011-P-A (GESTAR II)***. Also, prior to performing the shutdown margin demonstration test at the beginning of each cycle, the following additional design margin is adopted:

k_{eff} < 0.99% with the highest-worth rod fully withdrawn

4.3.2.4.2 Reactivity Variations

Reference section 3.2.4.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.5 Control Rod Patterns and Reactivity Worth

Below the low-power setpoint, control rod movements are constrained to the banked position withdrawal sequence (BPWS) rod patterns. The constraints prevent the development of high rod-worth patterns to limit the consequences of the control rod drop accident discussed in section 15.3.2.

The range of travel of the control rods is 144 in., which corresponds to the bottom twenty-four 6-in. nodes of the active fuel length.

4.3.2.6 Criticality of Reactor During Refueling

Criticality during refueling is discussed in section 3.2.5 of ***NEDE-24011-P-A (GESTAR II)***. Subsection 7.6.1 contains a description of the reactor design features that prevent criticality during refueling.

4.3.2.7 Stability

4.3.2.7.1 Xenon Transients

Reference section 3.2.6.1 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.2.7.2 Thermal-Hydraulic Stability

Reference paragraph 4.4.4.5.1.

4.3.2.8 <u>Vessel Irradiation</u>

A. HNP-1

Vessel irradiation for HNP-1 is described in HNP-1-FSAR appendix R, sections R.4 and R.5.

B. HNP-2

The lead factor was calculated using the one- and two-dimensional, discrete ordinates transport codes described in paragraph 4.1.4.4. The discrete ordinates calculations were performed in cylindrical geometry with fission neutron source density distributions as input. The geometry described seven regions with the core modeled as two homogenized regions. In each calculation, the source density distribution, the total source, and the coolant density in each of the two core regions were specified to be appropriate for the elevation under consideration. The two-dimensional calculation model was based on the reactor midplane elevation. One-dimensional calculations were performed for a number of additional elevations. The coolant water region between the core and the shroud contained saturated water. The region between the shroud and vessel was assumed to be filled with subcooled water. The presence of the jet pumps was ignored. The material compositions for the stainless-steel shroud and the carbon-steel vessel contained the mixtures by weight as specified in the ASME material specifications for ASME SA240, 304L, and ASME SA533 Grade B.

The source distribution, which can be separated in space and energy, was obtained from the incremental fuel exposures by axial fuel node and bundle for a typical cycle and from the fission neutron energy spectrum. The integral of source density over space and energy in the core region was normalized to the total fission neutron source rate in the region. In these calculations, the core region was treated as a 1-cm-thick cross-section of the core with no transverse leakage.

Dosimetry located on the inside surface of the vessel was removed with the first surveillance capsule and tested to determine the flux at that location. The lead factor, relating the dosimeter location to the peak location, was used to calculate the peak vessel inside surface flux. Assuming an 80% capacity factor or 32 effective full power years (EFPYs) in 40 years of operation, the fluence for this operating period was estimated. The measured dosimeter flux and calculated peak flux and fluence are shown in HNP-2-FSAR table 4.3-1.

4.3.3 ANALYTICAL METHODS

Reference section 3.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.3.4 FINAL LOADING PATTERN

The final loading pattern must meet the requirements described in section 3.4 of ***NEDE-24011-P-A, (GESTAR II)***.

<u>DOCUMENTS INCORPORATED BY REFERENCE INTO THE FSAR</u>

"GESTAR II - General Electric Standard Application for Reactor Fuel," <u>NEDE-24011-P-A</u>.

Unit 1 and Unit 2 Core Operating Limits Reports (located in each unit's Technical Requirements Manual, Appendix A).

REFERENCES

1. CENPD-300-P-A, "Reference Safety Report for Boiling Water Reactor Fuel," July 1996.

2. Westinghouse Report NF-BSN-10-10, "Supplemental Licensing Report, SVEA-96 Optima2 Lead Use Fuel Assemblies for Edwin I. Hatch Nuclear Plant, Unit 1," Revision 0, February 2010.

TABLE 4.3-1

ESTIMATED DOSIMETER AND VESSEL PEAK FLUX AND FLUENCE (HNP-2)

Time at Power	
EOC 8	*6.58 EFPYs (2.08 x 10^8 s)*
32 EFPYs	*1.01 x 10^9 s*
Lead Factors	
Inside surface (ID)	*0.79*
Dosimeter Flux (n/cm^2-s)	*1.12 x 10^8*
Fluence (n/cm^2)	
EOC 8 dosimeter	*2.3 x 10^{17}*
32 EFPYs peak ID	*1.4 x 10^{18}*

LEGEND:

EOC = end of cycle
ID = inside diameter
EFPYs = effective full power years

4.4 THERMAL AND HYDRAULIC DESIGN (HNP-1 AND HNP-2)

The information provided in this section is applicable to both HNP-1 and HNP-2, unless specified otherwise.

The description of fuel assemblies in the following sections pertains to fuel bundles supplied by Global Nuclear Fuel (GNF) and described in GESTAR II. In addition, four SVEA-96 Optima2 lead use assemblies (LUAs) supplied by Westinghouse Electric Company have been installed in the HNP Unit 1 core. The thermal-hydraulic design description for these LUAs is contained in reference 1.

4.4.1 DESIGN BASES

Design bases information is found in ***NEDE-24011-P-A, "GESTAR II - General Electric Standard Application for Reactor Fuel" (incorporated by reference into the FSAR)***. Additionally, design basis information is found in the lead-plant report "Hatch 2 Lead Assembly Compatibility Report: Mechanical, Thermal and Neutronic Design for ANF 9x9 Lead Assemblies," ANF-87-77(P), which is applicable to both HNP-1 and HNP-2.

4.4.1.1 Safety Design Bases

Reference section 4.1.1 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.1.2 Power Generation Design Bases

Reference section 4.1.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.1.3 Requirements for Steady-State Conditions

Reference section 4.1.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.1.4 Requirements for Anticipated Operational Occurrences (AOOs

Reference section 4.1.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.1.5 Summary of Design Bases

Reference section 4.1.4 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2 DESCRIPTION OF THERMAL-HYDRAULIC DESIGN OF REACTOR CORE

4.4.2.1 Critical Power Ratio

Reference sections 4.2.1 and 4.3.1 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.2 Average Planar Linear Heat Generation Rate (APLHGR

Reference section 4.2.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.3 Core Coolant Flow Distribution and Orificing Pattern

Reference section 4.2.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.4 Void Fraction Distribution

The distribution of void fractions in individual fuel assemblies and the core is a complex function of power level, fuel design, and rod pattern. Likewise, the bundle exit and the core steam quality depends upon the same factors. Void fraction distribution and steam quality are calculated by a 3-D core simulator code that incorporates thermal-hydraulic and neutronic feedback models.

4.4.2.5 Core Pressure Drop and Hydraulic Loads

Reference section 4.2.4 of ***NEDE-24011-P-A (GESTAR II).***

4.4.2.5.1 Friction Pressure Drop

Reference section 4.2.4.1 of ***NEDE-24011-P-A (GESTAR II).***

4.4.2.5.2 Local Pressure Drop

Reference section 4.2.4.2 of ***NEDE-24011-P-A (GESTAR II).***

4.4.2.5.3 Elevation Pressure Drop

Reference section 4.2.4.3 of ***NEDE-24011-P-A (GESTAR II).***

4.4.2.5.4 Acceleration Pressure Drop

Reference section 4.2.4.4 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.6 <u>Correlation and Physical Data</u>

Reference section 4.2.5 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.6.1 Pressure Drop Correlation

Reference section 4.2.5.1 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.6.2 Void Fraction Correlation

Reference section 4.2.5.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.6.3 Heat Transfer Correlation

Reference section 4.2.5.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.7 <u>Thermal Effect of AOOs</u>

Reference section 4.2.6 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.8 <u>Uncertainties in Estimates</u>

Reference section 4.2.7 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.9 <u>Flux Tilt Considerations</u>

Reference section 4.2.8 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.10 <u>Thermal-Hydraulic Uncertainties</u>

Reference sections 4.2 and 4.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.2.11 Gross Power Tilt Considerations

Reference section 3.2.2.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.3 DESCRIPTION OF THERMAL AND HYDRAULIC DESIGN OF REACTOR COOLANT SYSTEM

4.4.3.1 Plant Configuration Data

Table 4.4-1 provides the flow path length, height, liquid level, minimum elevations, and minimum flow areas for each major flow path volume within the reactor pressure vessel (RPV) and recirculation loops of the RCS.

4.4.3.2 Operating Restrictions on Pumps

A. Pump Characteristics

Limitations on pump performance are discussed in section S.5.2.1 of ***NEDE-24011-P-A (GESTAR II)***.

B. Performance Range for Normal Operation

A boiling water reactor (BWR) must operate with certain restrictions because of pump NPSH, overall plant control characteristics, and core thermal power limits.

Paragraph 4.4.3.3, together with the power-to-flow map (figure 15.1-3), describes the region where the plant may normally operate. This region is bounded by the considerations stated. Minimum power at high-forced circulation (bottom of map) is bounded to protect recirculation loop components from cavitation. Interlocks are provided to prevent operation below this bound. Maximum power is bounded for thermal margin considerations and protected by rod block and scram lines.

4.4.3.3 Power-to-Flow Map

As shown on figure 15.1-3, the licensed (analyzed) region of power operation is limited by the following power and flow relationships:

Core Flow (% of RTP)	Maximum Core Power (% of RTP)
0 to 10	24
10 to ~ 92.9	Maximum extended load line limit (MELL) (~ 104.9% rod line)
~ 92.9 to 105	100
105 to 110	decreasing to 69

4.4.4 EVALUATION

Reference section 4.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.4.1 Critical Power

Reference section 4.3.1 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.4.1.1 Fuel Cladding Integrity Safety Limit

Reference section 4.3.1.1 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.4.1.2 Operating Limit Minimum Critical Power Ratio Calculational Procedures (OLMCPR)

Reference section 4.3.1.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.4.2 Core Hydraulics

Reference section 4.3.2 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.4.3 Influence of Power Distribution

Reference section 4.3.3 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.4.4 Core Thermal Response

Reference section 4.3.4 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.4.5 Analytical Methods

Reference section 4.3.5 of ***NEDE-24011-P-A (GESTAR II)***.

4.4.4.5.1 Thermal-Hydraulic Stability Analysis

Thermal-hydraulic instabilities are unlikely to occur during normal power operations; however, some BWRs have experienced instabilities while operating at high power with little or no forced circulation. General Design Criterion (GDC) 12 of Title 10 Code of Federal Regulations (CFR) Part 50, Appendix A, requires that the reactor core and associated coolant, control, and protection systems be designed to ensure such power oscillations are not possible or can be reliable and readily detected and suppressed prior to the violation of the safety limit MCPR (SLMCPR) by the oscillation power range monitor (OPRM). (Reference HNP-1-FSAR subsection 7.5.10 and HNP-2-FSAR paragraph 7.6.2.2.7.)

The thermal-hydraulic stability analysis performed for reloads is described in HNP-2-FSAR subsection 15.4.1.

4.4.4.5.2 Power Test Program

Core performance is evaluated at or near rated temperature and pressure. Evaluations include a reactor heat balance at rated temperature. Local power range monitor (LPRM) calibrations, which include use of the flux mapping and calibration system, are made. Each LPRM is calibrated to read in terms of the local fuel rod surface heat flux. Axial power distribution is measured with the traversing incore probe (TIP) system after significant changes in power, control rod pattern, or flowrate. Core void distribution is determined by calculation at several node points.

Additional tests include measuring response to changes in reactor setpoints, flow control tests to substantiate load-following characteristics, and measuring core plate pressure drop.

The stability of the nuclear system is verified during startup testing by introducing the same near-step perturbations that were used during the analytical simulation. Compliance with the ultimate performance limit is demonstrated at selected responsive plant conditions by the absence of divergent or limit-cycle oscillations excluding those minor limit cycles that can be induced by controller deadband characteristics.

The detailed core power and critical power ratio (CPR) distributions are calculated periodically. The unit is operated as necessary to maintain MCPR and the maximum linear heat generation rate within the operating limits for the plant.

4.4.5 INSTRUMENTATION REQUIREMENTS

The RPV instrumentation monitors the key operating parameters during planned operation to ensure sufficient parameter control. The following RPV sensors are discussed in HNP-1-FSAR subsection 7.8.5 and HNP-2-FSAR subsection 7.6.7:

- Temperature (function of coolant temperature).
- Water level.
- Coolant flowrates and differential pressures.
- Internal pressure.

The nuclear incore monitoring system is discussed in HNP-1-FSAR section 7.5 and HNP-2-FSAR subsection 7.6.2.

<u>DOCUMENTS INCORPORATED BY REFERENCE INTO THE FSAR</u>

"GESTAR II - General Electric Standard Application for Reactor Fuel," <u>NEDE-24011-P-A</u>.

REFERENCE

1. CENPD-300-P-A, “Reference Safety Report for Boiling Water Reactor Fuel,” July 1996.

TABLE 4.4-1

PLANT CONFIGURATION DATA
(HNP-1 AND HNP-2)

	Flow Path Length	Height & Liquid Level (in.)	Elevation of Bottom of Each Volume[a] (in.)	Minimum Flow Areas (ft^2)
Lower plenum	209 in.	209 209	-161	80.0
Core	164 in.	164 164	47	106.0 (includes bypass)
Upper plenum and separators	179 in.	179 179	211	33.0
Dome (above normal water level)	280 in.	280 0	390	264.0
Downcomer area	328 in.	328 328	-52	91.0
Recirculation loops & jet pumps (one loop)	136 ft	497 497	-480	90.8

a. Reference point is recirculation nozzle outlet.

4.5 CONTROL ROD DRIVE HOUSING SUPPORTS (HNP-1 AND HNP-2)

The information provided in this section is applicable to both HNP-1 and HNP-2, unless specified otherwise.

4.5.1 SAFETY DESIGN BASES

The CRD housing supports meet the following safety design bases:

A. Following a postulated CRD housing failure, control rod downward motion is limited so that any resulting nuclear transient could not be sufficient to cause fuel damage.

B. The clearance between the CRD housings and the supports is sufficient to prevent vertical contact stresses caused by thermal expansion during plant operation.

4.5.2 DESCRIPTION

The CRD housing supports are shown in figure 4.5-1. Horizontal beams are installed immediately below the bottom head of the RPV, between the rows of CRD housings. The beams are bolted to brackets welded to the inner steel ring of the drive room in the reactor support pedestal.

Hanger rods, ~ 10 ft long and 1 3/4 in. in diameter, are supported from the beams on stacks of disk springs, which compress ~ 2 in. under the design load.

The support bars are bolted between the bottom ends of the hanger rods. The spring pivots at the top and the beveled, loose-fitting ends on the support bars prevent substantial bending movement in the hanger rods if the support bars are overloaded.

Individual grids rest on the support bars between adjacent beams. Because a single-piece grid is difficult to handle in the limited work space and because it is necessary that CRD position indicators and incore instrumentation components be accessible for inspection and maintenance, each grid is designed for in-place assembly or disassembly. Each grid assembly is made up of two grid plates, a clamp, and a bolt. The top part of the clamp guides the grid to its correct position directly below the respective CRD housing that it would support in the postulated accident.

When the support bars and grids are installed, a gap of ~ 1 in. at room temperature (~ 70°F) is provided between the grid and the bottom contact surface of the CRD flange.

During system heatup, this gap is reduced by a net downward expansion of the housings with respect to the supports. In the hot operating condition, the gap is ~ 1/4 in.

In the postulated CRD housing failure, the CRD housing supports are loaded when the lower contact surface of the CRD flange contacts the grid. The resulting load is then carried by two grid plates, two support bars, four hanger rods, their disk springs, and two adjacent beams.

The American Institute of Steel Construction (AISC) Manual of Steel Construction, "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," was used in designing the CRD housing support system. However, to provide structure that absorbs as much energy as practical without yielding, the allowable tension and bending stresses used were 90% of yield, and shear stress used was 60% of yield. These design stresses are 1.5 times the AISC allowable stresses (60% and 40% of yield, respectively).

For purposes of mechanical design, the postulated failure resulting in the highest forces is an instantaneous circumferential separation of the CRD housing from the RPV, with an internal pressure of 1250 psig (RPV design pressure) acting on the area of the separated housing. The weight of the separated housing, CRD, and blade, plus the pressure of 1250 psig acting on the area of the separated housing, gives a force of ~ 35,000 lb. This force is multiplied by a factor of 3 for impact, conservatively assuming that the housing travels through a 1-in. gap before it contacts the supports. The total force (105,000 lb) is the treated as a static load in design.

Except for the following items, all CRD housing support subassemblies are fabricated of American Society of Testing Materials (ASTM) A 36 structural steel:

- Grid - ASTM A 441.

- Disc springs - Schnorr, Type BS-125-71-8.

- Hex bolts and nuts - ASTM A 307.

4.5.3 SAFETY EVALUATION

For design purposes, the postulated failure resulting from an instantaneous circumferential separation of the CRD housing from the RPV, with an internal pressure of 1250 psig (RPV design pressure) acting on the area of the separated housing is the governing design condition. The vertical force (dead load) of the separated housing, CRD, and blade, plus the force of 1250-psig pressure acting on the area of the separated housing multiplied by an impact factor of three gives the design static load on the CRD housing support members. The effect of an earthquake on the design load is not considered in the design because the earthquake load is only 3% of the design load.

Downward travel of the CRD housing and its control rod following the postulated housing failure equals the sum of the following distances:

- The compression of the disk springs under dynamic loading.

- The initial gap between the grid and the bottom contact surface of the CRD flange.

If the reactor is cold and pressurized, the downward motion of the control rod is limited to the spring compression (~ 2 in.), plus a gap of ~ 1 in. If the reactor is hot and pressurized, the gap is ~ 1/4 in., and the spring compression is slightly less than in the cold condition. In either case, the control rod movement following a housing failure is substantially limited below one drive

notch movement (6 in.). Sudden withdrawal of any control rod through a distance of one drive notch at any position in the core does not produce a transient sufficient to damage any radioactive material barrier.

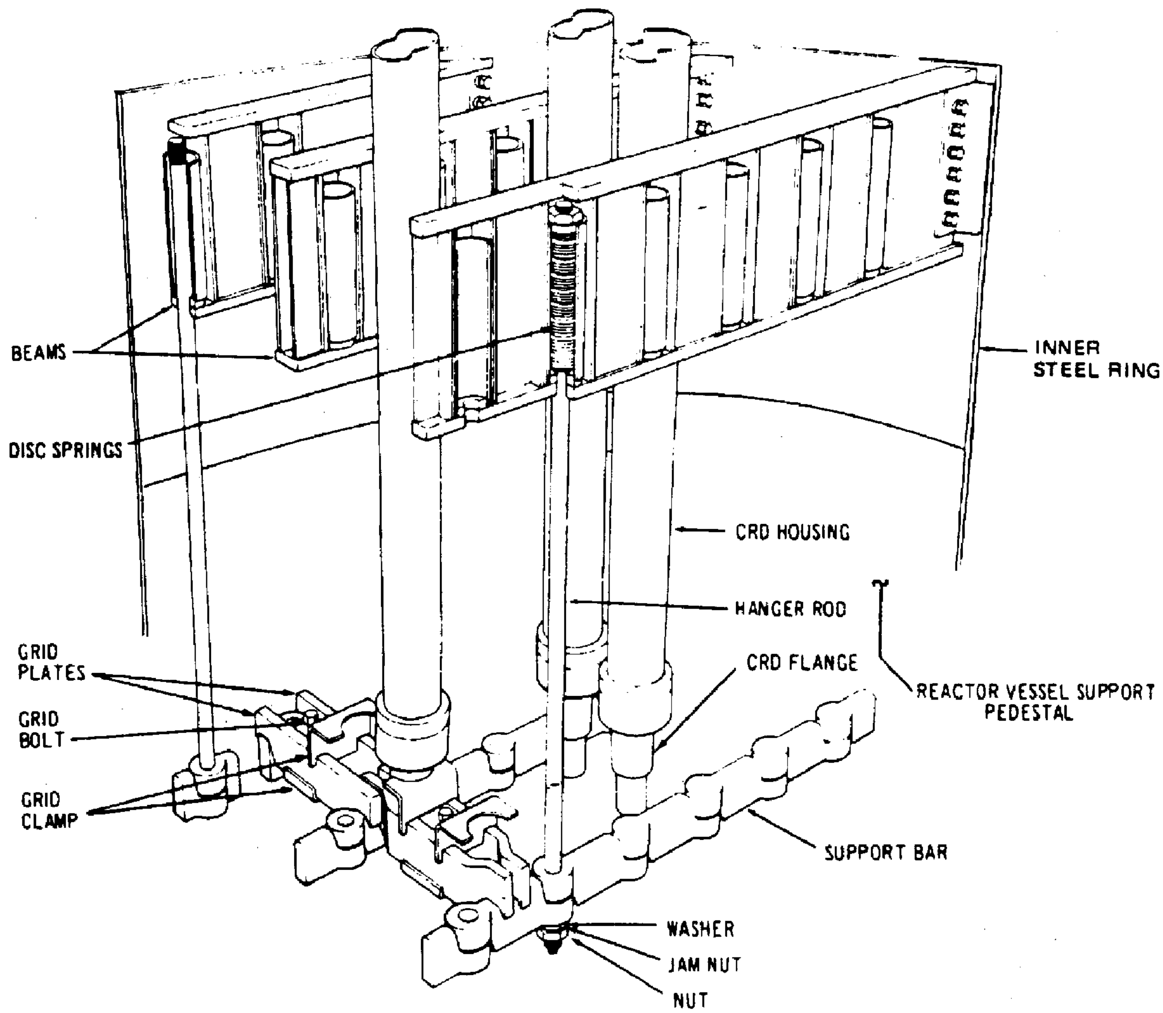
The stress criterion (1.5 times the AISC-allowable stresses) is considered desirable for this application and adequate for the once-in-a-lifetime loading condition. The effect of stress raisers in the structural support members is considered by designing the actual stress in areas of stress concentration to be less than the AISC-allowable flexural stresses at 60% of yield.

The CRD housing supports are in place during power operation and when the nuclear system is pressurized. If a control rod is ejected during shutdown, the reactor remains subcritical because it is designed to remain subcritical with any one control rod fully withdrawn at any time.

At plant operating temperature, a gap of ~ 1/4 in. exists between the CRD housing and the supports. At lower temperatures the gap is greater. Because the supports do not contact any of the CRD housing, except during the postulated accident condition, vertical contact stresses are prevented.

4.5.4 INSPECTION AND TESTING

CRD housing supports are removed for inspection and maintenance of the CRDs. The supports for one control rod can be removed during reactor shutdown, even when the reactor is pressurized, because all control rods are then inserted. When the support structure is reinstalled, it is inspected for correct assembly with particular attention to maintaining the correct gap between the CRD flange lower contact surface and the grid.



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UNIT 1 & UNIT 2

CONTROL ROD DRIVE
HOUSING SUPPORT

FIGURE 4.5-1

SUPPLEMENT 4A

INITIAL CORE
(HNP-1 AND HNP-2)

4A.1 GENERAL

The Edwin I. Hatch Nuclear Plant (HNP) Unit 1 (HNP-1) and Unit 2 (HNP-2) initial core fuel was irradiated in the initial core and subsequent reloads. The initial core no longer resides in the core, but does reside onsite. This supplement provides information consistent with its current status.

4A.2 FUEL CONFIGURATION

The initial core fuel configuration for HNP-1 and HNP-2 is described in the following paragraphs.

A typical cross-section of a fuel assembly for HNP-1 and HNP-2 is shown in figures 4A-1 and 4A-2, respectively. A summary of the core fuel assembly data for HNP-1 and HNP-2 initial core is provided in tables 4A-1 and 4A-2, respectively. The initial core fuel for HNP-1 was 7x7 matrix while HNP-2 was an 8x8 matrix with 2 water rods.

4A.3 NUCLEAR DESIGN

Each HNP-1 fuel assembly contains 49 fuel rods which are spaced and supported in a square (7x7) array by the lower and upper tie plates. End fittings were designed so that it was not mechanically possible to complete the assembly of a fuel element with any high enrichment rods in positions specified to receive a lower enrichment. Gadolinia bearing pellets were used for some of the highest enrichment rods. The gadolinia-uranium fuel rods were designed with characteristic extended end plugs for each rod type. The extended end plug permitted a positive visual check on the location of each gadolinia-uranium rod after assembly. The bundle average enrichment was 1.0.

Each HNP-2 fuel assembly contains 62 fuel rods and 2 water rods. The water rods have a slightly larger diameter than the fuel rods. The enrichment distribution in the high- and medium-enrichment bundles is designed to meet the design bases. Gadolinia in the form of Gd_2O_3 is selectively placed in fuel rods in the high- and medium-enrichment bundles to provide reactivity control and is distributed axially to flatten the axial power distribution. The reactivity variations of the high- and medium-enrichment bundles are designed to complement each other. The low-enrichment bundle is composed entirely of natural uranium rods. The bundle average enrichments, including the natural uranium at the top and bottom, are 2.21 and 1.76, respectively. These three bundle types combine for a core enrichment of 1.87. The natural uranium bundles do not contain gadolinia rods.

TABLE 4A-1

HNP-1 INITIAL CORE FUEL

	Value
Fuel Assembly Data	
Overall length (in.)	*175.83*
Nominal active fuel length (in.)	*144.00*
Fuel rod pitch (in.)	*0.738*
Space between fuel rods (in.)	*0.175*
Fuel channel wall thickness (in.)	*0.80*
Fuel bundle heat transfer area (ft^2)	*86.513*
Fuel Rod Data	
Outside diameter (in.)	*0.563*
Cladding thickness (in.)	*0.037*
Pellet outside diameter (in.)	*0.477*
Fission gas plenum length (in.)	*16.00*
Pellet immersion density (g/cc)	*10.420*

TABLE 4A-2 (SHEET 1 OF 2)

HNP-2 INITIAL CORE FUEL

	Value
Fuel Assembly Data	
Overall length (in.)	*176.16*
Nominal active fuel length (in.)	*150.0*
Fuel rod pitch (in.)	*0.640*
Space between fuel rods (in.)	*0.157*
Fuel channel wall thickness (in.)	*0.100 or 0.080*
Fuel bundle heat transfer area (ft^2)	*98.0*
Channel width (inside) (in.)	*5.278*
Fuel Rod Data	
Outside diameter (in.)	*0.483*
Cladding inside diameter (in.)	*0.419*
Cladding thickness (in.)	*0.032*
Fission gas plenum length (in.)	*10.0*
Pellet immersion density (% TD)	*95.000*
Pellet outside diameter (in.)	*0.410*
Pellet length (in.)	*0.410*
Water Rod Data	
Outside diameter (in.)	*0.591*
Inside diameter (in.)	*0.531*

TABLE 4A-2 (SHEET 2 OF 2)

Zircaloy-2 Cladding

Thermal conductivity $T = (600 - 800°F)$
$k = 9 - 10$ *(Btu/h-ft-°F)*

Coefficient of linear thermal expansion:

$$\frac{1}{L_o}\left(\frac{\Delta L}{\Delta T}\right) \sim 3 \times 10^{-6} \left(F^{-1}\right)$$

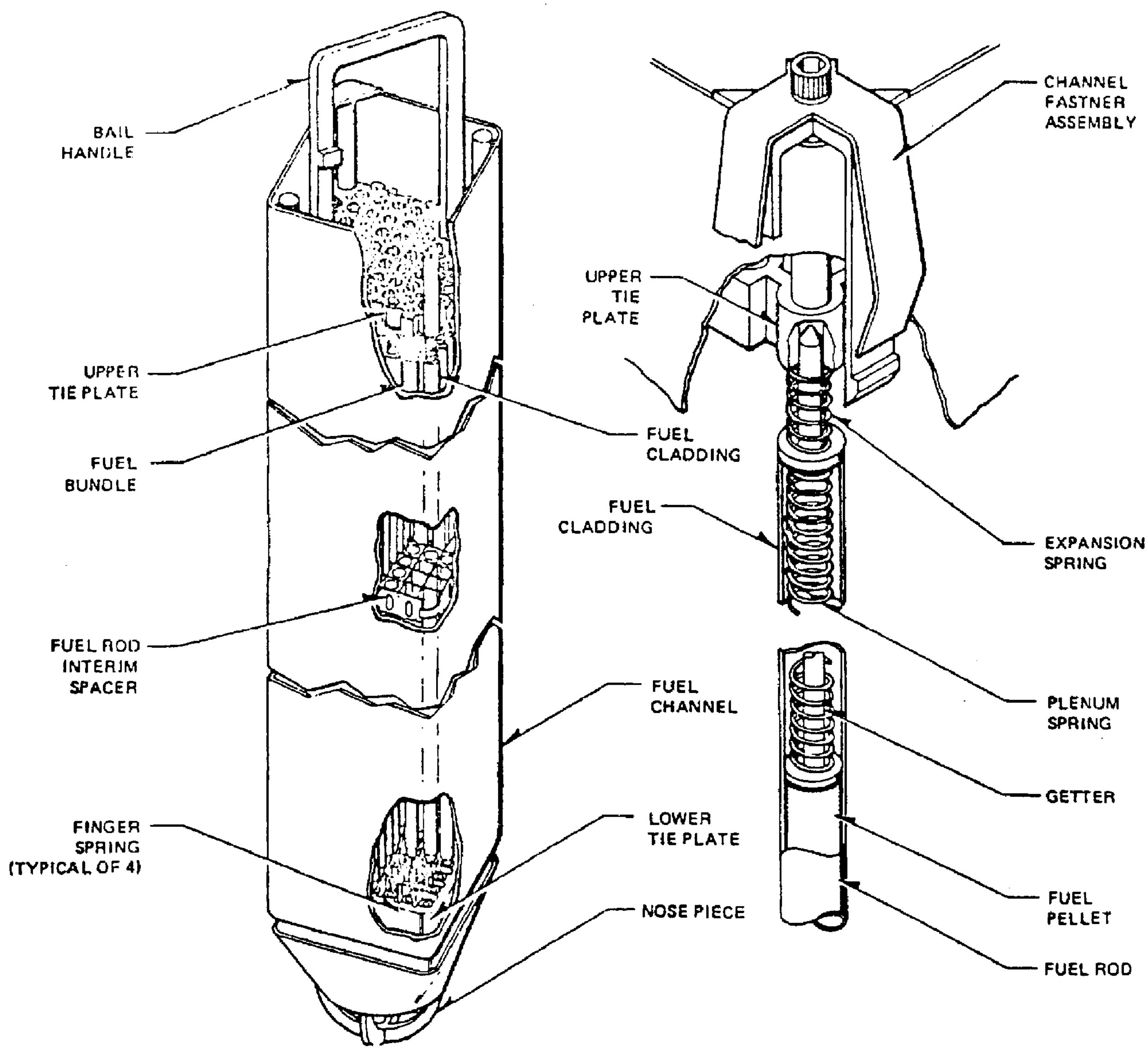
Total elongation (irradiated) $\geq 1\%$

UO_2 Pellets

$$\text{Thermal conductivity} = \left(\frac{3978.1}{692.61 + T}\right) + 6.02366 \times 10^{-12} (T + 460)^3 \text{ (Btu/h-ft-°F)}$$

Melting temperature $= 5080 - 63.5 \times 10^{-4} E$ *(°F)*

where: E *= exposure MWd/T*



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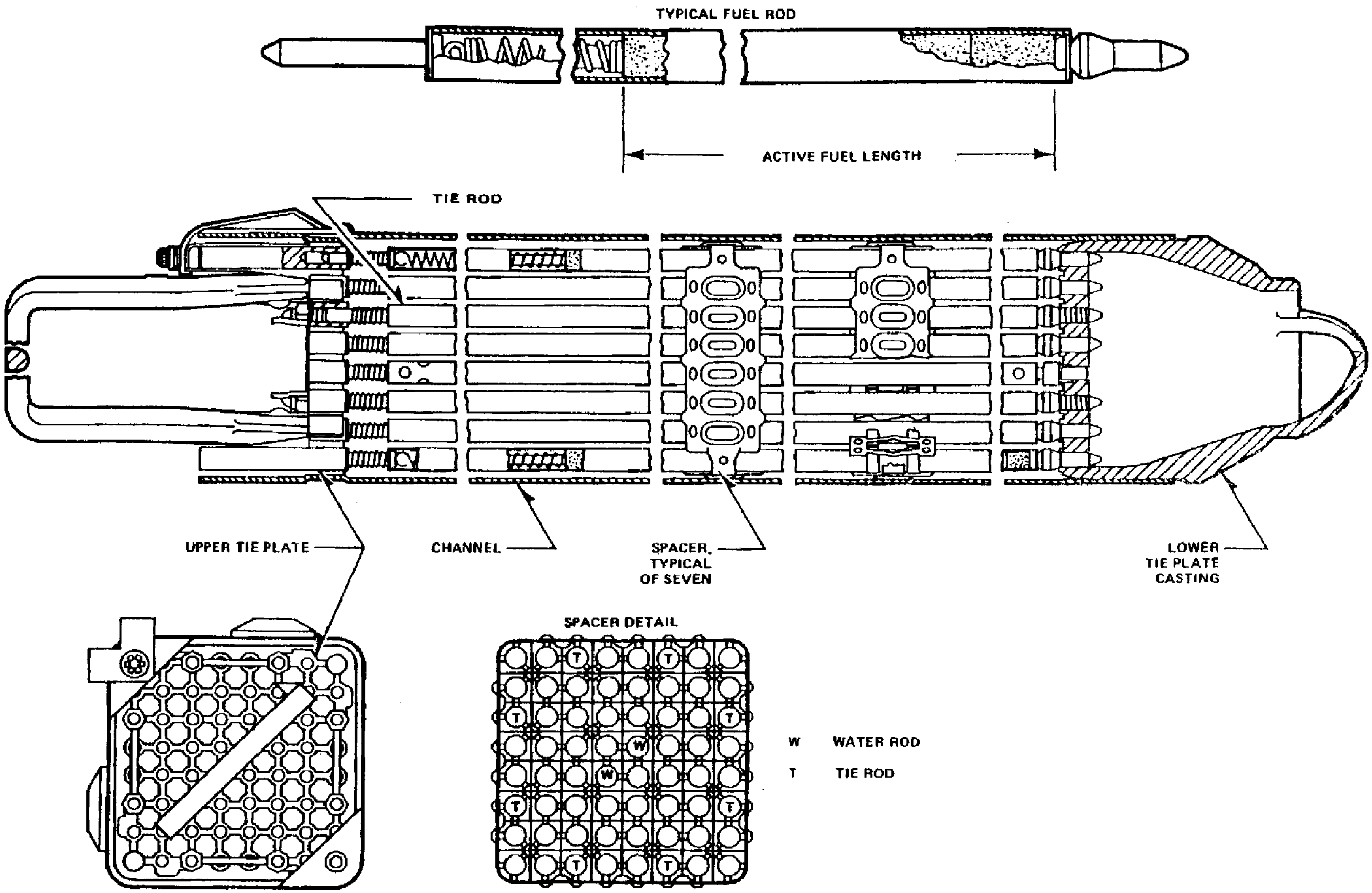
HNP-1 FUEL ASSEMBLY

FIGURE 4A-1



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EDWIN I. HATCH NUCLEAR PLANT
UNIT 2

HNP-2 FUEL ASSEMBLY

FIGURE 4A-2