APPENDIX E

Report of Environmental Studies
Geology, Hydrology and Seismology
Proposed Nuclear Power Plant
Prairie Island Site
Near Red Wing, Minnesota
for Northern States Power Company

by

Dames and Moore
Consulting Engineers in the Applied Earth Science

Incorporated into Updated Safety Analysis Report
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APPENDIX E

REPORT OF ENVIRONMENTAL STUDIES
GEOLOGY, HYDROLOGY AND SEISMOLOGY

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PART 1 - INTRODUCTION

GENERAL

This report presents the results of geological, hydrological and seismological environmental studies and a preliminary foundation evaluation conducted by Dames & Moore for the Proposed Nuclear Power Plant planned for construction by the Northern States Power Company near Red Wing, Minnesota.

The site is located adjacent to and west of the Mississippi River in Goodhue County, approximately 40 miles southeast of the Minneapolis-St. Paul area and approximately 6 miles northwest of Red Wing, Minnesota. The location of the site is shown on Plate 1.1, Map of Region. The site occupies most of Section 5 and part of Section 4 in Township 113 North, Range 15 West. The location of the site is shown in relation to surrounding topographic and cultural features on Plate 1.2, Vicinity Map. It is understood that most of the proposed facilities will be located in the central eastern portion of the site close to the Mississippi River.

PURPOSE

The purposes of these studies were as follows:

1. To investigate the geologic, hydrologic and seismologic features of the site and its environs,
2. To develop criteria for use in the aseismic design of structures to resist earthquake ground motion, and
3. To evaluate foundation requirements and develop preliminary foundation design criteria for static and dynamic loading.

In order to accomplish these purposes, the following program of site environmental studies has been performed and the results are presented in the subsequent sections of this report:

1. Geologic, Hydrologic and Seismologic Research and Site Reconnaissance;
2. Test Borings;
3. Geophysical and Seismic Explorations;
4. Ground and Surface Water Investigations;
5. Laboratory Tests;
6. Environmental Analyses;
7. Foundation Engineering Studies

PROJECT PERSONNEL

The following members of our firm provided the principal contributions to the information, conclusions and recommendations presented herein:

George D. Leal	- Supervising Partner
Joseph A. Fischer	- Participating Partner
William G. Paratore	- Project Manager
David J. Leeds	- Project Seismologist
Robert J. Wenzel	- Project Geologist
John M. Heckard	- Project Hydrologist
B.G. Randolph	- Field Geophysicist
William W. Moore, Jr.	- Field Engineer

LIST OF PLATES

The following Plates are attached and complete Part I.

Plate 1.1 - Map of Region
Plate 1.2 - Vicinity Map

PART 2 - GEOLOGY

GEOLOGICAL PROGRAM

A geological investigation of the site has been performed by Dames & Moore. The scope of the geological program consisted of:

A. A review of pertinent published literature and unpublished data, and discussions with local geologists, in order to describe the geology of the region and the site.

B. A study of the geologic features of the site and environs by means of visual field reconnaissance and interpretation of maps and aerial photographs.

C. A detailed test boring and laboratory test program performed to identify predominant soil and rock types and to evaluate pertinent physical properties of the soil and rock strata present at the site.

The results of our geologic studies are presented in the following sections. The results of the field explorations and laboratory tests, which form the basis of our conclusions, are presented in Part 6 of this report.

SUMMARY

Based on the results of our geologic studies, it is our opinion that there is no geologic feature of the site or the surrounding area which adversely affects the intended use of the site. A summary of the geologic conditions is presented in this section.

The overburden soils at the site consist of sandy alluvium. The sandy alluvium ranges in thickness from 158 to 185 feet and is variable with respect to engineering properties. Several hundred feet of sound sandstone underlie the alluvial soils.

There is no evidence of even ancient inactive faulting closer than about six miles to the site. Another inactive fault is located about 13 miles from the site. No activity has occurred along either of these faults in recent geologic times.

REGIONAL GEOLOGY

General

Precambrian granite, gneiss, schist, and volcanics comprise the oldest bedrock in the Minnesota-Wisconsin region. This basement rock is overlain by as much as 800 feet of Paleozoic sandstone, shale and dolomite. Younger formations originally present in the region have been removed by erosion, and an irregular topography has been developed on the exposed bedrock surface. Except for local areas in southeastern Minnesota and parts of Wisconsin, bedrock is concealed under 100 to 300 feet of pleistocene glacial drift. In contrast, the extreme southeastern tip of Minnesota, including the site vicinity, is covered only by a thin veneer of drift. It is therefore considered a part of the “driftless” area commonly referred to by glacial geologists. In this driftless area of Minnesota and central and southwestern Wisconsin, the unconsolidated materials consist primarily of loess, recent alluvium, and residual soil.

Drainage in the region is controlled by the Mississippi River. The Mississippi River originated as an outlet for early glacial meltwaters. Its major present day tributaries were developed by the draining of glacial lakes at the close of the Pleistocene.

A geologic column showing the thicknesses and age relationships of the various bedrock units and surficial deposits of the region is presented in the generalized stratigraphic column presented on Table 2.1.

The regional extent of the consolidated strata is shown on Plate 2.1, Regional Geologic Map of Bedrock Formations.

Structure and Faulting

The dominant structural feature in southeastern Minnesota and adjacent areas of Wisconsin and Iowa is the Keweenawan Basin. This basin was formed in Precambrian time and extended from Lake Superior into Iowa. It provided a site for the deposition of thick sequences of later Precambrian and Paleozoic strata consisting of volcanics and sediments. These beds were gently warped by subsequent compressive forces into several subordinate structures. A large basin in the Paleozoic rocks extended northward from Iowa into the southeastern corner of Minnesota. This basin is separated from a smaller basin in the Twin Cities area by the Afton-Hudson anticline. The anticline begins at Farmington and trends northeastward through Hastings, Minnesota and Hudson, Wisconsin. A syncline lies to the east of this structure in the vicinity of River Falls, Wisconsin. Near the southeastern corner of Minnesota, a second anticline, the Red Wing Anticline, extends from Rochester through Red Wing and is postulated to extend a short distance into Wisconsin.

Several major faults in the Minnesota-Wisconsin region have been inferred from geophysical surveys. The principal movements along these faults, which amounted to thousands of feet, appear to have been restricted to Precambrian time. The Douglas and Lake Owen faults penetrate Precambrian rocks and these faults are located on the North and South sides of the Keweenawan Basin, respectively. A southern extension of the Lake Owens fault known as the Hastings fault, trends southwest near the city of Hastings, about 13 miles northwest of the site. Minor activity occurred along the Hastings fault during both Precambrian and Paleozoic times. Minor movements occurred in the overlying Paleozoic strata six miles southeast of the site near the city of Red Wing. Other small movements in the Paleozoic strata have occurred in the River Falls syncline near Waverly, Wisconsin approximately 20 miles northeast of the site.

TABLE 2.1 GEOLOGIC FORMATIONS IN THE GENERAL AREA OF THE SITE

GEOLOGIC AGE ERA PERIOD	GEOLOGIC NAME	APPROX. THICKNESS IN FEET	DESCRIPTION	REMARKS
Cenozoic Quarternary	Recent Deposits	20 to 200	Unconsolidated clay, silt, sand and gravel	Largely Mississippi and Vermillion River deposits
	Pleistocene		Unconsolidated clay, silt, sand, gravel and boulders deposited as till, outwash and loess	Largely from Superior and Des Moines lobes of Wisconsin glaciation
Paleozoic Ordovician	Oneota	100	Dolomite	Exposed along river bluffs
Cambrian	Jordan	100	Sandstone	An important aquifer
	St. Lawrence Formation	43	Dolomite siltstone and silty dolomite	
	Franconia Formation (St. Croix Series)	180	Sandstone and shale	Aquifer zones. Uppermost bed-rock at site
	Dresbach Formation (St. Croix Series)	100+	Sandstone, siltstone and shale	Aquifer zones
Precambrian Keweenawan	Hinckley Formation	100+	Sandstone	An important aquifer
	Red Classic Series		Sandstone and Red Shale	May not be present under the site
	Volcanics		Mafic lava flow with thin layers of tuff and breccia	May be present under the site
	Granite and Associated Intrusives			Principal basement rock under the site

There is no evidence of recent activity along any of the known fault zones in the Minnesota-Wisconsin region.

The locations of the above-discussed structural features are shown on Plate 2.1, Regional Geologic Map of Bedrock Formations. Regional geology is further depicted on Plate 2.2 Regional Geologic Structure, and on Plate 2.3, Regional Geologic Cross Section A-A.

SITE GEOLOGY

General

Prairie Island is a low island terrace associated with the Mississippi River flood plain. It is separated from other parts of the lowland by the Vermillion River on the west, and by the Mississippi River on the east. Ground surface elevations range from approximately 675 to 706 feet. Most of Prairie Island is under cultivation. Other lowland areas near the site are forested or covered by swamp vegetation.

The Mississippi River flood plain in this area is confined within a valley about three miles wide. Rocky bluffs and heavily forested slopes rise abruptly from both sides of the valley to a height of some 300 feet. The uplands immediately surrounding the valley reach elevations ranging from approximately 1000 to 1200 feet. They are deeply trenched by numerous streams emptying into the Mississippi River.

The overburden materials at the site are permeable sandy alluvial soils which were deposited as glacial outwash and as recent river sedimentation. Test borings have shown that the thickness of the overburden soils at the site varies from 158 to 185 feet.

The uppermost bedrock unit at the site is sandstone and is believed be part of the Franconia formation (See Table 2.1). Its thickness at this location is unknown but we believe it to be much less than 180 feet; the total measured thickness of the Franconia formation in complete sections. Underneath the Franconia formation are several hundred feet of lower Cambrian and Precambrian sandstones with minor shale horizons.

The site is located on the west limb of the Red Wing anticline, as evidenced by a gentle westward dip of the bedrock.

Subsurface Conditions

The results of our geologic research and site reconnaissance were supplemented by the drilling of exploration test borings at the locations shown on Plate 2.4, Map of Area. The drilling program was performed to confirm the actual soil, rock and ground water conditions at the site of the Proposed Nuclear Power Plant. The borings penetrated to depths ranging from approximately 174 to 213 Feet below the existing ground surface. Samples of the overburden soils and cores of the underlying rock were obtained visual inspection. Selected soil and rock samples were subjected to laboratory tests to determine pertinent engineering properties. Detailed descriptions of the soils and rocks encountered at each boring location and the results of the laboratory tests are presented in Part 6 of this report.

Based on the results of the exploration test borings and the laboratory tests, we interpret the general subsurface conditions at the site as essentially a layered system containing three predominant soil zones, underlain by rock. These zones are described below:

APPROXIMATE ELEVATION (FEET)	DESCRIPTION
Ground Surface to 665	Predominantly loose granular soils which exhibit relatively low strength and moderately high compressibility characteristics. These soils consist of brown loose fine to medium grained sands which are partially saturated with the water table (approximately 674).
665 to 645	Predominantly medium dense to dense granular soils exhibiting moderate strength to compressibility characteristics. The soils consist of brown fine to medium sands containing varying amounts of coarse sand and gravel. This zone contains interspersed discontinuous layers of loose granular soils. These soils are located below the ground water table and are denser than the overlying sands.
645 to 515	Predominantly dense to very dense fine to medium grained granular soils containing interspersed discontinuous zones of coarse grained sands, gravels and cobbles. Generally, the lower 10 to 30 feet of this zone contains many cobbles and boulders. These soils exhibit moderately high strength and relatively low compressibility characteristics. These soils are saturated and are somewhat denser than the overlying sands.
515 to the depths penetrated by the borings	Paleozoic sandstone of the Franconia formation. The sandstone encountered in the borings consist predominantly of a gray fine and medium grained quartz sandstone containing loose and cemented zones.

To assist you in visualizing the general subsurface conditions at the site, a subsurface profile through the site in the east-west direction is presented on Plate 2.5, Generalized Subsurface Section A-A.

Shoreline

The shoreline along most of the site is characterized by relatively steep stable slopes covered with dense growths of surface vegetation and trees It is our opinion that the stability of the slopes is related to the presence of the surface vegetation and trees. The shoreline adjacent to the site has undergone minor erosion, principally by river action during high water levels. Should the shoreline trees and vegetation be removed, it is considered that protective measures could be initiated, if required, to protect the shoreline from excessive erosion.

The direction of current flow and the influence of wave action will be an important consideration in locating the discharge and intake structures, and the protection of these structures from sedimentation.

LIST OF PLATES

The following Plates are attached and complete Part 2.

Plate 2.1 - Regional Geologic Map of Bedrock Formations

Plate 2.2 - Regional Geologic Structure

Plate 2.3 - Regional Geologic Cross Section A-A

Plate 2.4 - Map of Area

Plate 2.5 - Generalized Subsurface Section A-A

PART 3 - HYDROLOGY

HYDROLOGICAL PROGRAM

A hydrological investigation of the site has been performed by Dames & Moore. The scope of the hydrological program consisted of:

A. A review 6f pertinent data pertaining to flow, usage, temperature, chemical and biological characteristics, and other properties of Mississippi River waters and other adjacent surface waters.

B. A study o the hydrological interrelationship between the Mississippi River, Sturgeon Lake, the ground waters underlying the site, and other adjacent surface waters.

C. A study of the infiltration characteristics of the near-surface soils at the site and the determination of the composition and permeability characteristics of the bottom sediments in Sturgeon Lake and the Slough areas located immediately east of the site.

D. The performance of a pumping test to obtain information regarding permeability, transmissibility, and storage characteristics of the deeper soil strata.

SUMMARY

The principal surface waters in the vicinity of the site are the Mississippi River, Sturgeon Lake, the Vermillion River, and the Cannon River. The levels of the Mississippi River and Sturgeon Lake are controlled by Lock and Dam #3 which is located approximately one mile downstream from the site. The Vermillion River enters the main stream of the Mississippi below the dam. The maximum flood of record occurred in 1965 when a peak stage of approximately 688 was recorded. Flood stages during the anticipated plant life are not expected to exceed levels attained during 1965.

The ground water table is normally within 5 to 20 feet, at approximately elevation 675, of the ground surface of the site and appears to generally slope southwest from the Mississippi River towards the Vermillion River. The nearest ground water consumption of important magnitude is in the town of Red Wing, six miles downstream.

SURFACE WATER

Surface Drainage

Surface drainage at the site is essentially non-existent, owing to the extremely sandy nature of the soils and the topography of the island. There are no well-established drainage lines, and because of the hummocky nature of the terrain there are many small internal drainage basins.

Stream Flow

The Mississippi River and its major tributaries, the Vermillion River and the Cannon River, are the principal streams of the area. The location of the major streams and gauging stations are shown on Plate 3.1. The Mississippi River is dammed at a point about one mile downstream from the proposed site by Lock and Dam #3, and its two tributaries enter the main stream below the dam. The normal pool upstream from the dam is at elevation 675.

There are no major withdrawals of river water for at least 37 miles downstream from the site. Minor withdrawals of river water for irrigation purposes do occur.

Stream flow records are available for the Mississippi River at Prescott and at Winona, and for the Cannon River at Welch. Plate 3.2 shows flow-duration curves for the two stations on the Mississippi River.

Table 3.1 summarizes the consecutive-day low-flow characteristics of the Mississippi River at Prescott.

TABLE 3.1 MINIMUM CONSECUTIVE-DAY LOW-FLOW, IN CFS, FOR THE FIVE LOWEST YEARS OF RECORD*

1 DAY	(YEAR)	7 DAYS	(YEAR)	14 DAYS	(YEAR)	30 DAYS	(YEAR)
1380	(1940)	2190	(1936)	2260	(1936)	2350	(1934)
2100	(1936)	2240	(1934)	2260	(1934)	2650	(1936)
2210	(1934)	2640	(1933)	2650	(1933)	2860	(1933)
2270	(1939)	3110	(1931)	3190	(1931)	3360	(1932)
2520	(1933)	3270	(1932)	3320	(1932)	3370	(1931)

* Note: Gauged at Prescott, about 15 miles upstream from proposed site. Since no severe low flows have occurred after 1940, it is believed that construction of facilities on the river since that time assist in augmenting low flow.

Table 3.2 summarizes the discharge characteristics of the Mississippi River and Cannon River:

TABLE 3.2 DISCHARGE OF MISSISSIPPI RIVER AND CANNON RIVER

	MISSISSIPPI RIVER		CANNON RIVER
	AT PRESCOTT	AT WINONA	AT WELCH
AVERAGE	15,020 cfs	24.520 cfs	475 cfs
MAXIMUM	228,000 cfs (4-18-65)	268,000 cfs (4-19-65)	36,100 cfs (4-10-65)
MINIMUM	1,380 cfs (7-13-40)	3,350 cfs (12-29-33)	2.5 cfs (1-3-50)

Because of the lack of reliable data regarding the flow in the slack water area of Sturgeon Lake and the slough area adjacent to the site, a current meter is being installed to observe the long-term velocity and directional characteristics of current movements in these areas. Data regarding the velocity and direction of water movements will be accumulated beginning in July 1967 and will be recorded until the surface water flow characteristics adjacent to the site can be adequately defined.

Floods

According to the U.S. Army Corps of Engineers, the 1965 flood, which is the highest of record, has a recurrence interval of 150 years. The peak stage at Lock and Dam #3 during this flood was about 688 feet above sea level. It is estimated that a flood having a 1000 year recurrence interval would have a peek stage of about 691.8 at Lock and Dam #3, with a discharge of about 335,000 cubic feet per second.

A stream profile of the section of the river near the proposed site is presented on Plate 3.3 and shows the high water marks of the two highest floods of record. An approximate longitudinal land profile of the Prairie Island site is also shown on Plate 3.3.

River Temperature

A temperature curve showing the average monthly temperature of the Mississippi River at St. Paul is presented on Plate 3.4. Indications are that, on the average, these temperatures are from two to three degrees higher than those at the site.

Chemical, Physical, and Bacteriological Conditions

A summary of chemical, physical, and bacteriological analyses for the water year extending from October 1965 to September 1966 is tabulated in Table 3.3.

Radioactivity

A record of radioactivity determinations in the water of the Mississippi River for the period extending from October 1965 to May 1966 is presented in Table 3.4.

TABLE 3.3 CHEMICAL, PHYSICAL AND BACTERIOLOGICAL ANALYSES AT LOCK AND DAM #3

Date of Sample			Temp. (degrees centi-grade)	Dissolved Oxygen mg/l	pH	B.O.D. mg/l	C.O.D. mg/l	Chlorine Demand		Ammonia-Nitrogen mg/l	Chlorides mg/l	Alkalinity mg/l	Hardness mg/l	Color (Scale Units)	Turbidity (Scale Units)	Sulfates mg/l	Phosphates mg/l	Total Dissolved Solids mg/l	Coliforms per 100 ml.
Month	Day	Year						1-hr mg/l	24-hr mg/l										
10	5	65	10.5	9.1	7.9	3.3	34	6.2	13.0	-	7	150	200	35	110	29	.3	253	-
10	13	65	8.5	9.0	8.0	3.5	35	6.5	14.2	-	7	143	202	70	70	40	.3	269	-
10	19	65	14.5	8.3	8.0	3.4	35	7.3	15.6	-	7	147	198	60	70	40	.2	270	-
11	2	65	8.3	10.3	8.3	3.4	34	6.7	16.6	-	7	142	184	60	40	34	.2	254	75000
11	9	65	5.7	10.7	8.3	3.3	34	6.7	16.8	-	8	155	202	60	30	37	.3	266	75000
11	16	65	3.5	11.1	8.2	3.7	30	6.6	16.3	-	9	154	202	50	25	36	.3	253	75000
11	30	65	.3	12.2	8.0	4.6	27	-	-	-	9	153	194	50	25	34	.3	257	100000
12	7	65	.6	12.2	8.1	3.9	29	-	-	-	11	170	215	40	25	35	.5	275	25000
12	14	65	.7	11.8	8.1	3.5	28	-	-	-	9	166	212	40	25	38	.4	261	100000
12	21	65	.3	11.8	8.1	2.6	28	-	-	-	8	150	194	45	25	30	.4	258	-
12	28	65	.1	12.5	8.0	2.1	26	-	-	-	10	179	232	45	25	41	.4	302	46000
1	4	66	.1	11.2	7.9	2.8	29	-	-	-	10	175	226	45	25	38	.5	290	-
1	11	66	.1	10.3	8.0	4.0	30	-	-	-	10	170	216	45	25	38	.5	289	82000
1	18	66	.2	9.4	7.9	2.9	31	-	-	-	10	176	210	45	25	33	.5	274	22000
1	25	66	.0	8.7	7.8	2.9	34	-	-	-	11	179	224	45	25	35	.6	301	10000
2	1	66	.1	8.8	7.8	2.7	29	-	-	-	10	184	222	45	25	33	.6	281	22000
2	8	66	.4	7.9	7.8	4.1	29	-	-	-	10	175	211	40	25	29	.6	275	15000
2	15	66	.2	10.4	7.9	4.9	32	-	-	-	9	119	152	45	60	25	.6	211	100000
2	23	66	.2	10.3	7.7	3.5	31	-	-	-	8	133	171	45	25	28	.6	234	5000
3	1	66	.2	9.3	7.8	2.5	29	-	-	-	12	143	176	45	25	28	.5	244	15000
3	8	66	.3	11.2	7.8	3.7	32	-	-	-	9	142	188	45	90	36	.6	248	80000
3	15	66	2.4	11.4	7.9	3.5	30	-	-	-	6	124	156	45	50	27	.4	205	65000
4	19	66	7.6	10.1	8.1	3.1	32	-	-	-	5	125	169	45	40	31	.1	221	60000
5	3	66	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	27000
5	18	66	11.6	9.3	8.2	3.4	35	-	-	-	6	136	191	45	50	37	.2	217	85000
5	25	66	15.6	8.0	8.2	3.3	35	-	-	-	6	137	188	50	75	42	.2	243	95000
6	15	66	19.1	7.5	8.0	3.8	41	-	-	-	6	132	188	60	90	35	.1	228	25000
6	22	66	22.8	6.2	7.9	3.9	30	-	-	-	7	140	188	45	50	33	.3	240	5600
6	29	66	24.0	5.0	7.9	4.1	34	-	-	-	6	139	208	40	110	41	.5	264	90000
7	7	66	25.2	5.9	8.0	3.1	25	-	-	-	8	146	198	40	50	32	.4	257	-
7	27	66	26.8	6.2	8.3	3.3	30	-	-	-	8	143	185	40	40	27	.4	239	6500
8	3	66	24.2	7.0	7.8	3.6	23	-	-	-	9	142	176	40	40	21	.4	217	8000
8	10	66	22.9	6.6	8.1	3.1	29	-	-	-	8	140	169	35	40	23	.4	224	4000
8	17	66	23.2	6.0	8.0	3.1	27	-	-	-	8	135	158	35	60	18	.3	211	20000
8	24	66	18.5	7.0	8.0	2.2	31	-	-	-	7	128	152	35	65	17	.4	201	15000
8	31	66	24.0	6.6	7.9	3.1	32	-	-	-	6	128	152	40	60	18	.2	211	20000
9	8	66	21.1	7.9	8.0	2.7	33	-	-	-	8	134	165	40	45	22	.3	215	-
9	14	66	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4000

TABLE 3.4 RADIOACTIVITY DETERMINATIONS AT LOCK & DAM #3

DATE SAMPLE TAKEN			DATE OF DETERMINATION		RADIOACTIVITY IN WATER											
					ALPHA						BETA					
					SUSPENDED		DISSOLVED		TOTAL		SUSPENDED		DISSOLVED		TOTAL	
MO.	DAY	YR.	MO.	DAY	μ μc/l	±	μ μc/l	±	μ μc/l	±	μ μc/l	±	μ μc/l	±	μ μc/l	±
10	19	65	12	9	0	0	1	1	1	1	2	1	23	2	25	2
11	24	65	2	28	0	0	1	1	1	1	2	1	17	2	19	2
12	21	65	2	16	0	0	2	1	2	1	0	2	16	4	16	4
1	25	66	3	17	0	0	1	1	1	1	1	1	18	2	19	2
2	23	66	4	18	0	0	0	1	0	1	0	1	16	2	16	2
3	15	66	5	20	0	0	1	1	1	1	2	1	19	2	21	2
4	26	66	7	10	0	0	2	1	2	1	2	1	16	2	18	2
5	18	66	7	25	0	0	2	1	2	1	2	1	16	2	18	2

Navigation

Records kept since 1931 indicate that the commercial navigation season at St. Paul extends from about the latter part of March to the latter part of November. The longest season on record is 266 days (1947) and the shortest is 210 days (1965)

The total lockages at Lock and Dam #3 from 1961 to 1966 are given below:

TABLE 3.5 TOTAL LOCKAGES AT LOCK AND DAM #3, 1961 TO 1966

1961	4,066	1964	3,951
1962	3,674	1965	3,388
1963	3,965	1966	3,945

Fishing

Fishing, both for pleasure and from a commercial standpoint, is an important activity on the Mississippi River and its backwaters. commercial fishing is limited to rough fish, and the amount of catch estimated for 1964 in pool #4, immediately below Lock and Dam #3 is shown in Table 3.6. The estimated number of fish caught by sport fishermen is also shown in Table 3.6.

TABLE 3.6 ESTIMATED FISH CATCH BY MINNESOTA FISHERMAN IN POOL #4 FOR THE YEAR 1964

KIND OF FISH	(COMMERCIAL FISHERMEN) POUNDS	(SPORT FISHERMEN) NO. OF FISH CAUGHT
Carp	2,379,000	2,685
Buffalofish	51,913	
Drum	51,240	
Carpsuckers	3,420	
Bluegill		55,275
Black Croppies		90,731
Lake Croppies		15,061
Sauger		55,685
Walleye		75,566
White Bass		78,384
Largemouth Bass		1,141
Smallmouth Bass		13,216
Rock Bass		591
Northern Pike		4,104
Channel Catfish	23,726	1,071
Flathead Catfish	147	164
Bullhead	234	
Sunfish		1,391

INFILTRATION CHARACTERISTICS

The infiltration characteristics of the near-surface soils at the site and the composition and permeability characteristics of the bottom sediments at the south end of Sturgeon Lake and the slough area adjacent to the site have been investigated.

The infiltration characteristics of the near-surface soils, present above the water table, were evaluated by the performance of field percolation tests. A total of Five Field percolation tests were performed in the immediate vicinity of Borings 9 through 12. The tests were performed by first installing a piece of casing or well screen to a predetermined depth and then pouring water into the casing on well screen and measuring the time required for the water level to drop a measured distance. The results of the percolation tests are presented below:

TABLE 3.7 PERCOLATION TEST RESULTS

DEPTH TO GROUND WATER (FEET)	TEST DEPTH (FEET)	PERMEABILITY (FEET PER YEAR)
17	2 to 4	5.8×10^3
17	10 to 13	19.4×10^3

Based on the results of the field percolation tests and our knowledge of the subsurface conditions, it is concluded that the near-surface soils have relatively high permeability characteristics and that fluids will percolate rapidly through these soils.

The bottom sediments at the south end of Sturgeon Lake and in the slough area adjacent to the site were sampled by use of a coring device handled from a raft. The upper two to six Feet of the bottom sediments were sampled and were visually examined. Approximately 23 locations were investigated as shown on Plate 3.5, Location of Bottom Sampling. Presented on Plate 3.6 are profiles which illustrate the general subsurface conditions encountered in Sturgeon Lake and the slough area.

The explorations indicate that the bottom of the south end of Sturgeon Lake is covered with one to five feet of impermeable soft clayey silt which is, in turn, underlain by permeable loose silty fine to medium sand. In the slough area, the bottom sediments consist of approximately one to four feet of relatively impermeable clayey silt which is underlain by permeable loose silty fine to medium sand. The exception to this is in the deep channel area located in the middle of the slough area where scour action has removed the clayey silts and has exposed the permeable loose silty fine to medium sands. Within the slough area there are several narrow islands which frequently extend above the river level. The surfaces of these islands are covered with an impermeable soft clayey silt topsoil approximately two feet thick, except near the shoreline where permeable loose silty fine to medium sands may be exposed.

Based on the results of our explorations, the areas where permeable sands are exposed provide a means for surface waters or other fluids to percolate into the ground water system underlying the site.

GROUND WATER

Regional Characteristics

Regionally, the movement of ground water is toward the Mississippi River and its main tributaries. The ground water table slopes from the higher, partially glaciated bedrock areas toward these surface streams, generally at low gradients. Ground water enters the river valley from along the base of the bordering bedrock bluffs in the form of springs or as subsurface flow.

Beneath the flood plains and low terraces which border the Mississippi River, ground water levels closely coincide with the elevations of the river surface, and vary in accordance with river fluctuations. The average ground water gradient in these bottomlands is downstream, and essentially parallel to the stream gradient.

Pool elevation on the Mississippi River adjacent to the site is controlled by Lock and Dam #3. The Vermillion River by-passes the dam and therefore is not directly controlled by it. Elevations on the Vermillion River and connected lakes are therefore lower than the Mississippi River and the ground water table slopes southwestward between the two rivers. Due to the permeable nature of the sandy alluvial soils forming Prairie Island, the ground water table responds quickly to changes in river stage.

There is only minor usage of ground water for domestic, agricultural and irrigation purposes near the site or immediately downstream. A deep well believed to penetrate bedrock aquifers exists at Lock and Dam #3. The nearest ground water consumption of important magnitude is in the town of Red Wing, six miles downstream. This community derives its water supply from four deep wells which penetrate sandstone aquifers of the Dresbach and Hinckley formations. The wells pump from depths of 400 to 730 feet, and each well is capable of providing the municipal requirements of about 1400 gpm. A high degree of hardness is characteristic of the water from these wells.

Several industries in the Red Wing area also utilize ground water in quantities exceeding the municipal consumption, and derive their supplies principally from the bedrock aquifers. Total well production from the bedrock at Red Wing probably exceeds 3000 gpm, and fairly large quantities also may be extracted from the alluvium for certain industrial uses.

Communities further downstream from the plant site which supply their water needs from wells in bedrock are Lake City, 25 river miles distant, and Wabasha, 37 river miles downstream.

Site Conditions

During and subsequent to our field explorations at the site, observations of the ground water table were made in the cased exploration test borings. These observations confirm that the regional ground water table slopes in the southwesterly direction. A typical set of ground water readings obtained from the test borings is presented below:

TABLE 3.8

BORING NUMBER	ELEVATION OF GROUND WATER (FEET) ON JUNE 20, 1967
3	674.3
4	673.1
5	673.0
6	673.1
7	674.0
8	673.0
9	674.1
10	Not Recorded
11	Not Recorded
12	673.6
Mississippi River	675.6

Presented on Plate 3.7 is a comparison of the Mississippi River level and the ground water level in Borings 4 and 10. This graphical representation confirms that the ground water level at the site is directly influenced by the elevation of the Mississippi River surface and will vary in accordance with the river fluctuations.

Pumping Test

A pumping test was performed approximately 600 feet from the Mississippi River at coordinates N 593,986 & E 2,356,030 to evaluate the transmissibility, permeability and storage capacity of the in-situ soils encountered below the ground water table (elevation 675) to a depth of approximately elevation 617. The data obtained from the pumping test provides information to contractors which will be of value to them in preparing cost estimates for dewatering operations. Five observation borings were installed as part of the pumping test program. The location of the pumped well and the observation borings are shown on Plate 3.8.

The test well was approximately 27 inches in diameter and was installed utilizing truck-mounted drilling equipment. The well penetrated to a depth of about 71 feet (elevation 617) below the existing ground surface. A 16-inch irrigator type (all round wire) well screen, approximately 20 feet long, with No. 100 slot openings was installed at the bottom of the well. Casing approximately 16 inches in diameter was connected to and installed above the well screen. The annular space between the casing, the well screen and the sides of the well was filled with clean rounded gravel with a uniformity coefficient of 2.5 and an effective size of approximately 0.100 inches. A vertical turbine pump was installed in the well at a depth of 68 feet (elevation 620).

The observation borings were installed at distances of 20, 100, 200 and 400 feet from the pump well. The observation borings extended to elevations ranging from 630 to 635, and two inch diameter metal pipe having a three foot drive point extension was installed in each observation boring.

Based on the results of the data accumulated during the pumping test, the following characteristics of the in-situ soils were determined:

TABLE 3.9

COEFFICIENT OF TRANSMISSIBILITY (GALLONS PER DAY PER FOOT)	COEFFICIENT OF PERMEABILITY (GALLONS PER DAY) (PER SQUARE FOOT)	COEFFICIENT OF STORAGE
500,000	1,000 to 4,000	0.20

Presented on plate 3.9 is the drawdown curve for steady state flow after 24 hours of pumping at an average rate of 1100 gallons per minute. The drawdown versus time for Observation Boring 1 and the drawdown versus distance from the pumped well are shown graphically on Plate 3.10.

During the course of the pumping test, samples of water were obtained from the pump well. Comprehensive chemical analyses of the water were made and the results of the analyses are presented in Table 3.10.

TABLE 3.10 CHEMICAL TEST RESULTS ON GROUND WATER

	PARTS PER MILLION
Total Dissolved Solids	453
Non-Carbonate Hardness (As $CaCO_3$)	51
Carbonate Hardness (As $CaCO_3$)	184
Total Hardness (As $CaCO_3$)	235
Bicarbonate Alkalinity (As $CaCO_3$)	185
Carbonate Alkalinity (As $CaCO_3$)	0
Total Alkalinity (As $CaCO_3$)	185
Calcium (As $CaCO_3$)	168
Magnesium (Mg)	67
Silica (SiO_2)	12.8
Iron (Fe)	0.08
Manganese (Mn)	less than 0.01
Chlorides (Cl)	10.0
Sulphates (SO_4)	31.0

pH equals 7.7

Movements of Effluents

Dispersion of effluents entering the ground water system from the proposed plant would take place principally in the upper portion of the saturated zone of the sandy alluvium. Due to the numerous surface waterways in the vicinity of the site, the majority of effluents would permanently leave the ground water environment and would commingle with surface waters at the borders of Prairie Island. It is very unlikely that significant amounts of effluents could succeed in reaching shallow wells at Red Wing by way of continuous ground water paths. Penetration of effluents to the depth of bedrock aquifers at Red Wing is not anticipated.

LIST OF PLATES

The following Plates are attached and complete Part 3:

Plate 3.1	-	Location of Major Streams and Gauging Stations
Plate 3.2	-	Flow Duration Curves for Mississippi River
Plate 3.3	-	Stream Profile For Mississippi River from Lock & Dam No. 2 to Lock & Dam No. 3 Including Site of Proposed Nuclear Plant
Plate 3.4	-	Average Temperature of Mississippi River at St. Paul, Minnesota
Plate 3.5	-	Location of Bottom Sampling
Plate 3.6	-	Profile of Sturgeon Lake and Slough Area
Plate 3.7	-	Comparison of Mississippi River Level and Ground Water Level in Borings 4 and 10
Plate 3.8	-	Location of Test Well and Observation Borings
Plate 3.9	-	Drawdown Curve
Plate 3.10	-	Pumping Test Data

PART 4 - ENGINEERING SEISMOLOGY

SEISMOLOGICAL PROGRAM

A seismological investigation of the site has been performed by Dames & Moore. The scope of the seismological program consisted of:

A. An evaluation of the seismicity of the area.

B. A study of geologic structure as related to earthquake activity.

C. The postulation of “design” and “maximum credible” earthquake accelerations and the preparation of recommended response spectra.

D. Field and laboratory measurements of the dynamic response, characteristics of the soil and rock strata underlying the site.

The results of our seismological studies are presented in this section. The results of the field explorations and laboratory tests, which form the basis of our conclusions, are presented in Part 6 of this report. The terminology used in the Engineering Seismology section is defined in Part 7 of this report.

SUMMARY

Based on the seismic history and the regional tectonics, it is our opinion that the site will not experience any significant earthquake motion during the economic life of the proposed nuclear facility. Historically, there is no basis for expecting ground motion of more than a few percent of gravity. However, on a conservative basis, we recommend that the nuclear power plant be designed to respond elastically, with no loss of function, to earthquake ground motion as high as six percent of gravity.

Provisions should also be made for a safe shutdown of the reactor if ground motions reach as high as 12 percent of gravity in the overburden soils at the site. We believe, however, that the possibility of such an occurrence is quite remote.

At the time of the preparation of this report, preliminary structural design data had been provided to us. Based on the anticipated loading conditions, we believe that satisfactory earth support for a mat foundation for the reactor units will be provided by the dense to very dense granular soils at and below approximately elevation 645. As a result of the looser condition of the upper granular soils, structures on spread foundations or mats above approximately elevation 645 would experience some settlement. In addition there is only a small margin of safety against liquefaction of the in-situ upper soils under earthquake loading. Therefore, we recommend that structures established above elevation 645 be supported on either pile foundations or on densified sand. Preliminary design data are presented for various foundation systems in Part 5 of this report. Additional detailed data pertaining to foundation design will be obtained and evaluated during the performance of a comprehensive foundation investigation at the site.

All foundations will be within the sandy soils above the bedrock. The design of the proposed structures and their foundations will take into account the dynamic effects of earthquake motion. Therefore, consideration will be given in design to maximum expected ground motions, response spectra, and elastic moduli and damping values of the various soils and rock.

SEISMIC GEOLOGY

General

From a seismic point of view, the most important geologic considerations are the type, structure, and physical properties of the foundation soils and rock, and the location and activity of nearby faults. These factors are discussed below.

Stratigraphy

A detailed description of regional and site geology is presented in Part 2 of this report. In summary, the site is underlain by approximately 158 to 185 feet of sandy alluvial deposits. These soils are immediately underlain by competent sandstone of the Franconia Formation. The surface of the sandstone dips gently to the west at a slope of about 30 feet per mile. The Franconia Formation has a thickness of less than 180 feet at the site and is underlain by other older sedimentary rocks.

Pertinent physical properties of the subsurface soils and rock were measured during our field explorations. Presented on Plate 4.1 is a generalized column showing the average thicknesses and properties of the bedrock and surficial deposits at the site.

Faulting

As discussed in the geologic section of this report, there are a number of geologic structures within 100 miles of the site. However, there is no surface expression of recent movement on these faults.

The closest known fault to the site is about six miles southeast, near the city of Red Wing. Our geologic and geophysical explorations of the site and its immediate environs did not disclose any faulting closer than this previously observed fault which is apparently associated with the Red Wing anticline. The known geologic structure in the region is depicted on Plate 4.2, Regional Geologic Structure.

SEISMIC HISTORY

Southeastern Minnesota and adjacent areas in Wisconsin and Iowa are considered among the least seismically active zones of the United States. The Philip P. King study on the number of recorded epicenters throughout the United States classifies the region in the "least active" category, i.e., having less than one epicenter per 10,000 square kilometers. However, earthquakes are not unknown in Minnesota and the adjacent states. At least six (1860, 1865-70, 1917, 1928, 1939 and 1950) have had origins in Minnesota, and certain others, with epicenters outside the state, have been felt within the borders of Minnesota.

The records of local earthquakes are "man" observations, rather than instrumental, made on the surface of the ground for local site conditions. The sites of observation of past local earthquakes could vary slightly and have not been fully investigated because of the uniformly low intensities of ground motion.

In relating historic local seismic reports to the prediction of ground motion at the Prairie Island site in the subsequent development of earthquake criteria, it has been assumed that the historic sites and the Prairie Island Site are roughly equivalent. This assumption therefore includes the utilization of a "built-in" soil amplification factor which represents the direct application of experience on surface ground motions in local towns with ordinary buildings. It is considered that this approach is more satisfactory than the direct computation of surface amplification ratios by site to site comparison, due to the accumulative uncertainties caused by lack of reliable local earthquake site data.

One of the best documented, of what could be considered a typical local earthquake, is the slight shock of September 3, 1917. This shock was rated as having maximum intensities[*] between V and VI at several points in Todd, Lincoln, and Crow Wing Counties. It was felt for a maximum distance of 120 miles along a line connecting Brainerd and Minneapolis, and was most severe at Staples, Minnesota. Damage in this 1917 shock was limited to cracking of a wall on one side of a brick building, and cracking of the concrete floor in the vault of the City Clerk's office in Staples. Several brick courses were dislodged from a chimney in Brainerd and a chimney was thrown down in Lincoln. No windows were reported broken. The higher intensities reported for this earthquake followed the Crow Wing - Mississippi Valleys between Staples and Brainerd, either reflecting less favorable soil conditions or, perhaps, vagaries of population distribution. The shock was not associated with recent faulting or with any of the known historic faults in Minnesota.

[*] All intensities in this report are identified in terms of the Modified Mercalli Intensity Scale of 1931, as described in Part 7 of this report.

An 1860 shock is known only through the testimony of old settlers, who stated that it was harder at Long Prairie than the 1917 shock there. The 1860 earthquake could possibly have done damage if the country had been more thickly settled.

A slight shock was reported sometime between 1865 and 1870 at Le Sueur.

On December 23, 1928, at Bowstring, a small shock was reported, where “the house seemed to sway in an east-west direction.”

On January 28, 1939, a shock with a maximum reported intensity of IV occurred at Detroit Lakes in west-central Minnesota. The shock was felt over an area within a radius of 50 miles from Detroit Lakes.

On February 15, 1950, a “sharp shock accompanied by a muffled boom awakened residents and startled night workers,” at Alexandria. Two 136-foot wells at a creamery were damaged.

Earthquakes originating outside the state have been felt in Minnesota, but with low intensities. The St. Lawrence Valley earthquake of March 1, 1925 has an intensity no more than I in Minneapolis as did the Timiskaming, Quebec earthquake of November 1, 1935. The May 26, 1909, Illinois shock had an intensity of III in the southeast corner of Minnesota. The greatest eastern earthquakes to date (1811 and 1812 New Madrid and 1886 Charleston) were not reported as being felt in Minnesota.

Several small earthquakes have occurred in the Keweenaw peninsula in northwestern Michigan. This area has been extensively mined and some of the small shocks are probably the result of mine collapse. Others may be associated with the Keweenaw system earthquakes in 1905 and 1906 which had reported epicentral intensities of VII and VIII, respectively. These high intensities are probably the result of mine collapses. We do not believe that the magnitude of any of this series of shocks was greater than about 4 on the Richter[*] Scale.

Significant earthquakes having epicenters in Minnesota together with certain out-of-state earthquakes felt in the state are presented in Part 7 of this report. Earthquake epicenters within an approximate 300 mile radius of the site are shown on Plate 4.3, Regional Earthquakes.

A review of all the earthquakes mentioned above reveals that no shock is known with an epicenter within a distance of 50 miles of the site, and only eight earthquakes have been recorded within 200 miles of the site. We therefore conclude that it is very doubtful that any Minnesota earthquakes, which have epicentral intensities of VI or less, or any out-of-state earthquakes have ever been felt at the site.

[*] The Richter Scale is described in Part 7 of this report.

The relationship of known earthquake epicenters to regional geologic faulting is shown on Plate 4.3, Regional Earthquakes. Minnesota's earthquakes appear to be centered in an area about 100 miles northwest of Minneapolis in the Cuyuna Range. There are known faults in this region, however, the last movements on these faults is believed to have occurred during Precambrian and Paleozoic times. No significant surface displacement has been demonstrated in Cenozoic times.

Other faults are known in the area but earthquakes are absent in these areas, and it is reasonable to consider the faults inactive and therefore not a potential source of earthquakes. There are no other anomalies present in the immediate vicinity of the site that would indicate a potential seismic hazard.

In summary, most earthquakes in the region occur in a limited area northwest of the Twin Cities Basin approximately 150 miles or more northwest of the site. There is no evidence to link these earthquakes to geologic structures near the site. No major earthquake has been experienced in the region, and the available history indicates a very low regional seismicity. However, the brevity of the historical record, the low quality of the historical seismic data and the lack of modern instrumentation in the region indicate that small earthquake event could possibly have been overlooked. The minor shocks which may have occurred throughout the region would have some significance respect to design of important engineering structures.

ASEISMIC DESIGN CRITERIA

Selection of Design Earthquakes

Design Earthquakes

On the basis of the seismic history of the area, it is our opinion that the site will not experience any significant earthquake motion during the economic life of the proposed nuclear facility.

The possibility of a recurrence of a shock of the same order of magnitude as the 1917 Staples, Minnesota shock (the largest of the region) close to the site is quite remote. On the basis of a statistical study of the seismic history of the region, it is estimated that a shock similar to the 1917 shock within a 50 mile radius of the site would occur about once in 3,000 years. The actual possibility is low since there is no evidence of any geologic structure continuous from the area of the 1917 shock to the site.

The occurrence of a minor shock (epicentral intensity of IV or V) within 25 miles of the site would be once in 2000 years.

Using the available knowledge of tectonics and seismic history of the region, we believe that the maximum expected ground motion to which the site may be subjected during its economic life would result from:

1. A magnitude 3 to b, shock (maximum epicentral intensity VI) about 100 miles northwest of the site or;

2. A magnitude 5 shock (maximum epicentral intensity VII) about 150 miles southeast of the site, related to faulting in the Madison and Beloit, Wisconsin areas.

We believe that ground motion at the site due to either of these possible occurrences would be barely perceptible. On a historical basis, therefore, it does not appear necessary to incorporate a seismic factor in the design of the proposed nuclear power plant. However, in view of the importance of the nuclear power plant facility, we believe that the critical structures should be conservatively designed for maximum ground acceleration of six percent of gravity.

Maximum Credible Earthquake

In order to provide for safe shutdown of the reactors, it is prudent to investigate the maximum credible earthquake which might occur in the region. The maximum credible earthquake is generally considered to be a recurrence of the largest recorded earthquake in the region, at the closest epicentral distance consistent with geologic structure.

It is very probable that, at this site, the ground accelerations postulated for the design of the reactors under essentially elastic strains would not be exceeded by any credible earthquake occurrence. However, the recommended design acceleration is based on records of shocks which have not been definitely related to known tectonics. The bedrock in the region is concealed by overburden and much regarding local and regional tectonics is unknown. It is possible therefore that further studies in the region would reveal yet unknown geologic structures in the vicinity of the site.

Based on the foregoing, we have investigated the possibility of a magnitude three to four earthquake very close to the site. We have also considered the possibility that the northeast-southwest trending fault system identified about 13 miles northwest of the site is related to faulting on the Keweenaw peninsula in Michigan. Based on this assumption, and considering that buried unknown faulting related to this system may extend closer to the site than 13 miles, we have investigated the possibility of an earthquake similar to one of the Keweenaw peninsula series of shocks (magnitude no more than 4.5) with its epicenter close to the site.

For the above exceedingly remote occurrence, we estimate that the maximum ground acceleration at foundation level at the site would be less than 12 percent of gravity. We therefore recommend that the reactor be designed for safe shutdown for a maximum ground acceleration of 12 percent of gravity.

Amplification Spectra

In evaluating the maximum ground motion which would occur at the site during the design and maximum credible earthquake, amplification spectra were developed. Amplification spectra are plots which indicate the amplification of earthquake wave motion between the basement rock and the foundation soils, for wave motion of varying periods. Recent developments permit the incorporation of values for soil damping into computed amplification spectra. The spectra are computer plotted, with both elastic and damped results plotted in log-log-format. Several models are utilized to enable the variation of properties over a range of observed or estimated values, and the effect on surface amplification is noted.

For the completely elastic case, computed spectra yield amplification ratios between surface (-20 to -50 feet) to geologic basement (-4000 feet) of approximately three but less than four. Five percent damping lowers the amplification ratio between surface to basement to less than two at all periods.

For comparison purposes, amplification spectra were also developed for the subsurface conditions existing at the locations of strong motion recording stations in Taft and El Centro, California. The amplification spectra for the Prairie Island Site for foundation conditions at a depth of 20 to 50 feet is compared to the amplification spectra for the Taft and El Centro sites on Plate 4.4, Earthquake Wave Amplification Ratios.

The ratios of amplification spectra are graphical comparisons of the response of the Prairie Island Site to both Taft and El Centro. The comparisons presented on Plate 4.4 indicate that the Prairie Island Site would respond less than El Centro and no more than Taft given the same basement rock input to all three locations.

The application of the amplification ratios described above to computed geologic rock basement motion yields acceleration values well within the recommended acceleration criteria for the design and maximum credible earthquakes.

Response Spectra

Recommended response spectra, presenting estimated structural responses for typical values of damping, are presented for the postulated design and maximum credible earthquake conditions on Plates 4.5 and 4.6, Recommended Response Spectra. The response spectra represent the maximum amplitudes of motion in structures, having a range of natural frequencies, subjected to earthquake ground motion.

EFFECTS OF EARTHQUAKE LOADING ON SOIL AND ROCK

General

Experience indicates that the strength properties of sound rock are unaffected by earthquake loading. Therefore, no problem is expected in the performance of the sound bedrock formations at the site during an earthquake.

In order to evaluate the effects of dynamic or oscillatory loading on the sandy overburden soils at the site, such as might be experienced during an earthquake, a series of static and dynamic triaxial compression tests and dynamic confined compression tests were performed. The test methods and test results are presented in Part 6 of this report. Engineering analyses were performed utilizing the results of these dynamic tests and the results of various other field and laboratory tests. A detailed discussion of the results of our studies Is presented in this section.

Liquefaction of Sand

Under the influence of dynamic or oscillatory loads, loose to medium dense sands tend to compact with a consequent reduction in the volume of the pore space. If the sand has saturated pore spaces, this tendency to compact will increase the pressure in the pore-water. If this excess pore-water pressure cannot be dissipated by drainage and increases until it is equal to the overburden pressure, the confinement pressure (effective stress) on the sand will be reduced to zero. When this occurs, the sand undergoes a complete loss of strength and a “liquefied” state is developed.

The phenomenon of liquefaction has been recognized for many years and some qualitative understanding has been developed. Information from recent research (see Part 8) presently allows some quantitative estimates of the probability of liquefaction occurring at a specific location, when the in-situ soils are subjected to postulated shaking motions.

The occurrence of liquefaction is affected by many site and earthquake motion characteristics and soil properties, all of which must be evaluated before an estimate of the liquefaction potential of any specific soil deposit can be made. Of these the principal variables are the density characteristics of the sand, the magnitude of the confining pressure, the magnitude, frequency and duration of the dynamic or oscillatory forces, the boundary conditions affecting drainage, and the particle size distribution of the sand. The general effects of each of these variables will be discussed before considering the application to the conditions at the site.

Density Characteristics

Sand may exist over a wide range of densities. In soil mechanics, this density condition is usually defined either by means of the void ratio (the ratio of the volume of the void space to the volume of solid or sand particles present) or the porosity (the ratio of voids to the total volume).

In classical soil mechanics, the shearing behavior of sands has been divided into two categories, depending on the behavior of the sand during shearing. Loose sands were observed to decrease in volume during shearing, while dense sands tended to dilate. The concept of critical void ratio was defined as a void ratio at which shearing took place with no volume change. Although successful in its application to static analysis, the critical void ratio approach has been found to have no relationship to the liquefaction phenomenon. Sands with void ratios less than “critical” can liquefy if other adverse conditions exist. Similarly, loose sands with void ratios higher than the critical void ratio may not liquefy as a result of more favorable conditions

All variables must be considered together in each case. However, when all other variables are held constant, a sand with a high void ratio will liquefy before an identical sand with a low void ratio.

Confining Pressure

An increase in confining pressure will make liquefaction of a sample more difficult. The properties of sand, such as elastic moduli and damping factors, are also dependent on confining pressure and can change the response of a system which may be subject to liquefaction.

Magnitude and Duration of Shearing Stresses

Increasing either or both the dynamic shearing stress and the number of applications of the cyclic loading will increase the probability that any given deposit will liquefy. The loading condition must be such that the shear stresses be reduced to zero or be reversed in direction during the loading to produce liquefaction. This condition will be satisfied under normal earthquake conditions only in uniform level deposits where the cyclic shearing stresses result principally from vertically progressing shear waves producing strains which reverse direction many times. Under structures, existing shear stresses may present reversal during shaking. In this case, liquefaction would have to take place by progression from adjacent external liquefaction, thereby increasing the time for liquefaction.

Boundary Drainage Conditions

If the increase in pore pressures can be dissipated as the shaking motion occurs, the pore water pressure may never rise to equal the confining pressure. If the pore water pressure did not equal the confining pressure, liquefaction would not occur. This situation might exist in a dike or similar structure with short drainage paths. For large areas, however the effective drainage during earthquake motion is expected to be small and any beneficial effects which might accrue from drainage can be ignored in a conservative approach.

Particle Size Distribution

Most research work on liquefaction of sands has been performed on clean sands of uniform grain size which would be classified as fine to medium fine. Therefore, when field samples meet these same requirements, the results of the published work may be utilized. Where sands have non-uniform grain size characteristics or appreciable amounts of silts or clay (particles passing the No. 200 sieve), the tendency towards liquefaction under cyclic loading is greatly reduced. Limits can be set for the amounts of these materials where liquefaction can be considered improbable. Where some doubt exists, the effect of varying particle size and varying amounts of fines can be resolved by resort to direct laboratory testing under dynamic conditions on samples from the site in question.

Engineering Analyses

Earthquake motion is a random vibration which is modified by such factors as distance from the epicenter, site conditions and other geologic features. Being a random vibration, the use of some past records must be considered as a guide to the probability of future events, but the complicated procedures involved to make direct application of earthquake records to laboratory testing are not warranted. Instead, the earthquake effects may be approximated by a simple cyclic loading procedure. The application of this procedure to the proposed site is described below.

For this site, a maximum credible ground acceleration of 12 percent of gravity has been selected. This earthquake would probably have a duration of strong shaking of not more than ten seconds. During this period, several cycles of large shearing stresses, together with many smaller cycles, may be expected. The strongest expected soil vibrations will be primarily in the first mode, with a fundamental period of about one-half second. This would give a total number of 20 oscillations during the ten second duration. Because of the random nature of the forcing vibration of the bedrock and the effect of soil damping, it is likely that less than half of these oscillations will have appreciable magnitudes. Thus, it is expected that there may be six to ten cycles of shearing stress of a sufficient magnitude to cause concern about liquefaction during this postulated earthquake.

The maximum amplitude of shearing stress at any elevation in the soil may be conservatively estimated as the acceleration value of 0.12 multiplied by the total vertical pressure at that depth. The results of this computation are tabulated for several depths in Table 4.1.

TABLE 4.1

DEPTH (FEET)	TOTAL PRESSURE σ_t	EFFECTIVE STRESS* σ_v	MAXIMUM SHEARING STRESS τ	RATIO OF SHEARING STRESS / EFFECTIVE STRESS $= \frac{\tau}{\sigma_v}$
5	550	430	± 66	.15
10	1,100	680	± 132	.19
15	1,650	930	± 198	.21
20	2,200	1,180	± 264	.22
25	2,790	1,470	± 335	.23
30	3,380	1,760	± 406	.23
40	4,560	2,340	± 457	.23
60	7,400	3,980	± 888	.22

* In computing the effective stress, the ground water level has been assumed at a depth of three feet below the existing ground surface.

The conservative nature of these estimates can be seen by a comparison with published data by Seed and Idriss (1967). In this publication the computed response, using a more correct mathematical model of a similar deposit using a past earthquake record, the average ratio of shearing stress to effective stress was 0.22 For a postulated maximum acceleration of 12.5 percent of gravity. Using a similar ratio, a ratio of 0.21 for an acceleration of 12 percent of gravity could be used to estimate the shearing stresses in Table 4.1.

Laboratory cyclic loading tests on sands from the site were performed at several different confining pressures and sufficient cycles of stress were imposed to liquefy the samples. The results of these tests and other classification tests (summarized in Part 6 of this report) indicate that the sand has grain-size and dynamic properties similar to the sand which has been extensively tested. The results of the tests were published by Lee and Seed (1967). Based on the results of Lee and Seed, the ratio of shearing stress to confining pressure which would cause liquefaction in 10 cycles may be expressed as

$$\frac{\tau}{\sigma_v} = \frac{\text{relative density}}{200}$$

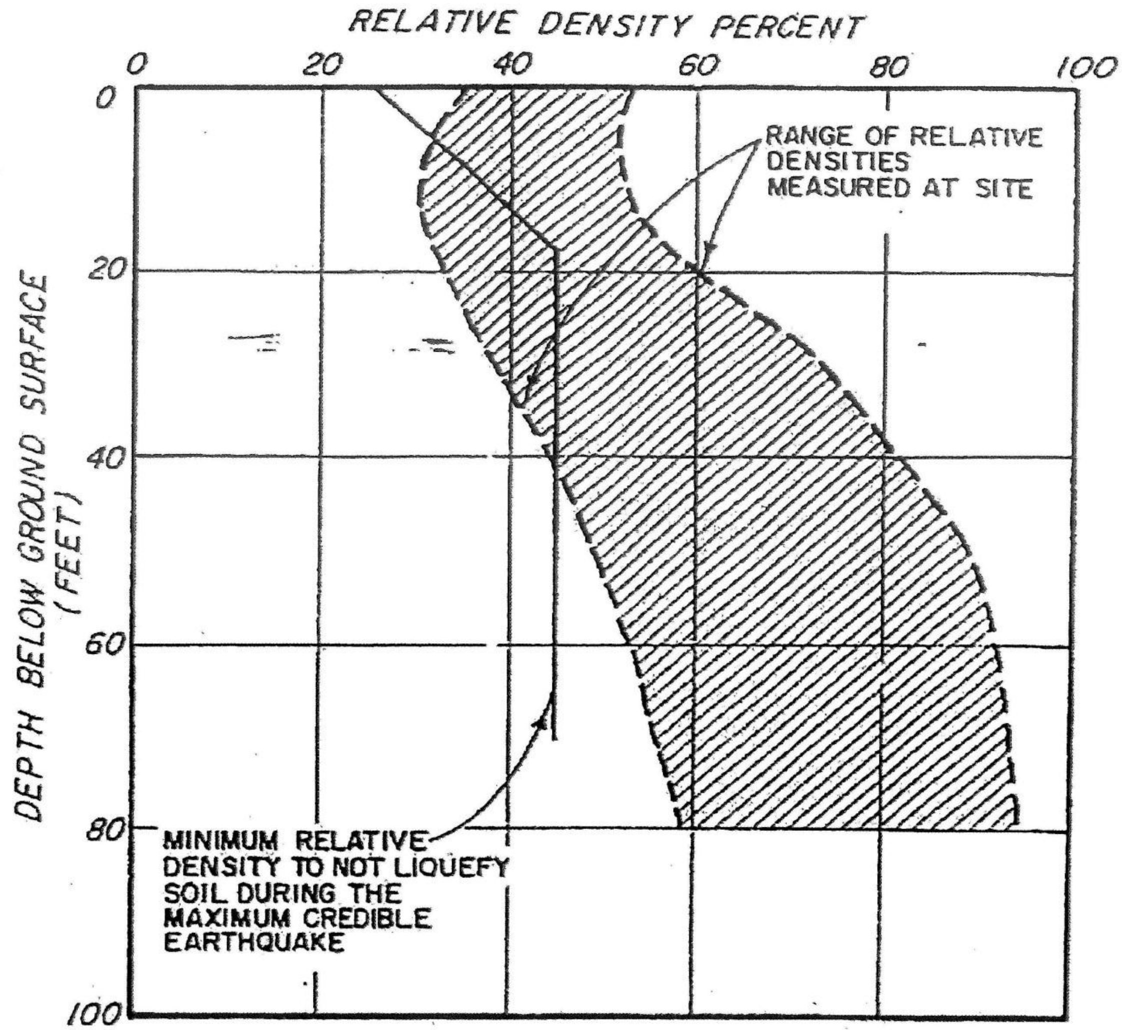
From this equation, the relative density to prevent liquefaction in the sands should exceed 31 percent in the upper five feet and should exceed 46 percent at and below 18 Feet. If only six cycles of strong motion were to occur the required relative densities would be less.

The blow count data obtained during the sampling operations have been converted to equivalent relative densities using the results of Gibbs & Holtz* together with values published by Terzaghi and Peck.** These have been compared with relative densities measured by the laboratory tests. The range of relative density values are plotted on Figure 4.1.

* Gibbs and Holtz, "Research on Determining the Density of Sand by Spool Penetration Testing," Proced. Fourth Internat'l Conf. on Soil Mechanics and Foundation Engr., London, Volume I, 1957.

** Tezaghi and Peck, "Soil Mechanics in Engr. Practice," John Wiley, New York, 1948 (eleventh printing 1960).

FIGURE 4.1



From Figure 4.1 it may be seen that the minimum relative densities measured at the site are slightly below this criterion for a range of depth. However, the average relative density would be above the minimum value throughout all the upper soils. Furthermore, the number of cycles required to cause liquefaction in the dynamic laboratory tests presented in Part 6 are well above the expected number of cycles which would be imposed by the maximum credible earthquake.

Therefore, we believe that liquefaction will not occur at the site during the postulated “maximum credible” earthquake. Reactor structures rounded at a depth of 50 to 60 feet appear to have an adequate margin of safety against liquefaction. For the critical structures founded at shallower depths, we do not believe the margin of safety against liquefaction is adequate for such an important structure.

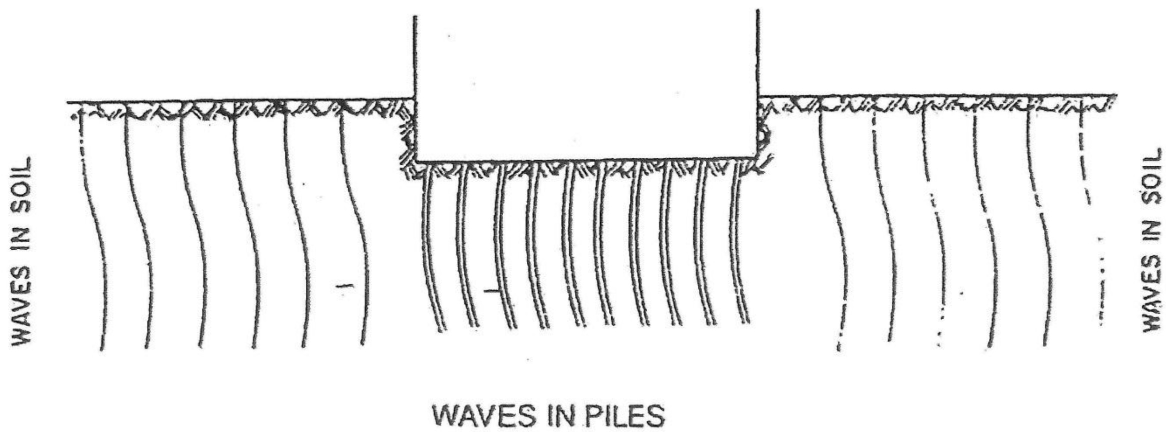
As will subsequently be recommended in Part 5 of this report, critical structures established above elevation 645 (approximately 50 feet below the existing ground surface) should be supported on pile foundations, or on spread or mat foundations established on sand which has been increased in density above elevation 645. If pile foundations are utilized, they should be displacement type piles which will increase the density of the sands in addition to providing structural support.

EFFECTS OF EARTHQUAKE ON SOIL-FOUNDATION SYSTEM

It is presently anticipated that the reactor structures will be earth supported on a mat foundation established at elevation 645, within the dense to very dense sandy soils. Should the reactor structures be established above elevation 645 they should be supported on either driven piles or on densified sand. Other critical structures will be established above elevation 645 and they should be supported on either driven piles or on densified sand.

As discussed previously, the results of the dynamic triaxial compression tests indicate an adequate' margin of safety against liquefaction for the reactor structures established at elevation 645. Additional dynamic triaxial compression tests and dynamic confined compression tests, the results of which are presented in Part 6, indicate that no significant loss in strength of the foundation materials will occur during an earthquake. Therefore, no reduction in the supporting capacity of the foundations will be required for earthquake loading conditions. We anticipate that these conclusions will be further substantiated during the performance of a comprehensive foundation investigation the site. Preliminary foundation design data have been formulated on this basis and are presented in Section 5 of this report.

Since it may be necessary to support certain structures on piles installed in the sandy soils, consideration should be given to possible additional stresses in piles caused by earthquake induced ground motion. The major stresses that would be introduced into piles would take place only if a void were assumed immediately below the pile cap. In this event the piles would have to transmit the base shear developed during an earthquake from the structure to soils surrounding the piles. It is unlikely that the void will occur between the supporting soils and the pile cap and the earthquake forces will be transmitted by friction and or lateral soil pressures. However, little data is available on this subject and a conservative approach should be used. Additional minor stresses could be developed in the piles as a result of the propagation of waves traveling at different velocities throughout the foundation soils and the piles. This effect is depicted on the sketch shown on the following page.



As a result of the interaction of soil and piles the increased stresses will be negligible. These stresses can be calculated after a pile type and size has been selected.

MODULI AND DAMPING VALUES

It is understood that deformation moduli and damping characteristics of the foundation soils may be used in developing the aseismic design of the proposed major structures.

Since soil is not a truly elastic medium, the commonly accepted terminology of modulus of elasticity and modulus of rigidity is not completely applicable. However, for ease of subsequent discussion, these terms will be used to describe soil properties which follow the general definitions used for elastic media. Although soils are not fully elastic media, the assumption of stress-strain linearity can usually be made for a particular stress level range. Thus, the assumption of elastic theory is fairly suitable for use in measuring moduli of elasticity and rigidity. For competent rock, the assumption of a linear stress-strain relationship is generally good.

The moduli and damping values are presented in Table 4.2, Moduli and Damping Values, and are believed to be applicable in the range of loading that might be experienced by the foundation materials during earthquake loading. The moduli of elasticity and rigidity and the damping values presented in this table were evaluated on the bases of both field and laboratory dynamic tests. In addition, damping values were correlated with data available from pertinent publications and from the results of large scale field tests of missile stands subjected to dynamic loading which were performed by Dames & Moore.

TABLE 4.2 MODULI AND DAMPING VALUES

			DAMPING FACTOR*	
MATERIAL	**MODULUS OF ELASTICITY LBS./SQ. FT.**	**MODULUS OF RIGIDITY LBS./SQ. FT.**	**DESIGN EARTHQUAKE PERCENT**	**MAXIMUM CREDIBLE EARTHQUAKE PERCENT**
Sandy Soil at Elevation 665	7×10^6	2.5×10^6	5 to 10	10 to 15
Sandy Soil at Elevation 645	9×10^6	3.1×10^6	5 to 7	7 to 12
Sandstone	250×10^6	100×10^6	1	1

* Expressed as percentage of critical damping.

LIST OF PLATES

The following Plates are attached and complete Part 4:

Plate 4.1 - Typical Geologic Column

Plate 4.2 - Regional Geologic Structure

Plate 4.3 - Regional Earthquakes

Plate 4.4 - Earthquake Wave Amplification Ratios

Plate 4.5 - Recommended Response Spectra – Design Earthquake - 6% Acceleration

Plate 4.6 - Recommended Response Spectra – Maximum Credible Earthquake - 12% Acceleration

PART 5 - FOUNDATION CONSIDERATIONS

COMPREHENSIVE FOUNDATION INVESTIGATION

A comprehensive foundation investigation has been performed by Dames & Moore. The scope and purposes of the investigation were as follows:

1. To evaluate the type or types of foundations which will be suitable for the support of the various plant structures.
2. To develop finalized foundation design data. These data include recommended bearing capacities and estimated settlements for mat and spread type foundations.
3. To establish criteria for the densification and/or removal of and recompaction of soils intended for the support of mat foundations.
4. To estimate the magnitude and time-rate of settlement of foundations under the anticipated static and dynamic loading conditions, including an evaluation of the differential settlements which may be expected between the various units.
5. To provide recommendations for the design of structures to withstand hydrostatic uplift and lateral soil and water pressures.
6. To provide recommendations regarding site preparation, dewatering, excavating, bracing and sloping of excavations, and methods of placing and compacting available earth materials in filling and back-filling operations.
7. To supervise and evaluate a test program of compaction of the on-site soils by the vibroflotation method.
8. To discuss possible design and construction problems associated with the installation of future units.

The results of our field explorations and laboratory tests are presented in Part 6 of this report. The location of plant facilities and the location of test borings drilled for the comprehensive foundation investigation are shown on Plate 5.1.

SUMMARY

The results of the field explorations, laboratory testing and engineering analyses performed in connection with the comprehensive foundation investigation have substantiated that the Prairie Island site is satisfactory, from a foundation standpoint, for the construction of the Proposed Nuclear Power Plant. These studies have also confirmed that the major problem associated with the foundation support of critical plant structures would be the low margin of safety of the supporting soils above elevation 645, against the possibility of liquefaction during ground accelerations associated with the postulated maximum credible earthquake.

Various methods of providing satisfactory foundation support together with a suitable margin of safety against liquefaction are possible. These methods are identified below and are considered equally satisfactory. The final choice of a method of foundation support may be based on consideration of cost and construction scheduling. Suitable methods are:

1. Dewatering the area to be occupied by the critical plant structures, excavating the granular soils to elevation 645, and subsequently replacing the granular soils as compacted fill under controlled conditions.
2. Densifying the in-place granular soils above elevation 645 by the vibroflotation method performed under controlled conditions.
3. Supporting plant structures on driven displacement type piles deriving their support primarily in friction from the soils below elevation 645.

We understand that cost studies have been performed which indicate Methods 1 and 2 (Soil Densification) to be much more economical foundation solutions than Method 3 (Pile Support). Our foundation investigation has therefore been primarily directed toward establishing criteria for providing a satisfactory margin of safety against liquefaction, assuming that the proposed structures will be earth-supported on mat or spread type foundations, and toward developing data required for site preparation and foundation design for either of these methods.

It has been concluded that satisfactory earth support can be provided for critical structures by densifying the granular soils above elevation 645 to a minimum relative density of 85 percent. This conclusion results from detailed liquefaction studies, the results of which are summarized in Part 4 of this report. The range of relative soil densities measured at the site are summarized in Figure 4.1, P. 4.19, and the in-plane densities are compared to the minimum relative densities required to prevent liquefaction. Achieving a minimum relative density of 85 percent above elevation 645 will, in our opinion, provide a satisfactory margin of safety against liquefaction at the Prairie Island site.

In connection with soil densification by dewatering, excavating and recompacting, laboratory and field test results are presented in this report which indicate the feasibility of this method. In addition, criteria are presented for proper placement and compaction of fill soils under controlled conditions.

In connection with soil densification by vibroflotation, the feasibility and effectiveness of this method were verified by the performance of a full scale field test compaction program. The program consisted of compacting the in-place granular soils between elevations 645 and 675 in two areas located near the northeast corner of the proposed power plant. The test compaction program was evaluated as satisfactory and the detailed results of our evaluation were provided in a separate report. Certain data summarizing the results of the vibroflotation test compaction program are also contained this report.

Assuming that satisfactory densification of on-site soils will be achieved in accordance with the criteria recommended herein, we have analyzed the bearing capacity and consolidation characteristics of the soils from which foundation support will be derived. The resulting foundation design criteria are presented in this section of the report.

SITE PREPARATION AND EARTHWORK

General

Site preparation operations required for the preparation of foundation soils for the support of critical structures will consist essentially of densifying the granular soils above elevation 645. Two methods of densification are considered suitable. These are: (1) densification of the natural soils by vibroflotation and, (2) densification of the natural soils by dewatering, excavating to elevation 645, and replacing and compacting the natural soils under controlled conditions. It is considered that either method would be equally satisfactory and that the final selection of the method of densification may be based upon economic scheduling criteria.

Compaction By Vibroflotation

<u>Vibroflotation Test Compaction Program</u> - In order to evaluate the feasibility of utilizing the vibroflotation compaction method at the Prairie Island Site, two test areas, approximately 17 feet by 19 feet in plan dimensions, were laid out near the northeast corner of the proposed power house. Surface grades in these areas were cut to approximately elevation 680. An attempt was made to compact the granular soils in each area between elevations 645 and 680. It was recommended by Dames & Moore that a minimum relative density of 85 percent be attained by the vibroflotation compaction process. The vibroflot was inserted at a minimum of eight penetration points in a symmetrical pattern within each area.

The compaction results achieved in each area were evaluated by drilling at least two borings in each area to an elevation below 645 prior to compaction by vibroflotation, and by drilling at least six borings in each area subsequent to compaction by vibroflotation. Relative densities before and after compaction were determined indirectly by converting blow count data obtained from standard penetration tests 1:0 relative densities utilizing the research results of Gibbs and Holtz, and by laboratory tests on relatively undisturbed samples extracted from borings with a Dames & Moore soil sampler.

The results of the compaction effort in each of the two test patterns were as follows:

> <u>Test Pattern 1</u> - The vibroflot could not be advanced to elevation 645. The upper five feet to seven feet of soil was not compacted to a relative density of at least 85 percent. The soils between the lowest elevation attained by the vibroflot and the upper soils were compacted to at least 85 percent relative density.
>
> <u>Test Pattern 2</u> - Based on the results achieved in Test Pattern the vibroflot was modified by increasing its weight and by adding additional water jets to the outside of the vibroflot in an attempt to penetrate to elevation 645. In addition, the vibroflot was withdrawn at a slower rate near the ground surface in an effort to compact the near surface soils.

The vibroflot attained the required elevation 645 in each of the eight penetrations. The upper five to seven feet of soil was not compacted to the required density. This is believed due to low confining pressures near the ground surface. The soils between elevation 645 and the upper five to seven feet of surface soils were compacted to densities in excess of 85 percent relative density.

Summaries of the results of the vibroflotation test compaction program are presented on Plates 5.2 through 5.5. The results of our evaluation can be summarized as follows:

1. The vibroflotation compaction method was successful in achieving a minimum relative density of 85 percent between approximately elevation 673 and the depths penetrated. This conclusion is substantiated by standard penetration test results as well as density determinations on relatively undisturbed soil samples.

2. Less than adequate densities were noted, in some cases, above elevation 673 for both test patterns. This is believed to be the results of too rapid withdrawal of the vibroflot at the conclusion of the penetrations and also due to insufficient confining pressures near the existing ground surface (elevation 680 during the time of the test.)

Recommended Design and Quality Control Criteria

If soil densification by vibroflotation is selected as the preferred method of site preparation, it is recommended that all soils between elevation 645 and foundation levels for critical structures be compacted to at least 85 percent relative density. The compacted zone of soil should extend laterally at least 30 feet outside the edge of the reactor foundations and at least 25 feet outside the edge of other major structure foundations. It should be anticipated that upper five to seven feet of the densified soil zone will probably not receive sufficient compaction by the vibroflotation method, and other mechanical means should be utilized to compact these soils if they extend above level of foundation support.

It is recommended that close and continuous supervision of the compaction operations be exercised by Dames & Moore engineering personnel in order to verify the following:

1. Penetration of the vibroflot to elevation 645.

2. Satisfactory compaction to a relative density of at least 85 percent within the entire zone specified.

3. Immediate correction of any deficiencies noted.

4. Compaction of the upper five to seven feet of soils by other mechanical means, if insufficient compaction by vibroflotation is noted.

The quality control program for the vibroflotation program should be based upon inspection by visual observation and by test borings drilled 25 to 50 feet on centers. Standard penetration tests are considered to be the most direct and expedient means of determining whether the required relative densities are attained. However, other means of density evaluation, such as direct density measurement on relatively undisturbed samples extracted from test borings, should also be used periodically for check and correlation purposes or should be used if standard penetration tests yield unsatisfactory or marginal results.

When insufficient compaction is noted by observation or testing, the supervising field engineers should direct that additional compactive effort be applied until the specified degree of compaction is achieved and verified.

Compaction by Dewatering Excavating Filling and Compacting

Dewatering - If compaction by dewatering, excavating, filling and compacting operations is utilized, it will be necessary to excavate, replace and compact all of the granular soils above elevation 645 in areas to be occupied by critical structures. Since the ground water level is at approximately elevation 675, it will be necessary to dewater the excavation, and to maintain the ground water at a sufficiently low level to allow for subsequent replacement and compaction of granular materials under dry conditions. It is recommended that the ground water level be lowered to an elevation at least three feet below the bottom of the excavation, and that it be maintained at least three feet below the working surface during filling operations.

In order to evaluate the feasibility of dewatering the excavation to the depth and lateral extent required, a pumping test was performed. The test well was located approximately 600 feet from the Mississippi River. Five observation borings were also installed as part of the pumping test program. A description of the pumping test and our evaluation of the data obtained are presented in Part 3 of this report.

The estimated hydraulic properties presented in Part 3 of this report are not intended as recommended design values for a dewatering system at the Prairie Island site. These values are intended only for feasibility and cost studies.

Data obtained from the pumping test indicate that a deep well system or a multi-stage wellpoint system will be required to dewater the site. The design of the dewatering system should provide for periods of high water in the Mississippi River, when the ground water level at the site will rise in conjunction with the river level.

It is suggested that standby generating capacity be provided to operate the dewatering system so as to prevent flooding of the excavation in the event of a power interruption at the site.

<u>Excavating</u> - Excavations in the areas to be occupied by critical structures should extend to elevation 645. It is recommended that the bottom of the excavations extend laterally at least 30 feet outside the edges of the reactor foundations and at least 25 feet outside the edges of other major structure foundations.

It is considered that the banks of the dewatered temporary excavations will be essentially stable on a slope of one vertical to one and one quarter horizontal. However, it is anticipated that localized sloughing and erosion of the banks will occur.

Should it be desired to cut the banks of excavations at steeper slopes, a bracing system will be required. The design of the bracing system will depend on the depth and lateral extent of the excavation and the proximity of external loads which could induce lateral forces against the bracing. Dames & Moore would be pleased to provide design data for lateral bracing at such time as the geometric configurations of the excavations and the locations and magnitudes of external loads are determined.

Filling and Compacting - It is recommended that all fills and backfills which will be subjected to structural or vehicular loads be composed of clean granular material such as the soils which will be removed from on-site excavations.

All fills placed for the support of the major structures should extend laterally at least 30 feet outside the edges of the reactor foundations and at least 25 feet outside the edges of other major structure foundations. It is recommended that fill materials intended for structural support be compacted to a dry density of at least 100 percent of the maximum dry density as determined by the American Association of State Highway Officials Test Designation T 180-57*.

Fills placed for the support of non-critical structures, roadways and parking areas, should be compacted to a dry density equal to at least 95 percent of the maximum dry density. Fills placed to merely raise the grade, and which will not be subjected to structural or vehicular loads, should be compacted to a dry density equal to at least 90 percent of the maximum dry density.

It is recommended that all filling operations be performed under dry conditions. Fill materials should be placed in layers approximately 8 to 12 inches in loose thickness and each layer of fill should be compacted in accordance with the criteria outlined above.

It is recommended that all filling and compacting operations be performed under the technical supervision of Dames & Moore engineering personnel. The quality control program should consist of continuous observation of filling and compacting operations and the performance of in-place density tests in the compacted fill to verify that each layer of fill material has been compacted to the required densities. When insufficient compaction is noted by observation or testing, the supervising field engineers should direct that additional compactive effort be applied until the specified degree of compaction is achieved and verified.

FOUNDATION DESIGN CRITERIA - MAJOR STRUCTURES

Design Considerations

The major plant structures are the power house and the screen house. The power house will include two reactors, two turbines, fuel storage facilities and condenser pits. The layout and loading of the plant facilities are shown on Plate 5.6. Loading on major units is summarized in the following paragraphs. The plant grades have been established based on a high water elevation of 693.5.

* The 100 percent AASHO density criterion is essentially equivalent, in terms of weight per unit volume, to the 85 percent relative density criterion specified in the previous section for compaction by vibroflotation. However, the AASHO criterion is utilized herein because it is a more conservative term in earthwork operations and is more readily understood by earthwork contractors.

Reactors - The structures housing the reactors will be cylindrical in shape and will extend 100 feet or more above the ground surface. Each reactor structure will be approximately 130 feet in diameter and will be supported on a mat foundation established at elevation 674. The mat foundations will impose loads dead plus live on the order of 6,000 pounds per square foot. The maximum toe pressure will be 11,000 pounds per square foot due to the total of dead, plus live, plus seismic loads.

Fuel Storage Facilities - The fuel pit will be established at elevation 691 and will impose dead plus live loads on the order of 3,200 pounds per square foot.

The refueling tanks will be established at elevation 691 and will impose dead plus live loads on the order of 4,000 pounds per square foot.

Increases in foundation loading due to seismic effects have not yet been established.

Turbines - Each of the two turbines will be housed in structures approximately 150 feet by 230 feet in plan dimensions. The turbine support foundations will impose dead plus live loads on the order of 3,800 pounds per square foot. Exterior wails and interior and exterior columns of the turbine building may be supported on individual spread foundations, or may be carried on a common mat foundation. It is estimated that a mat foundation would be established at approximately elevation 688 and that it would impose a dead plus live load pressure on the order of 3,000 pounds per square foot. If individual spread foundations are utilized, foundation loads would be as indicated on Plate 5.6.

The condenser pit area within each turbine building will be approximately 60 feet by 120 feet in plan dimensions and will be established at elevation 674.

Increases in foundation loading due to seismic effects have not as yet been established.

Auxiliary Building - The auxiliary building is located south of the turbine structures. It is planned to support the auxiliary building on a mat foundation which will impose dead plus live loads on the order of 4,000 pounds per square foot.

Increases in loading due to seismic effects have not yet been established.

Recommended Design Data

Reactors - It is recommended that each of the proposed reactor structures be supported on mat foundations established on compacted granular soils. Engineering analyses have been performed to evaluate the ultimate bearing capacity which can be developed by the supporting soils and to estimate the settlement which will occur under the design loads. The results of our analyses are tabulated below:

	FACTOR OF SAFETY			
ULTIMATE BEARING CAPACITY (LBS./SQ. FT.	**DL + LL**	**DL + LL + SEISMIC**	**ESTIMATED SETTLEMENT FOR DL + LL CONDITION (INCHES)**	**ESTIMATED ADDITIONAL SETTLEMENT FOR SEISMIC LOAD CONDITION (INCHES)**
100,000	17	9	1/4	1/4 to 1/2

The above estimated settlements will occur practically simultaneously with the initial application of the loads.

The estimated settlements under static loads were computed utilizing an average coefficient of subgrade reaction equal to 500 tons per cubic foot for the densified granular soils above elevation 645, and a very conservative average value of coefficient of subgrade reaction equal to 150 tons per cubic foot for the natural granular soils below elevation 645.

Under seismic loads, possible additional settlements are expected to be on the order of one to two times the values estimated for static loading. These additional settlements would possibly occur due to densification of the natural soils below elevation 645. Due to the presence of a 30 foot layer of densely compacted materials underlying the foundations above elevation 645, the effect of the additional settlements would be uniform for all structures. No differential settlements between structures are expected under seismic loading except slight elastic deformations that may be calculated by assuming a soil deformation modulus (Pv)* equal to 2,700,000 pounds per cubic foot.

* (Pv) is the pressure corresponding to unit deflection of the ground. See paper by R.G. Merritt and G.W. Housner: “Effect of Foundation Compliance on Earthquake Stresses in Multistory Buildings.”

Bulletin of Seismological Society of America, Volume 44, p. 551-570.

Other Major Structures

It is considered that the auxiliary building, turbines, condenser pits, fuel storage facilities, and columns and walls of the turbine structures may be satisfactorily supported on mat foundations at the planned elevations.

If desired, columns and wails of the turbine structures may be supported on individual spread foundations. All foundations should be established on compacted granular soils.

Assuming that major structures will be supported on mat foundations, engineering analyses have been performed to evaluate the ultimate bearing capacity which can be developed by the supporting soils and to estimate the settlement which will occur under the design loads. The results of our analyses are tabulated below:

DESIGN DATA
MAJOR STRUCTURES SUPPORTED ON MAT FOUNDATIONS

FACILITY	ULTIMATE BEARING CAPACITY (LBS./SQ. FT.)	INDICATED BEARING PRESSURE* (LBS./SQ. FT.)	FACTOR OF SAFETY	ESTIMATED SETTLEMENT FOR DL + LL CONDITION (INCHES)
Auxiliary Building	60,000	4,000	15	0.2
Turbine Support Pad	30,000	3,800	7.9	0.2
Turbine Structure	100,000	3,000	33.3	0.1
Fueling Pit	30,000	3,200	9.4	0.1
Refueling Tanks	30,000	4,000	7.5	0.2
Screen House	40,000	6,000	6.7	0.2

* The indicated bearing pressures include dead and live loads.

Seismic load data for the above facilities have not as yet been provided to us. However, the additional settlements which would occur due to seismic loads would probably be on the order of one to two times the settlements estimated for static conditions as tabulated above. In no case are additional settlements under seismic loads expected to exceed one-half inch. As discussed in the preceding section, the effects of additional settlements under seismic loads would be relatively uniform for all structures. No differential settlements between structures are expected under seismic loading except slight elastic deformations.

The above tabulated settlements due to static and dynamic loads will occur practically simultaneously with the initial application of the loads.

The estimated settlements were computed utilizing the static and dynamic subgrade coefficients presented in the previous section.

If columns and walls of the turbine structures are supported on individual spread foundations, it is recommended that the foundations be proportioned utilizing a bearing pressure of 8,000 pounds per square foot for dead and live loads. The recommended bearing pressure may be increased to 10,000 pounds per square foot for maximum edge pressures under dead, live and seismic loads. The recommended bearing pressures have been computed assuming that the minimum depth of foundation embedment will be six feet. The recommended values contain a factor of safety on the order of 3 for dead plus live load, and a factor of safety on the order of 2.5 for dead, live and seismic loads. The recommended bearing pressures are net values, and the weight of concrete in the Foundations and the weight of backfill over the foundations may therefore be neglected in proportioning the foundations.

We estimate that settlements of individual spread foundations, designed in accordance with the above recommendations will be on the order of one-quarter inch or less. The settlements will occur essentially simultaneously with the application of the loads.

<u>Walls Below Grade</u> - Building and pit wails extending below grade will be subjected to lateral loads imposed by backfill and hydrostatic pressure. It is recommended that all walls be backfilled with clean granular material in accordance with the criteria presented in a previous section. In the design of the walls to resist the lateral loads imposed by a saturated granular backfill during flood conditions, it is recommended that the backfill be considered to act as an equivalent fluid with a density of 80 pounds per cubic foot below the design high water level. Above the design high water level, the backfill may be considered to act as an equivalent fluid with a unit weight of 40 pounds per cubic foot.

<u>Earth Supported Floor Slabs</u> - In the design of earth supported floors and pit floors, consideration should be given to hydrostatic uplift pressures which will act beneath the floors. For purposes of design, it is recommended that all floor slabs established below the design high water level be designed to resist full hydrostatic uplift pressures. Due to the very permeable nature of the granular soils at the site, we do not believe that it would be feasible to attempt to provide underfloor drainage facilities to prevent or dissipate the development of hydrostatic pressures.

DESIGN CRITERIA - APPURTENANT FACILITIES

Design Considerations

The appurtenant structures include the water intake canal, dikes and cooling towers.

Water Intake Canal - It is presently planned to dredge or otherwise excavate an open canal from the Mississippi River to the proposed screenhouse. The banks and bottom of the canal would be formed in essentially granular soils.

The water intake canal will be required to function during the safe shutdown of the plant under the maximum credible earthquake condition.

Dikes - The locations of two proposed dikes are shown on Plate 5.1. It is presently planned to construct the dikes with side slopes of one vertical to three horizontal. The crest of the dikes will be at elevation 694 and will be at least 15 feet in width.

River Bank - The stability of a typical section of existing river bank was investigated. Factors of safety for various reconstructed slope sections were also considered.

Cooling Towers - The locations of the proposed cooling towers have not as yet been provided to us. Therefore, borings have not yet been drilled to ascertain the subsurface conditions for these structures. It is anticipated that the cooling towers will impose relatively light foundation loads.

Recommended Design Data

Water Intake Canal - It is considered that ground accelerations caused by the postulated maximum earthquake could cause loss of strength, caving and sloughing of the banks of the water intake canal, resulting in a reduction in capacity of the canal. Consideration should be given to densifying the soils in the vicinity of the intake canal so that no appreciable loss of strength would occur during earthquake loading. Alternately, some other positive means of water supply would be required to facilitate safe shutdown of the plant.

Dikes - It is considered that the proposed dikes will be stable under normal conditions at the planned slopes of one vertical to three horizontal. The dikes, if composed of compacted materials, would probably also remain intact under earthquake loading. However, the underlying foundation materials would have a low margin of safety against liquefaction and a foundation Failure could occur during the maximum credible earthquake. The dikes should therefore not be considered to remain in operation during maximum earthquake loading.

River Bank - An analysis has been performed to evaluate the stability of the existing river bank at coordinate N594,000. In our analysis, we assumed that the bank was devoid of all trees and surface vegetation, that a high water condition was present, and that the water level in the river had dropped rapidly. The analyses indicates that the existing bank, for the assumed conditions, has a factor of safety less than 1.0. We attribute the present stability of the bank to the deep root system of the trees. Should these trees be removed, the bank would be unstable and remedial measures to stabilize the slope would be required. Factors of safety for various reconstructed slopes are presented in tabular form on Plate 5.7, Slope Stability Studies.

Cooling Towers - Provided the subsurface conditions at the locations of the proposed cooling towers are similar to the subsurface conditions in the vicinity of the proposed power house, we presently anticipate that the cooling towers may be supported on mat foundations established at a minimum depth of two feet below the lowest adjacent grade and may be designed to impose a bearing pressure of up to 2,500 pounds per square foot. No soil densification will be required for cooling tower support, provided that liquefaction does not control Foundation design and provided that moderate settlements can be accepted.

Factors of safety and estimated settlements can be provided at such time that foundation sizes and configurations are provided to us and after subsurface conditions in the area have been evaluated.

PROPOSED FUTURE UNIT NUMBER 3

The results of our subsurface investigation on land indicate that the subsurface conditions across the site are relatively uniform. On the assumption that Future Unit 3 is similar to Proposed Units 1 and 2, it is anticipated that construction procedures and the behavior of foundations would be similar.

To accomplish subgrade preparation for Future Unit 3, it will be necessary to dewater, excavate to elevation 645, and recompact the granular soils to foundation level, or to densify the soils above elevation 645 by the vibroflotation method. Excavating immediately adjacent to Units 1 and 2 would require that excavations be braced to avoid possible loss of ground under in-place foundations. Vibroflotation may cause undesirable vibrations during plant operations.

In order to avoid difficult construction operations during foundation preparation for Future Unit 3, it is suggested that all or part of the area to be occupied by the Future Unit 3 be prepared in advance by compacting the granular soils above elevation 645 in conjunction with the site preparation operations for Proposed Units 1 and 2. If only a part of Future Unit 3 area is prepared, it is recommended that the prepared area extend from the edge of the Foundations of Proposed Units 1 and 2 a lateral distance of at least 70 feet. Recommendations for soil densification presented in foregoing sections of this report are considered applicable for a future adjacent unit.

LIST OF PLATES

The following Plates are attached and complete Part 5:

Plate 5.1 - Plot Plan

Plate 5.2 - Summary of Standard Penetration Tests Test Pattern 1

Plate 5.3 - Summary of In-Place Density Tests Test Pattern 1

Plate 5.4 - Summary of Standard Penetration Tests Test Pattern 2

Plate 5.5 - Summary of In-Place Density Tests Test Pattern 2

Plate 5.6 - Proposed Layout

Plate 5.7 - Slope Stability Studies

PART 6 - FIELD EXPLORATIONS AND LABORATORY TESTS

FIELD EXPLORATIONS

General

Field explorations were performed to evaluate the geologic, hydrologic, seismologic, and foundation engineering characteristics of the site. The field exploration program consisted of the following:

1. A geologic reconnaissance of the general area and site,

2. A test boring program,

3. Geophysical explorations which included geophysical refraction surveys, shear wave velocity surveys, an uphole velocity survey, and micromotion measurements.

Descriptions of the field exploration program are presented in subsequent paragraphs.

The field exploration program was conducted under the technical direction and supervision of Dames & Moore Geologists, Engineering Seismologists, Geophysicists, and Soil Mechanics Engineers. Surveying services necessary to determine the locations and surface elevations related to the field explorations were provided by John . Gorman, Inc. of Minneapolis, Minnesota and the Northern States Power Company.

Geologic Reconnaissance

A geologic reconnaissance of the general area surrounding the site was undertaken for the purpose of examining surface features which would aid in the evaluation of the geologic characteristics of the area. The site was inspected with respect to topography, shoreline features, surface soils, drainage and other related surface features.

Geologic literature and aerial photographs of the site were studied. Representatives of local, state and federal agencies, private organizations and universities were interviewed to obtain all available geologic data.

The results of the geologic investigation are discussed and evaluated in Part 2 of this report.

Test Boring Program

The subsurface conditions at the site were initially investigated by drilling 10 test borings, Borings 3 through 12, at the locations shown on Plate 2.4 (in Part 2 of the report) to depths ranging from 174 to 213 feet below the existing ground surface. The subsurface conditions at the locations of the proposed facilities were subsequently investigated by drilling 25 test borings, Borings 13 through 38 (with the exception of Boring 33) at the locations shown on Plate 5.1 (in Part 5 of the report) to depths ranging from 74 feet to 219 feet below the ground surface. All of the borings were drilled utilizing truck-mounted rotary wash and rotary auger type drilling equipment.

Borings 1 and 2 were drilled at the site in 1959 by and under the supervision of Soil Exploration Company of Minneapolis, Minnesota and the results of these borings have been provided to Dames & Moore for purposes of correlation. The logs of these borings are not presented herein. The soils penetrated by these borings were comparable to those in Borings 3 through 12. Standard penetration tests were performed during drilling, utilizing a two-inch diameter standard split-spoon sampler.

The drilling operations were supervised by a Dames & Moore Soils Engineer, who maintained a log of the borings, obtained undisturbed samples of the soil utilizing a Dames & Moore soil sampler, obtained standard penetration samples utilizing the two-inch diameter standard split-spoon sampler, and supervised the diamond core drilling operations performed to extract cores of the underlying rock. Graphical representations of the soils and rock encountered in these borings are shown on Plates 6.1 through 6.35, Log of Borings. The method utilized in classifying the soils encountered by the borings is defined on Plate 6.36, Unified Soil Classification System. 01367047

Undisturbed samples of the soils penetrated by the Dames & Moore borings were obtained in a Dames & Moore Soil Sampler of the type illustrated on Plate 6.37, Soil Sampler Type U. The Dames & Moore Sampler was driven utilizing a 350 pound weight falling approximately 30 inches. The method of obtaining the samples is indicated and explained on the Dames & Moore Log of Borings. Rock cores were obtained from these borings utilizing NX size coring equipment.

The ground surface elevation is shown above the 109 of each boring and refers to Mean Sea Level Datum - 1929 Adjustment.

Geophysical Explorations

General - geophysical explorations were made to determine dynamic properties of the underlying soils and rock. The explorations conducted included geophysical refraction surveys, shear wave velocity surveys, an uphole velocity survey, and micromotion measurements. The purposes of the explorations were to measure compressional and shear wave velocities, interval velocities, and any possible predominant period of ground motion of the site. The locations of these surveys and observations are shown on Plate 6.38.

Geophysical Refraction Surveys - A 12-channel Electrotech Refraction Seismograph was used to record the results of the deep refraction surveys. The geophysical refraction surveys were performed along three lines for a total length of approximately 3,900 feet. Explosive charges (Nitramon) were placed in drill holes at the ends of these lines at depths of approximately 20 to 25 feet, which is below the water table. Standard geophones were located at 10, 50, or 100 foot intervals along these lines. The time-distance data resulting from the surveys were plotted. and average straight line slopes were drawn through the plotted points. The velocity of compressional wave propagation in the upper soils and underlying rock was computed from the plotted data. The results of the deep geophysical refraction surveys are presented on Plate 6.39, Geophysical Refraction Survey - Compressional Wave Velocities.

Shear Wave Velocity Survey - Shear wave velocities were computed from the recordings of the 12-Channel Electrotech Refraction Seismograph. Hall-Sears one-second horizontal seismometers, oriented transverse to the direction of propagation of the shock waves were used. Seismometers were located at 50 or 100 foot intervals along a portion of refraction survey lines. The shot holes were located at varying distances of up to 1400 feet from the farthest geophone. The survey indicated that the sand deposits below a depth of 20 feet have a shear wave velocity of approximately 1900 to 3000 feet per second and the sandstone bedrock has a shear wave velocity of approximately 5,000 feet per second.

Uphole Velocity Survey - An uphole velocity survey was performed in Borings 11 and 12. The test borings penetrated approximately 30 feet into the underlying sandstone to a depth of approximately 210 feet. The survey was performed with a 12-Channel Electrotech Refraction Seismograph using caps and boosters as the source of energy. Repeated shots were recorded of the explosions of the caps at closely spaced intervals in the test boring.

The uphole velocity survey was made to determine vertical interval compressional velocities of the underlying alluvial deposits and sandstone. The results of this survey are presented in Table 6.1.

TABLE 6.1 SUMMARY OF UPHOLE INTERVAL VELOCITY SURVEY

MATERIAL	DEPTH FEET	INTERVAL VELOCITY FT./SEC.
Sand	30 to 60	4750
Sand With Gravel	60 to 80	7280
Sand	80 to 110	6570
Sand	110 to 130	4760
Sand With Gravel	130 to 150	7100
Sand With Gravel	150 to 180	6650
Soil-Rock Interface	180 to 200	4300
Sandstone	200 to 210	10,000

Details of compressional wave velocity distributions are revealed by comparison of the results of this uphole interval velocity survey with the results of the deep refraction surveys. An occasional layer of hard (higher velocity) material is indicated by the deviation from a gradual increase in velocity with depth. The average velocity of 5500 feet per second is indicated by the deep refraction survey below a depth of 15 to 20 feet. Actually two zones are indicated: (1) a shallow layer from approximately 20 to 55 feet with a velocity of 4750 feet per second, and (2) a deeper layer extending to the bedrock surface with a velocity of 6300 feet per second. An erosional surface with boulders is indicated by the quite low velocity noted near the bedrock contact. The average value for the deep refraction velocity is probably more representative for the deep bedrock than the value shown for the interval near the top of the bedrock.

Micromotion Measurements - Micromotion measurements were made in the proposed plant area using the Dames & Moore Microtremor equipment. This equipment is a highly sensitive electronic vibration recording device capable of magnification of up to 150,000 times and is accurate over a range of periods of 0.03 to 2.5 seconds, or a frequency range of about 0.4 to 30 cycles per second. Micromotions of the overburden materials were recorded at the locations shown on Plate 6.38. The principal background motions measured had periods of 0.08, and from 0.32 to 0.46 seconds (12, and 2 to 3 cycles per second). Analyses of the microtremor records were utilized in our engineering seismology studies. The original microtremor vibration records are retained in our files.

LABORATORY TESTS

General

Samples extracted from the test borings were subjected to laboratory tests to evaluate the physical properties of the soils present at the site. The laboratory program included the following tests:

1. Static Tests
   a. Direct Shear
   b. Consolidation
   c. Rock Compression
2. Dynamic Tests
   a. Triaxial Compression
   b. Liquefaction
   c. Confined Compression
   d. Shockscope
3. Other Physical Tests
   a. Moisture and Density Tests
   b. Particle Size Analyses
   c. Maximum and Minimum Density Determinations
   d. Specific Gravity
   e. Compaction Tests

Static Tests

Direct Shear Tests - Selected representative soil samples recovered from the borings were tested to evaluate their strength characteristics. These tests were performed in order to evaluate the bearing characteristics of the soils underlying the site. The direct shear tests were performed on Dames & Moore samples in the manner described on Plate 6.40, Methods of Performing Direct Shear and Friction Tests.

A load-deflection curve was plotted for each strength test and the yield shearing strength of the soil was determined from this curve. Determinations of the field moisture content and dry density of the soil were made in conjunction with each strength test. The results of the strength tests and the corresponding moisture content and dry density determinations are presented on the Log of Borings included in Part 6. The method of presenting the Dames & Moore test data is described by the Key to Test Data shown on Plate 6.36

Consolidation Tests - Relatively undisturbed and recompacted samples of the natural sands obtained using the Dames & Moore sampler were subjected to consolidation tests. These tests were performed in order to evaluate the compressibility characteristics of these soils. The method of performing consolidation tests is described on Plate 6.41, Method of Performing Consolidation Tests. The results of these tests and the associated moisture content and dry density determinations are presented on Plates 6.42, 6.43 and 6.44, Static Consolidation Test Data.

Rock Compression Tests - Rock compression tests were performed on selected samples of the sandstone which immediately underlies the overburden soils. The rock compression tests were performed to evaluate the strength and elasticity characteristics of the bedrock. The tests were performed by the Robert W. Hunt Company on cores from Borings 4, 11 and 12, and the results of the rock compression tests are presented in Table 6.2, Rock Compression Test Results.

TABLE 6.2 ROCK COMPRESSION TEST RESULTS

BORING NUMBER	DEPTH FEET	ULTIMATE COMPRESSIVE STRENGTH LBS./SQ. IN.
4	173	1,051
4	178	2,229
4	181	1,828
11	193	446
12	193	389

Dynamic Tests

Triaxial Compression Tests - In order to evaluate the effect of vibratory motion on the strength of the in-situ soils, selected soil samples were subjected to dynamic triaxial compression tests. The test procedure used is similar to that for static triaxial compression tests, which is described on Plate 6.45, Methods of Performing Unconfined Compression and Triaxial Compression Tests. Each sample was subjected to a predetermined chamber pressure and deviator stress. At the specified stress, a series of oscillating loads were applied axially to the sample. The additional deformation or strain of the soil sample on each oscillating load was recorded. The results of the dynamic triaxial compression tests are presented in Table 6.3 Dynamic Triaxial Compression Test Results.

Liquefaction Tests - Additional dynamic triaxial compression tests were also performed in addition to those presented on Table 6.3. The purpose of these additional tests was to determine the liquefaction potential of the in-situ soils present at the site. These tests were performed on reconstituted samples having a relative density equal to or greater than the in-situ conditions.

The dynamic triaxial compression tests for evaluating liquefaction potential were performed as described in the preceding section; however, at the specified stress, the oscillating deviator stress was continuously applied until the soil either liquefied or a significant number of stress cycles were applied providing data that indicated liquefaction under the particular stress and oscillating deviator stress condition was difficult or impossible. The results of these tests are presented on Tables 6.4+ and 6.5. Presented on Plate 6.46 is a typical laboratory record of a sample of soil which has been liquefied.

Confined Compression Tests - In order to evaluate the effects of vibratory motion on the compressibility characteristics of the in-situ soils, selected soil samples were subjected to dynamic confined compression tests. The samples were initially allowed to consolidate under a predetermined load representative of those which would be imposed by the structures of the proposed nuclear power plant. After compression under the static load was essentially completed, the sample was subjected to an oscillating load. The additional deformation (compression) of the sample under the oscillating load was recorded. The results of the dynamic confined compression tests are presented in Table 6.6.

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TABLE 6.3 DYNAMIC TRIAXIAL COMPRESSION TEST RESULTS

(Page 1 of 2)

		STATIC CONDITIONS				DYNAMIC CONDITIONS		
BORING NUMBER	**DEPTH FEET**	**CONFINING PRESSURE LBS./SQ. FT.**	**DEVIATOR STRESS LBS./SQ. FT.**	**PERCENT STRAIN**	**APPLIED OSCILLATING DEVIATOR STRESS LBS./SQ. FT.**	**FREQUENCY CPS**	**TIME APPLIED SEC.**	**INCREASE IN PERCENT STRAIN FROM STATIC CONDITION**
3	30	1800	3510	1.1	2600 - 3480	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	0.2
					1950 - 3900	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	0.5
					1950 - 4120	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	0.9
					1950 - 4340	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	1.7
					1950 - 4760	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	4.0
					1950 - 4990	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	6.0
					2170 - 5420	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	8.5
					2000 - 6500	1/2	5*	-
3	50	3000	6720	2.0	6080 - 7150	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	0.2
					5200 - 7290	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	0.3
					5640 - 8020	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	0.5
					4780 - 8460	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	0.9
					4560 - 8460	1/2 - 1	15 - 1**	3.2

* Sample failed abruptly after 2 cycles of oscillation at 1/2 cycle per second. Maximum strain before failure not recorded.

** Sample failed abruptly after 1 cycle of oscillation at 1 cycle per second. Strain indicated is maximum before failure.

TABLE 6.3 DYNAMIC TRIAXIAL COMPRESSION TEST RESULTS
(Page 2 of 2)

		STATIC CONDITIONS			DYNAMIC CONDITIONS			
BORING NUMBER	**DEPTH FEET**	**CONFINING PRESSURE LBS./SQ. FT.**	**DEVIATOR STRESS LBS./SQ. FT.**	**PERCENT STRAIN**	**APPLIED OSCILLATING DEVIATOR STRESS LBS./SQ. FT.**	**FREQUENCY CPS**	**TIME APPLIED SEC.**	**INCREASE IN PERCENT STRAIN FROM STATIC CONDITION**
10	22	12000	2670	3.8	2930 - 3470	1/2 - 1 - 2	30 - 30 - 30	0.3
					2930 - 3580	1/2 - 1 - 2	30 - 30 - 30	0.3
					3360 - 4120	1/2 - 1 - 2	30 - 30 - 30	0.4
					3360 - 4340	1/2 - 1 - 2	30 - 30 - 30	0.4
					2600 - 4340	1/2 - 1 - 2	30 - 30 - 30	0.5
					2060 - 4560	1/2 - 1 - 2	30 - 30 - 30	0.7
					1840 - 4880	1/2 - 1 - 2	30 - 30 - 30	1.1
					1840 - 5210	1/2 - 1 - 2	30 - 30 - 30	1.5
					1950 - 5430	1/2 - 1 - 2	30 - 30 - 30	2.3
					1840 - 5530	1/2 - 1 - 2	30 - 30 - 30	4.2
					1520 - 6080	1/2 - 1	30 - 5**	4.2
10	66	4000	10410	0.4	10410 - 13020	1/2 - 1 - 2	30 - 30 - 30	3.8
					10200 - 14100	1/2 - 1 - 2	30 - 30 - 30	4.2
					9765 - 14750	1/2 - 1 - 2	30 - 30 - 30	5.1
					9110 - 15510	1/2 - 1	30 - 17*	8.3
11	29	1800	4770	0.7	4120 - 4770	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	0.2
					3250 - 5420	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	0.5
					3250 - 6070	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	1.2
					3250 - 6500	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	1.6
					3250 - 6710	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	1.9
					3250 - 7150	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	3.0
					3470 - 7380	1/2 - 1 - 2 - 4	15 - 15 - 15 - 15	4.0
					3250 - 7380	1/2 - 1	15 - 15**	6.5

* Sample failed abruptly after 17 cycles of oscillation. Strain indicated is maximum strain before failure.
** Sample failed abruptly after 5 cycles of oscillation. No additional strain was recorded before failure.
*** Sample failed abruptly after 15 cycles of oscillation. Strain indicated is maximum strain before failure.

TABLE 6.4 DYNAMIC TRIAXIAL COMPRESSION TEST RESULTS

BORING NUMBER	DEPTH FEET	TEST DENSITY LBS./CU. FT.	FIELD DENSITY LBS./CU. FT.	RELATIVE DENSITY PERCENT	CONFINING PRESSURE LBS./SQ. FT.	APPLIED OSCILLATING DEVIATOR STRESS LBS./SQ. FT.	CYCLES TO LIQUEFACTION
9	9	96	96	46	2500	± 900	>100
					2500	± 000	52
9	9	99	96	60	1000	± 100	>75
					1000	± 200	4
9	26	95	95	38	4000	± 200	>200
					4000	± 400	110
9	26	99	95	56	2000	± 400	>100
					2000	± 600	2
9	66	113	113	72	5000	± 1500	>100
					5000	± 2000	2

TABLE 6.5 DYNAMIC TRIAXIAL COMPRESSION TEST RESULTS LIQUEFACTION TESTS – COMPACTED SAMPLES

(Page 1 of 2)

SOIL TYPE	DRY DENSITY (P. C. F.)	CONFINED PRESSURE (P. S. F.)	MINIMUM DEVIATOR STRESS (P. S. F.)	MAXIMUM DEVIATOR STRESS (P. S. F.)	PERCENT STRAIN	NUMBER OF CYCLES
Fine-Medium Sand	99	1,500	+471	+1,256	.02	20
			-188	+157	.01	11
			-282	+282	.01	10
			-392	+392	.02	10
			-487	+502	.04	10
			-589	+667	.06	11
			-706	+785	.08 did not qualify	10
	100	4,000	+471	+1,256	.03	10
			+628	+2,435	.06	10
			-157	+157	.01	10
			-502	+377	.02	11
			-706	+706	.06	17
			-942	+942	.08	11
			-1,178	+1,178	.10	13
			-1,335	+1,335	Liquified	4

TABLE 6.5 DYNAMIC TRIAXIAL COMPRESSION TEST RESULTS LIQUEFACTION TESTS – COMPACTED SAMPLES

(Page 2 of 2)

SOIL TYPE	DRY DENSITY (P. C. F.)	CONFINED PRESSURE (P. S. F.)	MINIMUM DEVIATOR STRESS (P. S. F.)	MAXIMUM DEVIATOR STRESS (P. S. F.)	PERCENT STRAIN	NUMBER OF CYCLES
Fine-Coarse Sane	109	1,500	+471	+1,276	.04	12
			-164	+164	.02	10
			-298	+267	.02	10
			-377	+377	.02	11
			-518	+518	.04	11
			-648	+667	.06	11
			-746	+785	.07	10
			-1,315	+1,315	Liquefied	25
	109.5	4,000	+471	+1,296	.01	10
			+628	+2,356	.02	11
			-173	+165	.002	11
			-408	+377	.01	11
			-628	+565	.02	11
			-824	+864	.02	11
			-1,060	+1,099	.03	11
			-1,296	+1,335	.04	11
			-2,120	+2,277	Liquefied	20

TABLE 6.6 DYNAMIC CONFINED COMPRESSION TEST RESULTS

BORING NUMBER	DEPTH FEET	APPLIED AXIAL LOAD LBS./SQ. FT.	PERCENT STRAIN	APPLIED OSCILLATORY PRESSURE LBS./SQ. FT.	FREQUENCY CPS	TIME APPLIED SEC.	INCREASE IN PERCENT STRAIN*
9	16	2000	0.79	2000 - 1000	1/2 - 1 - 2	15 - 15 - 15	0.04
				2000 - 3000	1/2 - 1 - 2	15 - 15 - 15	0.17
				2000 - 4000	1/2 - 1 - 2	15 - 15 - 15	0.42
				2000 - 6000	1/2 - 1 - 2	15 - 15 - 15	0.83
				2000 - 10000	1/2 - 1 - 2	15 - 15 - 15	1.48
9	61	6000	0.66	6000 - 5000	1/2 - 1 - 2	15 - 15 - 15	0.02
				6000 - 4000	1/2 - 1 - 2	15 - 15 - 15	0.10
				6000 - 2000	1/2 - 1 - 2	15 - 15 - 15	0.43
				6000 - 7000	1/2 - 1 - 2	15 - 15 - 15	0.47
				6000 - 8000	1/2 - 1 - 2	15 - 15 - 15	0.50
				6000 - 10000	1/2 - 1 - 2	15 - 15 - 15	0.60
				6000 - 12000	1/2 - 1 - 2	15 - 15 - 15	0.90

* Increase from static condition.

Shockscope Tests - Several samples of the sands and sandstone underlying the site were tested in a schockscope instrument developed by Dames & Moore to measure the velocity of propagation of compression waves through the materials tested. The velocity of compressional wave propagation, observed in the laboratory is used for correlation purposes with the field velocity measurements obtained in the geophysical refraction surveys.

In the shockscope tests performed, samples were subjected to a physical shock under a range of confining pressures, and the time necessary for the shock wave to travel the length of the sample was measured using an oscilloscope. The velocity of compressional wave propagation was then computed. Since this velocity is proportional to the dynamic modulus of elasticity of the sample, the data are also used in evaluating the dynamic elastic properties. The results of the tests are presented in Tables 6.7 and 6.8.

Other Physical Tests

Moisture-Density Tests - In addition to the moisture content and dry density determinations made in conjunction with the strength and consolidation tests, moisture-density tests were also performed on other undisturbed soil samples for correlation purposes. The results of all moisture and density determinations are presented on the boring logs.

Particle Size Analyses - A number of selected soil samples were analyzed in order to determine their grain-size distribution. Grain-size curves presenting the results of the particle-size analyses are presented on Plates 6.47 through 6.53 Particle Size Distribution.

TABLE 6.7 SHOCKSCOPE TEST RESULTS ON SAND SOILS

BORING NUMBER	DEPTH (FEET)	CONFINING PRESSURE (LBS./SQ. FT)	VELOCITY OF COMPRESSIONAL WAVE PROPAGATION (FT./SEC.)	DRY DENSITY (LBS./CU. FT.)	MOISTURE CONTENT PERCENT
4	21	0	520	103	15.7
		2,000	640		
		4,000	830		
		6,000	1040		
4	61	0	2080	111	18.0
		2,000	2380		
		4,000	2780		
		6,000	2780		
11	13	0	600	92	6.4
		2,000	700		
		4,000	830		
		6,000	1040		
12	91	0	2080	116	13.6
		2,000	2380		
		4,000	2780		
		6,000	3330		

TABLE 6.8 SHOCKSCOPE TEST RESULTS ON SANDSTONE

BORING NUMBER	DEPTH (FEET)	CONFINING PRESSURE (LBS./SQ. FT.)	VELOCITY OF COMPRESSIONAL WAVE PROPAGATION (FT./SEC.)
4	174	0	2690
		2000	2870
		4000	2870
		6000	2870
4	182	0	5880
		2000	6250
		4000	6250
		6000	6250
12	187	0	1550
		2000	1700
		4000	1880
		6000	1940
12	199	0	2360
		2000	2480
		4000	2480
		6000	2580
12	202	0	3600
		2000	3820
		4000	4250
		6000	4250

Maximum and Minimum Density Determinations - In order to evaluate the relative density of the in-situ soils, a number of selected samples were analyzed to determine their minimum and maximum density. The minimum density of a sample was determined by first drying the sample and then lightly pouring the dried material into a container of known volume. The maximum density was determined by vibrating a saturated sample in a container to a constant density. During the vibrating operation, a surcharge load was applied to the top of the sample to prevent expansion of the soil mass. The results of the maximum and minimum density determinations and the in-situ relative density of the selected samples are presented on Table 6.9.

TABLE 6.9

BORING NUMBER	DEPTH (FEET)	MINIMUM DENSITY (LBS./CU. FT.)	MAXIMUM DENSITY (LBS./CU. FT.)	FIELD DENSITY (LBS./CU. FT.)	RELATIVE DENSITY (PERCENT)
9	6	83	112	102	72*
9	6	87	110	96	46
9	16	84	110	109	98*
9	26	88	110	95	38
9	33	90	112	104	68
9	36	92	113	108	79
9	66	95	122	113	72
10	21	85	110	96	51
10	66	96	121	112	69

* Samples tested contain a large percentage of fines passing the No. 200 sieve.

Specific Gravity - The specific gravity of three samples of soil determined in accordance with standard ASTM specifications. The results of these tests are presented in Table 6.10.

TABLE 6.10 SPECIFIC GRAVITY TEST RESULTS

BORING NUMBER	DEPTH FEET	SPECIFIC GRAVITY
9	9	2.57
9	26	2.61
9	66	2.63

Compaction Tests - Compaction tests were performed on bulk samples of the on-site soils to evaluate the moisture-density relationship of the soils and to establish criteria for their placement and compaction in the construction of compacted fills. The tests were performed in accordance the American Association of State Highway Officials Test Designation: T 180-57 as described on Plate 6.54, Methods of Performing Compaction Tests. The results of the tests are presented on Plate 6.55, Compaction Test Data.

LIST OF PLATES

The following plates are attached and complete Part 6:

Plate 6.1 - Log of Borings (Boring 3)

Plate 6.2 - Log of Borings (Boring 4)

Plate 6.3 - Log of Borings (Boring 5)

Plate 6.4 - Log of Borings (Boring 6)

Plate 6.5 - Log of Borings (Boring 7)

Plate 6.6 - Log of Borings (Boring 8)

Plate 6.7 - Log of Borings (Boring 9)

Plate 6.8 - Log of Borings (Boring 10)

Plate 6.9 - Log of Borings (Boring 11)

Plate 6.10 - Log of Borings (Boring 12)

Plate 6.11 - Log of Borings (Boring 13)

Plate 6.12 - Log of Borings (Boring 14)

Plate 6.13 - Log of Borings (Boring 15)

Plate 6.14 - Log of Borings (Boring 16)

Plate 6.15 - Log of Borings (Boring 17)

Plate 6.16 - Log of Borings (Boring 18)

Plate 6.17 - Log of Borings (Boring 19)

Plate 6.18 - Log of Borings (Boring 20)

Plate 6.19 - Log of Borings (Boring 21)

Plate 6.20 - Log of Borings (Boring 22)

Plate 6.21 - Log of Borings (Boring 23)

Plate 6.22 - Log of Borings (Boring 24)

Plate 6.23 - Log of Borings (Boring 25)

Plate 6.24 - Log of Borings (Boring 26)

Plate 6.25 - Log of Borings (Boring 27)

Plate 6.26 - Log of Borings (Boring 28)

Plate 6.27 - Log of Borings (Boring 29)

Plate 6.28 - Log of Borings (Boring 30)

Plate 6.29 - Log of Borings (Boring 31)

Plate 6.30 - Log of Borings (Boring 32)

Plate 6.31 - Log of Borings (Boring 34)

Plate 6.32 - Log of Borings (Boring 35)

Plate 6.33 - Log of Borings (Boring 36)

Plate 6.34 - Log of Borings (Boring 37)

Plate 6.35 - Log of Borings (Boring 38)

Plate 6.36 - Unified Soil Classification System

Plate 6.37 - Soil Sampler Type U

Plate 6.38 - Seismic Survey Plot Plan

Plate 6.39 - Geophysical Refraction Survey – Compressional Wave Velocities

Plate 6.40 - Method of Performing Direct Shear and Friction Tests

Plate 6.41 - Method of Performing Consolidation Tests

Plate 6.42 - Static Consolidation Test Data

Plate 6.43 - Static Consolidation Test Data

Plate 6.44 - Static Consolidation Test Data

Plate 6.45 - Methods of Performing Unconfined Compression and Triaxial Compression Tests

Plate 6.46 - Typical Laboratory Record of Liquefaction

Plate 6.47 - Particle Size Distribution

Plate 6.48 - Particle Size Distribution

Plate 6.49 - Particle Size Distribution

Plate 6.50 - Particle Size Distribution

Plate 6.51 - Particle Size Distribution

Plate 6.52 - Particle Size Distribution

Plate 6.53 - Particle Size Distribution

Plate 6.54 - Methods of Performing Compaction Tests

Plate 6.55 - Compaction Test Data

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PART 7 - SEISMIC TERMINOLOGY AND REGIONAL EARTHQUAKES

DEFINITION OF TERMS

TERM	DEFINITION
Focus	Point within the earth at which the earthquake starts.
Epicenter	The point on the surface of the earth directly above the focus of an earthquake.
Intensity	A term to describe earthquakes by the degree of shaking at a specified place. This is not based upon measurement, but is a rating assigned by an experienced observer using a descriptive scale. The descriptive scale now in use is the Modified Mercalli Scale which is described on Page E.7.2.
Magnitude	The rating of an earthquake based on a measure of the energy released. The rating scale is called the Richter Scale and is described on Page E.7.3.
Site	The proposed site of the Nuclear Power Plant.
Strong Motion Stations	Locations of instruments which record strong earthquake motions.
Ground Motion Spectrum	A plot of the maximum amplitudes of the simple harmonic components of ground motion against the period of the ground motion. The spectrum may be prepared from records or may be calculated.
Response Spectrum	A plot of the maximum amplitudes of simple oscillators (of varying natural periods) for a recorded or calculated ground motion.
Active Fault	A tear or break in the bedrock which historical records or observable geologic indications show to be recent.
Particle Velocity	Velocity at which a specific particle of the soil or rock mass moves as the result of wave motion.
Velocity of Wave Propagation	Velocity at which energy moves through soil or rock in the form of wave motion.
Amplification Spectrum	The plot of the maximum amplification of bedrock earthquake waves in a geologic column, versus the period of wave motion.

MODIFIED MERCALLI INTENSITY SCALE OF 1931

I. Not felt except by a very few under specially favorable circumstances. (I Rossi-Forel Scale.)

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing. (I to II Rossi-Forel Scale.)

III. Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motorcars may rock slightly. Vibration like passing of truck. Duration estimated. (III Rossi-Forel Scale.)

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motorcars rocked noticeably. (IV to V Rossi-Forel Scale.)

V. Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (V to VI Rossi-Forel Scale.)

VI. Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight. (VI to VII Rossi-Forel Scale.)

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 motorcars. (VIII Rossi-Forel Scale.)

VIII. Damage slight in specially designed structures; considerable with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factor stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motorcars disturbed. (VIII+ to IX- Rossi-Forel Scale.)

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. (IX+ Rossi-Forel Scale.)

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. (X Rossi-Forel Scale.)

XI. Few, if any, (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines 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 air.

THE RICHTER SCALE

Dr. C. F. Richter developed a magnitude scale which is based on the maximum recorded amplitude of a standard seismograph located at a distance of 100 kilometers from the source of a shallow earthquake. The magnitude is defined by the relationship $M = \log A - \log A_o$. In this equation, A is the recorded trace amplitude for a given earthquake at a given distance written by the standard instrument, and A_o is the trace amplitude for a particular earthquake selected as a standard. The zero of the scale is arbitrarily fixed to fit the smallest recorded earthquakes. The largest known earthquake magnitudes are on the order of 8-3/4; however, this magnitude is the result of observations and not an arbitrary scaling. The upper limit to magnitude is not known. It is estimated that it may be about 9.

An approximate relationship between Magnitude M and the Energy E liberated has been given by Richter in the form $\log E = C + BM$. The constants C and B have been revised a number of times. For large magnitude shocks, C = 7.5 and B = 2.0 can be used.

REGIONAL EARTHQUAKES

					EPICENTER LOCATION		
YEAR	**DAY**	**TIME**	**INTENSITY**	**LOCALITY**	**N. LAT.**	**W. LONG**	**AREA SQ. MILES**
1804	8-24		VI	Ft. Dearborn, Illinois	42.0	87.8	30,000
1860			V+	Long Prairie, Minnesota			
1865-70			?	Le Sueur, Minnesota			
1905	3-13		V	Menominee, Minnesota	45.0	87.7	
1905	7-26		VII	Calumet, Michigan	47.3	88.4	
1906*	5-26		VIII	Keweenaw, Peninsula, Michigan	47.3	88.4	1,000
1909	1-22		V	Houghton, Michigan	47.2	88.6	
1909	5-26	08:42	VII III in Minn.	Dixon, Illinois	42.5	89	500,000
1912	1-2		VI	N.E. Illinois	41.5	88.5	40,000
1917	9-3	15:30	V - VI	Staples, Minnesota	46.3	94.5	10,000
1925	2-28	09:19	IX I in Minn.	Quebec City, Canada	47.1	70.0	1,000,000
1928	12-23	00:10	III	Bowstring, Minnesota	47.4	94.0	Local
1931	10-18		II	Madison, Wisconsin			
1933	12-6		IV	Stoughton to Putland, Wisconsin			
1934	11-12		VI	Rock Island, Illinois	41.5	91.5	
1935	11-1	01:04	VI I in Minn.	Timiskaming, Canada	46.8	79.1	1,000,000

REGIONAL EARTHQUAKES [Continued]

					EPICENTER LOCATION		
YEAR	**DAY**	**TIME**	**INTENSITY**	**LOCALITY**	**N. LAT.**	**W. LONG**	**AREA SQ. MILES**
1938	2-2		IV	South Shore of Lake Michigan			
1938	10-11	03:37	V IV in Minn.	Sioux City, S. D.	43.3	96.4	3,000
1939	1-28	11:55	IV	Detroit Lakes, Minnesota	46.9	95.5	2,000
1943			II	Nenonisee, Michigan			
1944	11-16		II	Escanaba, Michigan			
1945	5-18		II	Escanaba, Michigan			
1947	5-6		V	S. E. Wisconsin			
1950	2-15	04:05	V-VI	Alexandria, Minnesota	45.7	94.8	Local
1956	7-18		IV	Oostburg, Wisconsin			
1955	1-6		V	Hancock, Michigan	47.2	88.3	
1956	9-13		IV	Milwaukee-Racine, Wisconsin			

* Believed to have been caused by a mine collapse.

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PART 8 - PRINCIPAL SOURCES OF DATA

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Geology

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* The Areal Geology of The Red Wing Quadrangle, Unpublished M.S. Thesis by W.E. Crain, Dated 1957.

* A contribution to the Study of the Pleistocene History of the Upper Mississippi River, Unpublished Ph.D. Thesis by J.H. Todd, Dated 1942.

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* Available at Minnesota State Geological Agency
** Available at Northern States Power Company

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Triaxial Compressive Strength of Saturated Sands Under Seismic Loading Conditions, Unpublished Ph.D. Thesis by K.L. Lee University of California at Berkeley, Dated 1965.

Cyclic Stress Conditions Causing Liquefaction of Sands, Journal of The Soil Mechanics and Foundation Division, ASCE Volume 93, No. SMI, Proceedings Paper 5058, by K.L. Lee and H.B. Seed, Dated January 1967.

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PERSONS CONTACTED

Geology

P.K. Sims, Minnesota Geological Survey

R.E. Sloan, Minnesota Geological Survey.

Charles Matsch, Professor of Geology, University of Minnesota.

John S. Fryberger, UOP-Johnson Division.

William L. Jungmann, UOP-Johnson Division.

H.M. Mooney, Professor of Geophysics, University of Minnesota.

G.F. Hanson, State Geologist, Wisconsin Geological & Natural History Survey.

Dr. Merideth Ostrom, Wisconsin Geological & Natural History Survey.

Hydrology

Henry Herrich, U.S. Army, Corps of Engineers, Minneapolis.

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PART 9 - PREPARATION OF FOUNDATION SOILS

METHOD OF PREPARATION

After a thorough investigation of the in-place soils, Dames & Moore, Consultants in Applied Earth Science, recommended that any one of three methods of foundation treatment is acceptable. The methods are given on Page 5.3 of Appendix H. Subsequent cost studies were performed which indicated method (1) (excavation-controlled backfill) to be the most economical solution. The excavation-controlled backfill method was, therefore, chosen for the project and will consist, essentially, of the method outlined in Pages 5.9-5.12 of Appendix H.

DEWATERING

The proposed dewatering system is shown on Page E.9-3. The system consists of thirty-six 12-inch diameter deep wells, 30 of which will be equipped with 25 horsepower, 440 volt, three phase, 60 cycle submersible pumps, each capable of approximately 1,100 gpm at approximately 65 ft. head. Six of the wells will be equipped with 20 HP submersible pumps capable of approximately 850 gpm. This pumping system will have an installed pumping capacity of 38,000 gpm. The dewatering contractor, American Dewatering Corporation, calculates that the required pumping capacity is approximately 32,000 gpm. Consequently, this system should have a factor of safety of approximately 20%. American Dewatering Corporation's experience at Lock and Dam No. 3, (approximately two miles downstream from the site) formed the basis for their design calculations. Because the dewatering is under a performance contract, the dewatering Contractor will be required, and has agreed, to provide the necessary steps to accomplish any additional dewatering necessary to maintain the water table at least three feet below the lowest working surface. The dewatering contractor is further required to install piezometers at the locations indicated on the drawing on Page 9.3. The piezometers will consist of steel pipe at least two inches in diameter with a well point at the bottom to prevent infiltration of soil into the piezometer. The contractor will be responsible for installing and maintaining all piezometers and observing and recording the elevation of the ground water levels in all of the piezometers daily. The contractor will replace, within 24 hours, any piezameters that become inactive, damaged, or destroyed. Periodically, the contractor will, by adding water to or removing water from all piezometer risers, demonstrate that the piezometers are functioning properly.

The daily record of information obtained from the piezameter readings will be reviewed by Dames & Moore and the project superintendent to assure that the water table is maintained at least three feet below the lowest working surfaces at all times. Should the ground water elevation rise above the minimum permissible level, earthwork operations will be suspended until the ground water table is adequately drawn down. In this event, Dames & Moore will run sufficient density tests to confirm that the higher water level has not disturbed previously compacted material. Any disturbed materials will be re-compacted.

Electric power used by the dewatering wells will be provided by Northern States Power Company. In addition, the contractor will provide 100% diesel standby generator capacity with appropriate switchgear in order to provide a safe dependable standby power source. Adequate standby diesel fuel will be provided by Northern States Power Company. The contractor will also provide at least two complete pumping units should any of the installed units require replacement. The contractor has additional pumping units in stock in both Minneapolis and New Jersey should any additional units be required.

INSPECTION

Because of their proven abilities with the methods to be employed and their familiarity with the project, Dames & Moore has been retained to provide inspection of the dewatering, excavation, and backfill procedures. Dames & Moore will maintain a sufficient number of qualified soils engineers at the site who will exercise continuous technical supervision during earthwork operations. Dames & Moore's responsibility will include the following:

1. Approval of all materials and procedures used during the earthwork operations.

2. Classification of materials obtained from on-site excavations and borrow areas and approval before their subsequent use as fill or backfill.

3. Approval of excavated surfaces prior to the placement of fill and backfill materials.

4. Daily review of water levels taken at the deep wells and piezometers and periodic checks of water level readings.

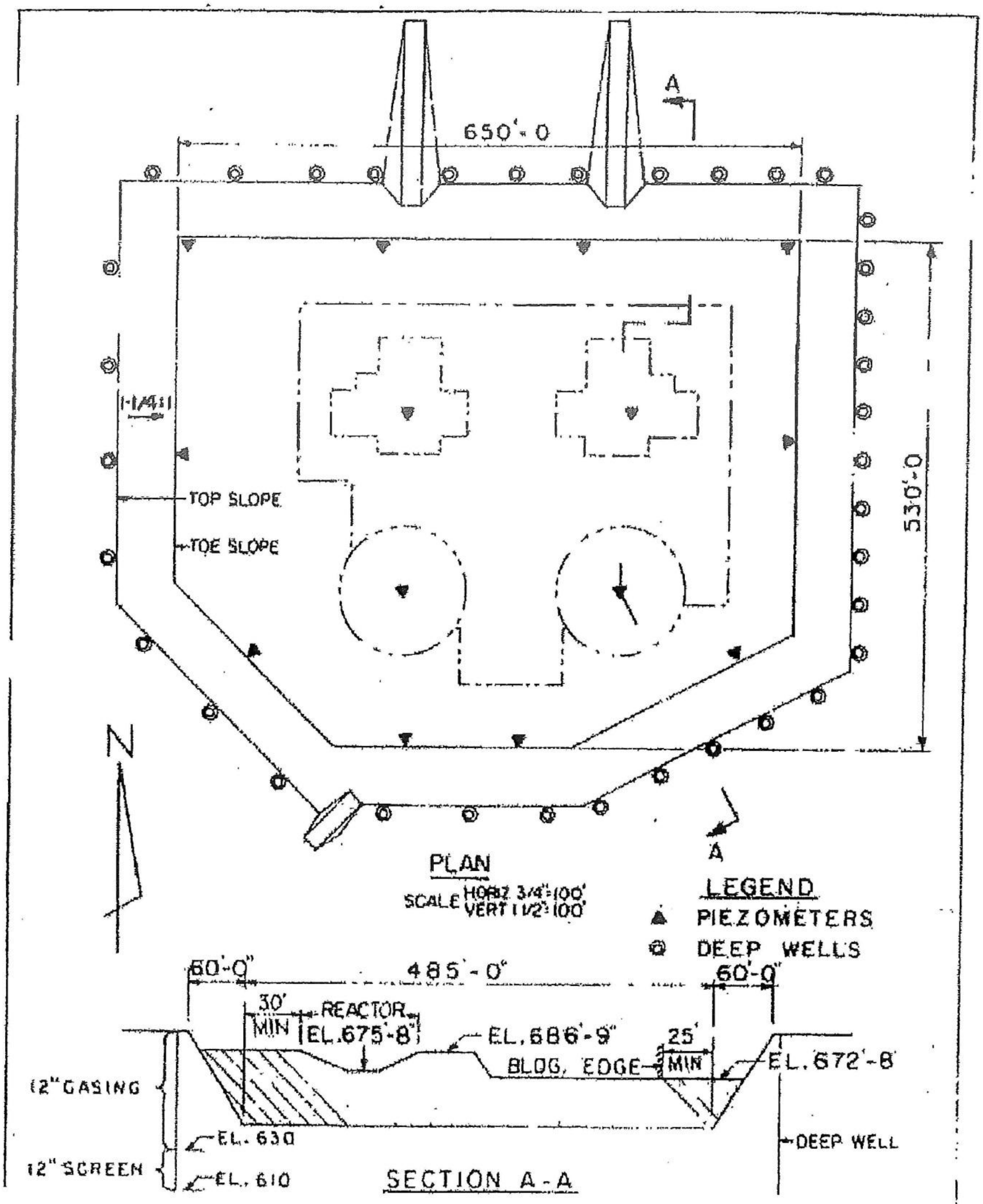
5. Evaluation of the placement and compaction of fill and backfill materials to determine, by means of visual inspection and performance of field density tests, that the materials are placed in 8-inch maximum layers and that compaction to 100 percent of maximum dry density as determined by AASHO T 18057 method is achieved. Areal distribution of tests will be determined by Dames & Moore on the basis of observation of the compaction operations and inspections of the compacted layers.

6. Advising the Northern States Power Company superintendent and Quality Assurance Engineer regarding all questions of quality or acceptability of materials and work performed that may arise during the earthwork operations.

7. Reporting observations and test results to the Northern States Power Company superintendent and Quality Assurance Engineer and contractor representatives designated by the superintendent.

Should Dames & Moore find any area which is not adequately compacted, the contractor will be required to apply additional compactive effort to that area. Should that area still fail to meet the required density, the material will be removed, replaced with selected granular material, and re-compacted to meet the required density.

Evacuation – Backfill – Dewatering Layout



EXCAVATION - BACKFILL - DEWATERING LAYOUT
PRAIRIE ISLAND NUCLEAR GENERATING PLANT UNITS 1 & 2

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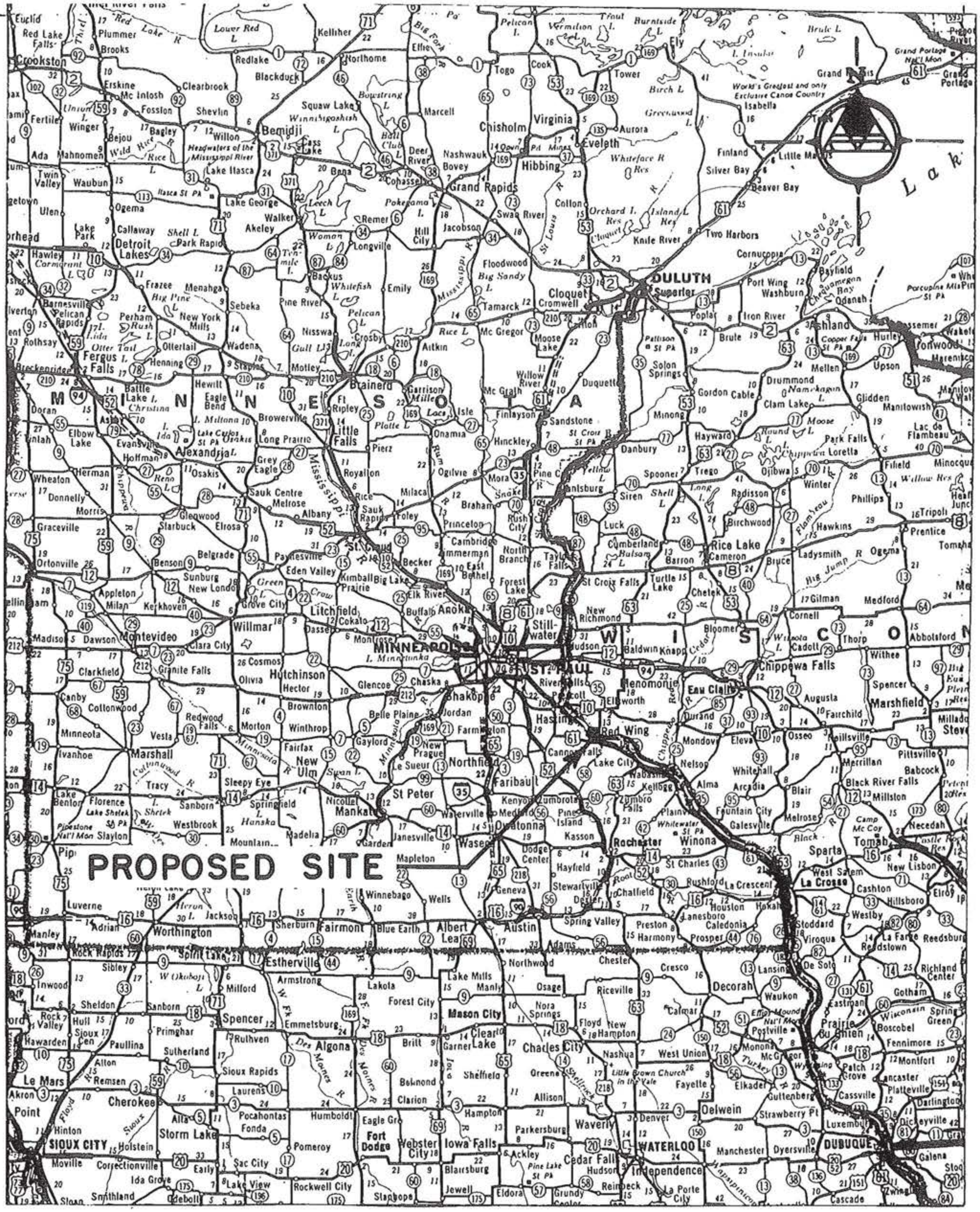

MAP OF REGION

0 50 100 MILES

REFERENCE:
ROADMAP-CENTRAL & WESTERN UNITED STATES
PREPARED FOR MOBILE OIL COMPANY
BY RAND McNALLY, 1965

12

DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE 1.1

VICINITY MAP

CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL

REFERENCE:
RED WING, MINN.-WISC.
U.S.G.S. QUADRANGLE
DATED 1952

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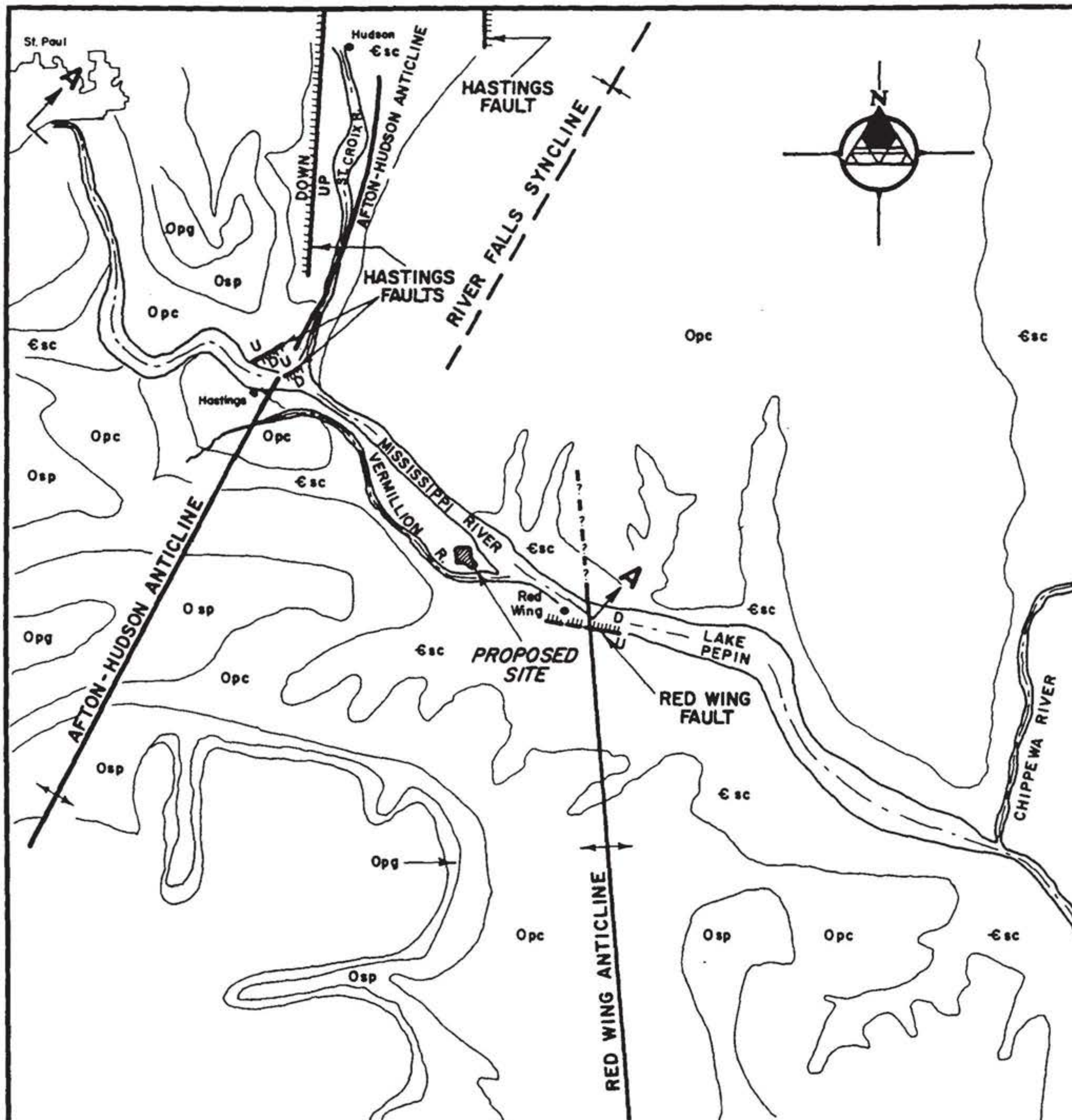
DAMES & MOORE

PLATE 1.2



REGIONAL GEOLOGIC MAP OF BEDROCK FORMATIONS

MILES

5 0 5 10 15 20

LEGEND

Opg LIMESTONE AND DOLOMITE - PLATTVILLE Fm.
SHALE - GLENWOOD Fm.

Osp SANDSTONE - ST. PETER Fm.

Opc DOLOMITE - SHAKOPEE Fm.
SANDSTONE - NEW RICHMOND Fm.
DOLOMITE - ONEOTA Fm.

€sc SANDSTONE - JORDAN Fm.
DOLOMITE AND SILTSTONE - ST. LAWRENCE Fm.
SANDSTONE - FRANCONIA Fm., DRESBACH Fm.

FAULT - KNOWN AND INFERRED

ANTICLINE AXIS

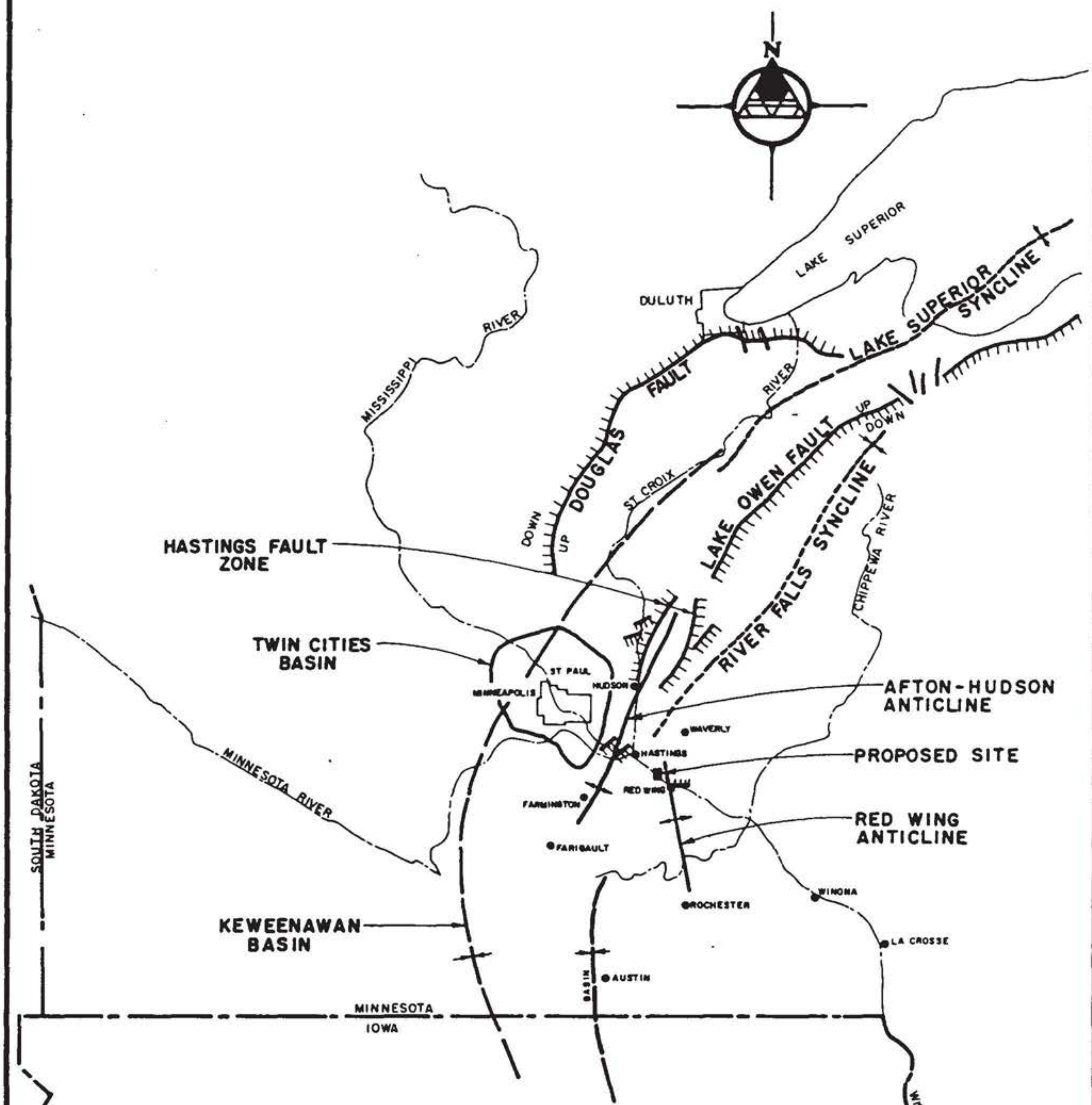
A A CROSS SECTION A-A - SEE PLATE 2.3

SYNCLINE AXIS

REFERENCE: GEOLOGIC MAP OF MINNESOTA
ST. PAUL SHEET 1966

DAMES & MOORE

PLATE 2.1



REGIONAL GEOLOGIC STRUCTURE

MILES

10 0 10 20 30 40 50 60 70 80 90 100

LEGEND

— - FAULT-KNOWN AND INFERRED

ANTICLINE AXIS

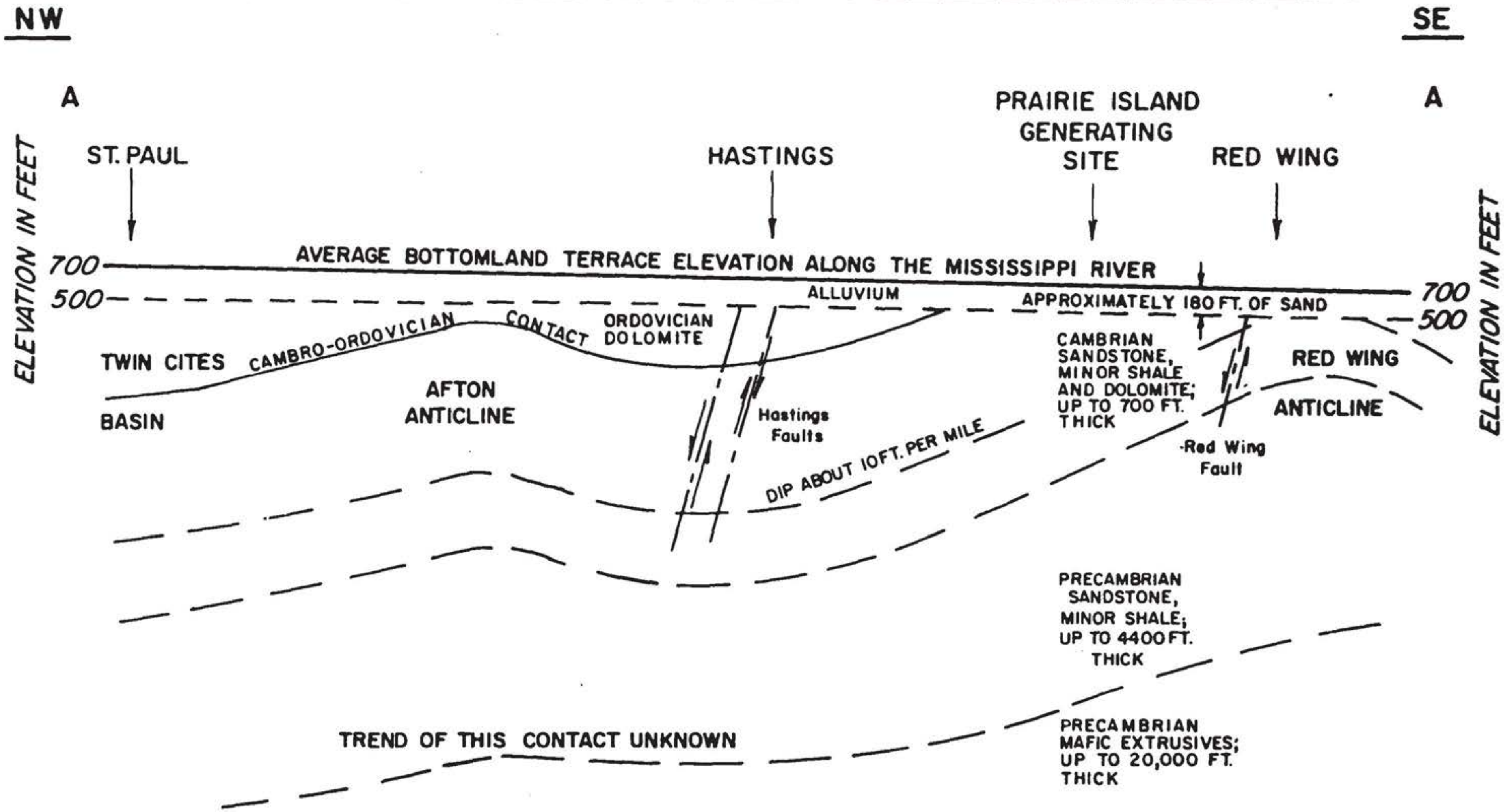
SYNCLINE AXIS

REFERENCES:
1-TECTONIC MAP OF THE UNITED STATES BY U.S.G.S. AND THE AMERICAN ASSOCIATION OF PETROLEUM GEOLOGISTS 1961

2-UNPUBLISHED STRUCTURE MAP BY MINNESOTA GEOLOGICAL SURVEY

DAMES & MOORE

PLATE 2.2



REGIONAL GEOLOGIC CROSS SECTION A-A*
PRAIRIE ISLAND GENERATING SITE
RED WING, MINNESOTA

HORIZONTAL SCALE: 1 IN. = APPROX. 6 MILES
VERTICAL SCALE: VARIABLE AND HIGHLY EXAGGERATED

* SEE PLATE 2.1

DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE 2.3

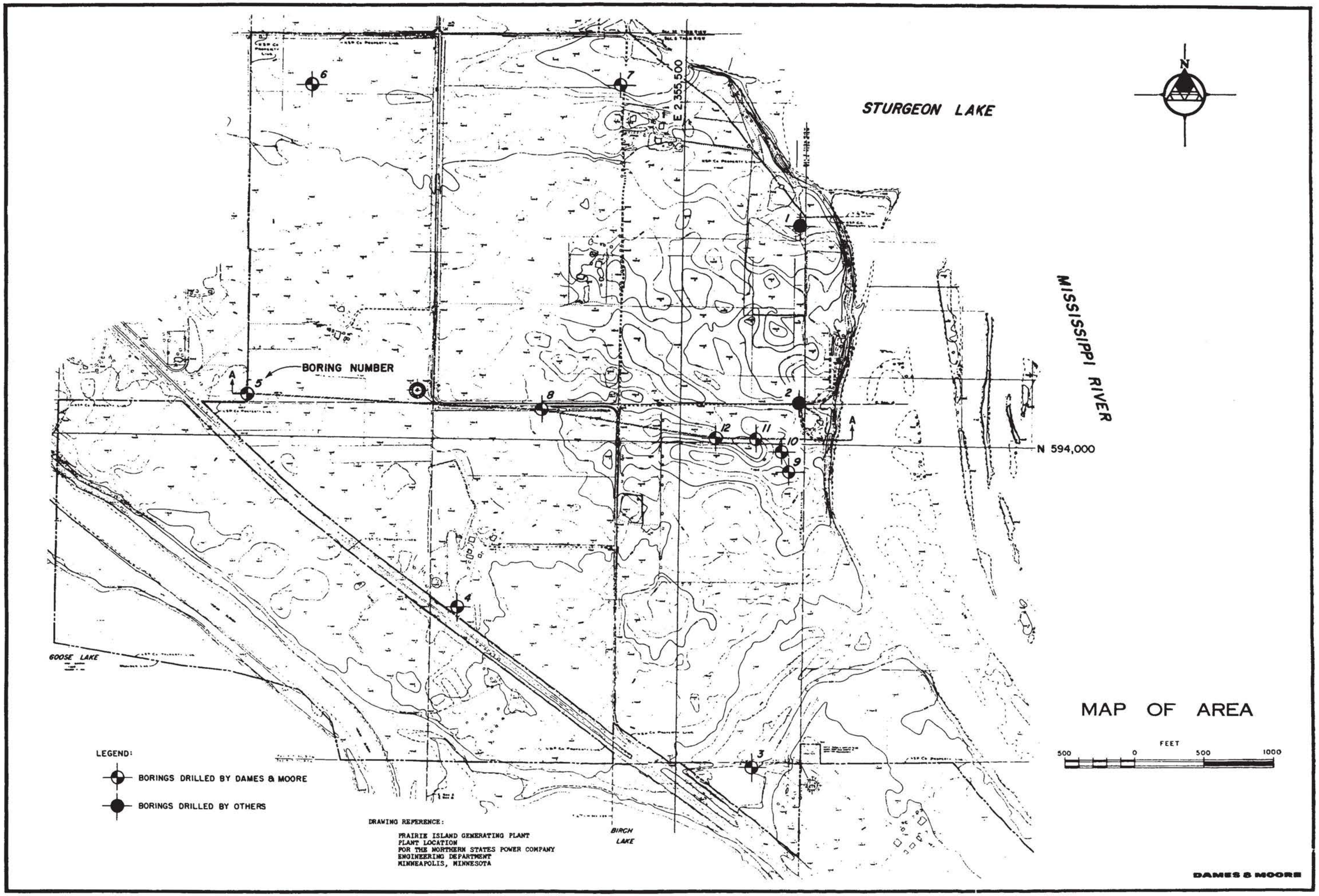
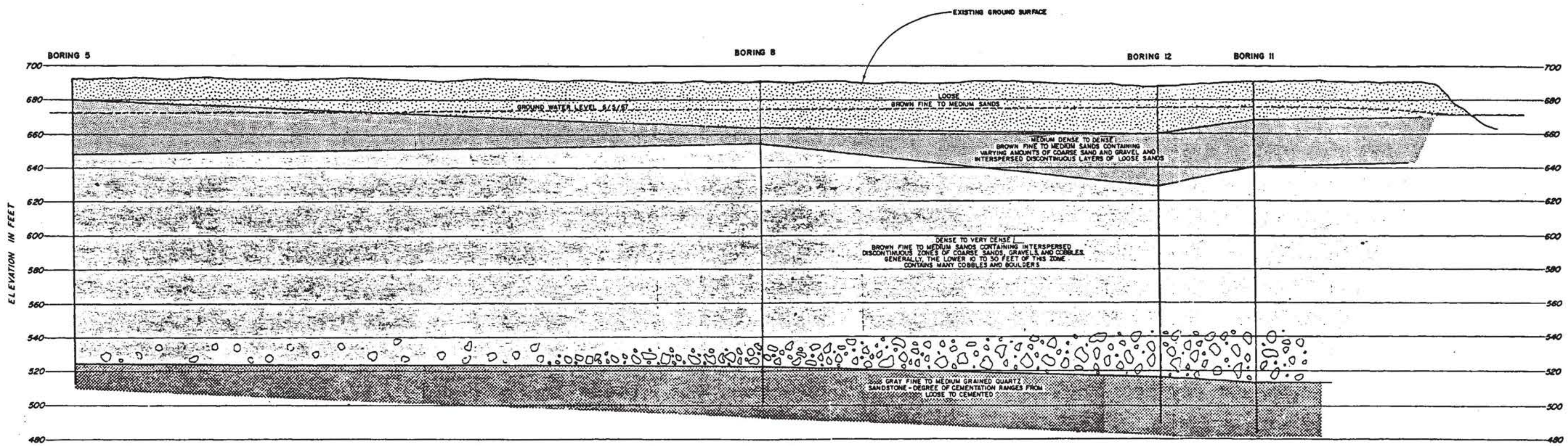

STURGEON LAKE
MISSISSIPPI RIVER
N
E 2,355,500
N 594,000
BORING NUMBER
A
A
1
2
3
4
5
6
7
8
9
10
11
12
GOOSE LAKE
BIRCH LAKE
MAP OF AREA
FEET
500 0 500 1000
LEGEND:
BORINGS DRILLED BY DAMES & MOORE
BORINGS DRILLED BY OTHERS
DRAWING REFERENCE:
PRAIRIE ISLAND GENERATING PLANT
PLANT LOCATION
FOR THE NORTHERN STATES POWER COMPANY
ENGINEERING DEPARTMENT
MINNEAPOLIS, MINNESOTA
DAMES & MOORE

PLATE 2.4



GENERALIZED SUBSURFACE SECTION A-A

FEET
100 0 100 200

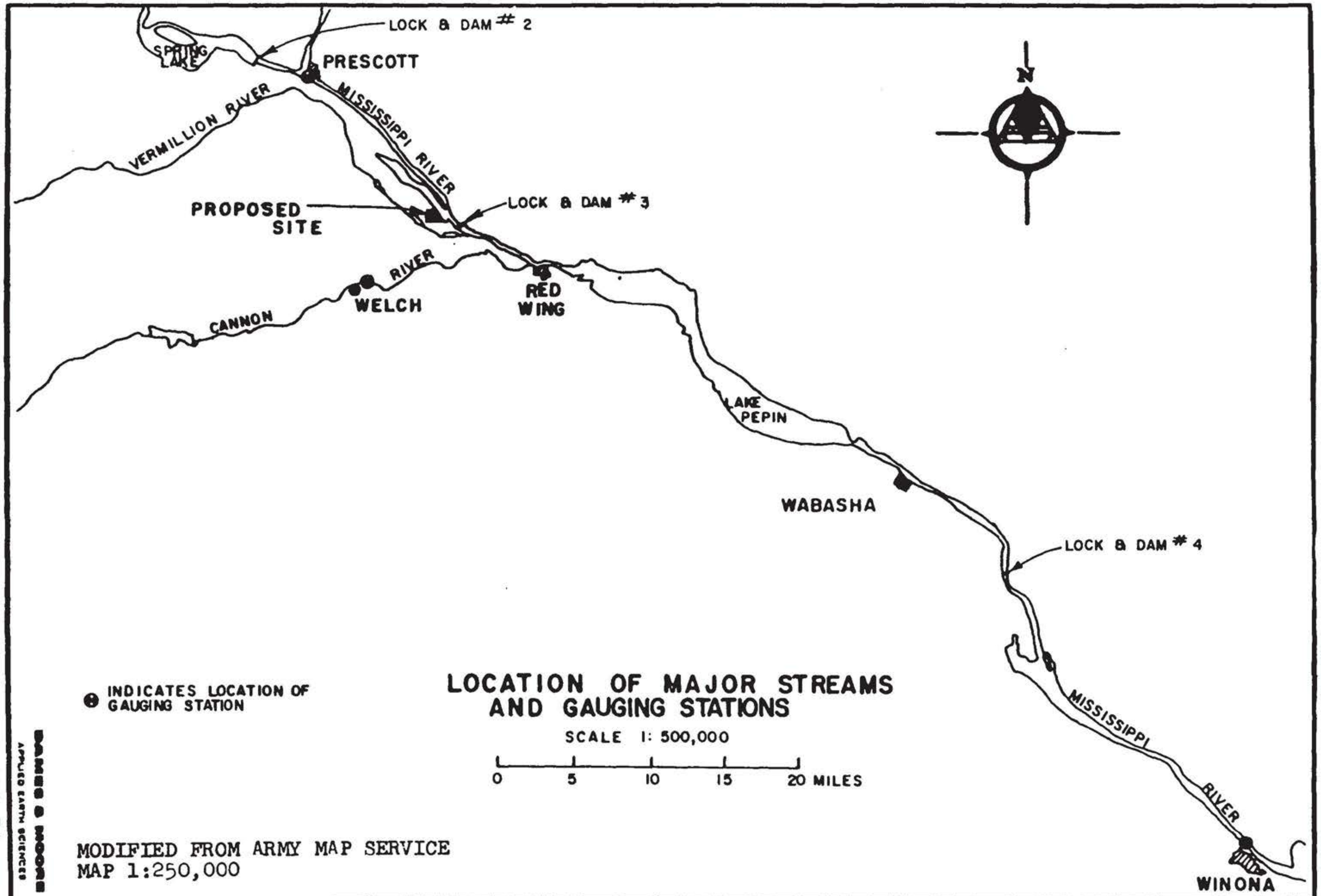
NOTES:

GROUND SURFACE ELEVATIONS REFER TO MEAN SEA LEVEL DATUM - 1929 ADJUSTMENT

GROUND SURFACE ELEVATIONS ARE CORRECT ONLY AT TEST BORING LOCATIONS.

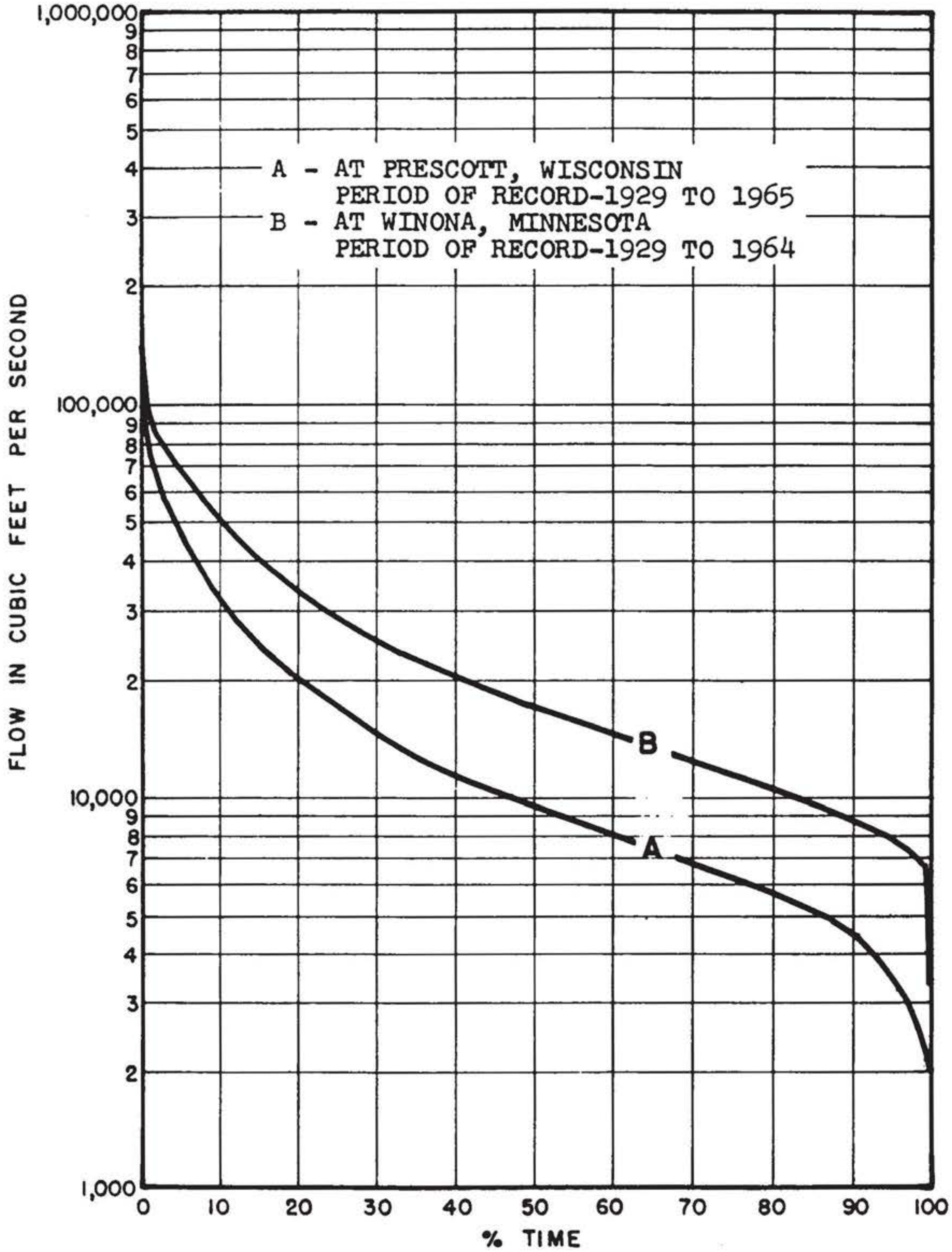
THE DEPTH AND THICKNESS OF THE SOIL AND ROCK STRATA INDICATED ON THE GENERALIZED SUBSURFACE SECTION WERE OBTAINED BY INTERPOLATING BETWEEN TEST BORINGS. INFORMATION ON ACTUAL SOIL AND ROCK CONDITIONS EXISTS ONLY AT THE TEST BORINGS AND IT IS POSSIBLE THAT THE SOIL AND ROCK CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

PLATE 2.5



LOCK & DAM # 2
SPRING LAKE
PRESCOTT
MISSISSIPPI RIVER
VERMILLION RIVER
N
PROPOSED SITE
LOCK & DAM # 3
CANNON RIVER
WELCH
RED WING
LAKE PEPIN
WABASHA
LOCK & DAM # 4
MISSISSIPPI RIVER
WINONA
INDICATES LOCATION OF GAUGING STATION
LOCATION OF MAJOR STREAMS AND GAUGING STATIONS
SCALE 1: 500,000
0 5 10 15 20 MILES
DAMES & MOORE
APPLIED EARTH SCIENCES
MODIFIED FROM ARMY MAP SERVICE
MAP 1:250,000

PLATE 3.1

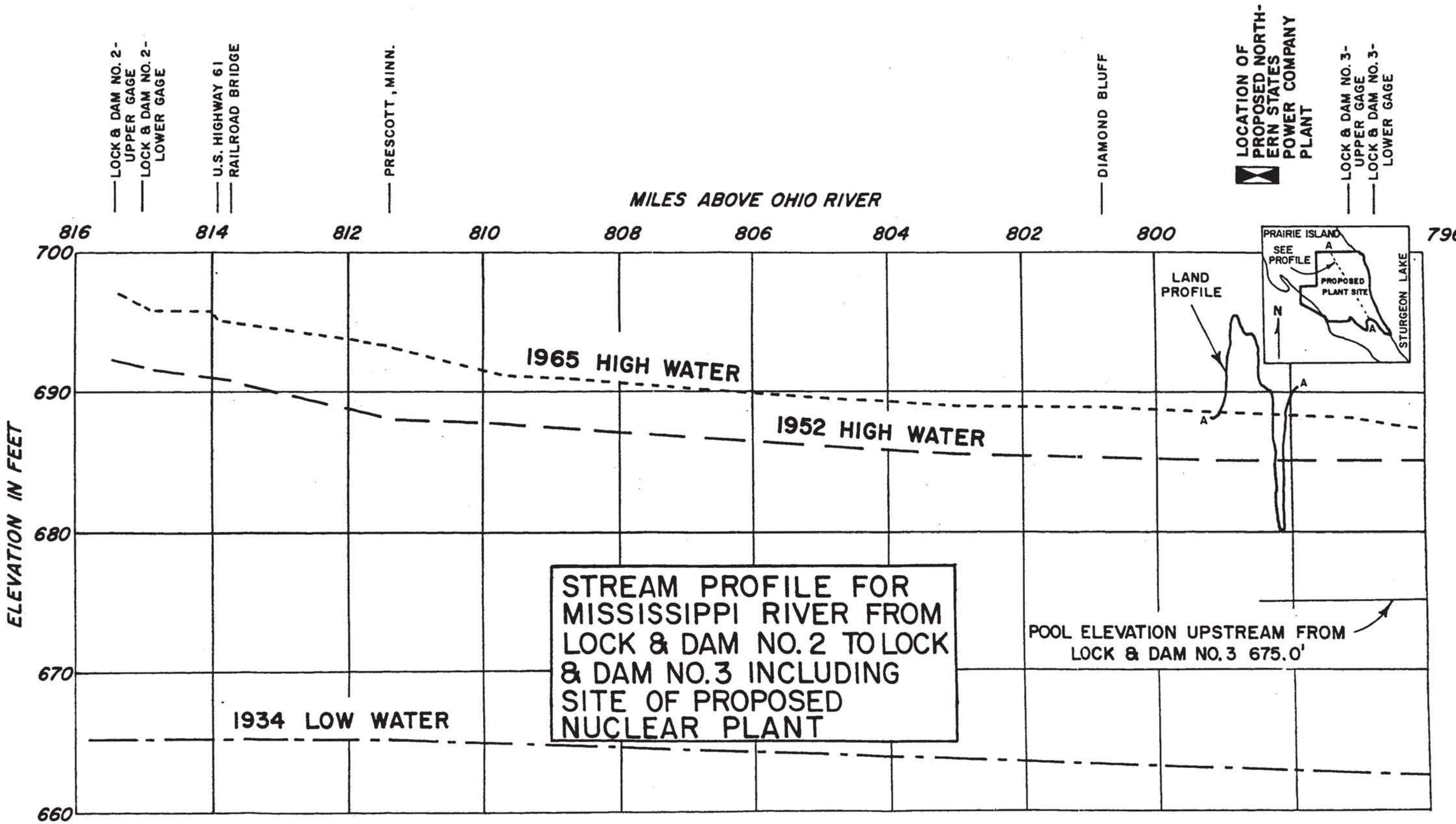


FLOW DURATION CURVES FOR MISSISSIPPI RIVER

GRAPH COMPILED FROM DATA OBTAINED
FROM U.S.GEOLOGICAL SURVEY
STATISTICAL SUMMARIES

DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE 3.2

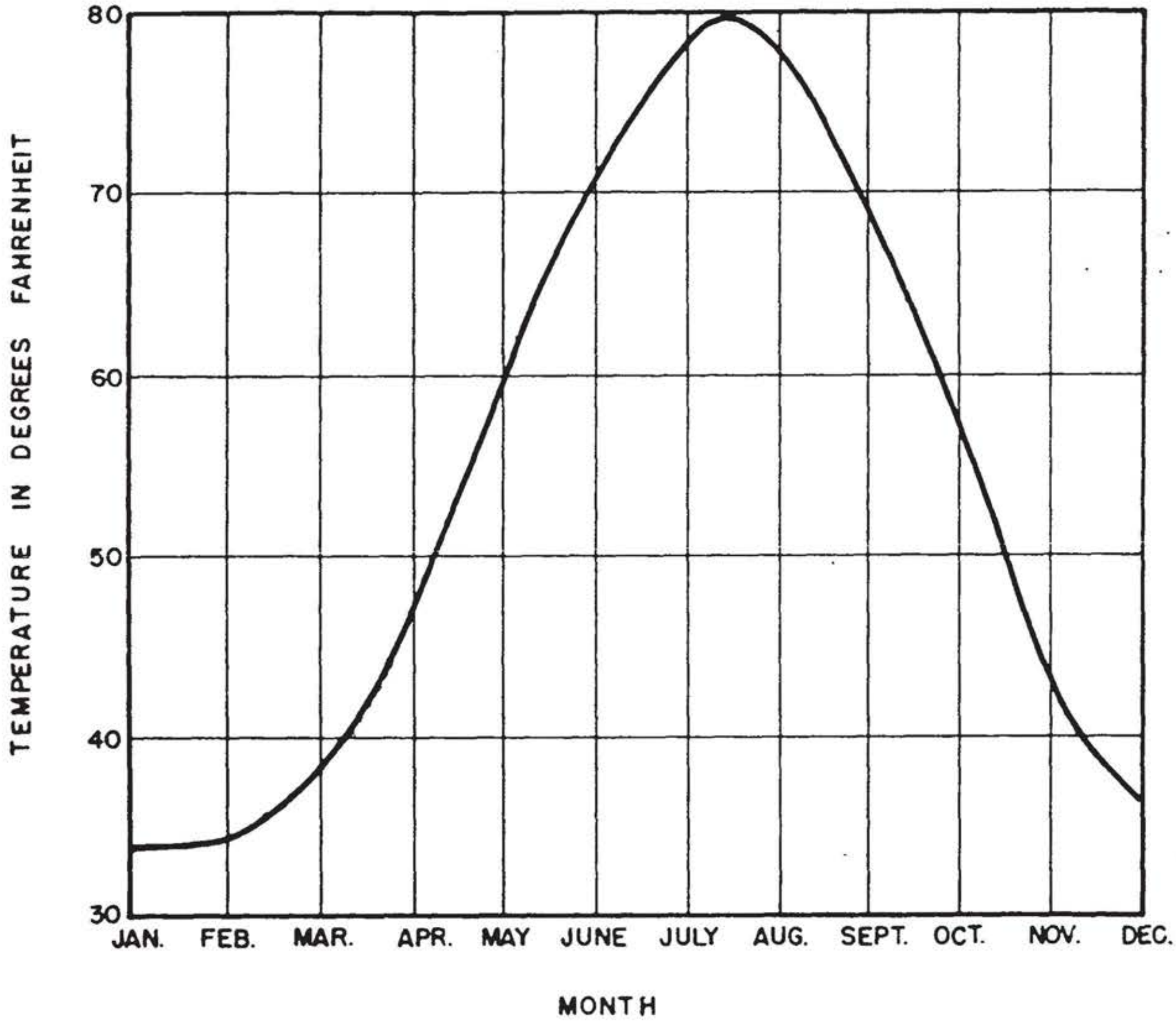


NOTE:

MODIFIED FROM U.S. CORPS OF ENGINEERS
STREAM PROFILE CURVES

DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE 3.3



AVERAGE TEMPERATURE
OF MISSISSIPPI RIVER
AT ST. PAUL, MINNESOTA

PERIOD OF RECORD 1956 TO 1964

REFERENCE:
UNPUBLISHED DATA ON STREAM TEMPERATURES FOR THE MISSISSIPPI RIVER COMPILED BY THE U. S. ARMY CORPS. OF ENGINEERS, ST. PAUL MINNESOTA.

DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE 34

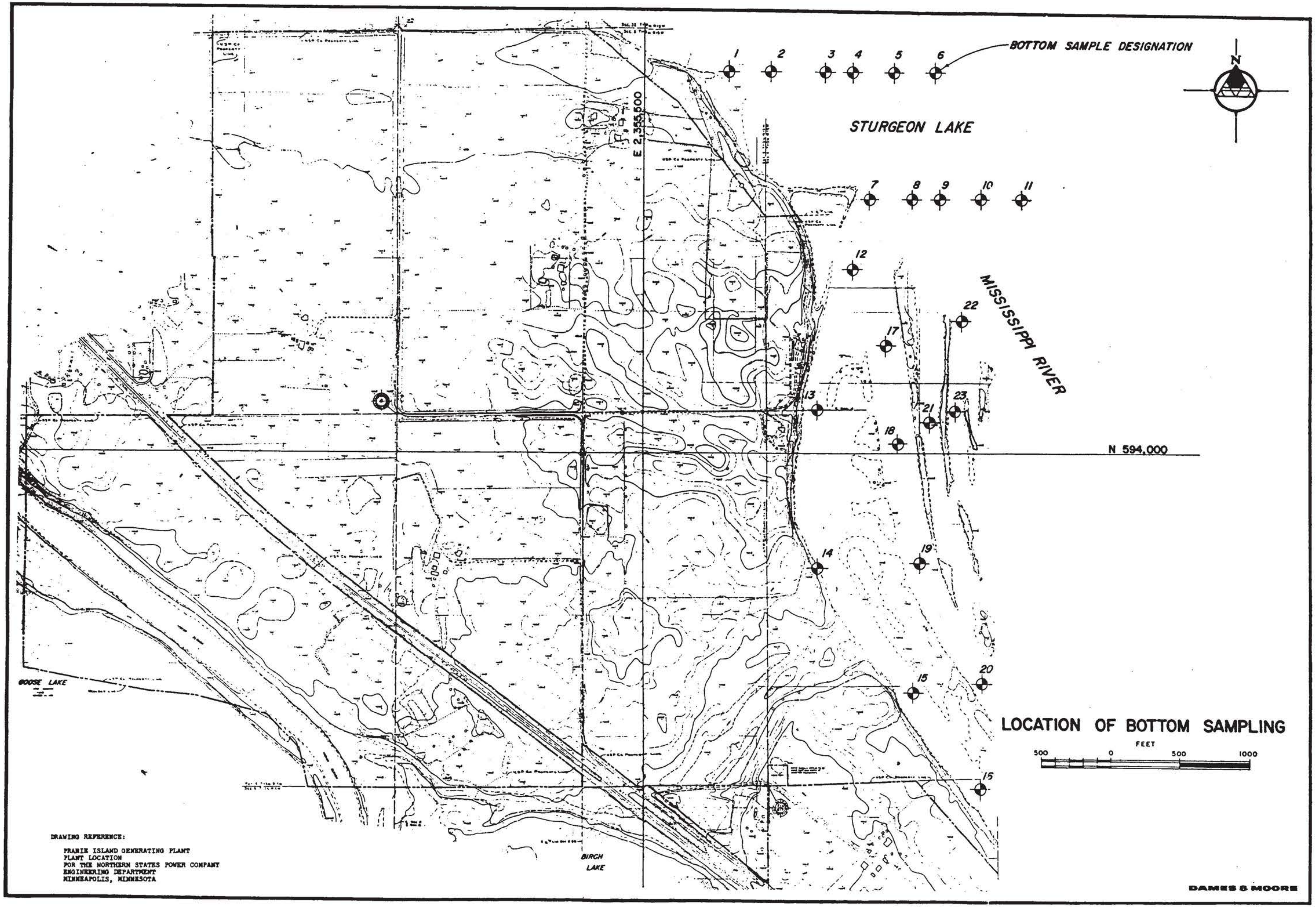

BOTTOM SAMPLE DESIGNATION
N
STURGEON LAKE
MISSISSIPPI RIVER
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23
E 2,355,500
N 594,000
GOOSE LAKE
BIRCH LAKE
LOCATION OF BOTTOM SAMPLING
FEET
500 0 500 1000
DRAWING REFERENCE:
PRARIE ISLAND GENERATING PLANT
PLANT LOCATION
FOR THE NORTHERN STATES POWER COMPANY
ENGINEERING DEPARTMENT
MINNEAPOLIS, MINNESOTA
DAMES & MOORE

PLATE 3.5

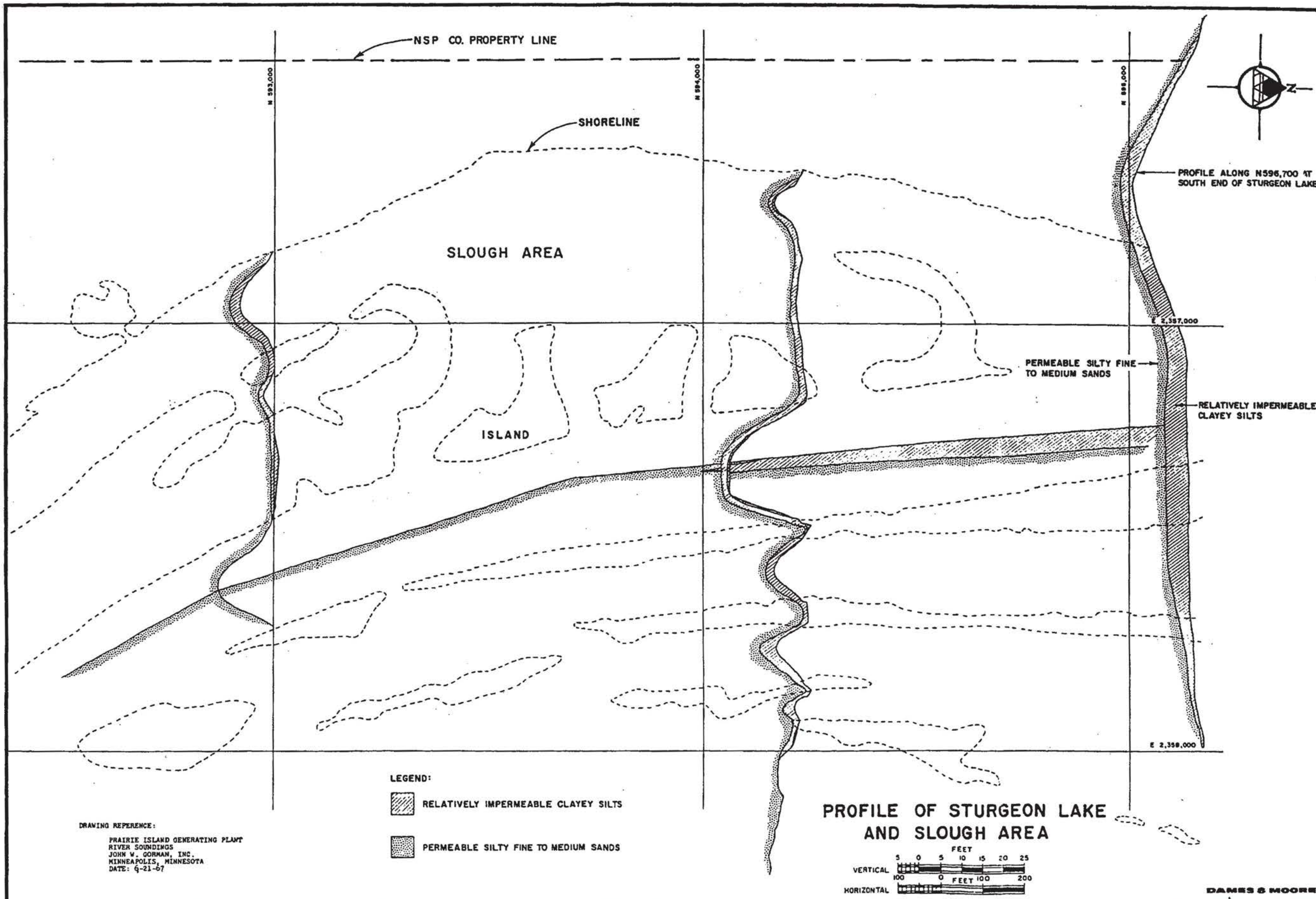
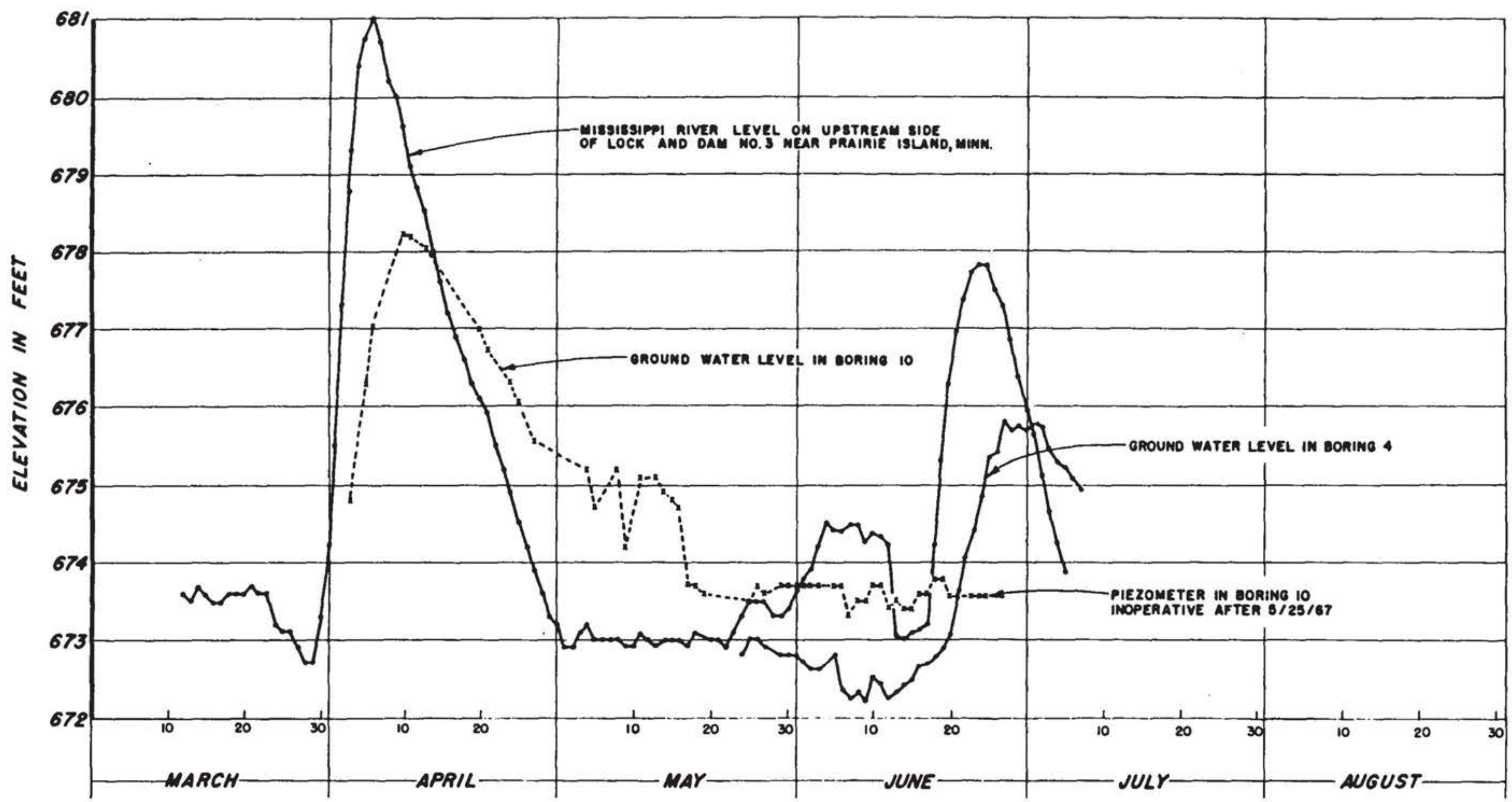

NSP CO. PROPERTY LINE
N 593,000
N 594,000
N 595,000
SHORELINE
SLOUGH AREA
ISLAND
PROFILE ALONG N596,700 AT SOUTH END OF STURGEON LAKE
E 2,357,000
PERMEABLE SILTY FINE TO MEDIUM SANDS
RELATIVELY IMPERMEABLE CLAYEY SILTS
E 2,358,000
LEGEND:
RELATIVELY IMPERMEABLE CLAYEY SILTS
PERMEABLE SILTY FINE TO MEDIUM SANDS
DRAWING REFERENCE:
PRAIRIE ISLAND GENERATING PLANT
RIVER SOUNDINGS
JOHN W. GORMAN, INC.
MINNEAPOLIS, MINNESOTA
DATE: 6-21-67
PROFILE OF STURGEON LAKE
AND SLOUGH AREA
FEET
5 0 5 10 15 20 25
VERTICAL
100 0 FEET 100 200
HORIZONTAL
DAMES & MOORE

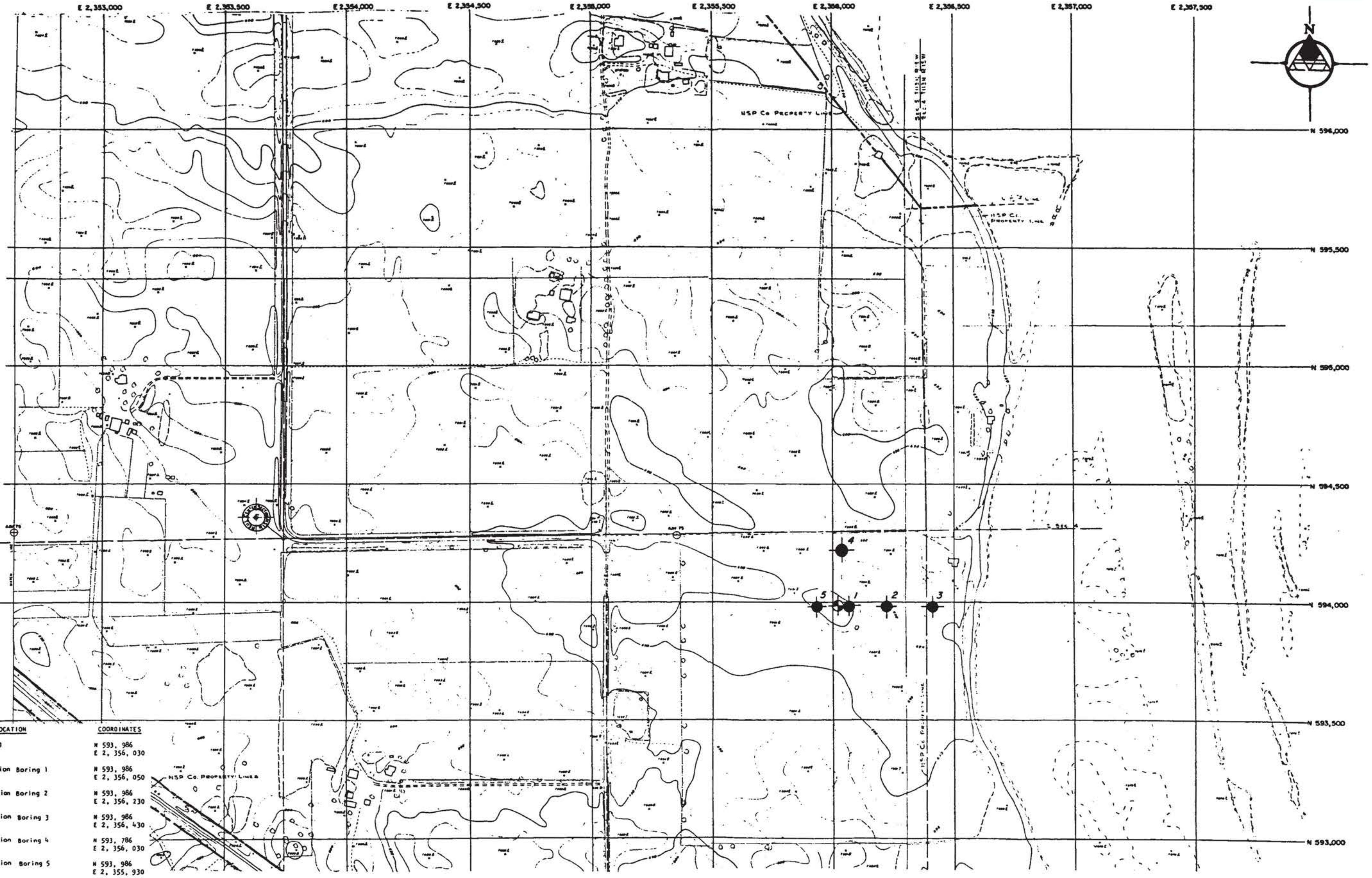
PLATE 3.6



DAMES & MOORE

COMPARISON OF MISSISSIPPI RIVER LEVEL
AND
GROUND WATER LEVEL IN BORINGS 4 AND 10

PLATE 3.7



LOCATION	COORDINATES
Test Well	N 593, 986 E 2, 356, 030
Observation Boring 1	N 593, 986 E 2, 356, 050
Observation Boring 2	N 593, 986 E 2, 356, 230
Observation Boring 3	N 593, 986 E 2, 356, 430
Observation Boring 4	N 593, 786 E 2, 356, 030
Observation Boring 5	N 593, 986 E 2, 355, 930

LEGEND:

TEST WELL

OBSERVATION BORINGS

DRAWING REFERENCE:

MARK HURD AERIAL SURVEYS, INC.
MINNEAPOLIS, MINNESOTA
PRAIRIE ISLAND GENERATING PLANT
TOPOGRAPHIC MAP OF PLANT SITE
SHEETS 3,4,7 AND 8
DRAWING NO. NF-30656-8
DATE: 1-28-65

LOCATION OF TEST WELL AND OBSERVATION BORINGS

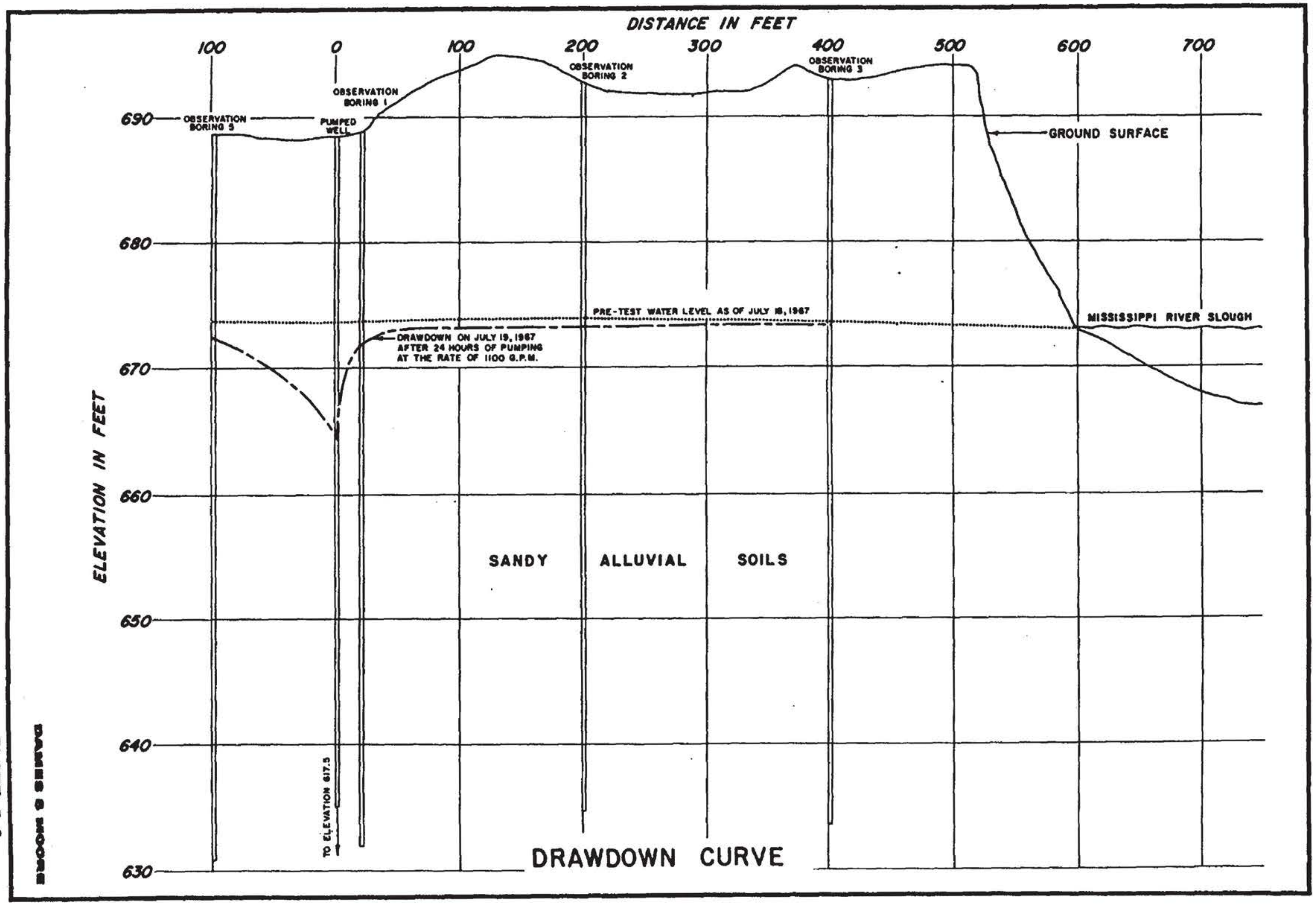
FEET
100 0 100 200 300 400 500

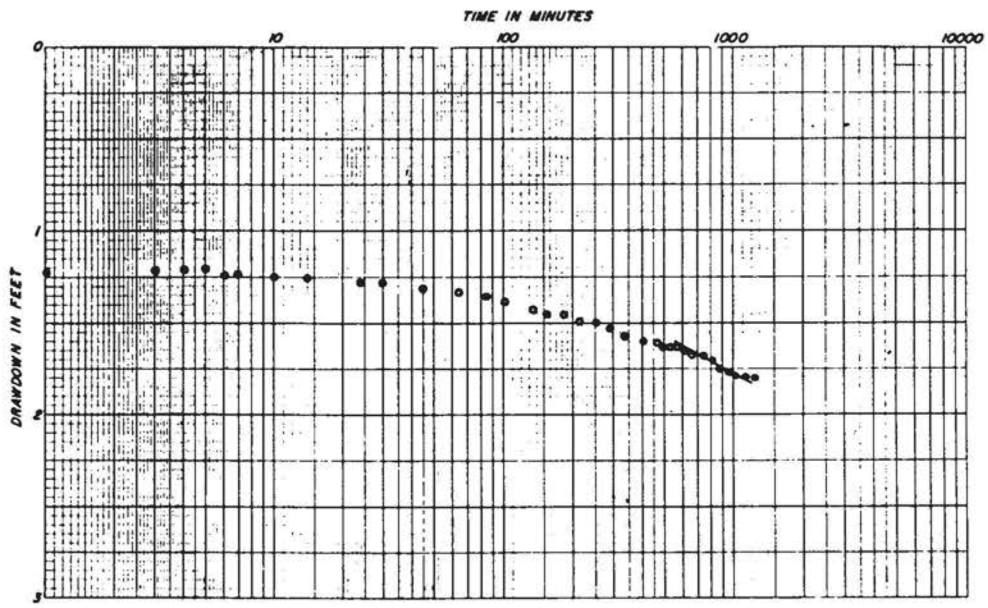
DAMES & MOORE

PLATE 3.8

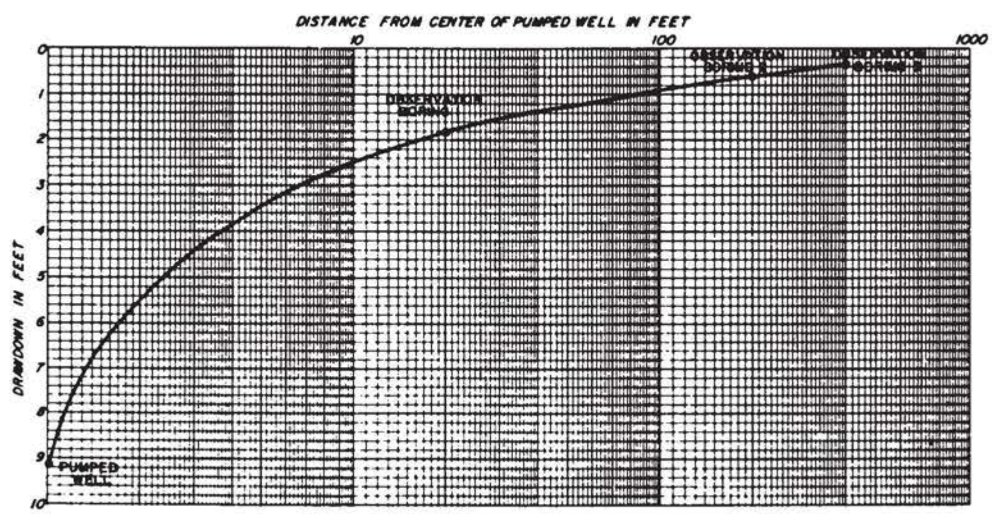
PLATE 3.9

DAMES & MOORE





DRAWDOWN Vs TIME CURVE FOR OBSERVATION BORING I



DRAWDOWN Vs DISTANCE CURVE AFTER 24 HOURS OF PUMPING AT 1100 GPM

PUMPING TEST DATA

DAMES & MOORE

PLATE 3.10

DEPTH IN FEET	LOG SYMBOLS	DESCRIPTION	COMPRESSIONAL WAVE VELOCITY (FT./SEC.)	POISSON'S RATIO	SHEAR WAVE VELOCITY (FT./SEC.)	UNIT WEIGHT* (LBS./CU.FT)
0		LOOSE SAND	1125	0.39	470	110
20		MEDIUM DENSE TO DENSE SAND WITH DISCONTINUOUS LOOSE ZONES OF SAND	4750	0.37	2150	125
50		DENSE TO VERY DENSE SAND WITH GRAVEL	6300	0.37	2860	130
180		SANDSTONE	9200	0.28	5020	148
4100		GRANITIC BASEMENT ROCK	(18,000)	(0.18)	(11,200)	(170)

TYPICAL GEOLOGIC COLUMN

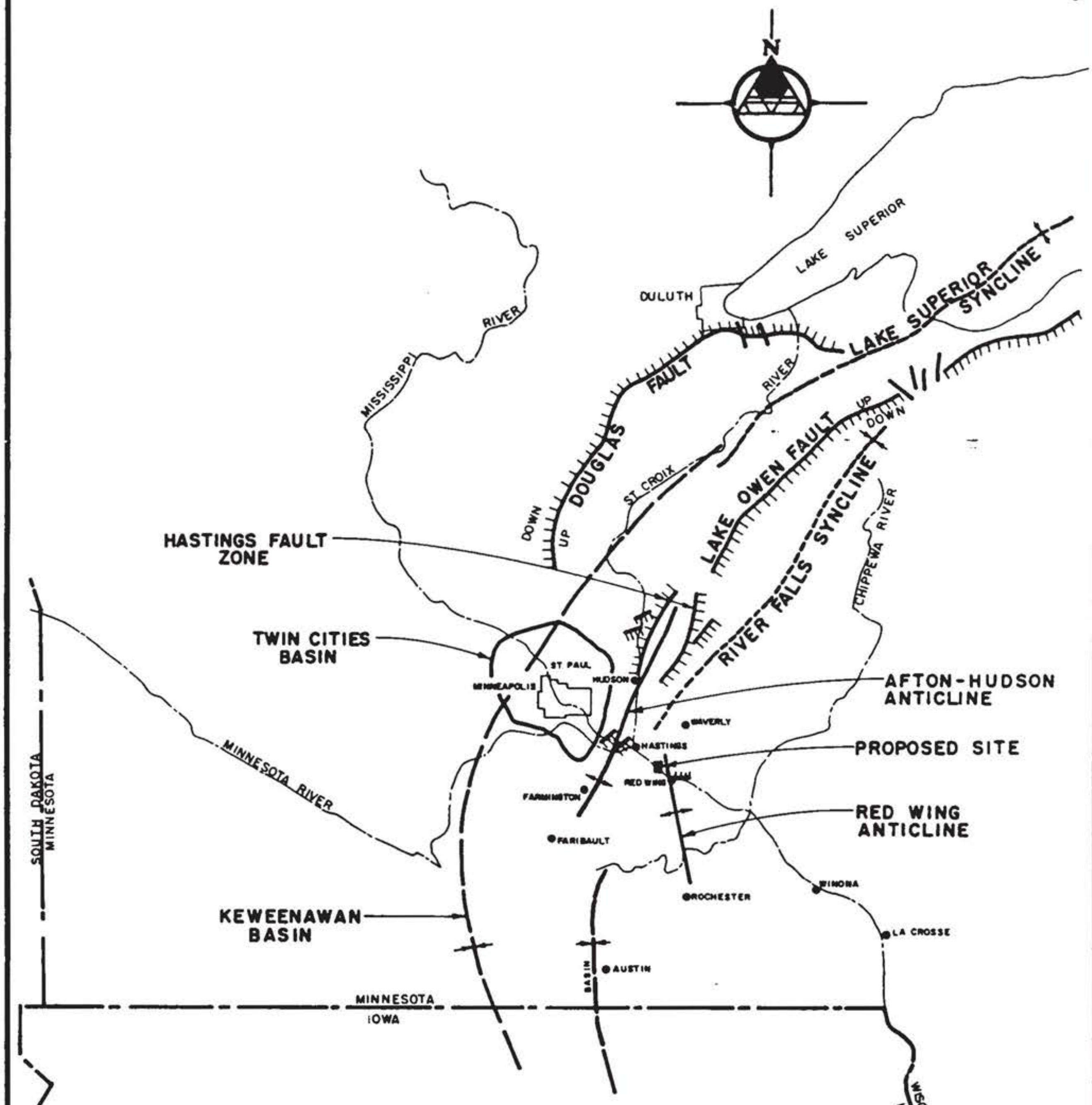
NOTE:

( ) ASSUMED

* WET DENSITY

DAMES & MOORE

PLATE 4.1



REGIONAL GEOLOGIC STRUCTURE

MILES

10 0 10 20 30 40 50 60 70 80 90 100

LEGEND

FAULT-KNOWN AND INFERRED

ANTICLINE AXIS

SYNCLINE AXIS

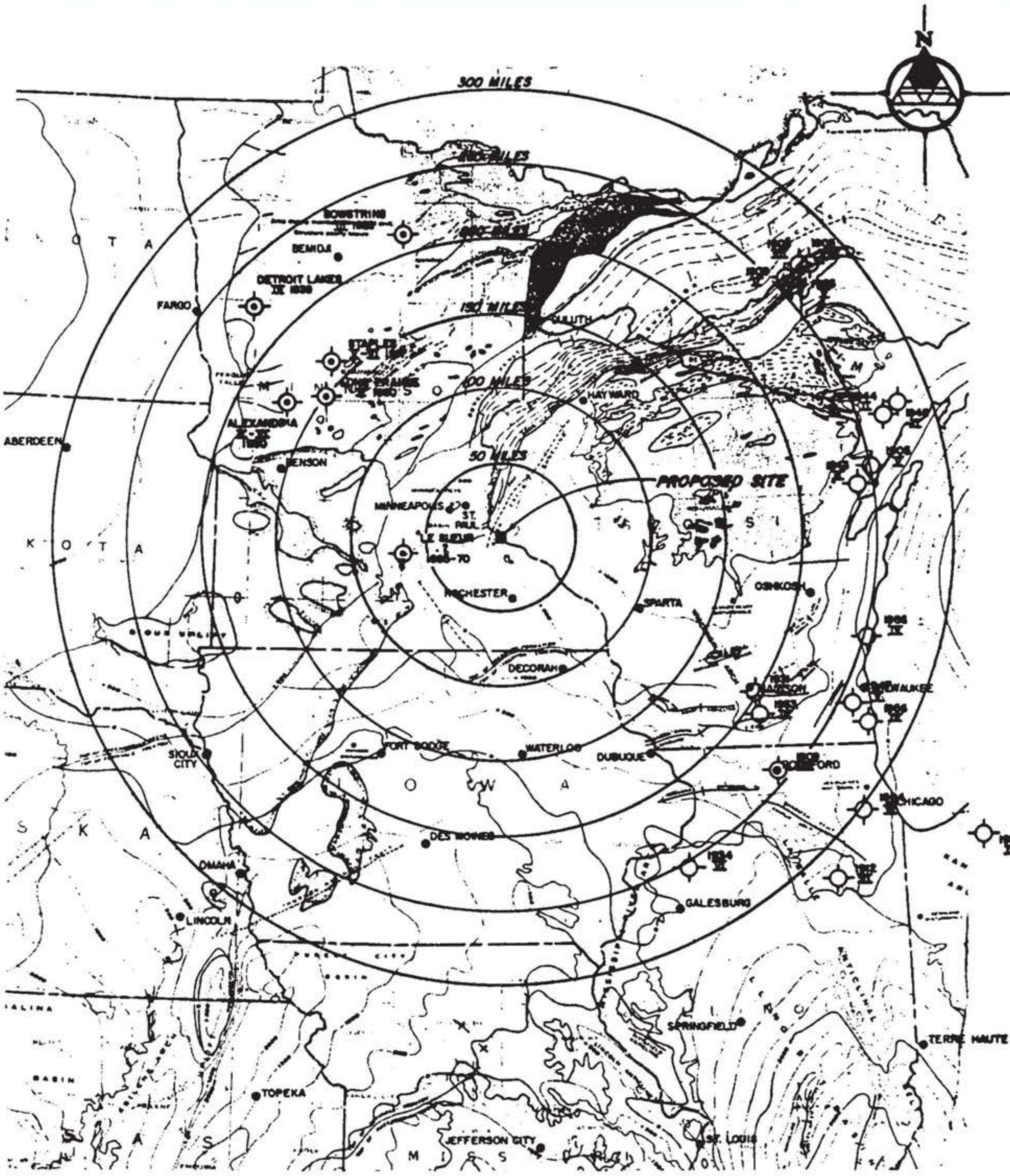
REFERENCES:

1-TECTONIC MAP OF THE UNITED STATES BY U.S.G.S. AND THE AMERICAN ASSOCIATION OF PETROLEUM GEOLOGISTS 1961

2-UNPUBLISHED STRUCTURE MAP BY MINNESOTA GEOLOGICAL SURVEY

DAMES & MOORE

PLATE 4.2



REGIONAL EARTHQUAKES

MILES
50 0 50 100

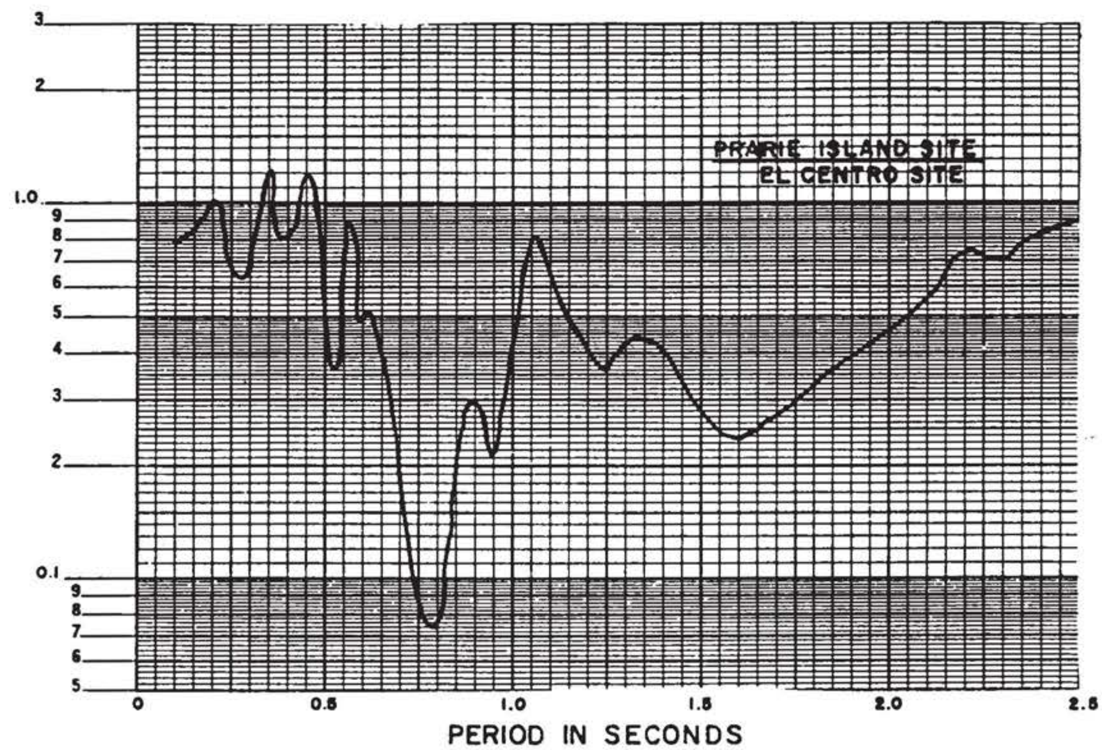
LEGEND:
LOCATION OF EPICENTER
1966 DATE OF EARTHQUAKE
III INTENSITY OF EARTHQUAKE

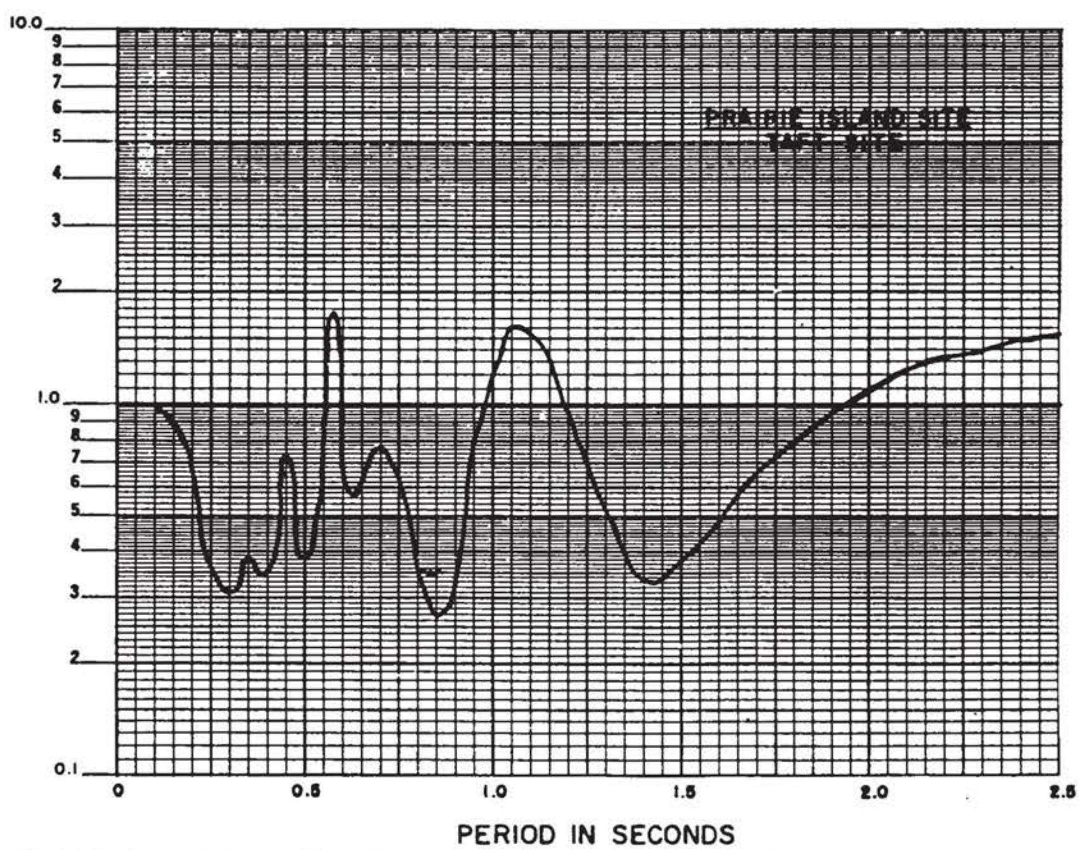
NOTE:

THIS MAP WAS PREPARED FROM A PORTION OF TECTONIC MAP OF THE UNITED STATES, BY THE UNITED STATES GEOLOGICAL SURVEY AND THE AMERICAN ASSOCIATION OF PETROLEUM GEOLOGISTS, 1962.

DAMES & MOORE

PLATE 4.3

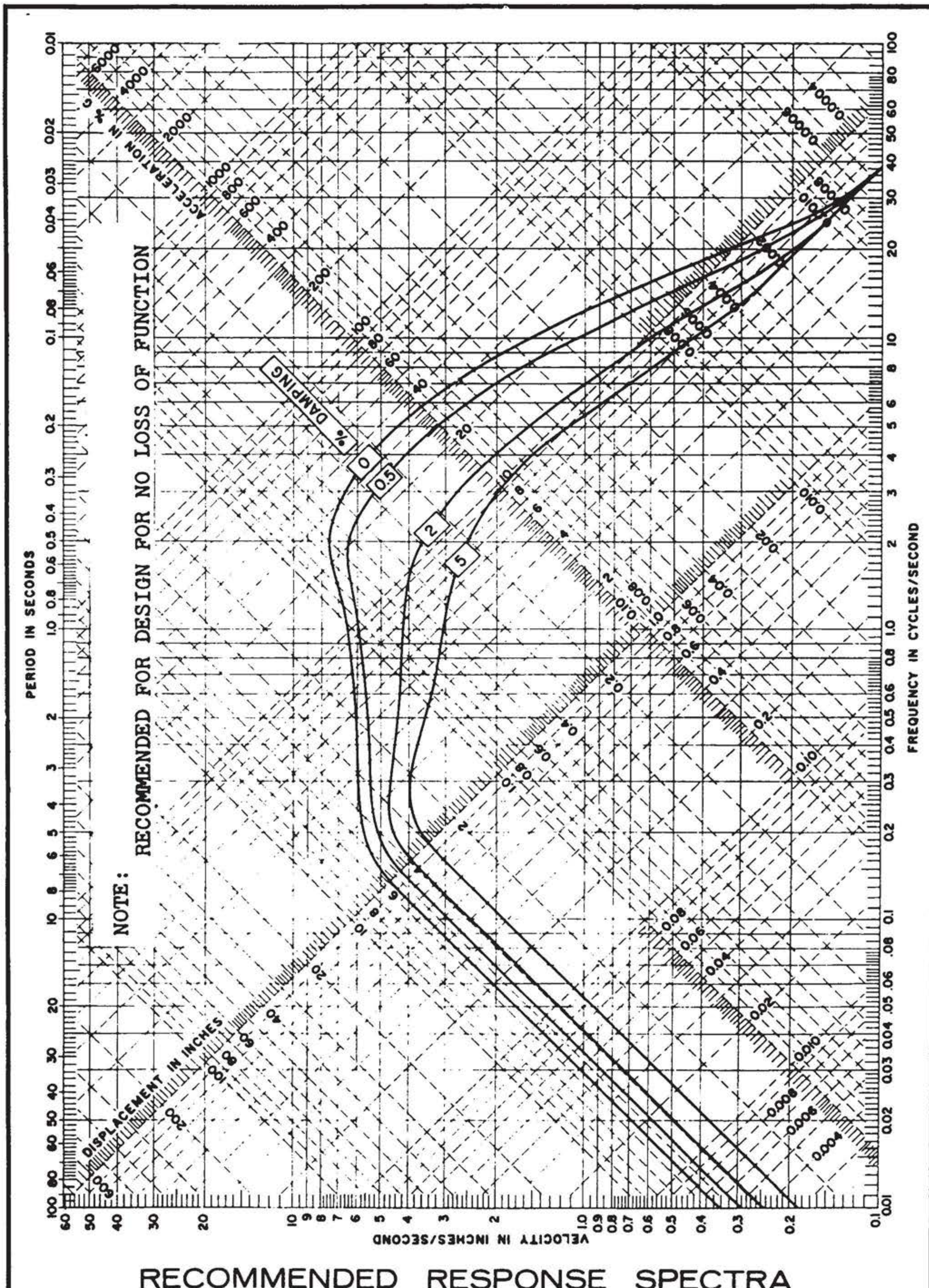




EARTHQUAKE WAVE AMPLIFICATION RATIOS

DAMES & MOORE
APPLIED EARTH SCIENCES

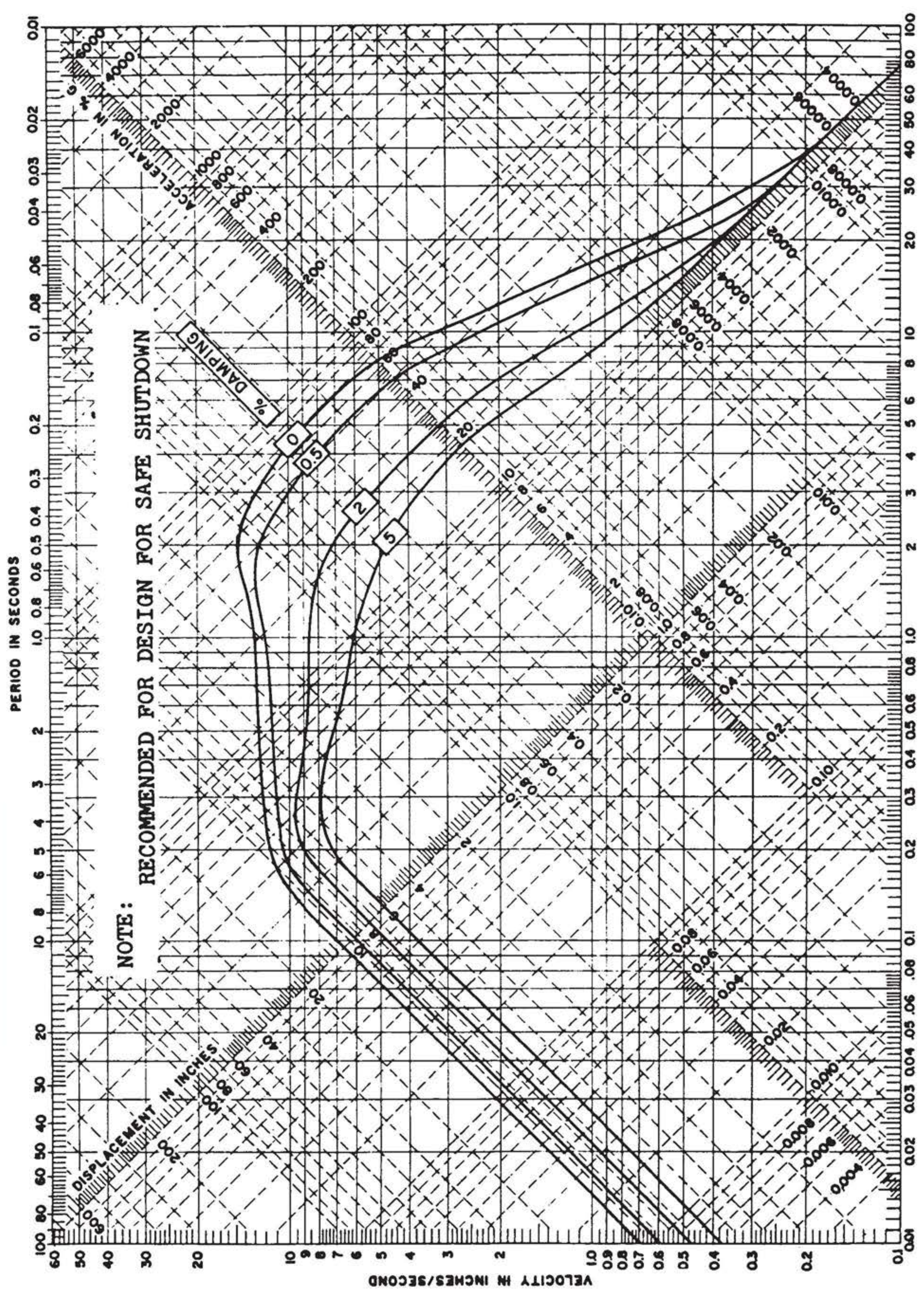
PLATE 4. 4



RECOMMENDED RESPONSE SPECTRA

DESIGN EARTHQUAKE - 6% ACCELERATION

DAMES & MOORE



RECOMMENDED RESPONSE SPECTRA

MAXIMUM CREDIBLE EARTHQUAKE - 12% ACCELERATION DAMES & MOORE

PLATE H 4.6 (Rev. I)

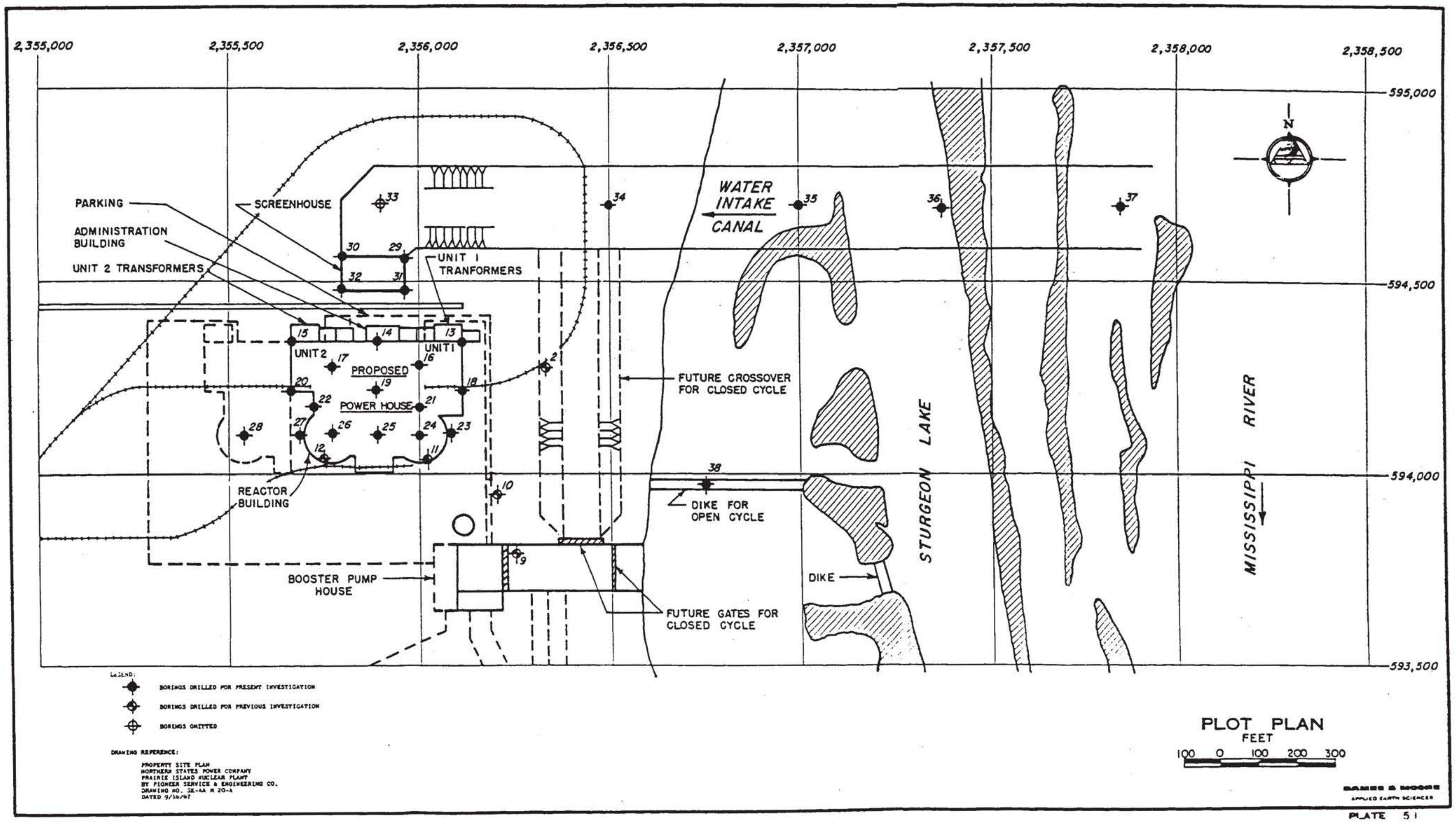
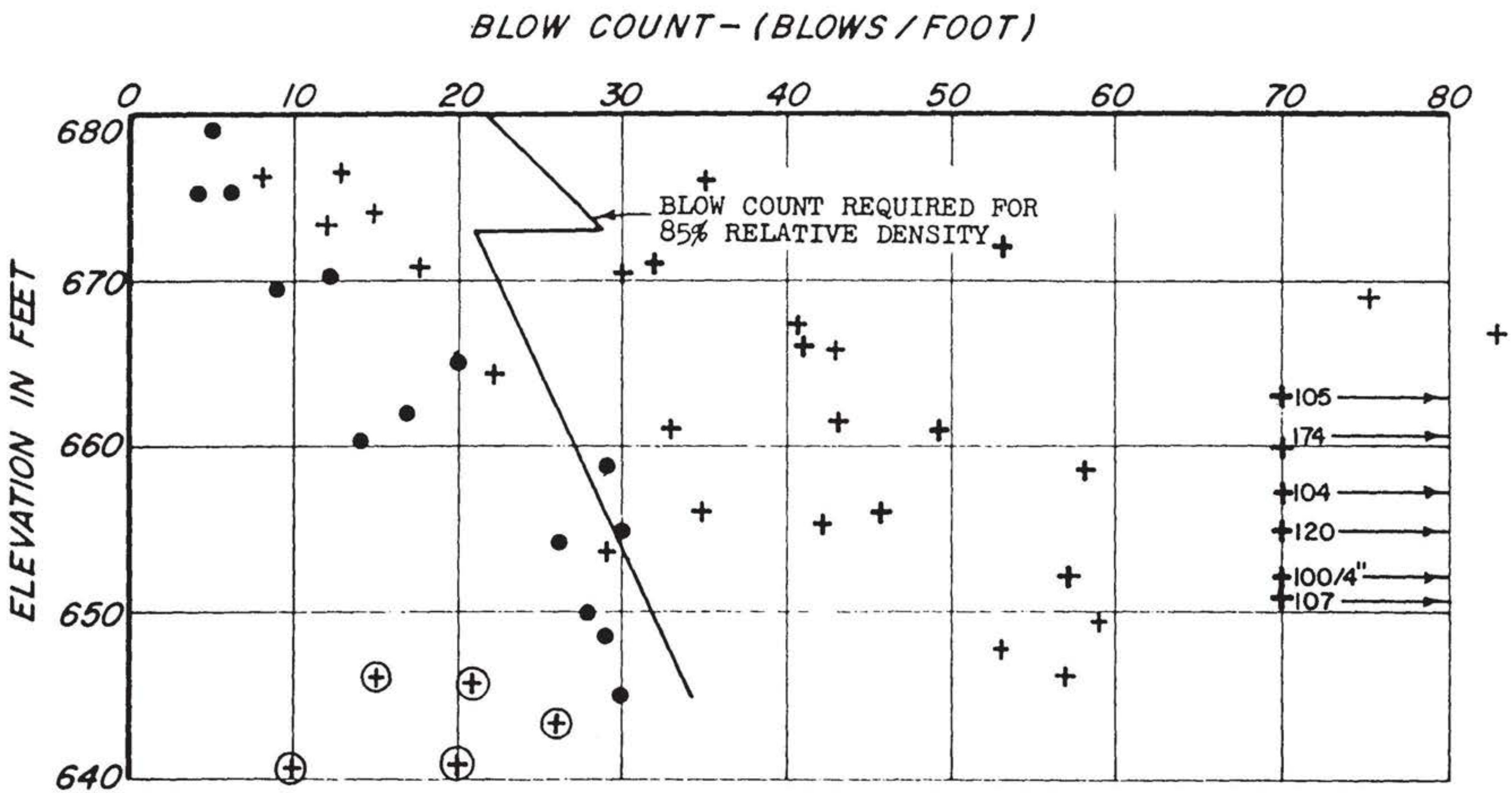
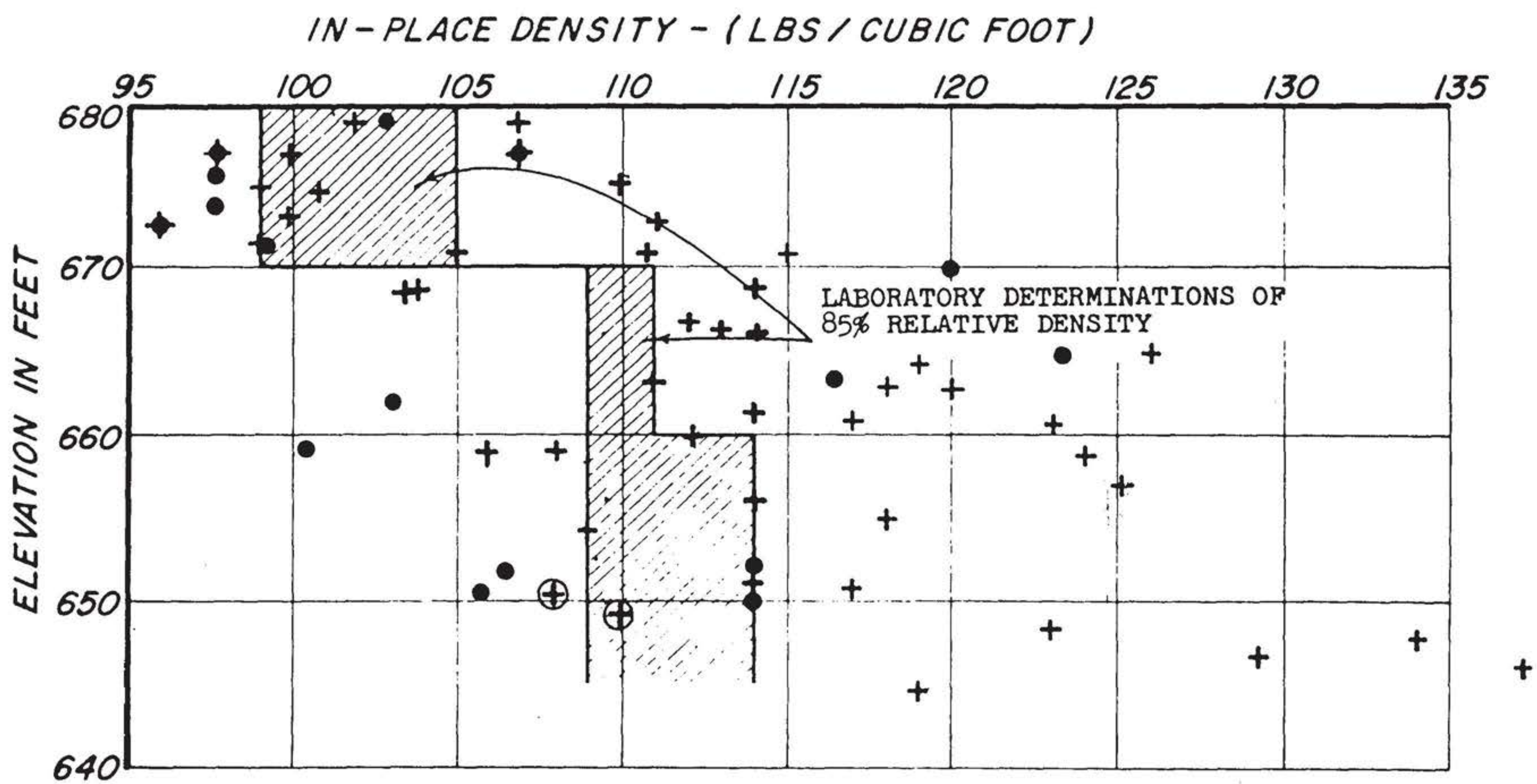

2,355,000
2,355,500
2,356,000
2,356,500
2,357,000
2,357,500
2,358,000
2,358,500
595,000
594,500
594,000
593,500
N
PARKING
SCREENHOUSE
ADMINISTRATION BUILDING
UNIT 2 TRANSFORMERS
UNIT I TRANFORMERS
WATER INTAKE CANAL
UNIT 2
UNIT I
PROPOSED
POWER HOUSE
REACTOR BUILDING
FUTURE CROSSOVER FOR CLOSED CYCLE
DIKE FOR OPEN CYCLE
DIKE
BOOSTER PUMP HOUSE
FUTURE GATES FOR CLOSED CYCLE
STURGEON LAKE
MISSISSIPPI RIVER
2
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
37
38
LEGEND:
BORINGS DRILLED FOR PRESENT INVESTIGATION
BORINGS DRILLED FOR PREVIOUS INVESTIGATION
BORINGS OMITTED
DRAWING REFERENCE:
PROPERTY SITE PLAN
NORTHERN STATES POWER COMPANY
PRAIRIE ISLAND NUCLEAR PLANT
BY PIONEER SERVICE & ENGINEERING CO.
DRAWING NO. SK-AA # 20-A
DATED 9/16/67
PLOT PLAN
FEET
100 0 100 200 300
APPLIED EARTH SCIENCES

PLATE 51



SUMMARY OF STANDARD PENETRATION TESTS
TEST PATTERN I

KEY:
● BEFORE VIBROFLOTATION
\+ AFTER VIBROFLOTATION
⊕ BELOW DEPTH ATTAINED BY VIBROFLOTATION



SUMMARY OF IN-PLACE DENSITY TESTS
TEST PATTERN I

KEY:

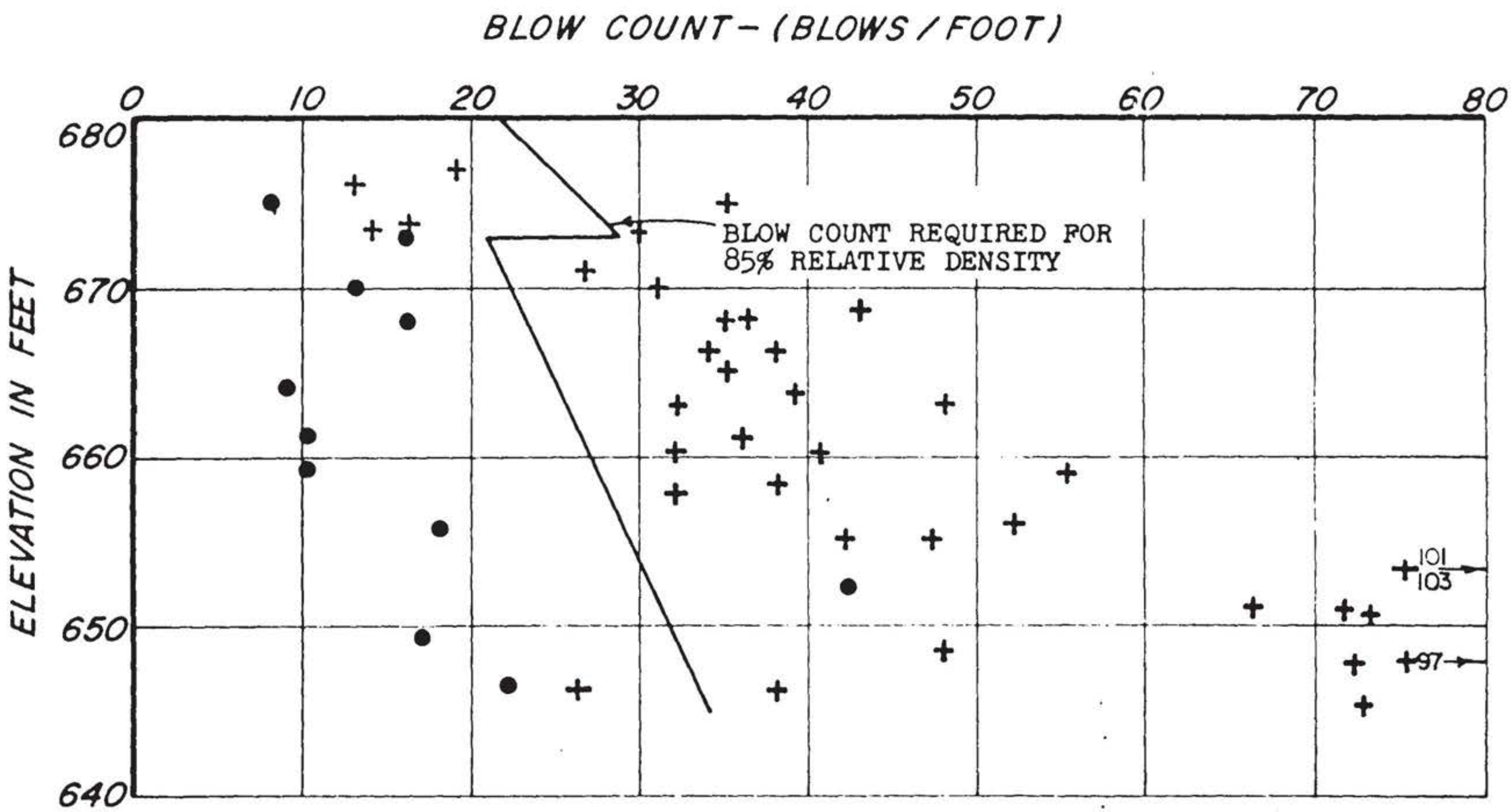
● BEFORE VIBROFLOTATION

+ AFTER VIBROFLOTATION

⊕ BELOW DEPTH ATTAINED BY VIBROFLOTATION

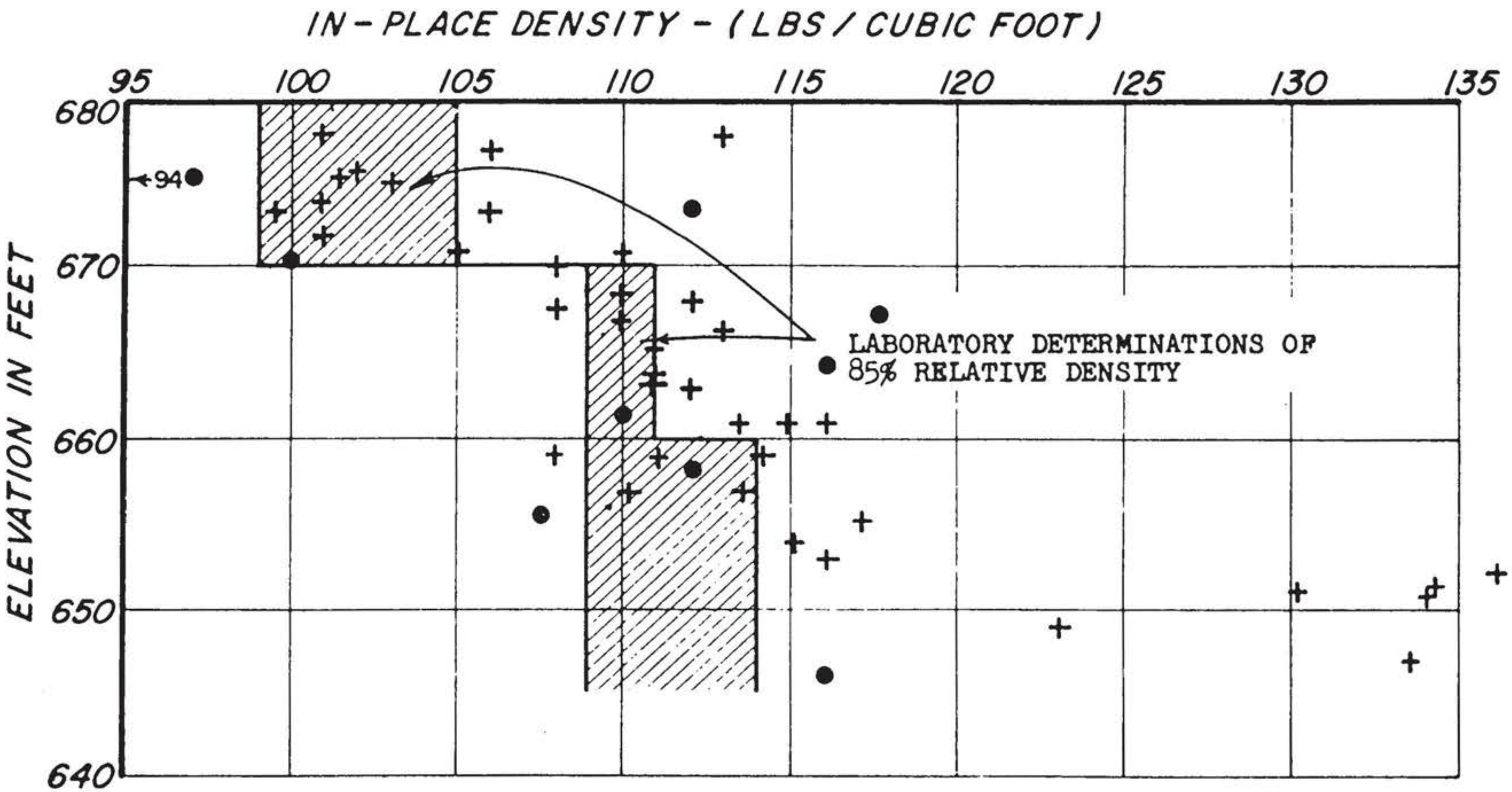
DAMES & MOORE

PLATE 5.3



SUMMARY OF STANDARD PENETRATION TESTS
TEST PATTERN 2

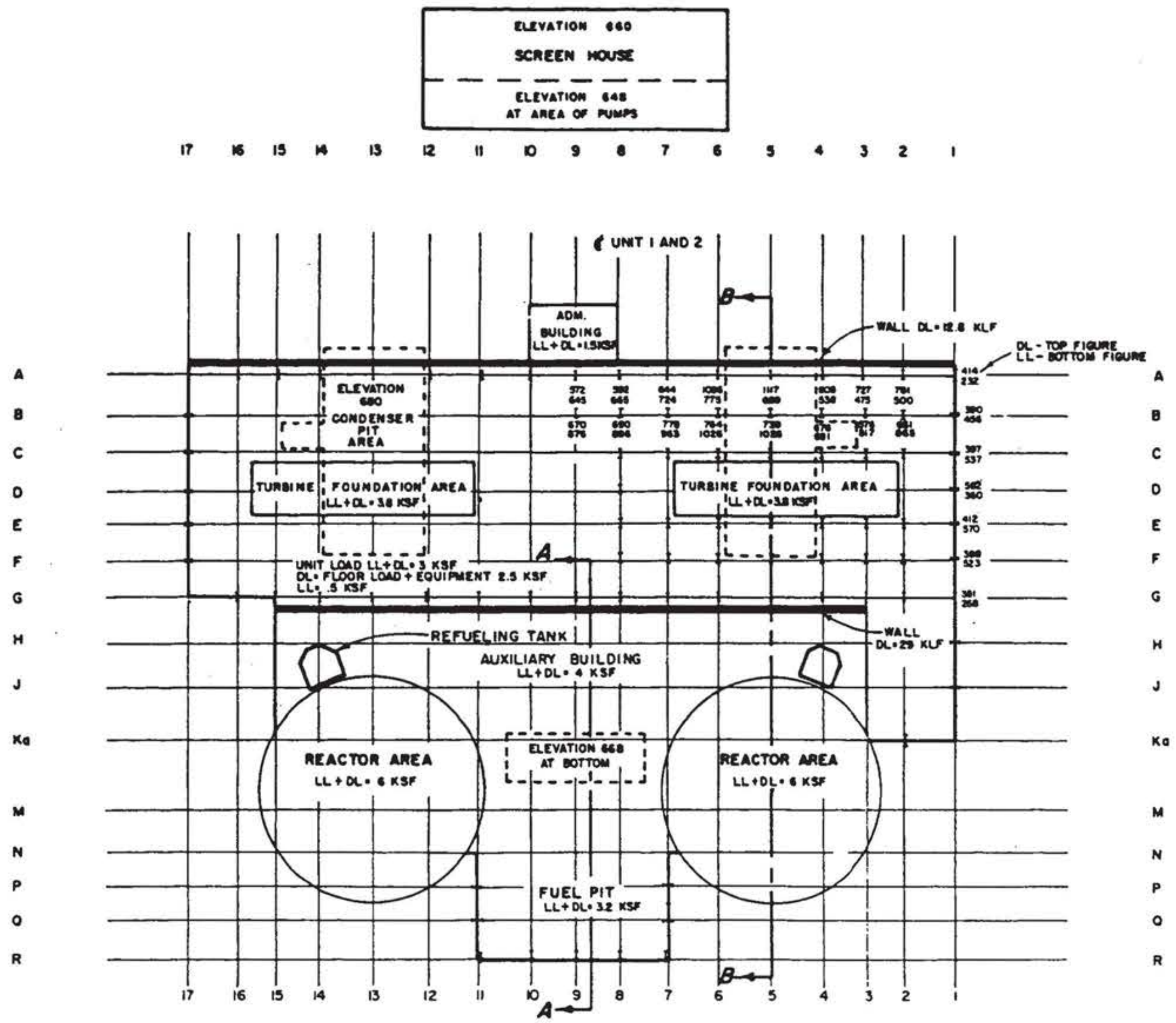
KEY:
● BEFORE VIBROFLOTATION
+ AFTER VIBROFLOTATION



SUMMARY OF IN-PLACE DENSITY TESTS
TEST PATTERN 2

KEY:
● BEFORE VIBROFLOTATION
+ AFTER VIBROFLOTATION

DAMES & MOORE



PROPOSED LAYOUT

FEET
50 0 50 100

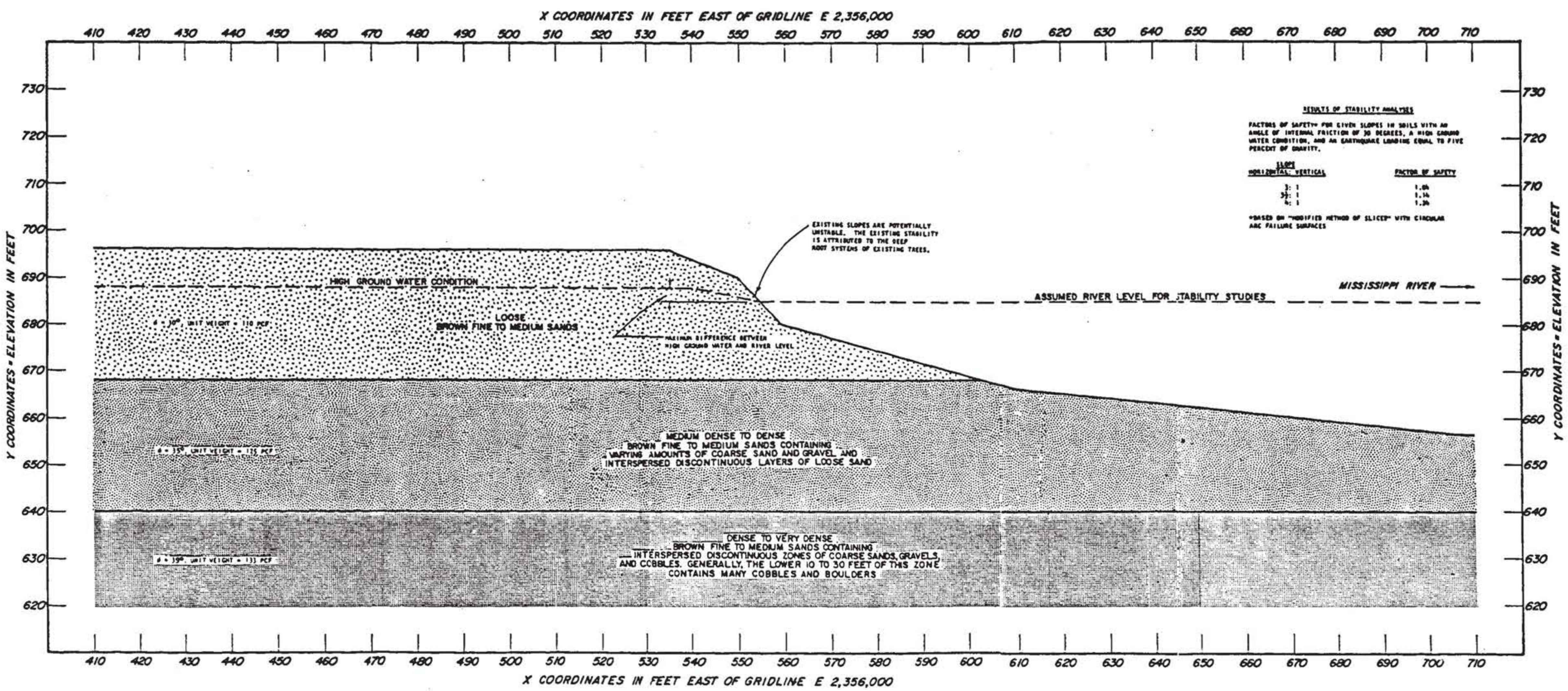
NOTE:
LOADS SHOWN @ COLUMN ROW A & G
DO NOT INCLUDE CRANE LOAD (+400K)

DRAWING REFERENCE:

SKETCH FOR FOUNDATION LOADS
PREPARED BY PIONEER SERVICE & ENGINEERING CO.
FOR THE NORTHERN STATES POWER COMPANY
PRAIRIE ISLAND NUCLEAR GENERATING PLANT
UNITS 1&2 RED WING, MINNESOTA.

DAMES & MOORE

PLATE 5.6



CROSS SECTION ALONG STATION N 594,000

SLOPE STABILITY STUDIES

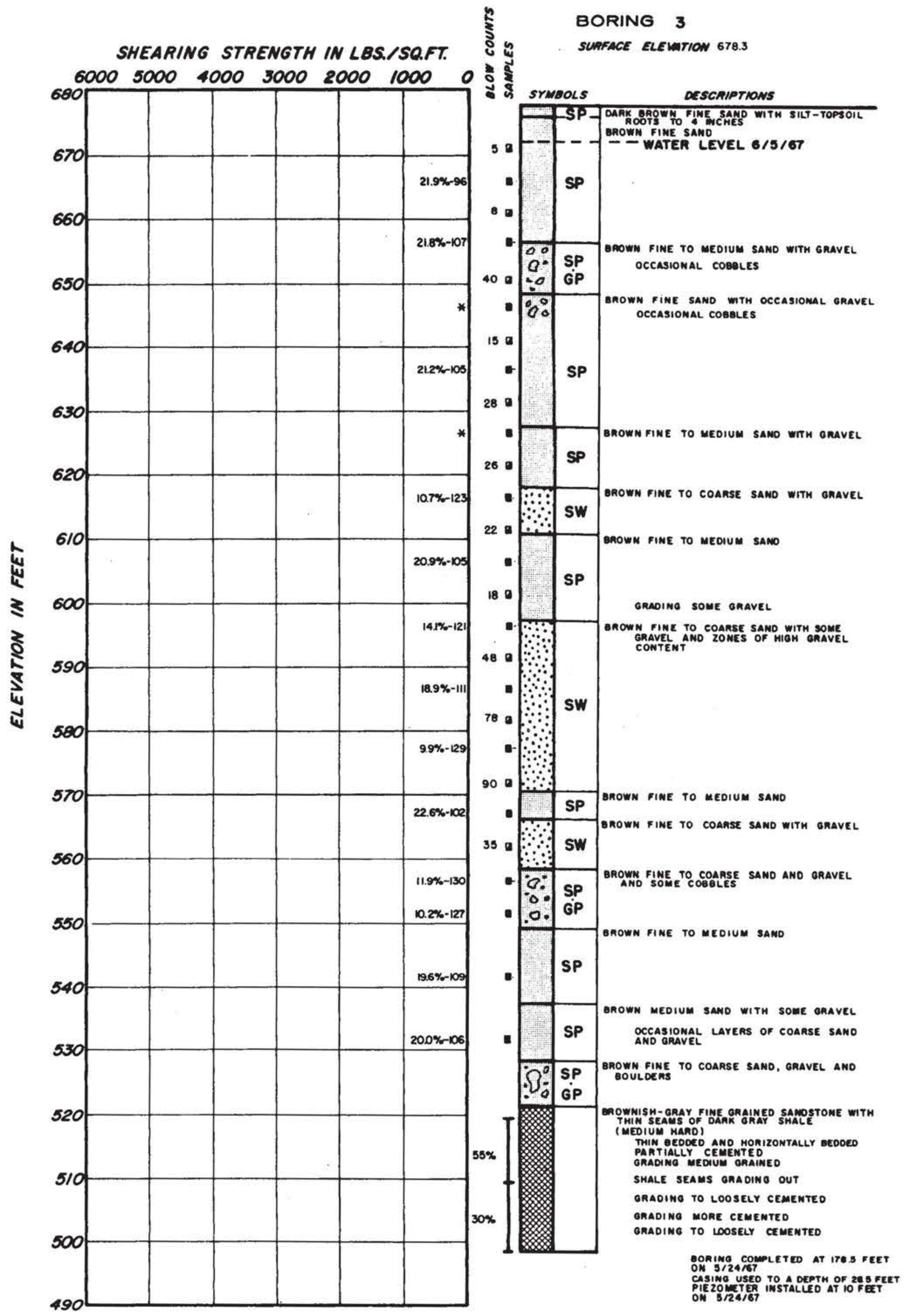
NOTES:

ELEVATIONS REFER TO MEAN SEA LEVEL DATUM - ADJUSTED 1929.

THE DEPTH, THICKNESS AND COMPOSITION OF THE SOIL STRATA DEPICTED WERE EXTRAPOLATED FROM BORINGS 9 AND 10.

DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE 5.7



NOTES:

GROUND SURFACE ELEVATIONS REFER TO MEAN SEA LEVEL DATUM - 1929 ADJUSTMENT

■ INDICATES A DAMES & MOORE UNDISTURBED SAMPLE (TYPE U SAMPLER)

▣ INDICATES STANDARD PENETRATION TEST. FIGURES UNDER THE BLOW COUNT COLUMN INDICATE THE NUMBER OF BLOWS REQUIRED TO DRIVE A SAMPLER, WITH AN OUTSIDE DIAMETER OF TWO INCHES, ONE FOOT WITH A 140 POUND WEIGHT FALLING 30 INCHES.

100% I INDICATES DEPTH, LENGTH, AND PERCENT OF CORE RUN RECOVERED.

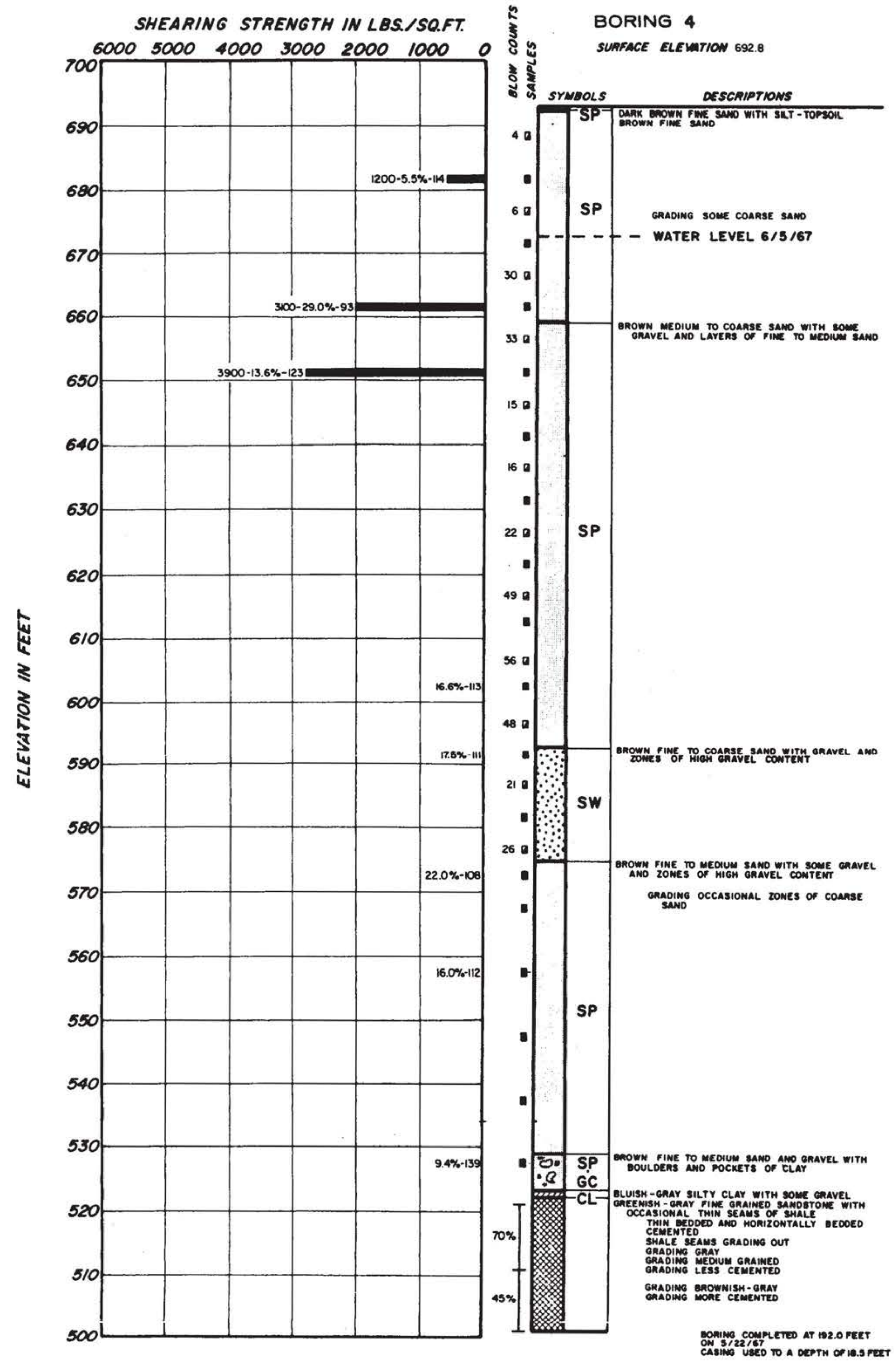
* INDICATES SAMPLE SUBJECTED TO DYNAMIC TRIAXIAL COMPRESSION TEST, DATA REGARDING TEST IS LOCATED IN PART 6 OF REPORT.

** INDICATES SAMPLE SUBJECTED TO DYNAMIC CONFINED COMPRESSION TEST. DATA REGARDING TEST IS LOCATED IN PART 6 OF REPORT.

LOG OF BORINGS

DAMES & MOORE

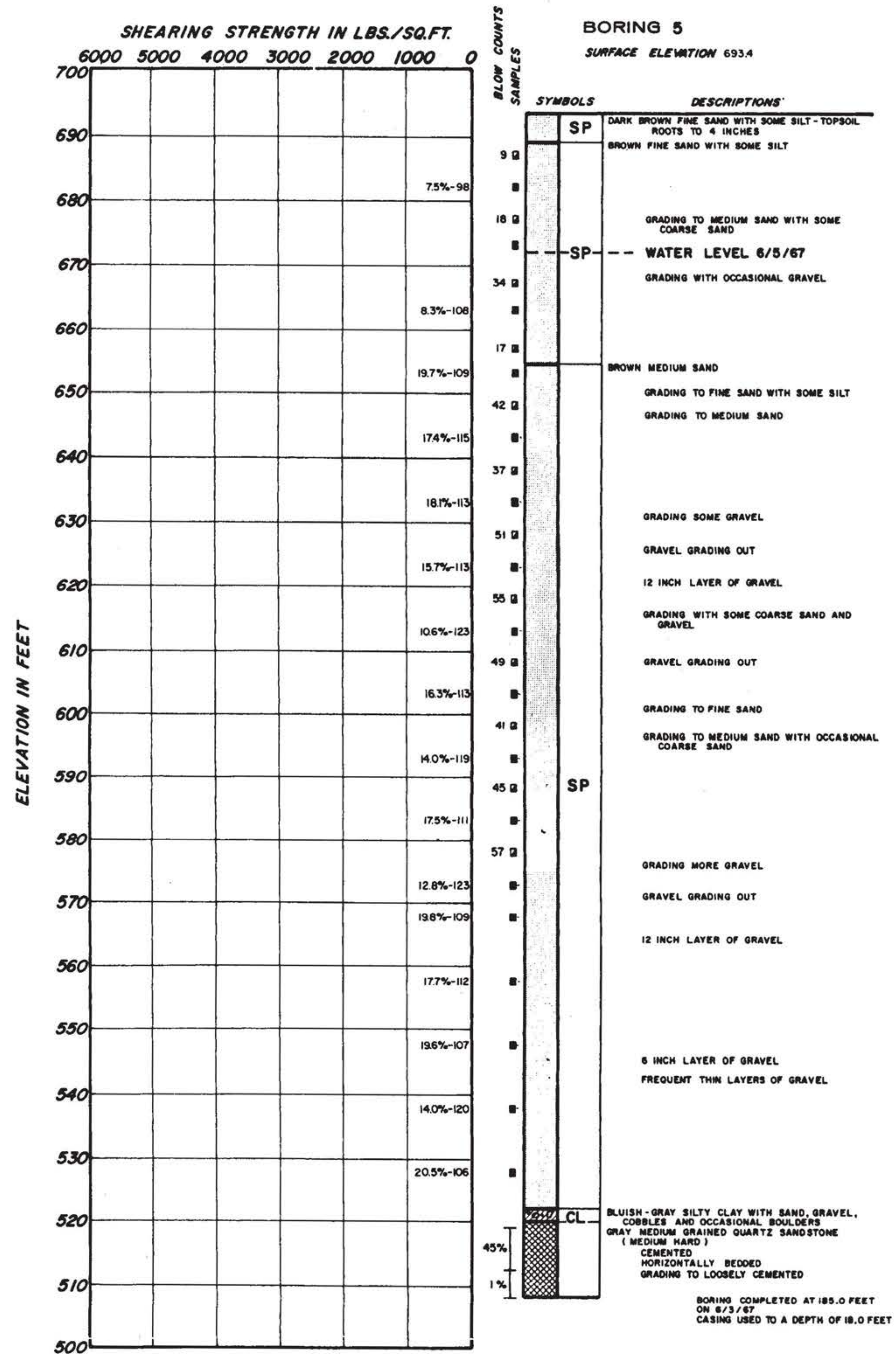
PLATE 6.1



LOG OF BORINGS

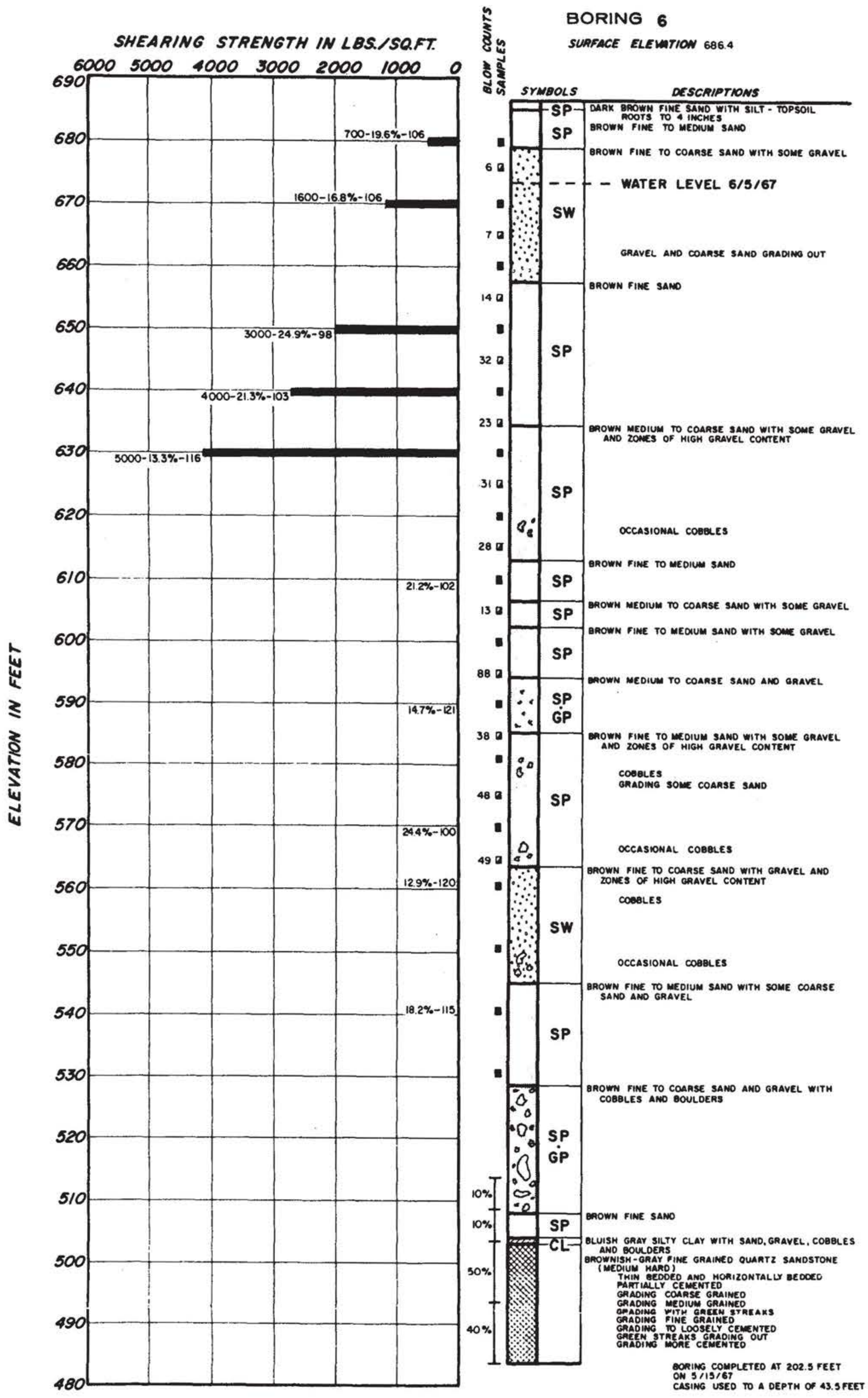
DAMES & MOORE

PLATE 6.2



LOG OF BORINGS

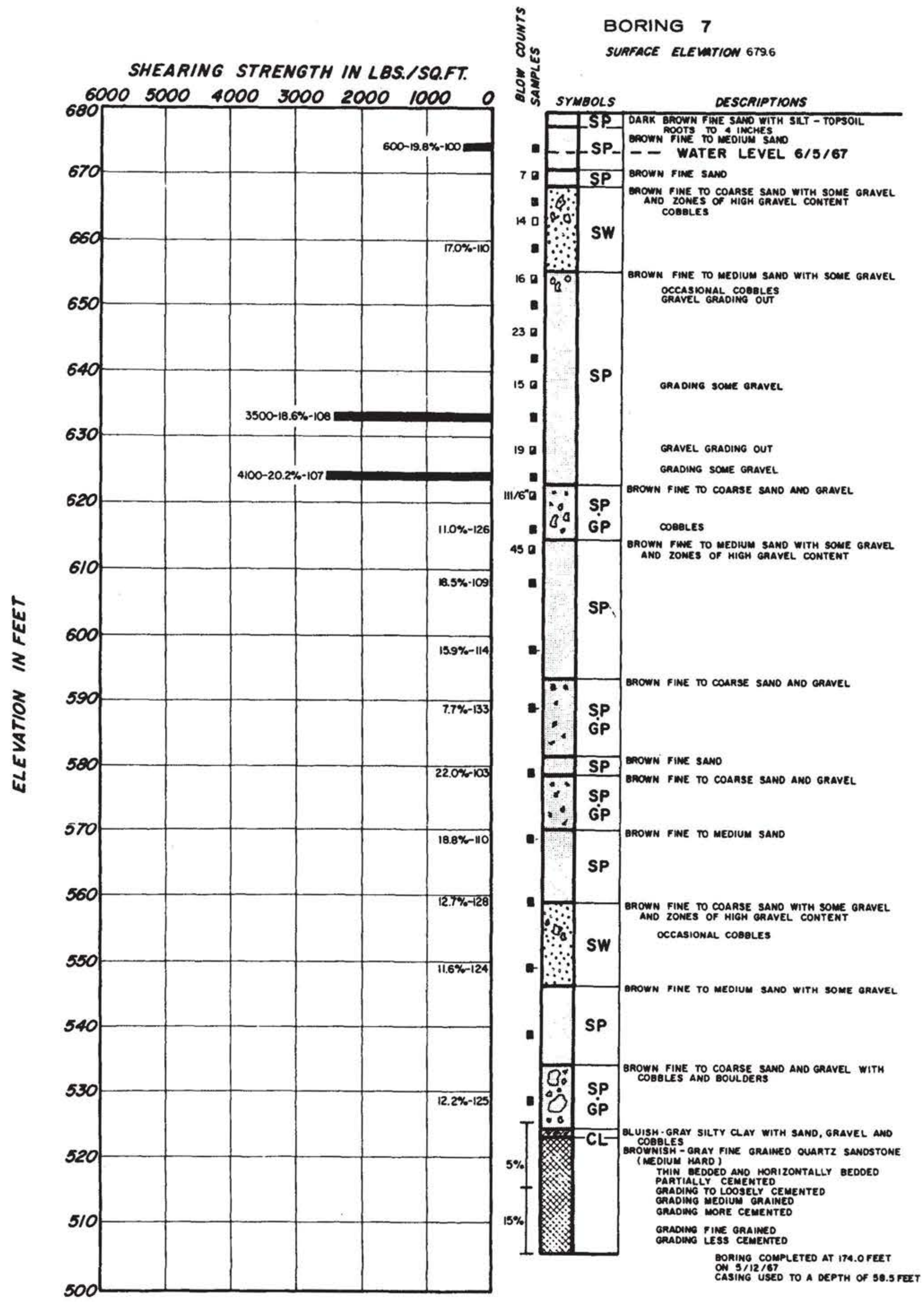
DAMES & MOORE



LOG OF BORINGS

DAMES & MOORE

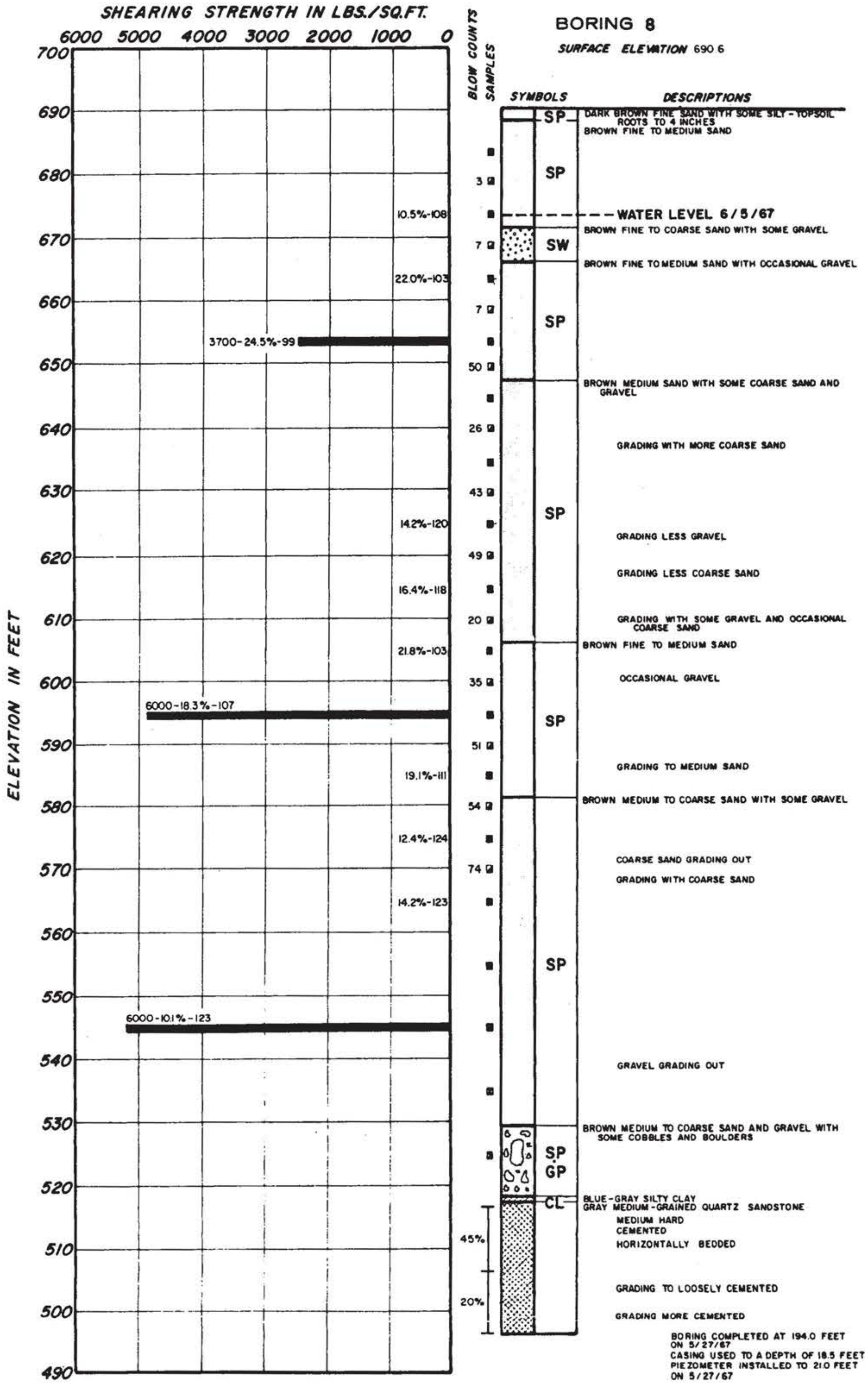
PLATE 6.4



LOG OF BORINGS

DAMES & MOORE

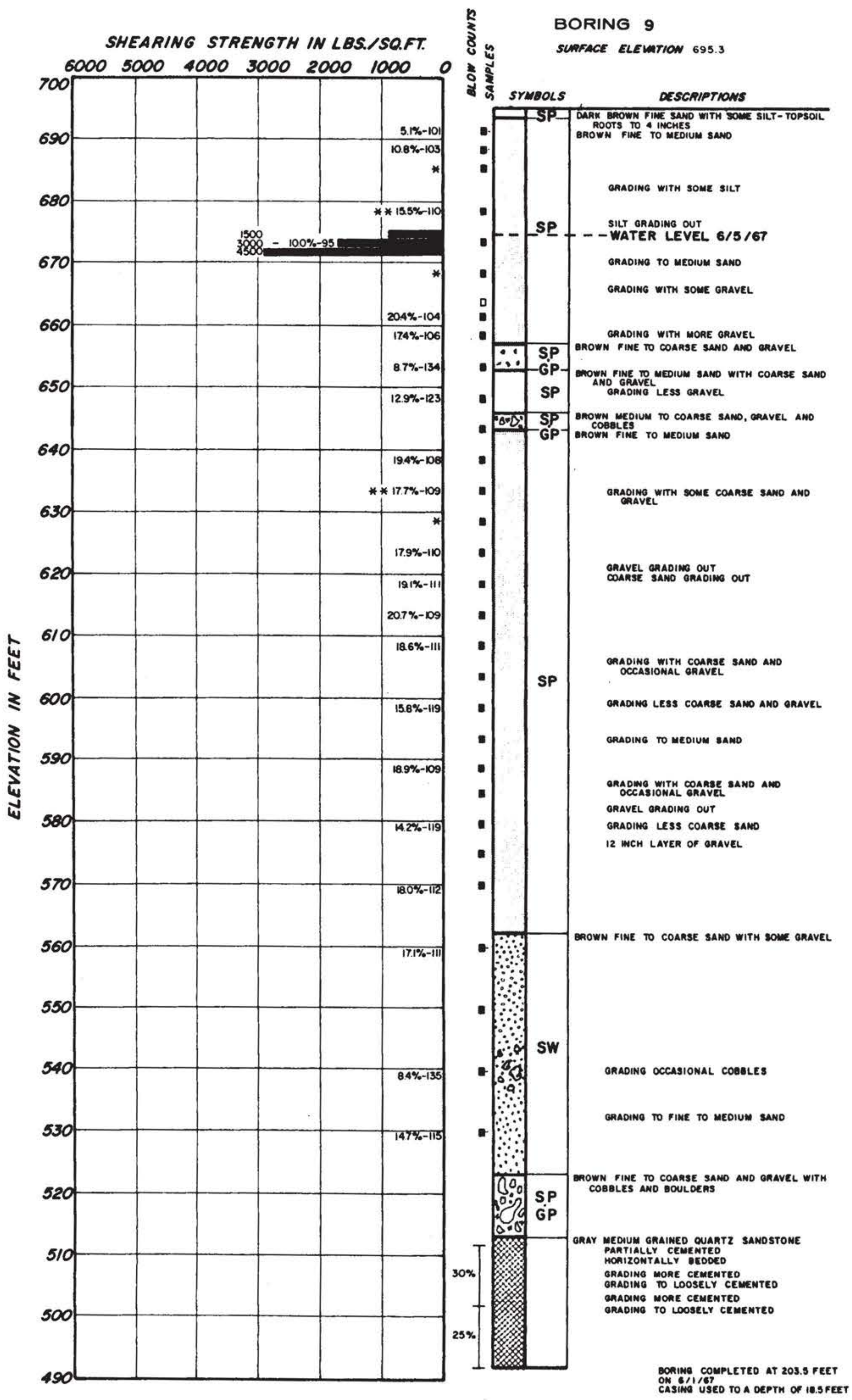
PLATE 6.5



LOG OF BORINGS

DAMES & MOORE

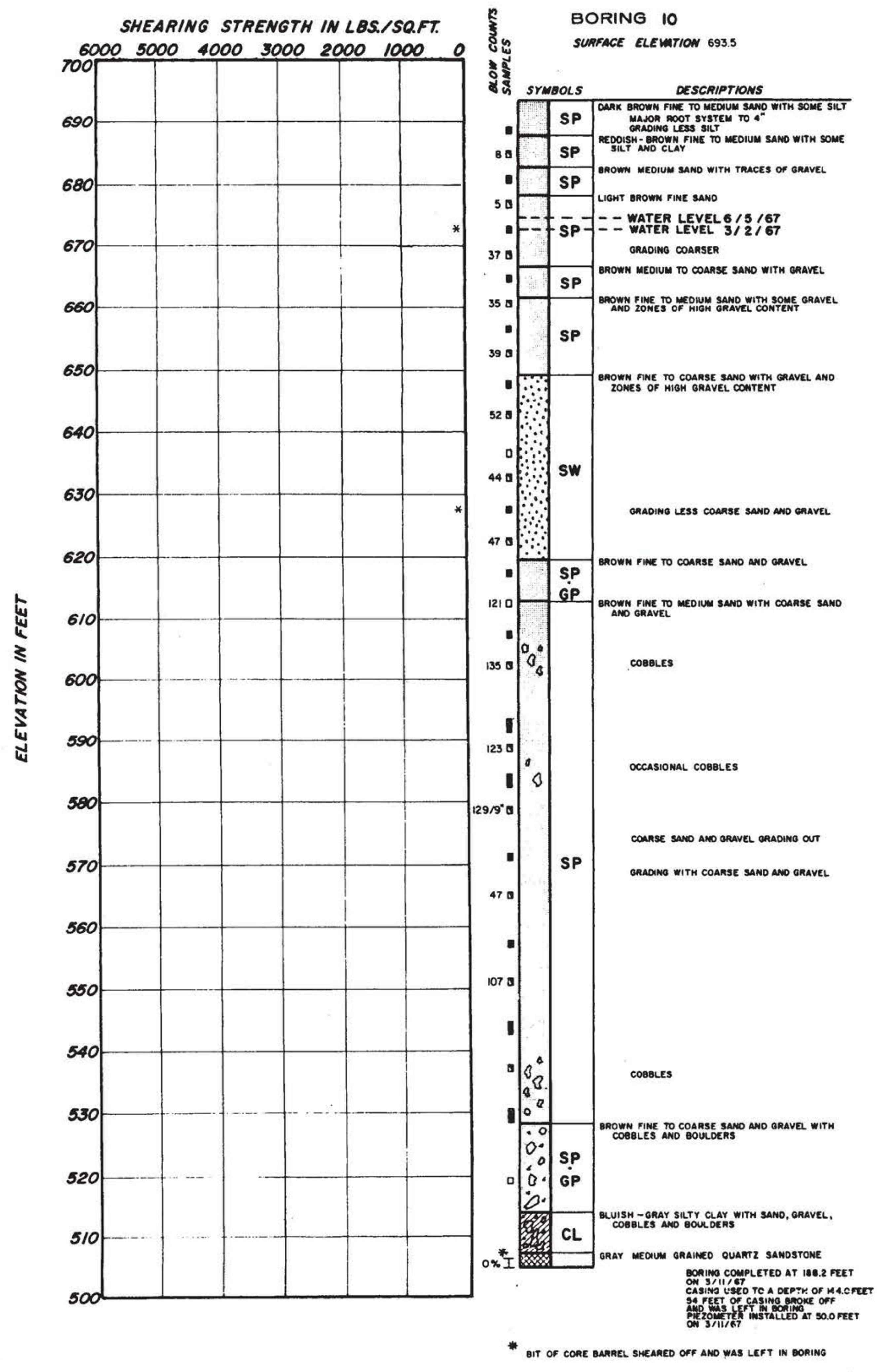
PLATE 6.6



LOG OF BORINGS

DAMES & MOORE

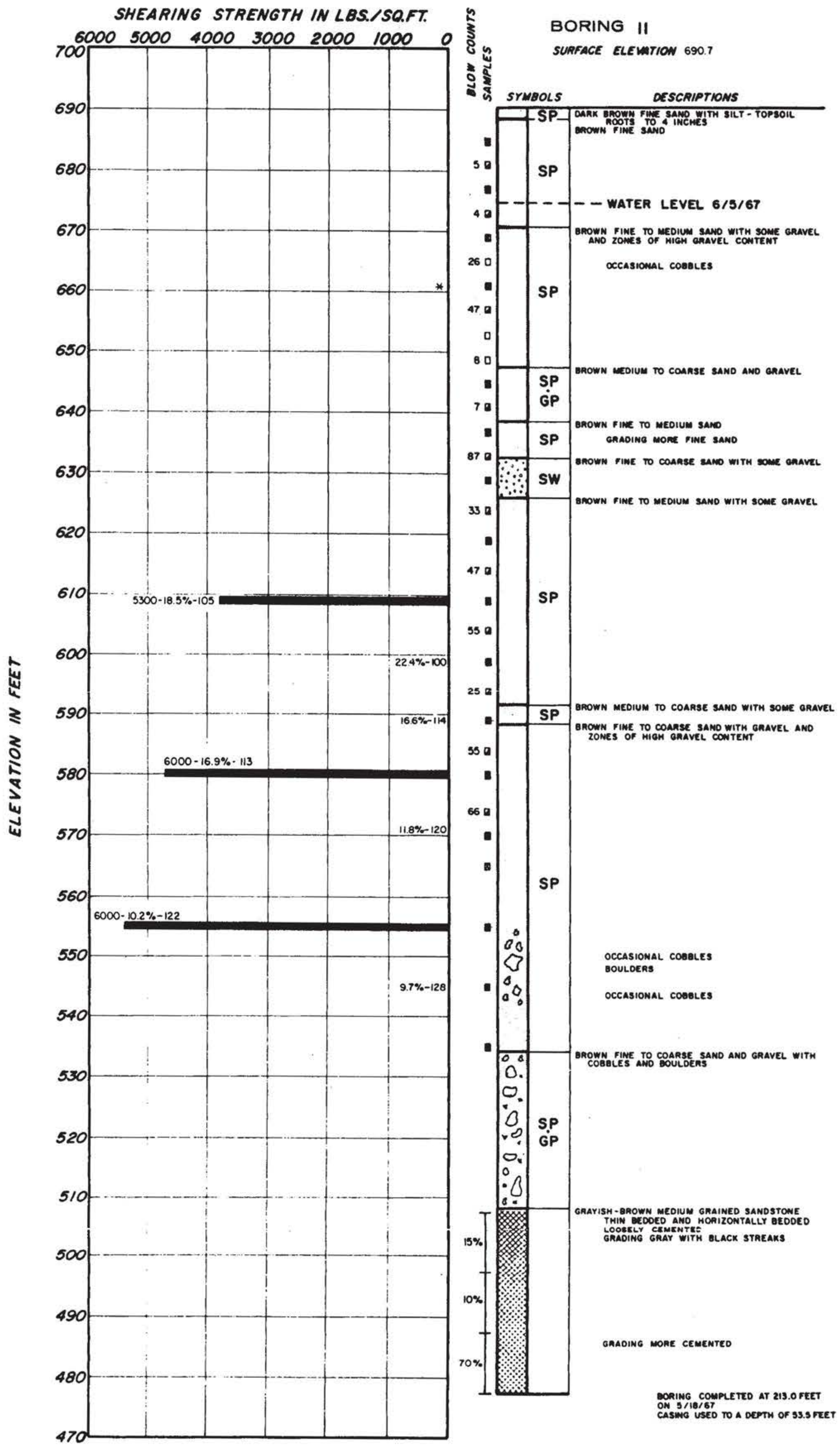
PLATE 6.7



LOG OF BORINGS

DAMES & MOORE

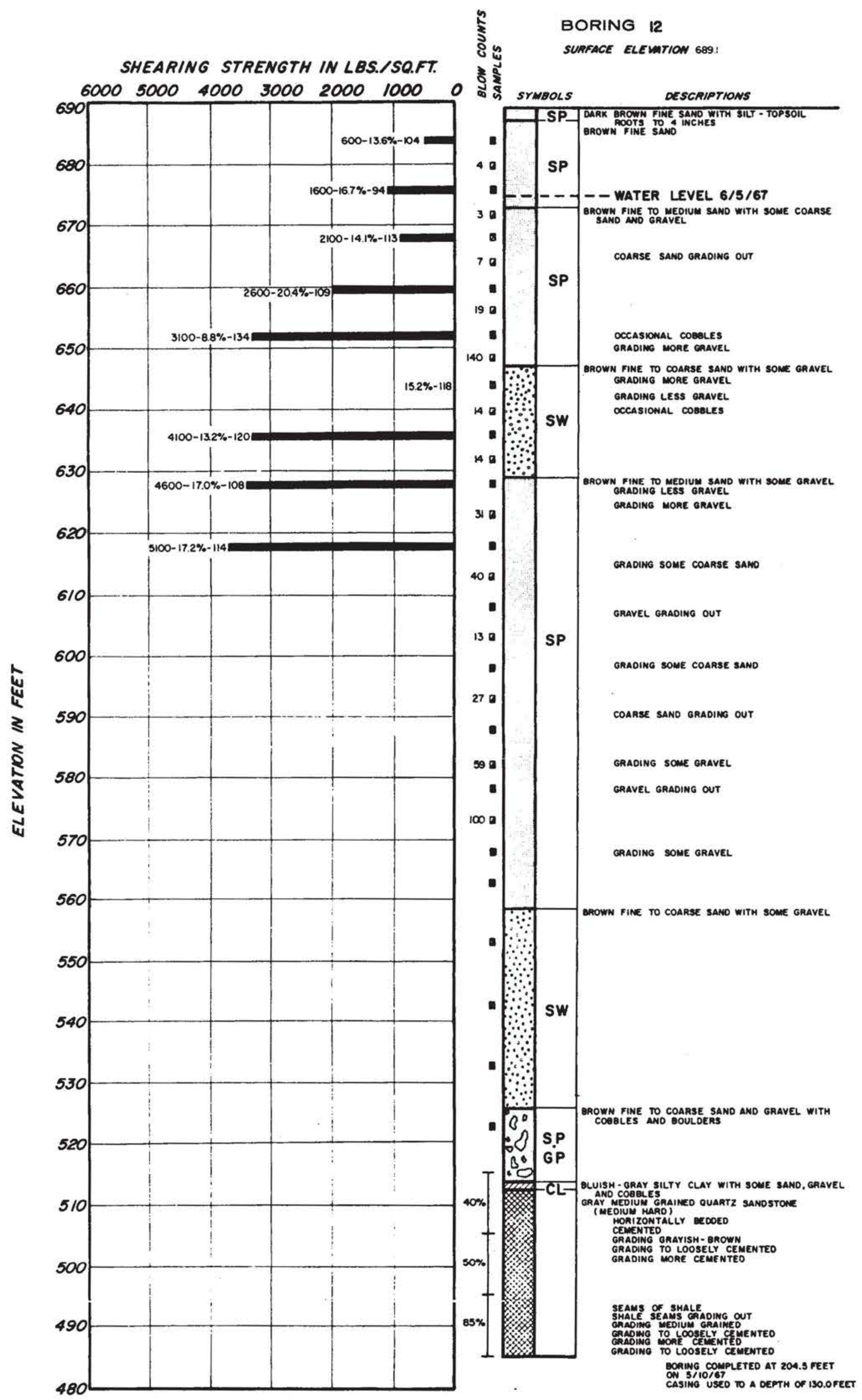
PLATE 6.8



LOG OF BORINGS

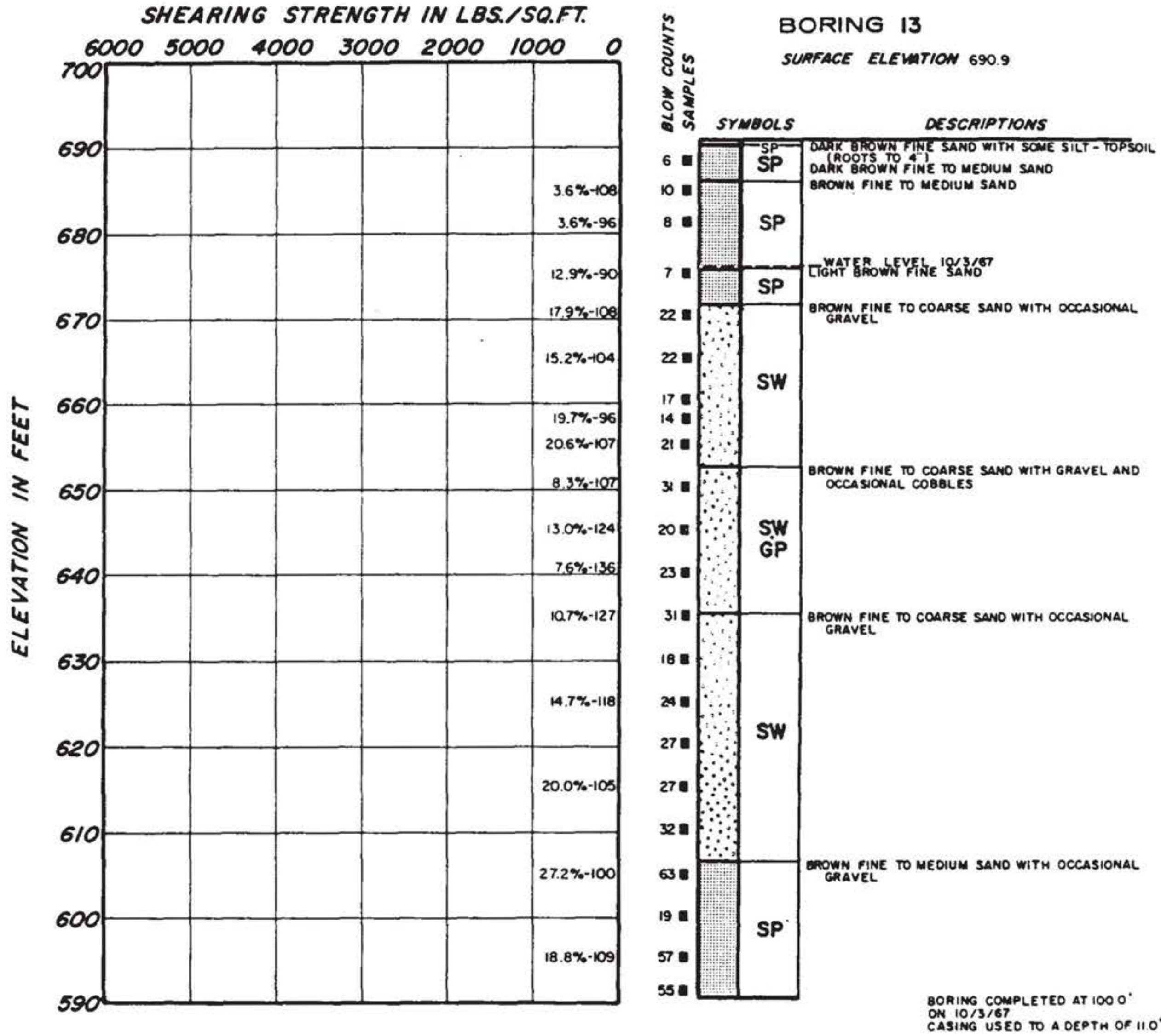
DAMES & MOORE

PLATE 6.9



LOG OF BORINGS

DAMES & MOORE



NOTES:

GROUND SURFACE ELEVATIONS REFER TO MEAN SEA LEVEL DATUM - 1929 ADJUSTMENT

■ INDICATES A DAMES & MOORE UNDISTURBED SAMPLE (TYPE U SAMPLER). FIGURES UNDER THE BLOW COUNT COLUMN INDICATE THE NUMBER OF BLOWS REQUIRED TO DRIVE A SAMPLER, WITH AN OUTSIDE DIAMETER OF 3.25 INCHES, ONE FOOT WITH A 350 POUND WEIGHT FALLING 18 INCHES.

P INDICATES SAMPLER WAS ADVANCED HYDRAULICALLY.

◘ INDICATES STANDARD PENETRATION TEST. FIGURES UNDER THE BLOW COUNT COLUMN INDICATE THE NUMBER OF BLOWS REQUIRED TO DRIVE A SAMPLER, WITH AN OUTSIDE DIAMETER OF TWO INCHES, ONE FOOT WITH A 140 POUND WEIGHT FALLING 30 INCHES.

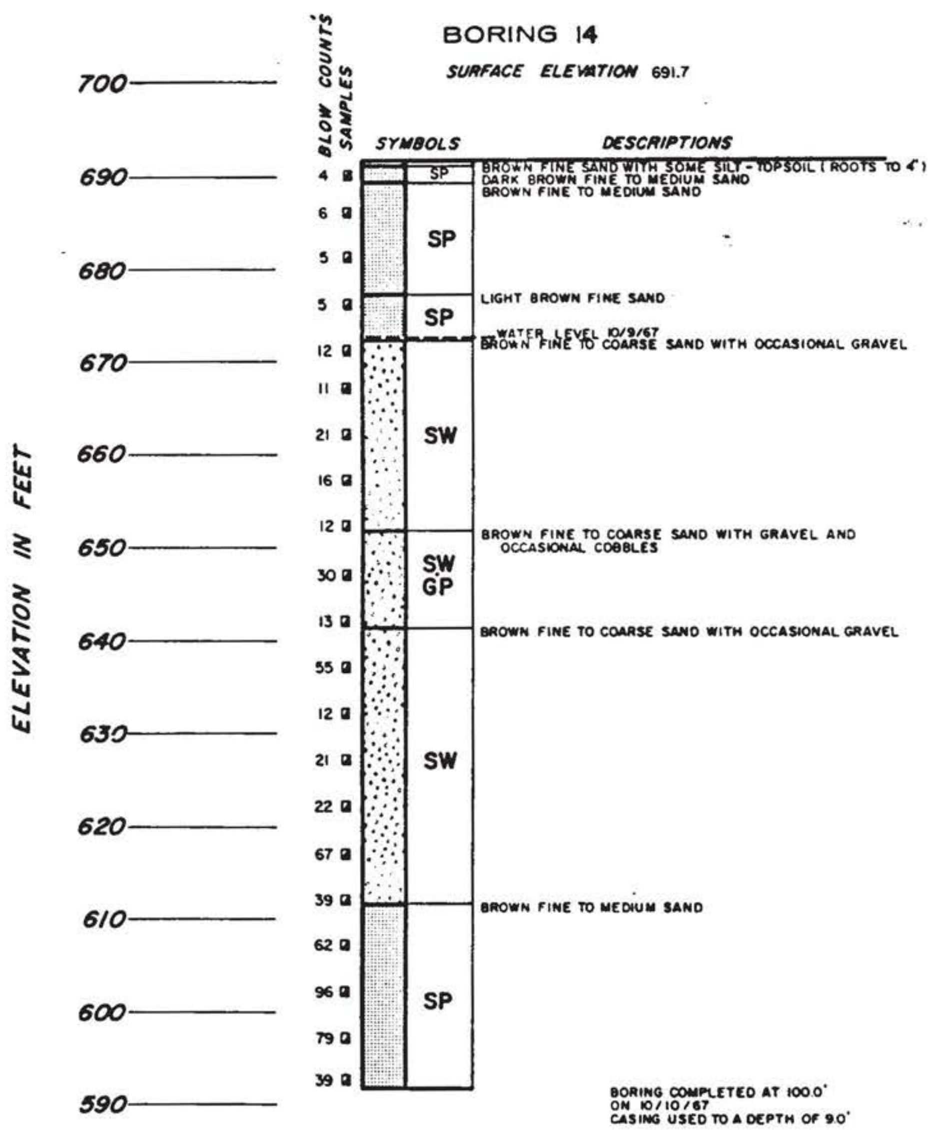
100% INDICATES DEPTH, LENGTH, AND PERCENT OF CORE RUN RECOVERED.

* INDICATES TEST WAS PERFORMED ON DENSIFIED SOIL SAMPLE.

LOG OF BORINGS

DAMES & MOORE

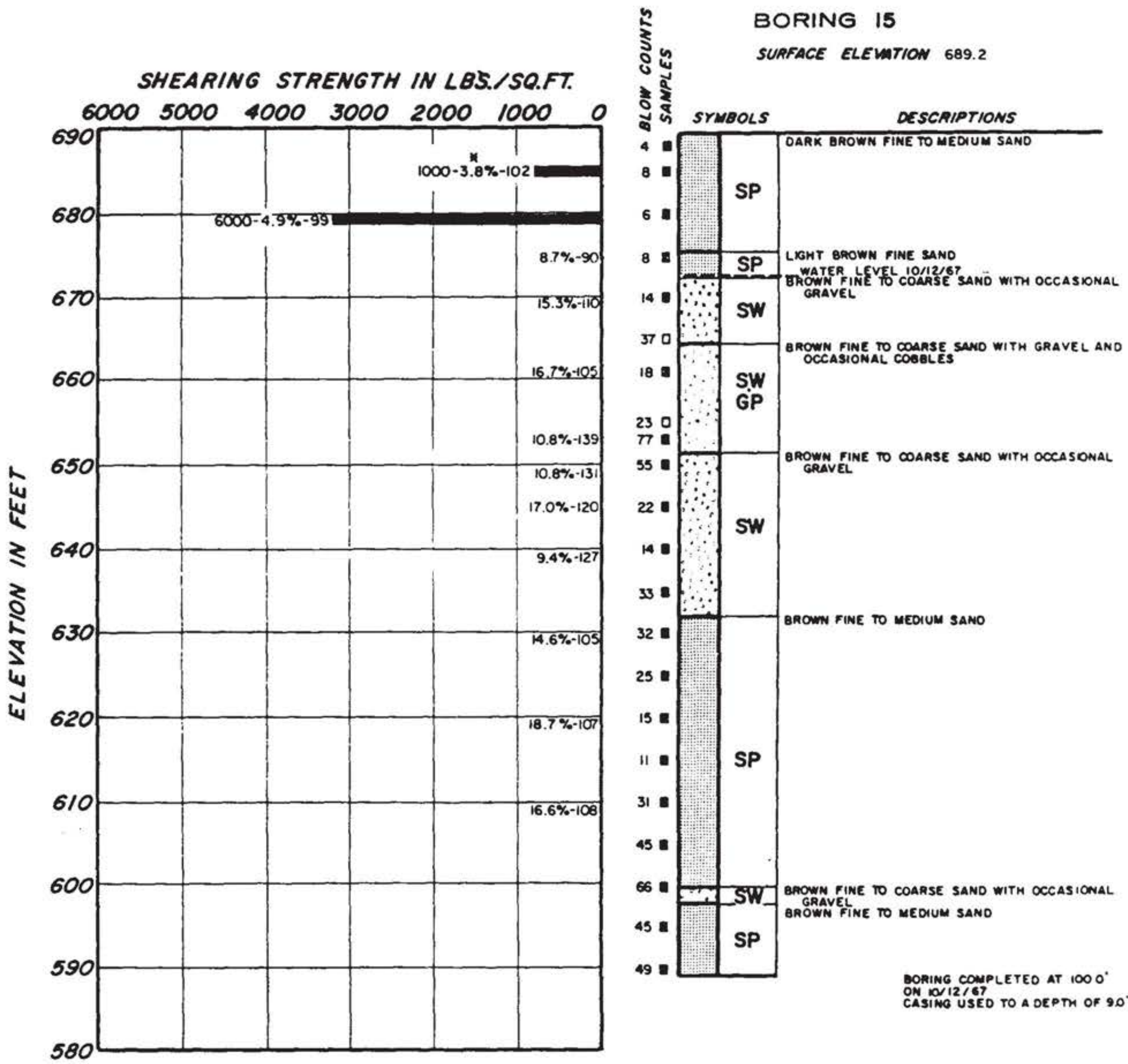
PLATE 6.11



LOG OF BORINGS

DAMES & MOORE

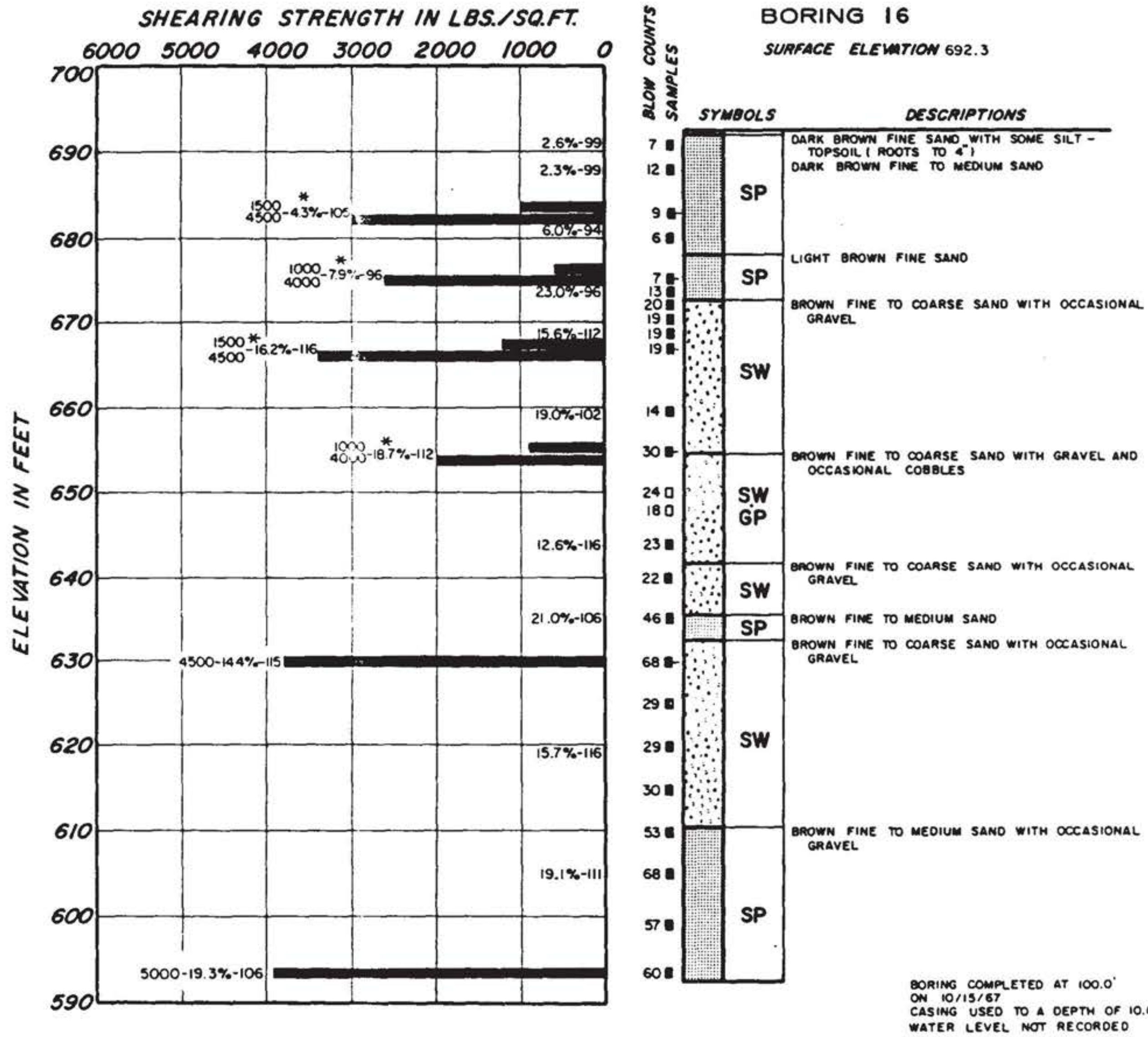
PLATE 6.12



LOG OF BORINGS

DAMES & MOORE

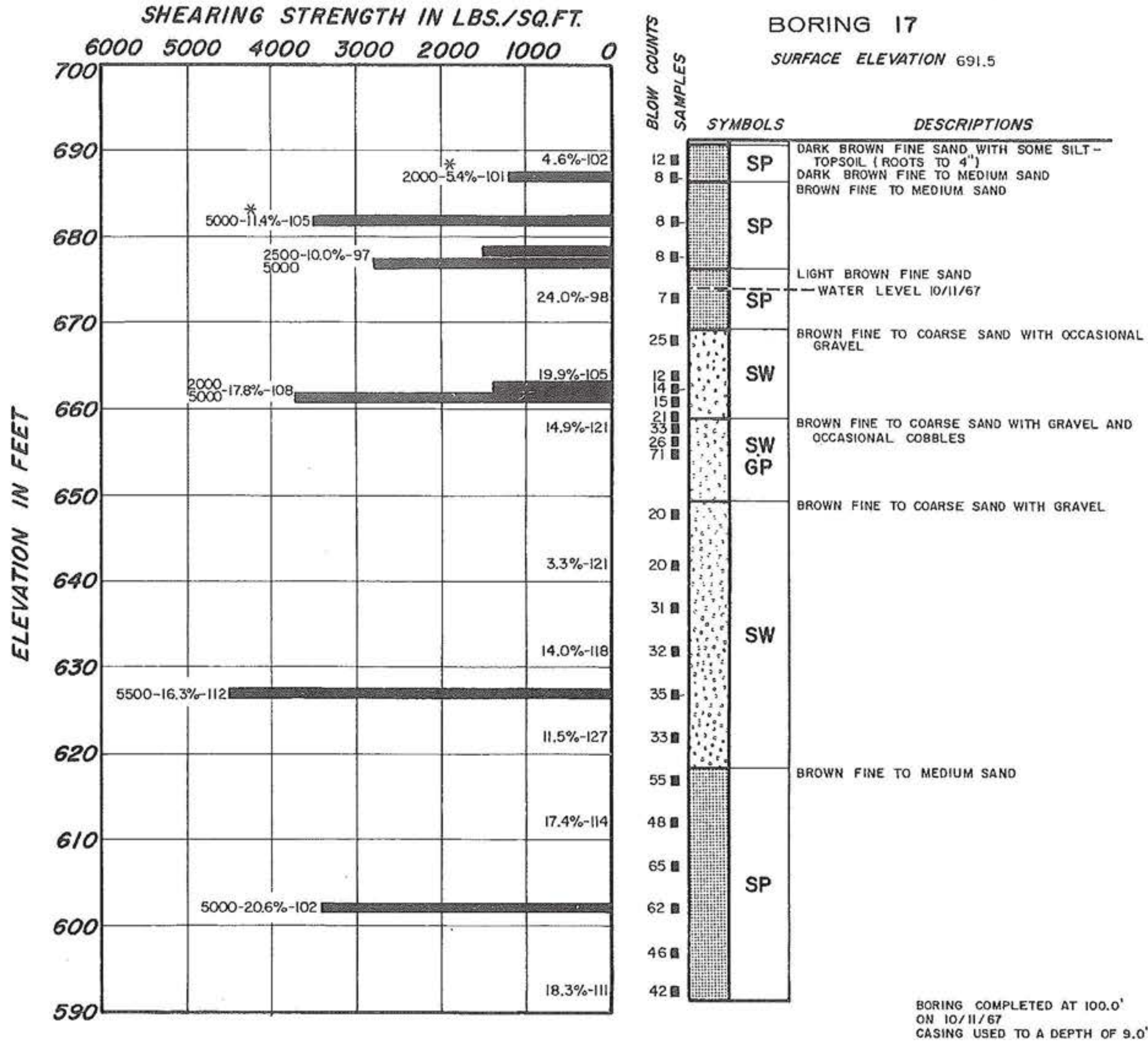
PLATE 6.13



LOG OF BORINGS

DAMES & MOORE

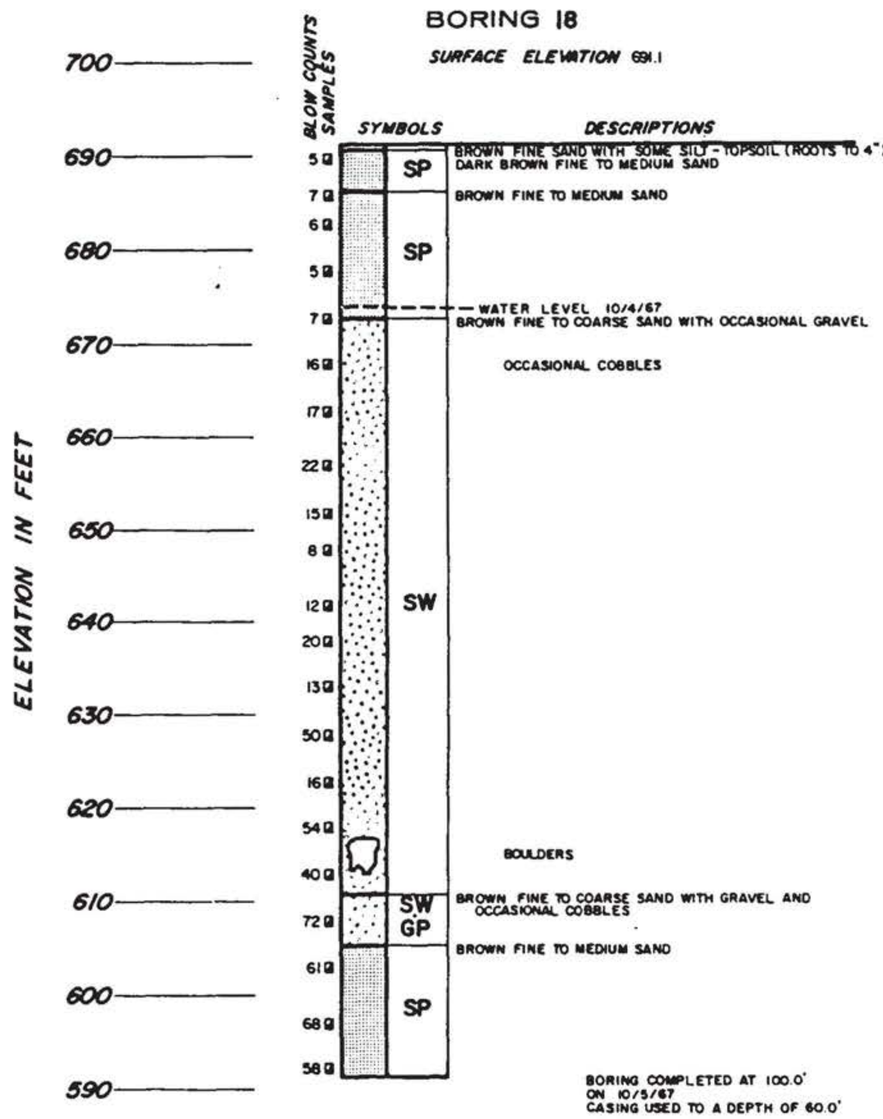
PLATE 6.14



LOG OF BORINGS

151

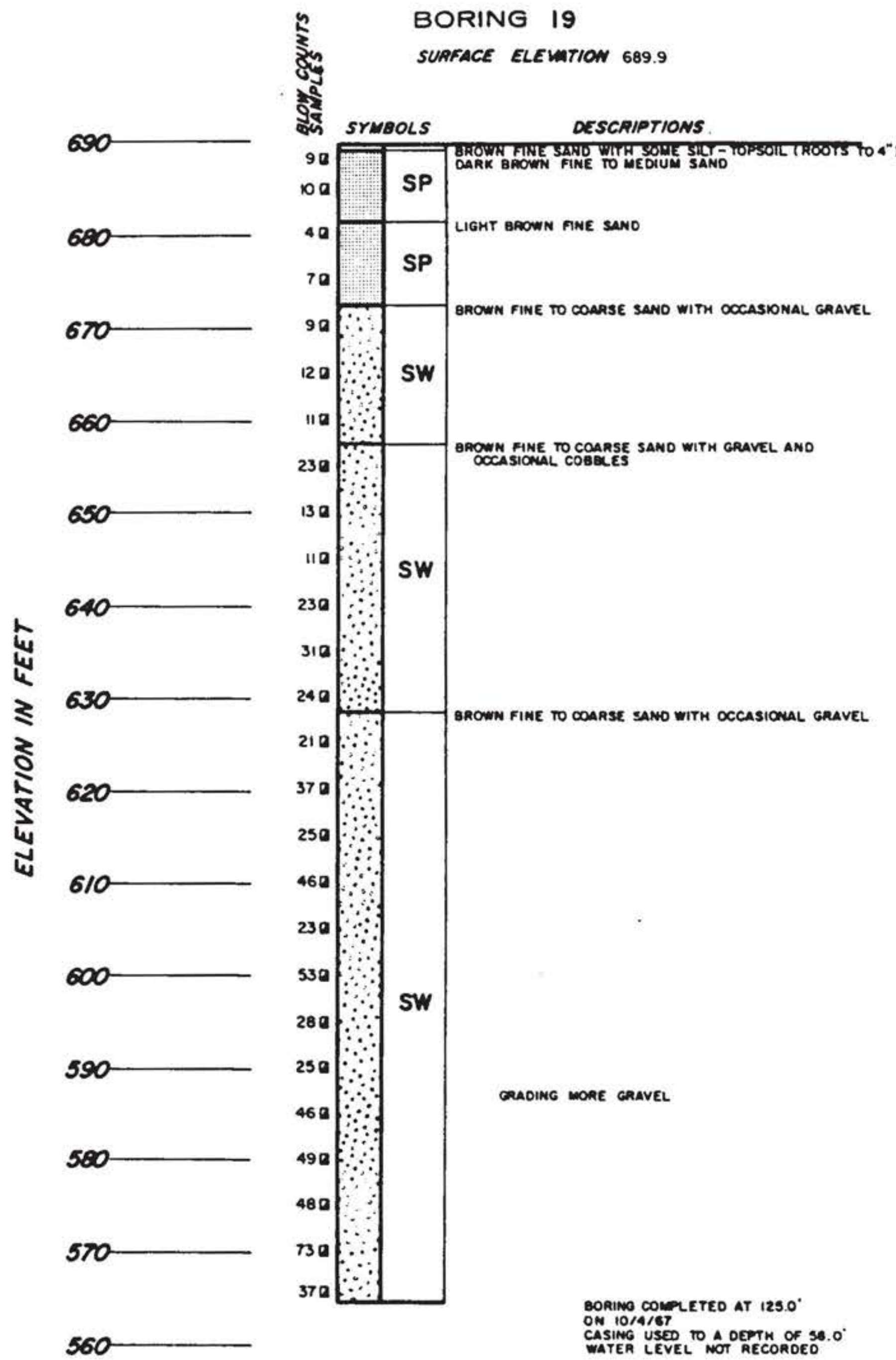
DAMES & MOORE



LOG OF BORINGS

DAMES & MOORE

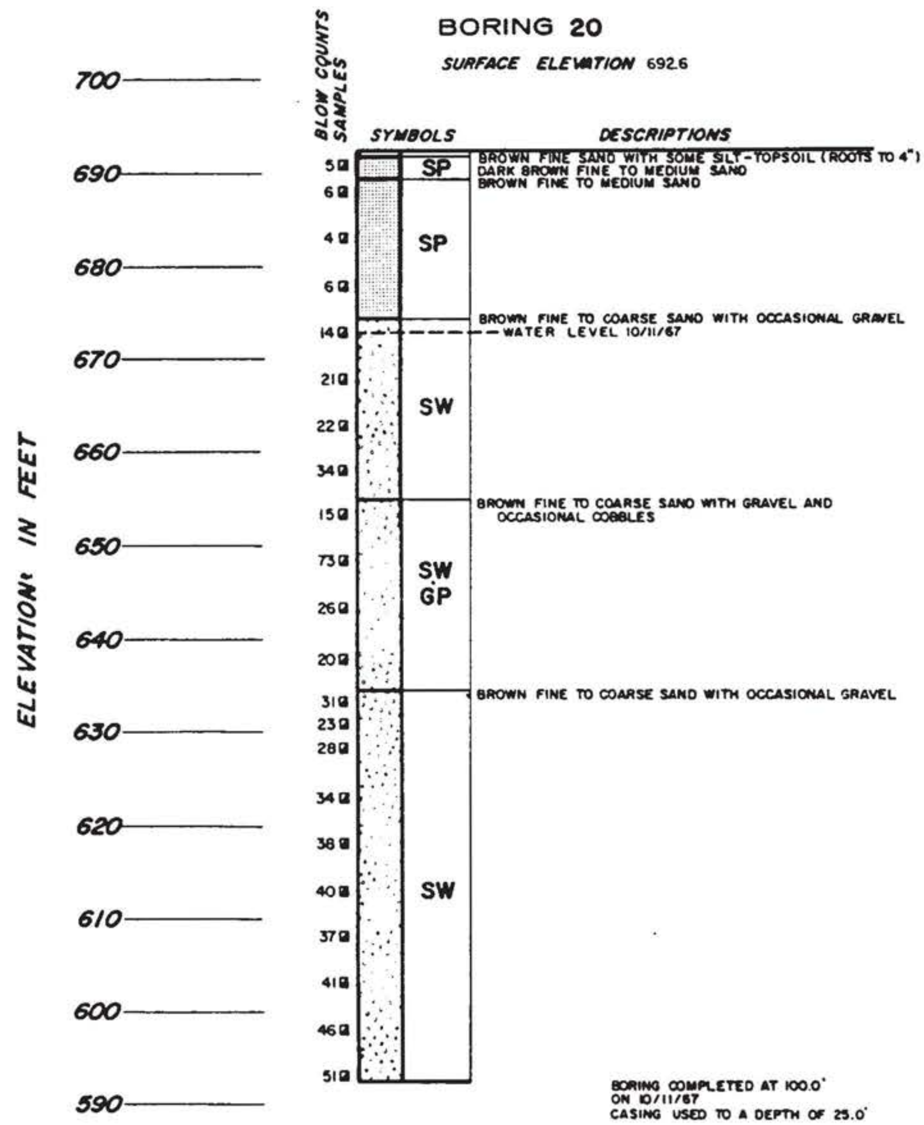
PLATE 6.16



LOG OF BORINGS

DAMES & MOORE

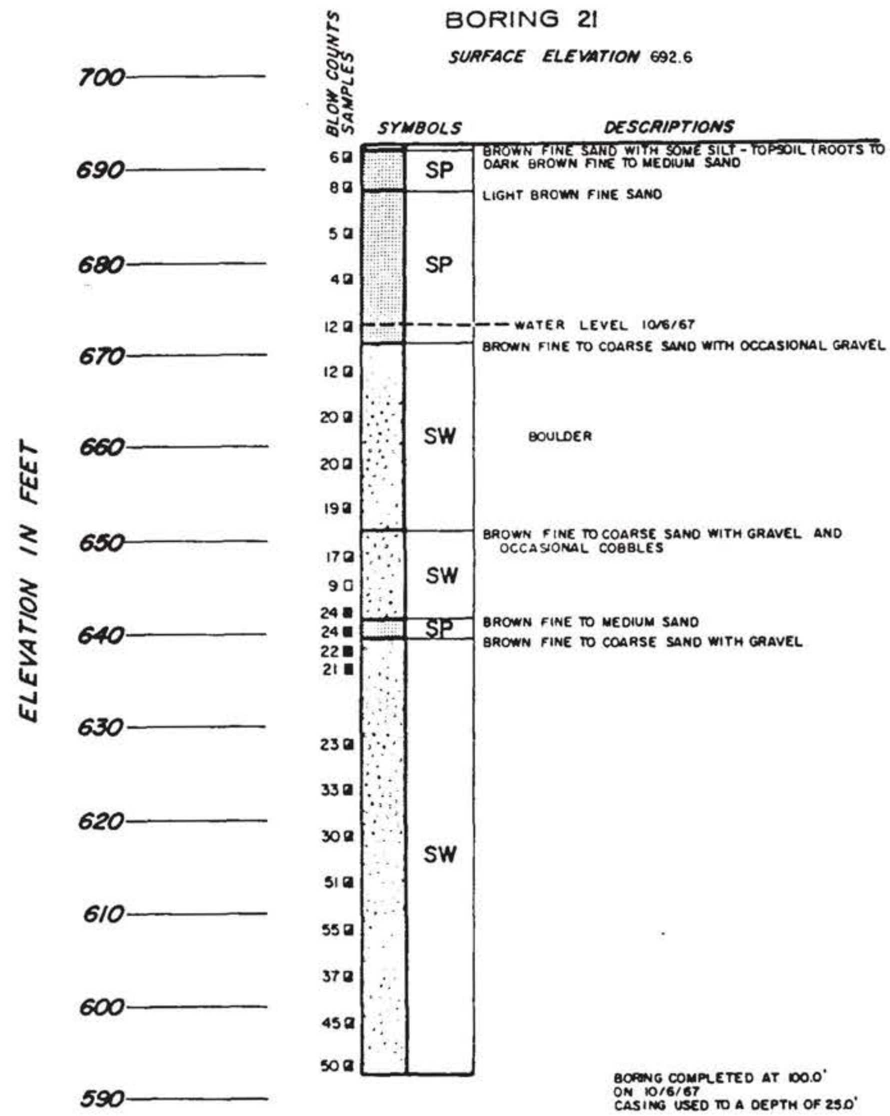
PLATE 6.17



LOG OF BORINGS

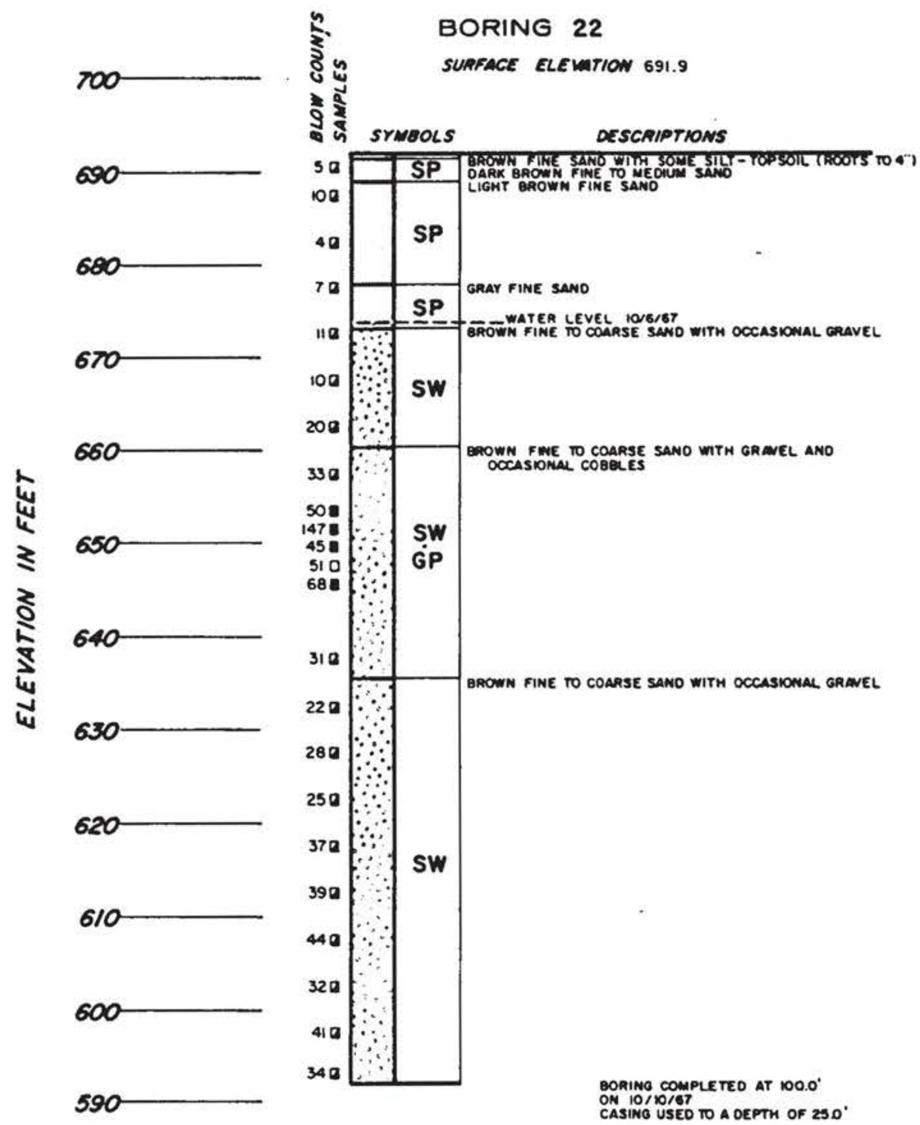
DAMES & MOORE

PLATE 6.18



LOG OF BORINGS

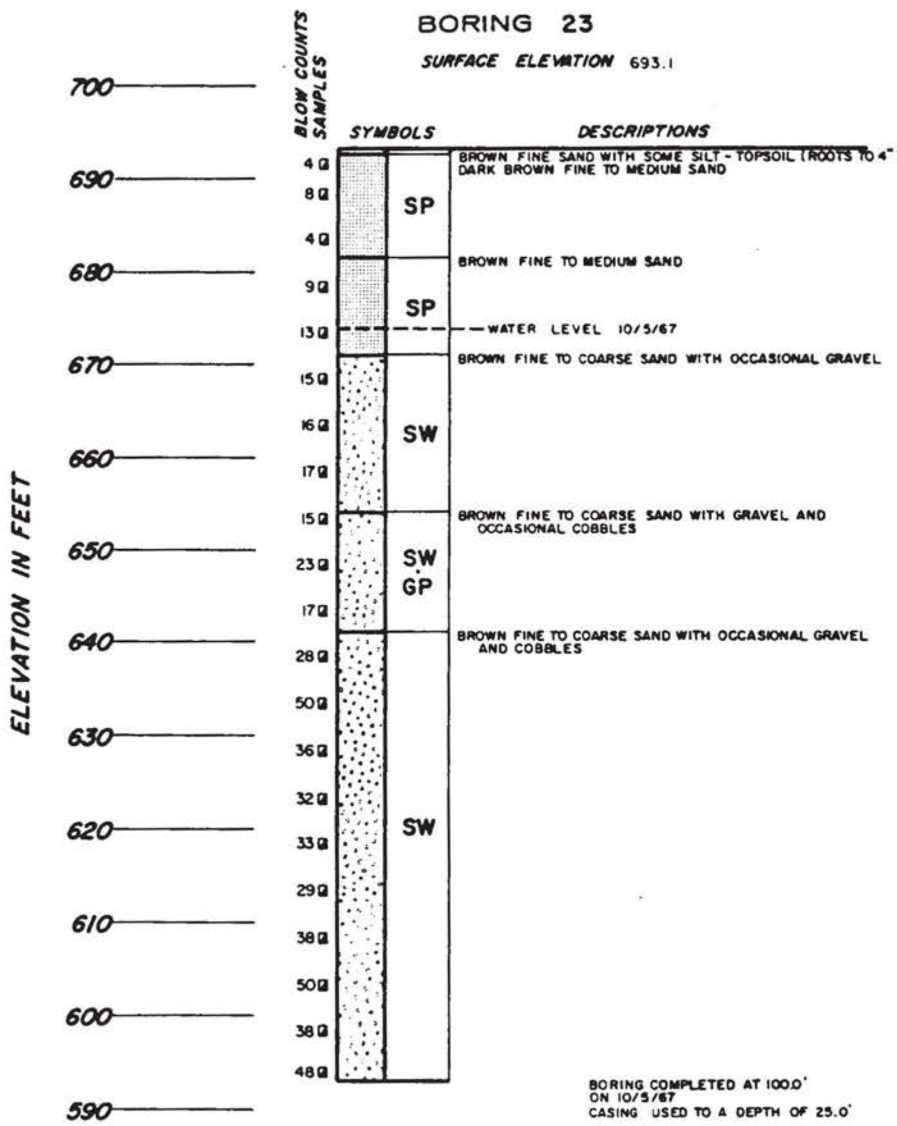
DAMES & MOORE



LOG OF BORINGS

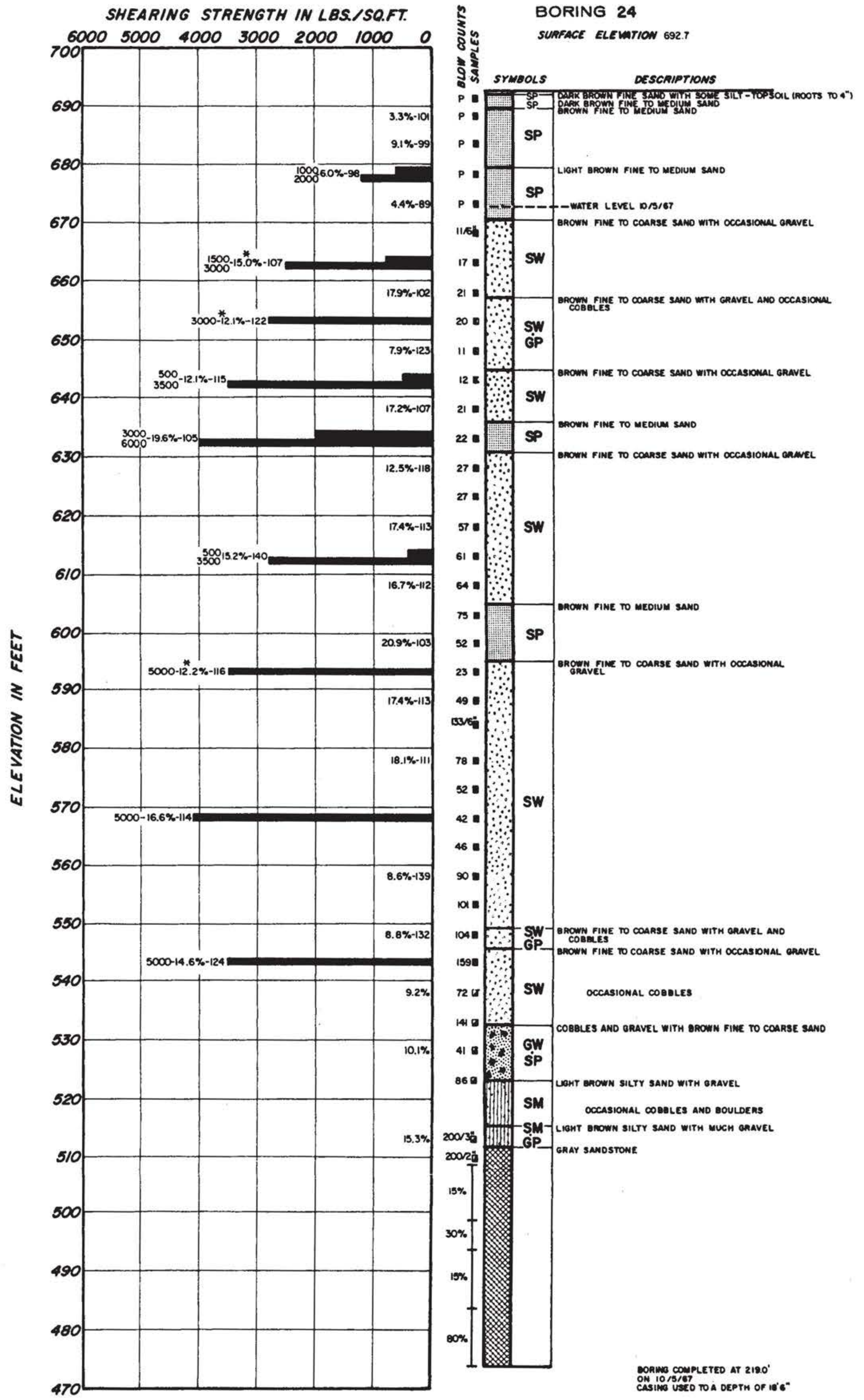
DAMES & MOORE

PLATE 6.20



LOG OF BORINGS

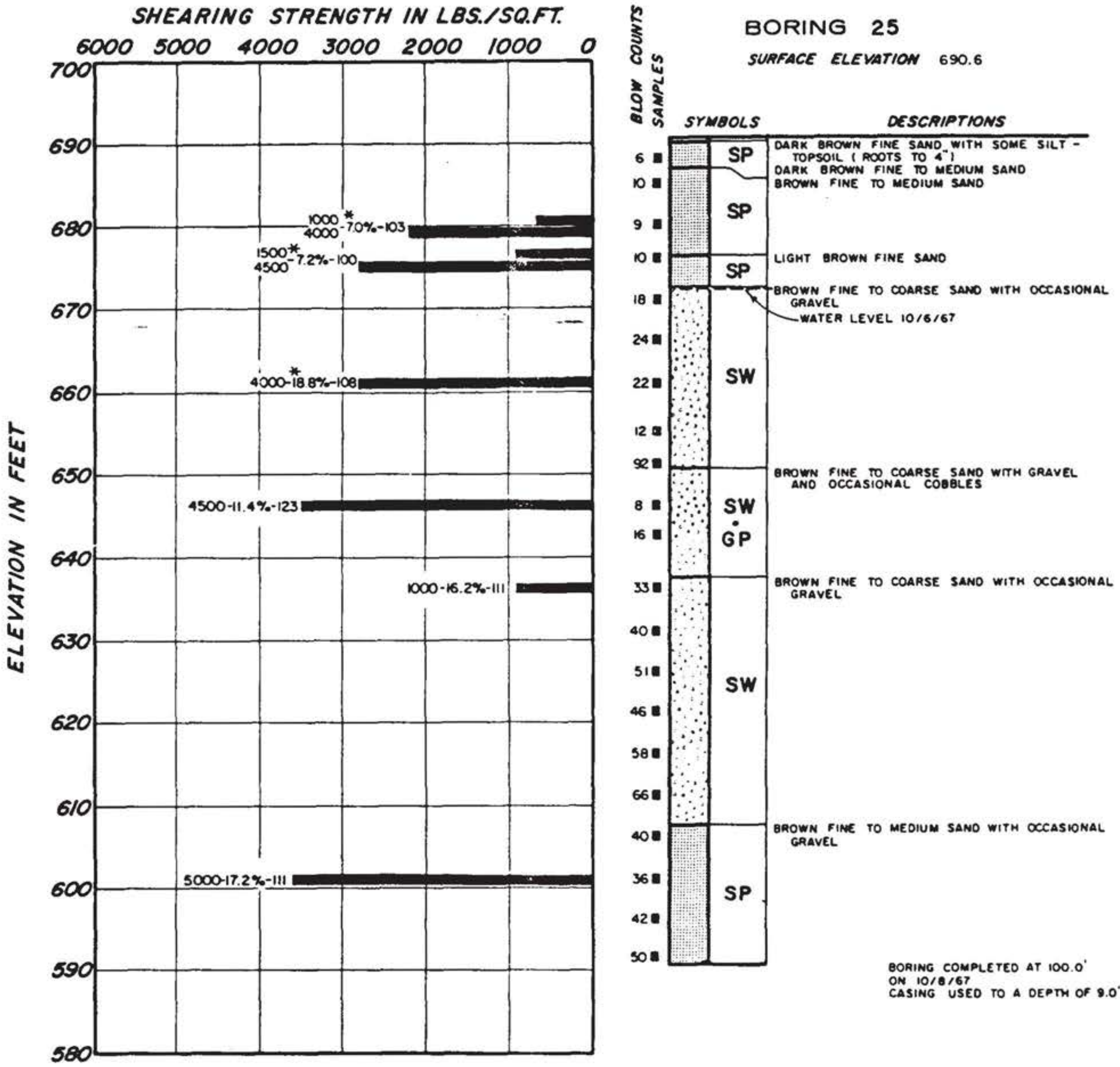
DAMES & MOORE



LOG OF BORINGS

DAMES & MOORE

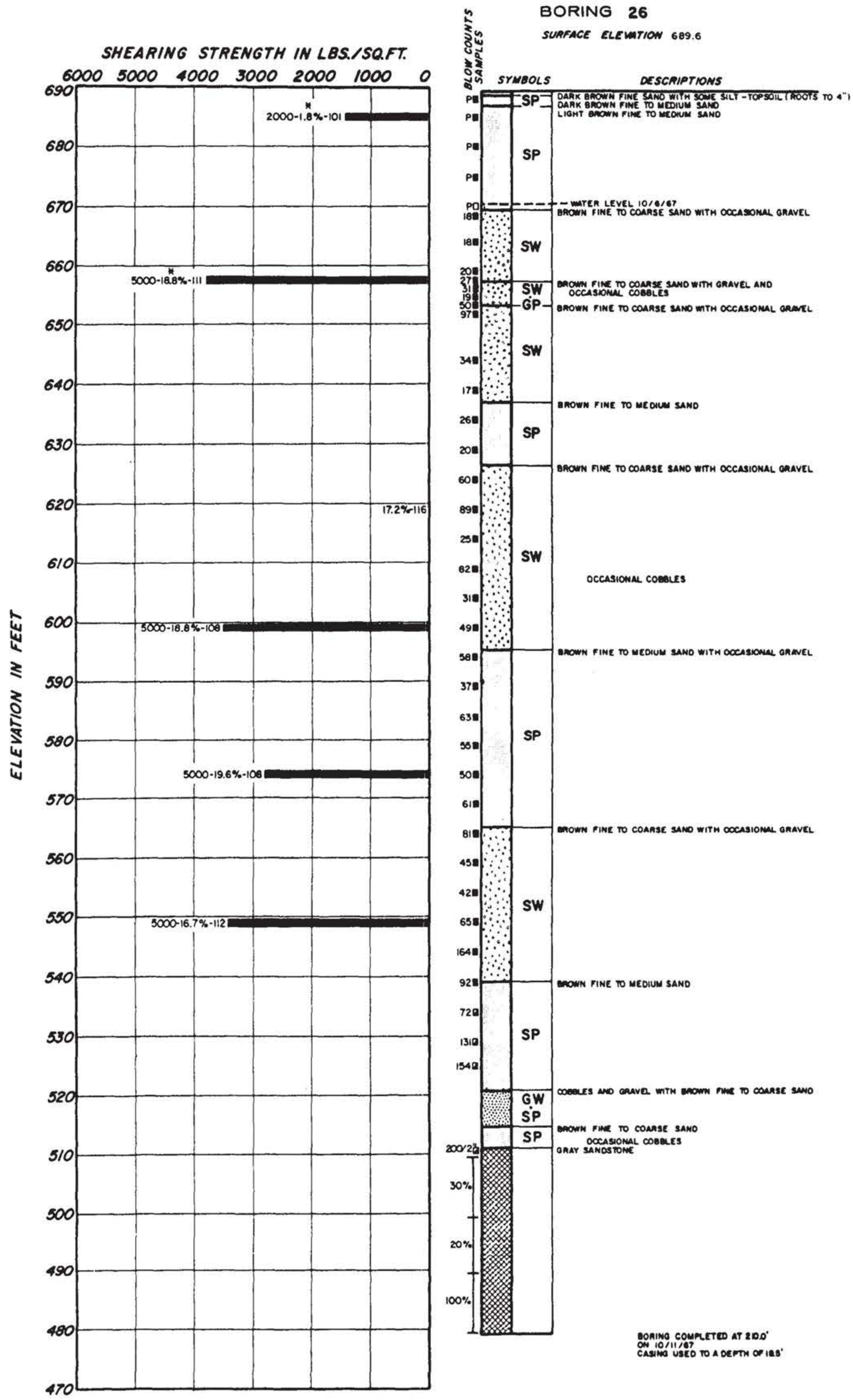
PLATE 6.22



LOG OF BORINGS

DAMES & MOORE

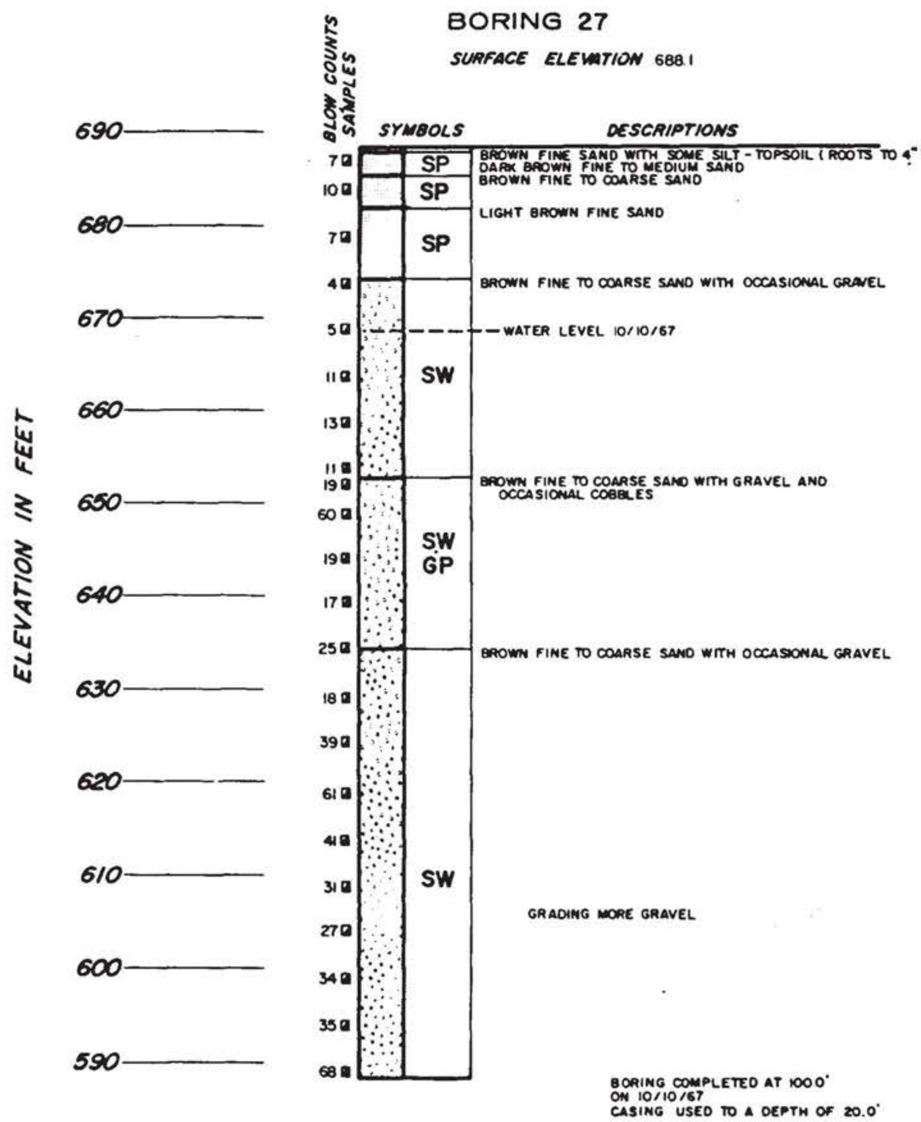
PLATE 6.23



LOG OF BORINGS

DAMES & MOORE

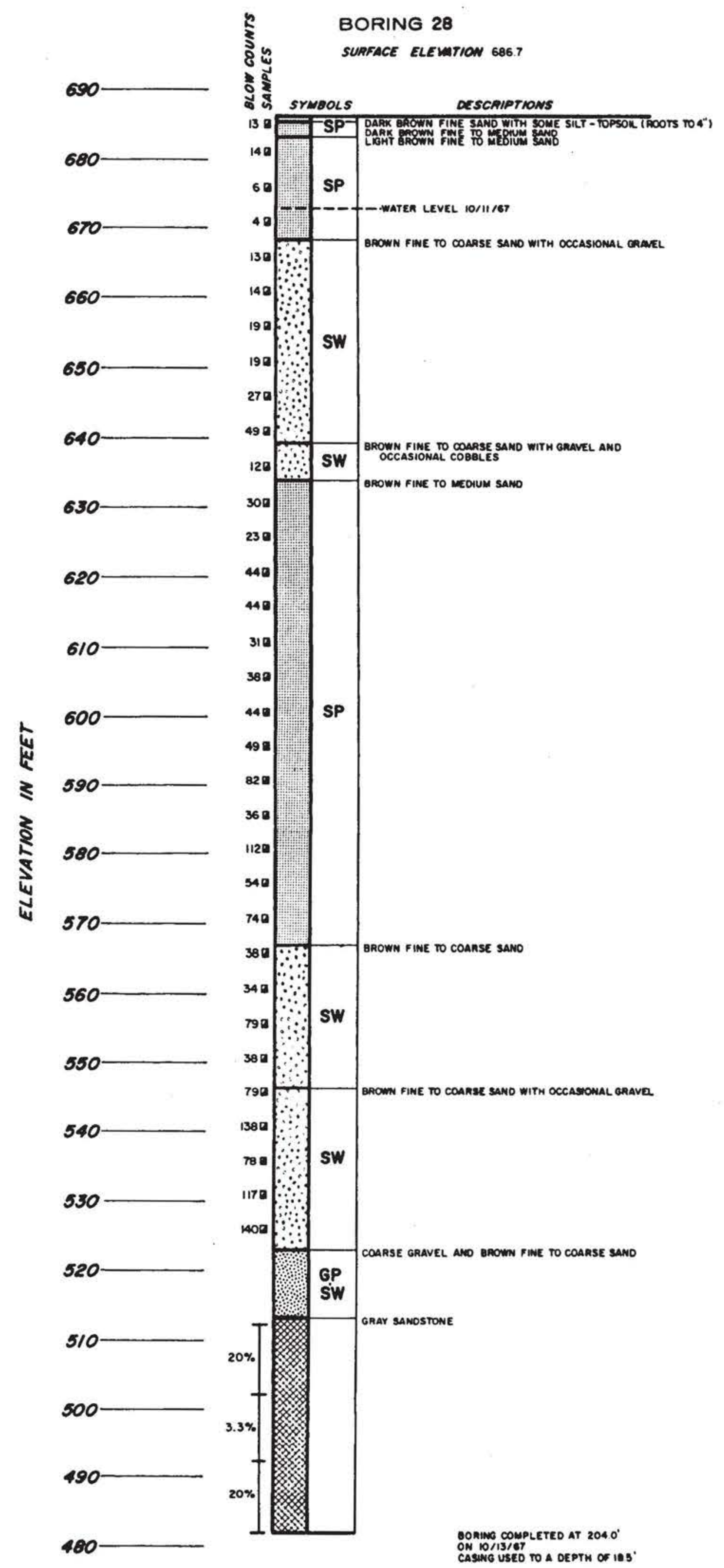
PLATE 6.24



LOG OF BORINGS

DAMES & MOORE

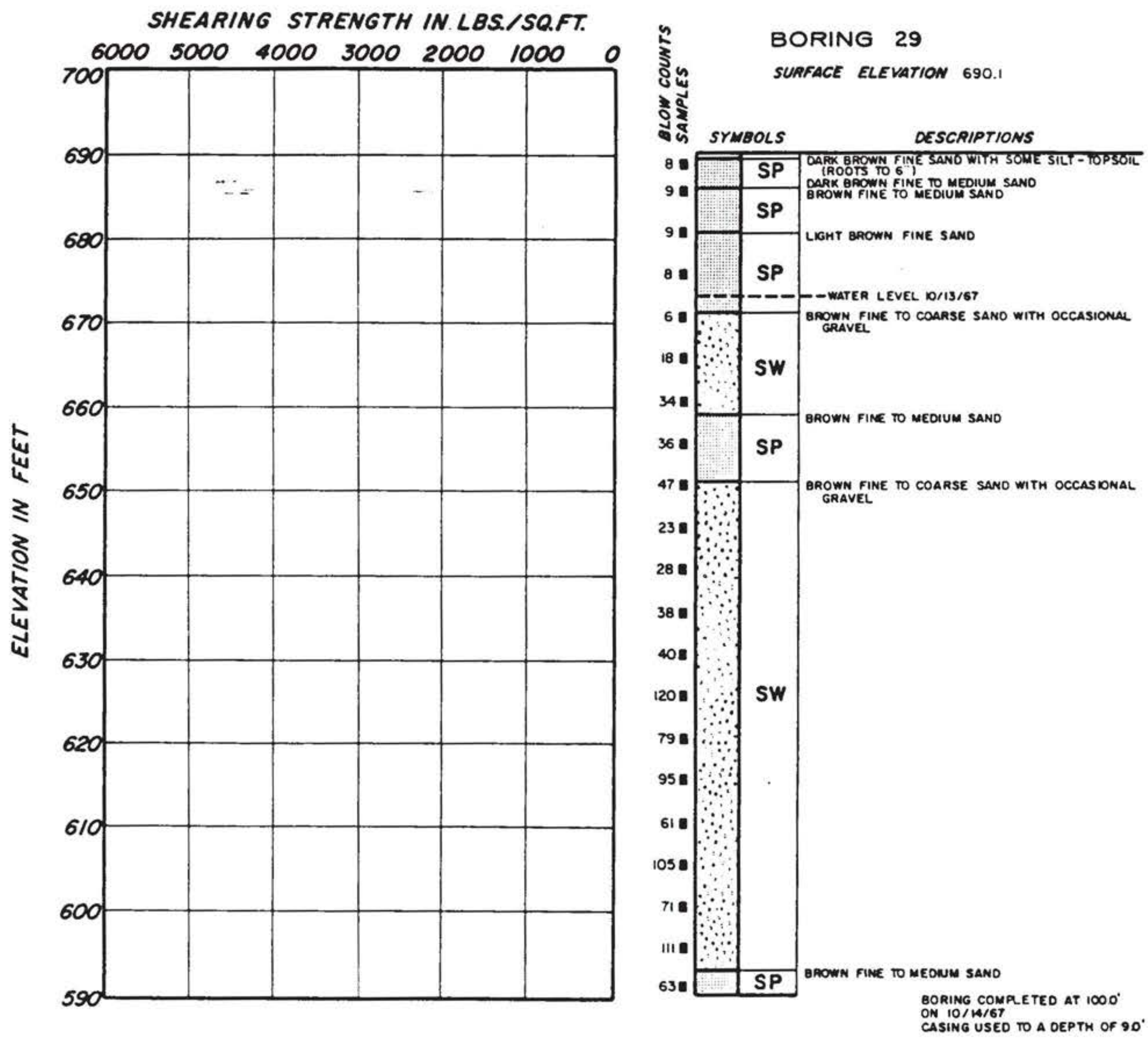
PLATE 6.25



LOG OF BORINGS

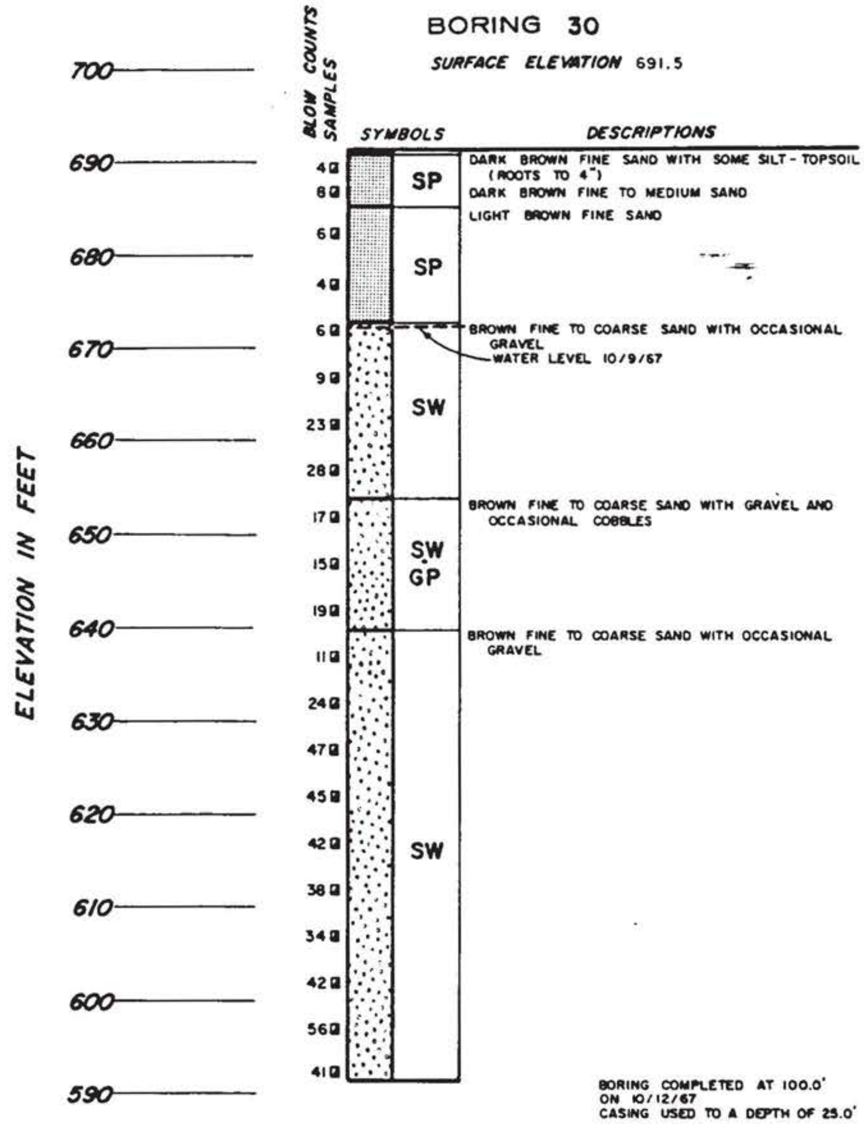
DAMES & MOORE

PLATE 6.26



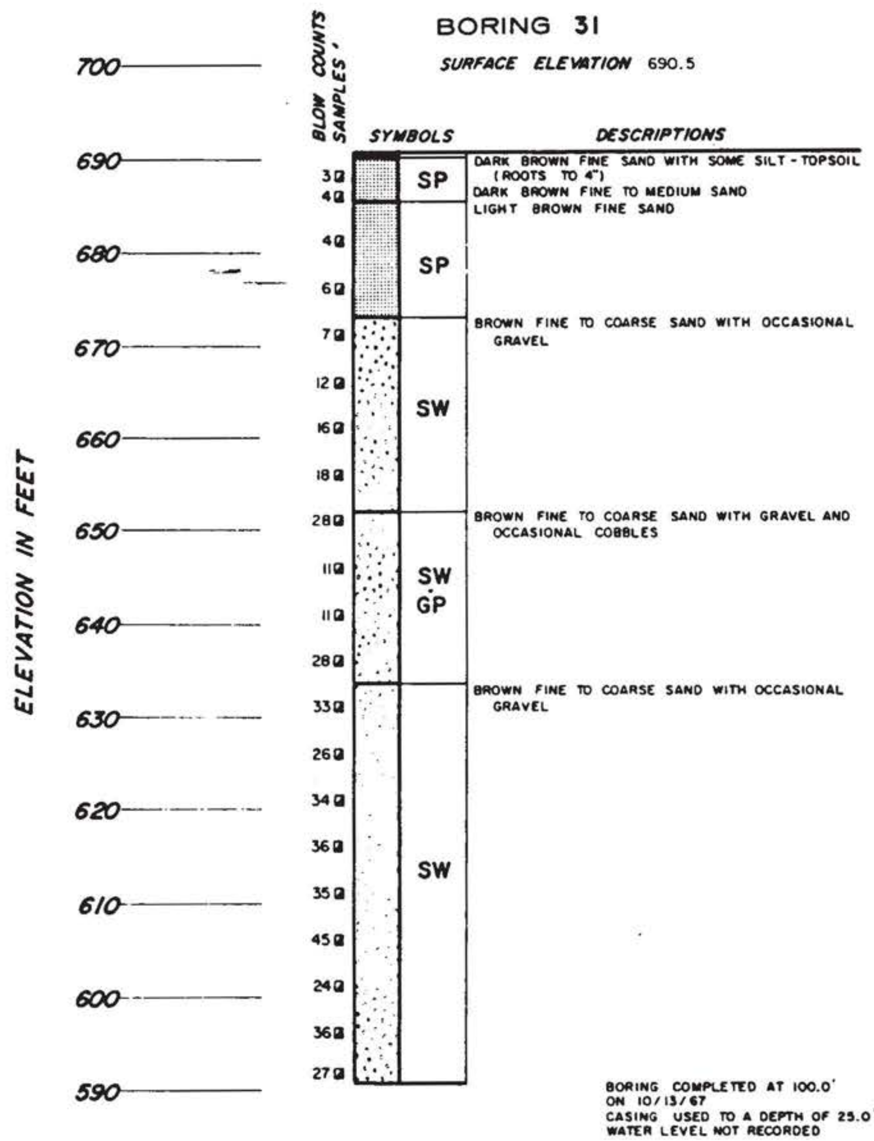
LOG OF BORINGS

DAMES & MOORE



LOG OF BORINGS

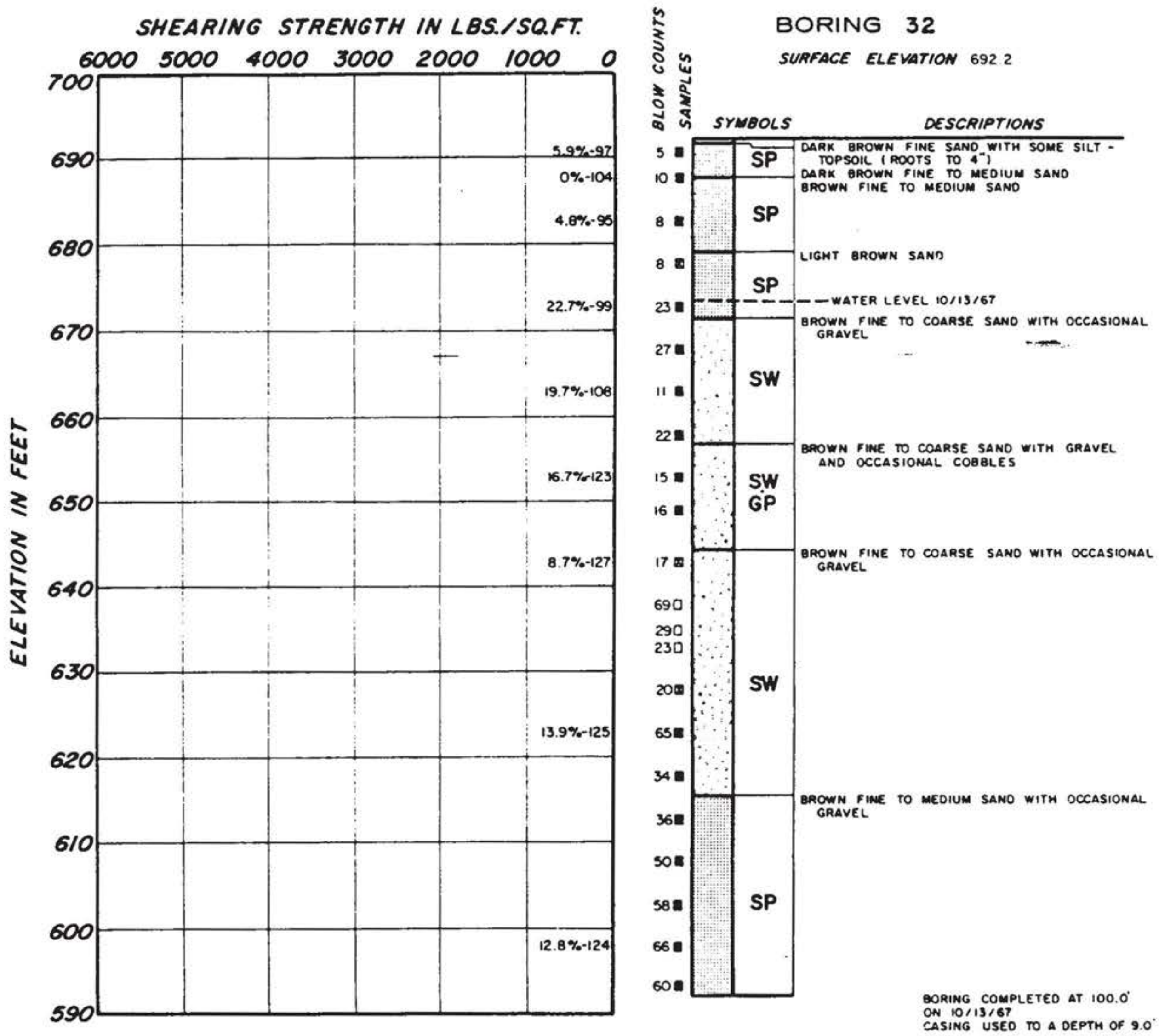
DAMES & MOORE



LOG OF BORINGS

DAMES & MOORE

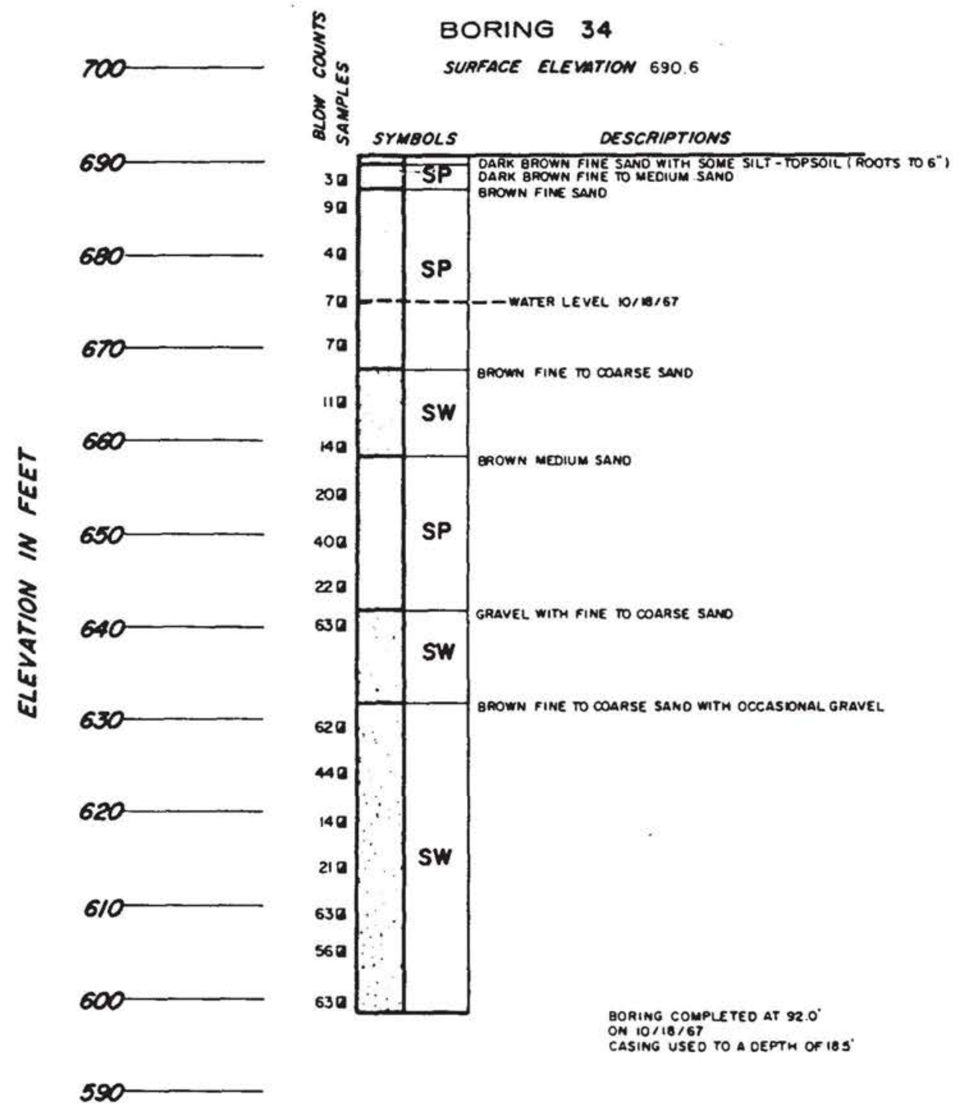
PLATE 6.29



LOG OF BORINGS

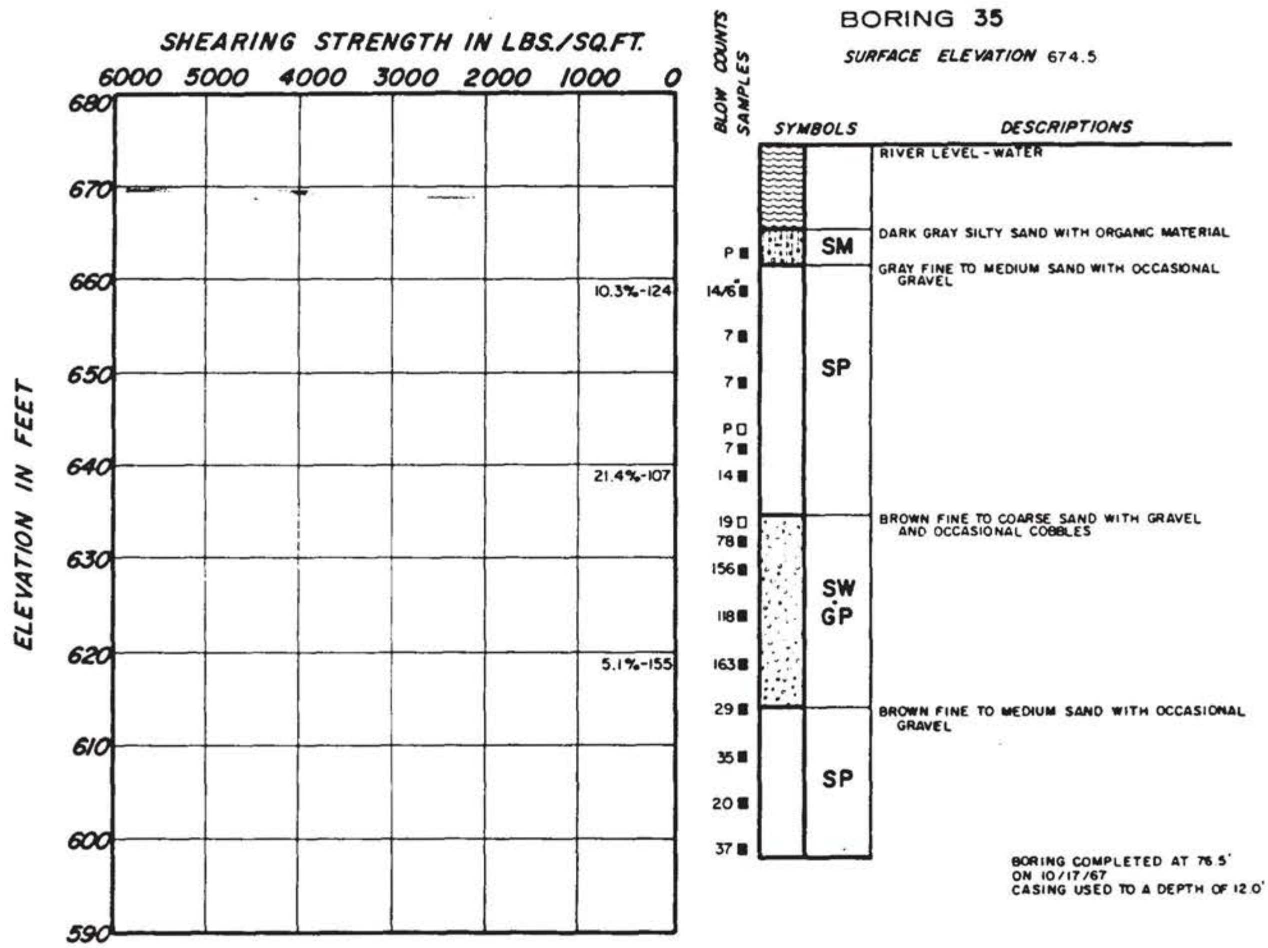
DAMES & MOORE

PLATE 6.30



LOG OF BORINGS

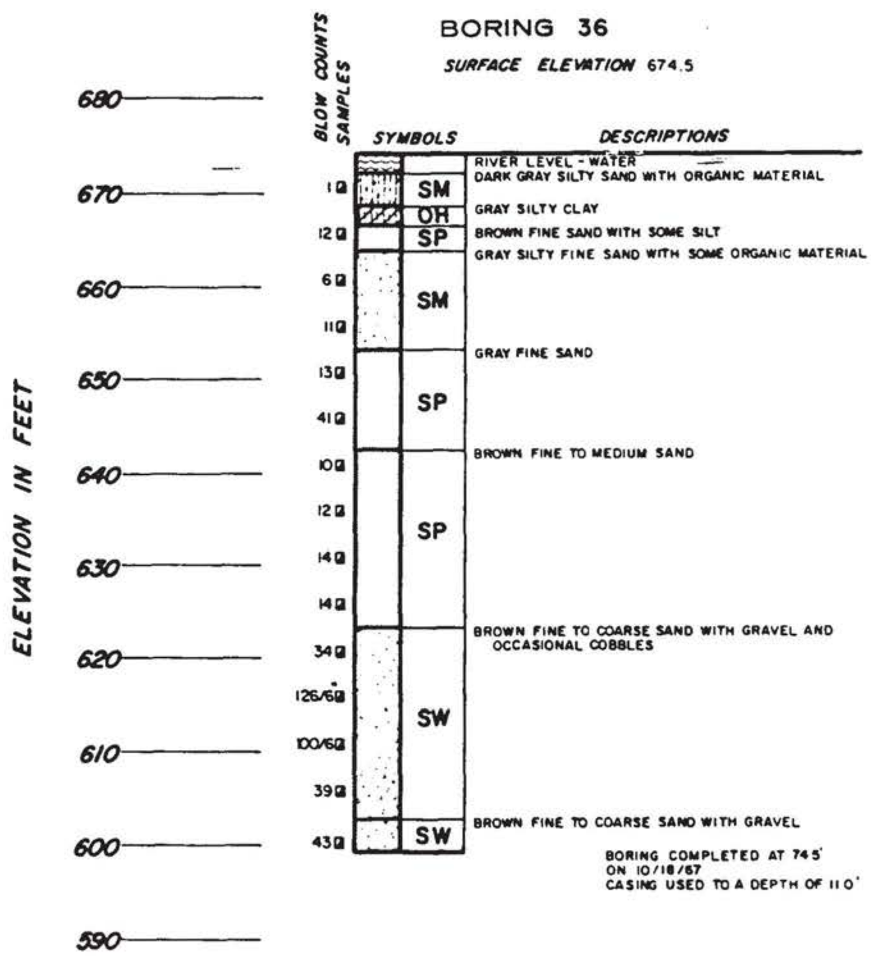
DAMES & MOORE



LOG OF BORINGS

DAMES & MOORE

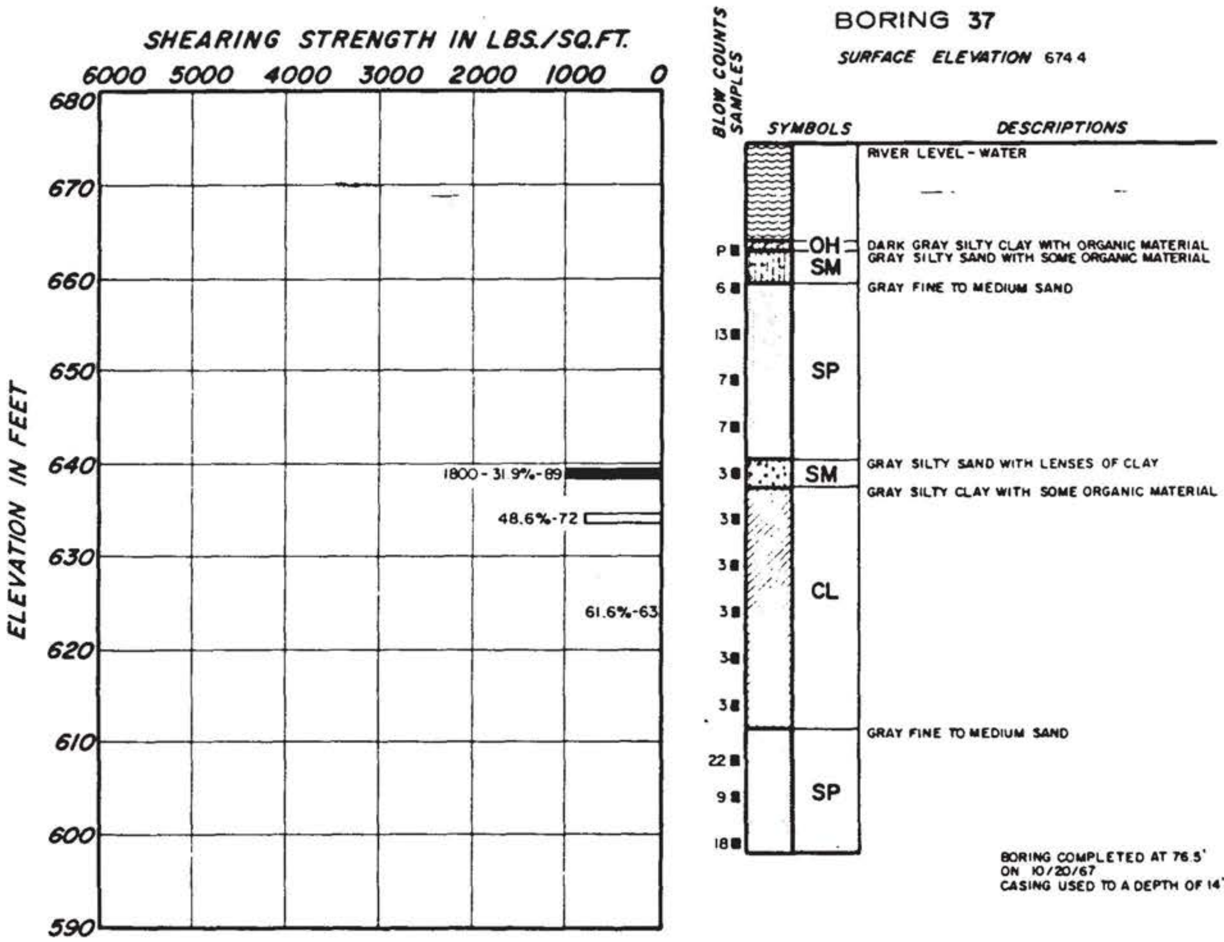
PLATE 6.32



LOG OF BORINGS

DAMES & MOORE

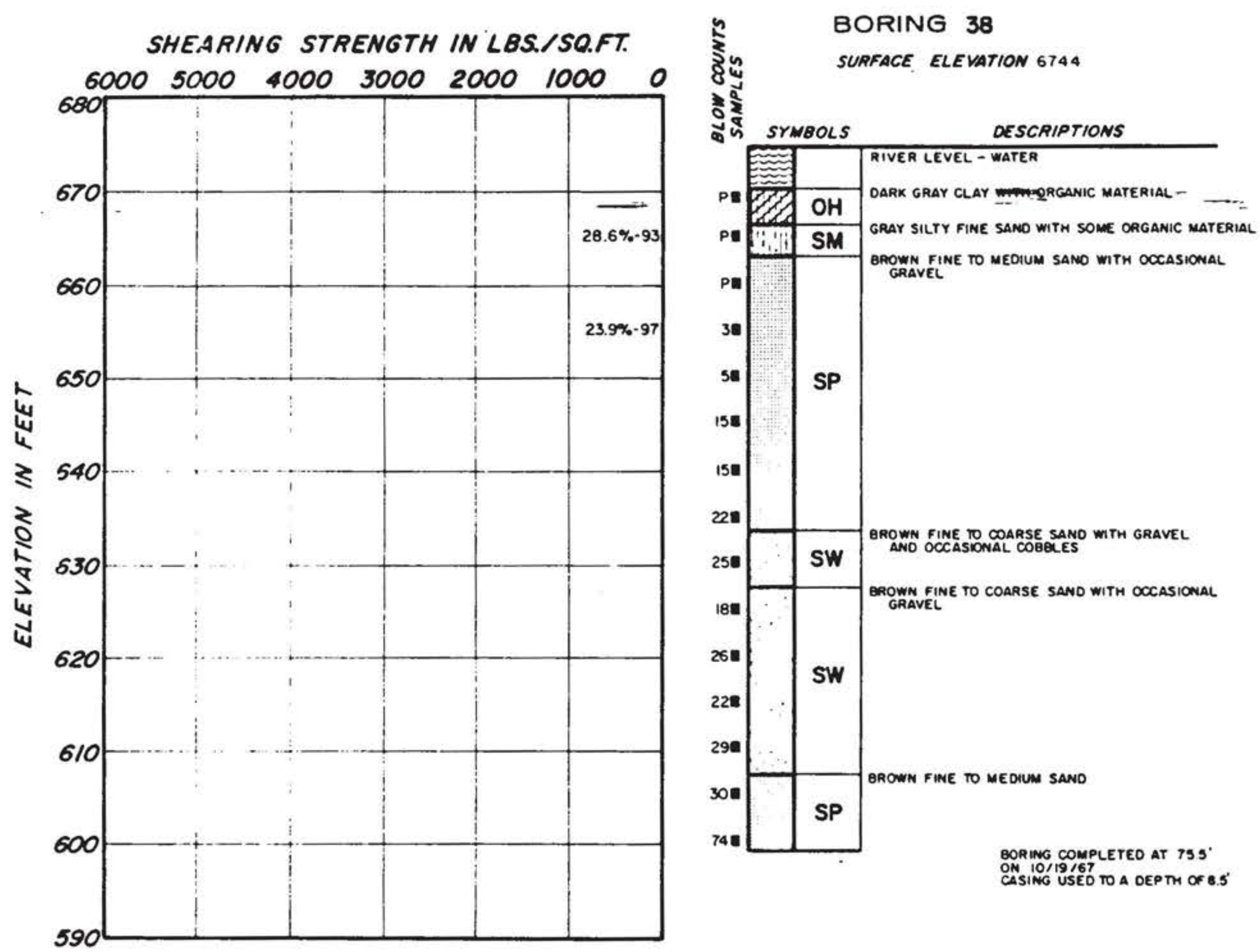
PLATE 6.33



LOG OF BORINGS

DAMES & MOORE

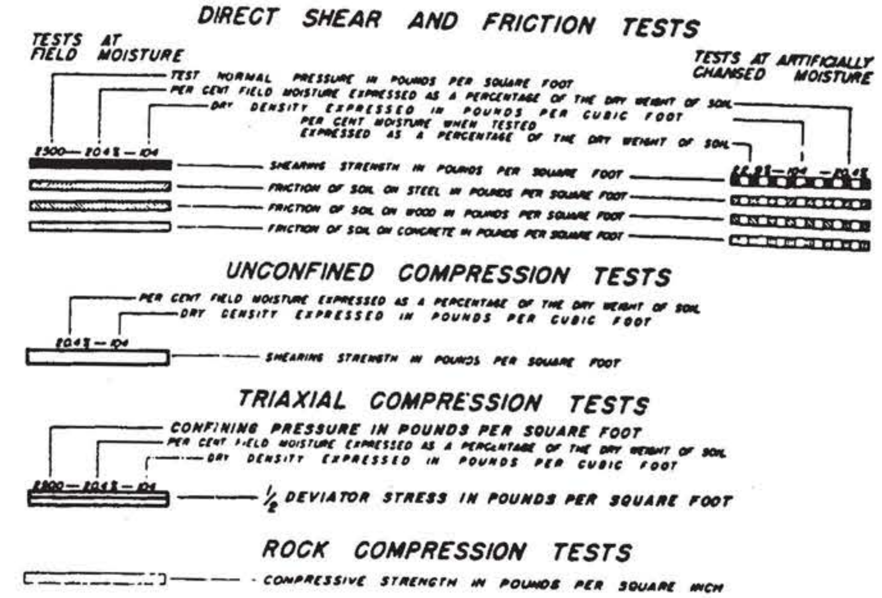
PLATE 6.34



LOG OF BORINGS

DAMES & MOORE

PLATE 6.35



KEY TO TEST DATA

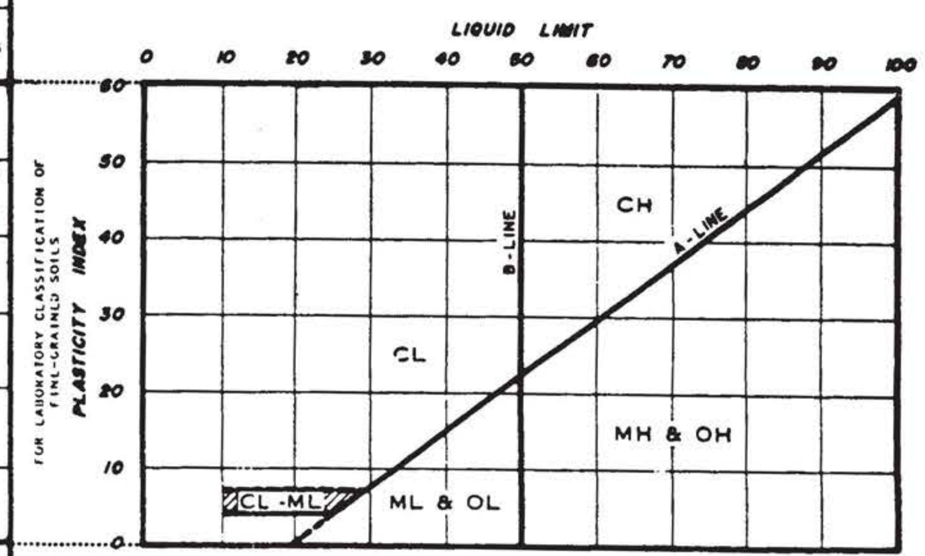
- INDICATES DEPTH OF UNDISTURBED SAMPLE
- INDICATES DEPTH OF DISTURBED SAMPLE
- INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY
- INDICATES DEPTH OF SPLIT-SPOON SAMPLE
- INDICATES DEPTH AND LENGTH OF CORING RUN

KEY TO SAMPLES

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

SOIL CLASSIFICATION CHART



PLASTICITY CHART

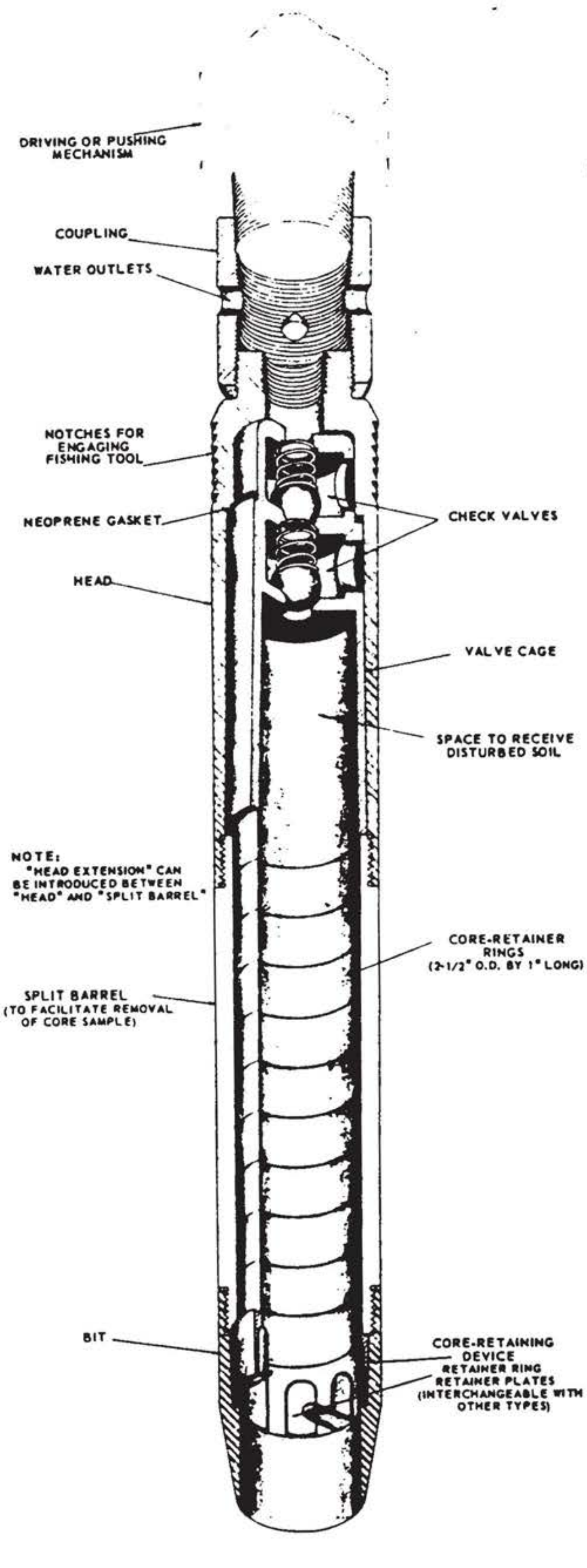
UNIFIED SOIL CLASSIFICATION SYSTEM

DAMES & MOORE
SOIL MECHANICS ENGINEERS

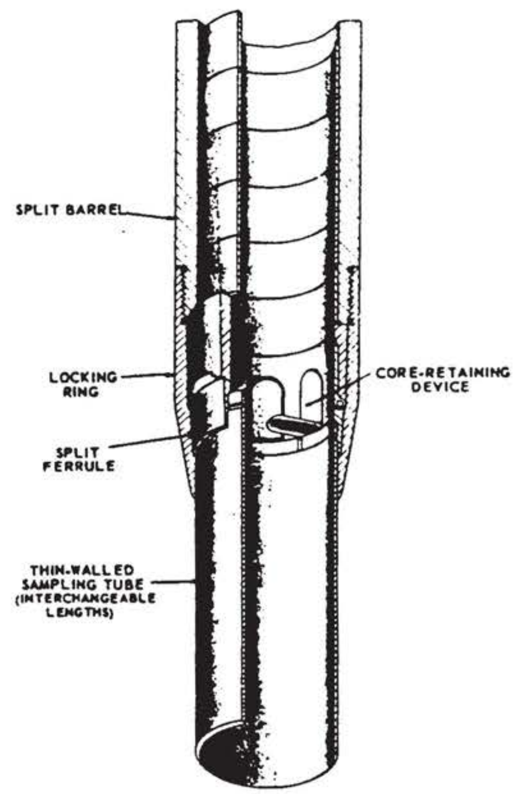
PLATE 6.36

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER
U. S. PATENT NO. 2,318,062

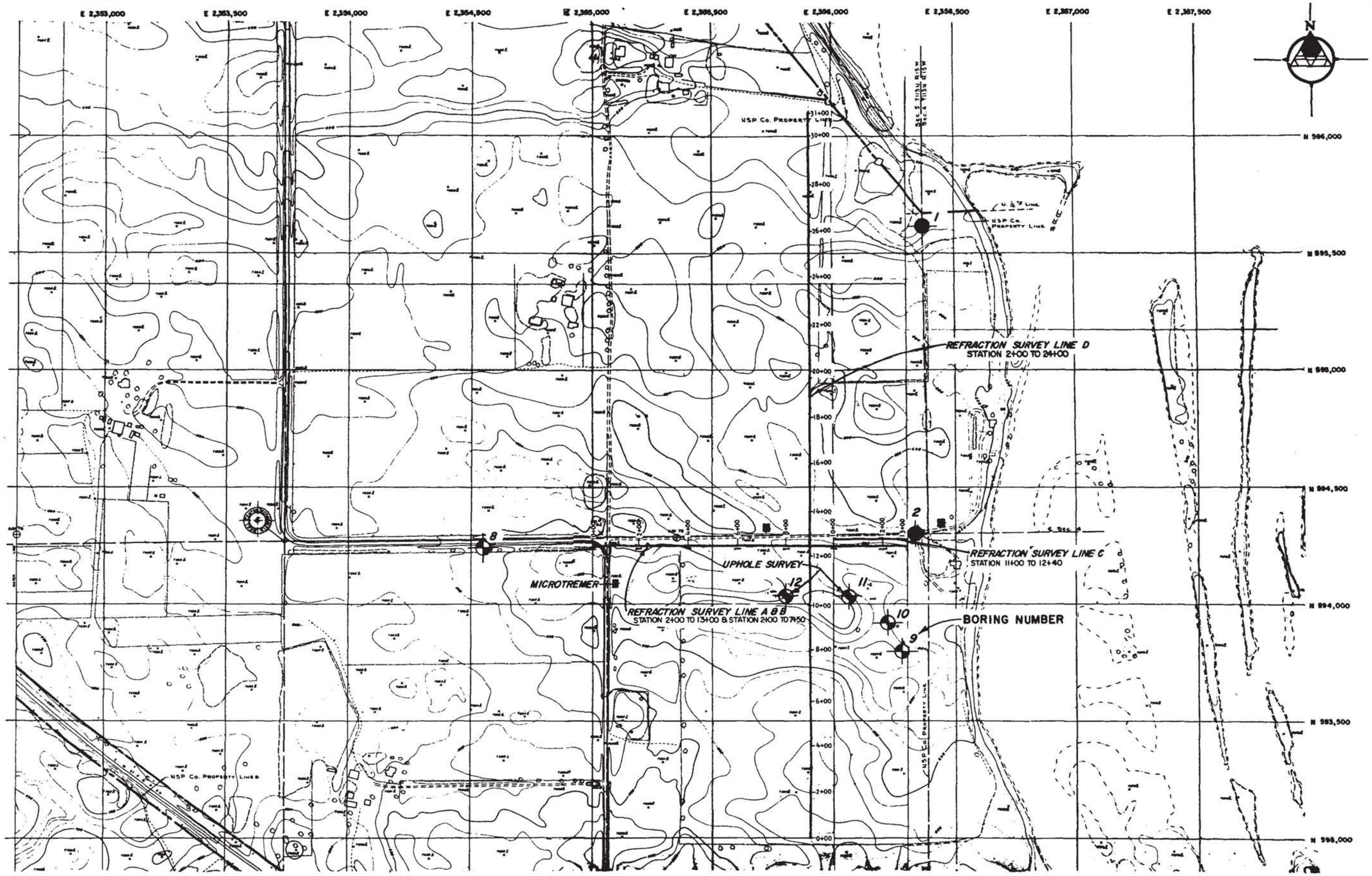


ALTERNATE ATTACHMENTS



DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE 6.37



BORINGS DRILLED BY DAMES & MOORE

BORINGS DRILLED BY OTHERS

■ MICROTREMER

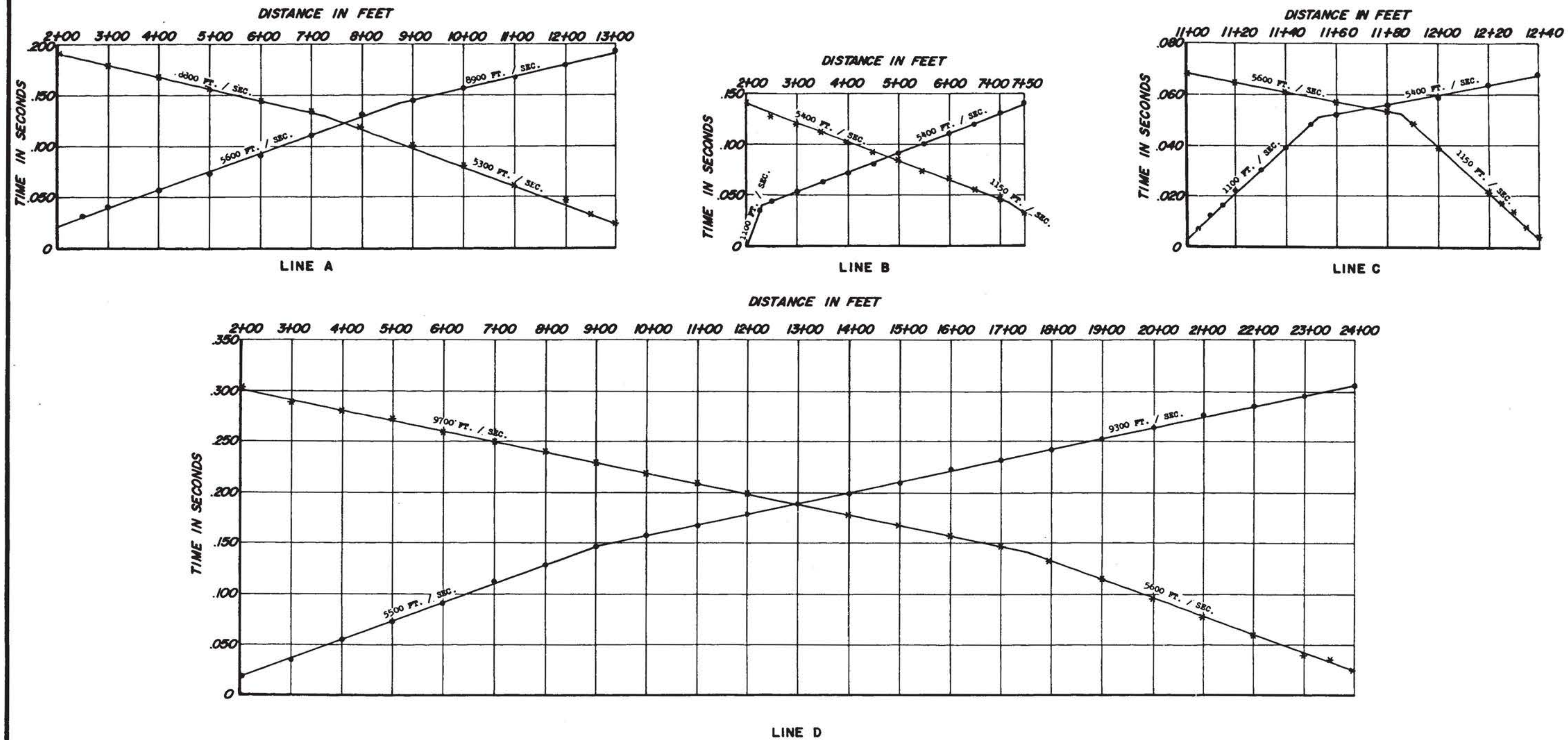
DRAWING REFERENCE:

MARK HURD AERIAL SURVEYS, INC.
MINNEAPOLIS, MINNESOTA
PRAIRIE ISLAND GENERATING PLANT
TOPOGRAPHIC MAP OF PLANT SITE
SHEETS 3,4,7 AND 8
DRAWING NO. NF-30656-8
DATE: 1-28-65

SEISMIC SURVEY PLOT PLAN

FEET
100 0 100 200 300 400 500

DAMES & MOORE



GEOPHYSICAL REFRACTION SURVEY

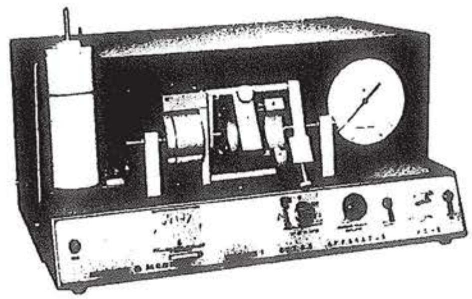
COMPRESSIONAL WAVE VELOCITIES

DAMES & MOORE

PLATE 6.39

Method Of Performing Direct Shear and Friction Tests

Direct shear tests are performed to determine the shearing strengths of soils. Friction tests are performed to determine the frictional resistances between soils and various other materials such as wood, steel, or concrete. The tests are performed in the laboratory to simulate anticipated field conditions.



DIRECT SHEAR TESTING & RECORDING APPARATUS

Each sample is tested within three brass rings, two and one-half inches in diameter and one inch in length. Undisturbed samples of in-place soils are tested in rings taken from the sampling device in which the samples were obtained. Loose samples of soils to be used in constructing earth fills are compacted in rings to predetermined conditions and tested.

Direct Shear Tests

A three-inch length of the sample is tested in direct double shear. A constant pressure, appropriate to the conditions of the problem for which the test is being performed, is applied normal to the ends of the sample through porous stones. A shearing failure of the sample is caused by moving the center ring in a direction perpendicular to the axis of the sample. Transverse movement of the outer rings is prevented.

The shearing failure may be accomplished by applying to the center ring either a constant rate of load, a constant rate of deflection, or increments of load or deflection. In each case, the shearing load and the deflections in both the axial and transverse directions are recorded and plotted. The shearing strength of the soil is determined from the resulting load-deflection curves.

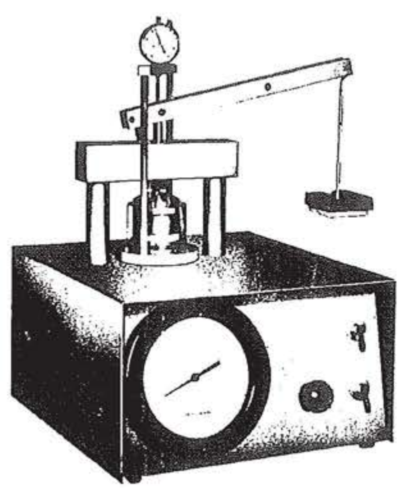
Friction Tests

In order to determine the frictional resistance between soil and the surfaces of various materials, the center ring of soil in the direct shear test is replaced by a disk of the material to be tested. The test is then performed in the same manner as the direct shear test by forcing the disk of material from the soil surfaces.

METHOD OF PERFORMING CONSOLIDATION TESTS

CONSOLIDATION TESTS ARE PERFORMED TO EVALUATE THE VOLUME CHANGES OF SOILS SUBJECTED TO INCREASED LOADS. TIME-CONSOLIDATION AND PRESSURE-CONSOLIDATION CURVES MAY BE PLOTTED FROM THE DATA OBTAINED IN THE TESTS. ENGINEERING ANALYSES BASED ON THESE CURVES PERMIT ESTIMATES TO BE MADE OF THE PROBABLE MAGNITUDE AND RATE OF SETTLEMENT OF THE TESTED SOILS UNDER APPLIED LOADS.

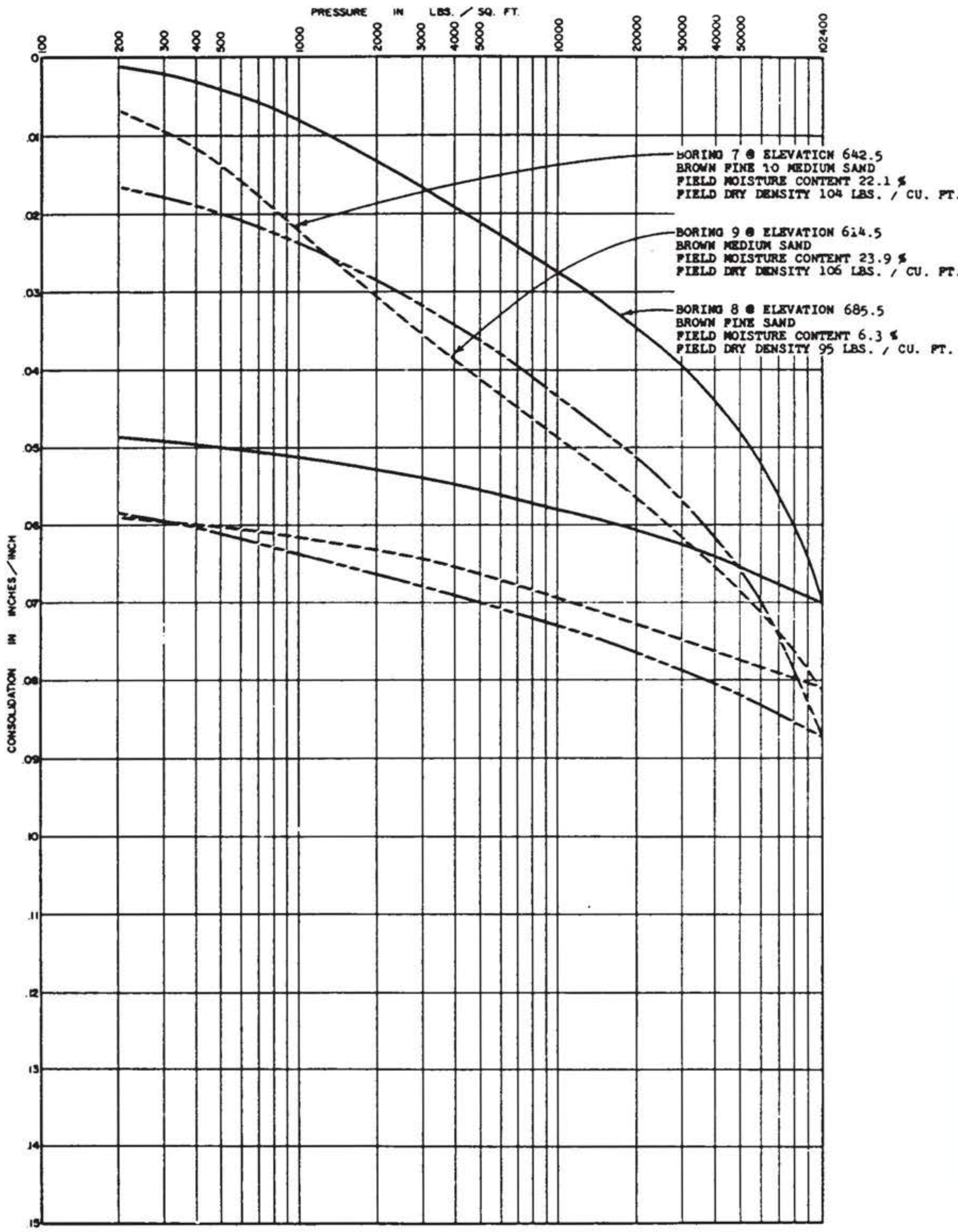
EACH SAMPLE IS TESTED WITHIN BRASS RINGS TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED IN RINGS TO PREDETERMINED CONDITIONS AND TESTED.



DEAD LOAD-PNEUMATIC CONSOLIDOMETER

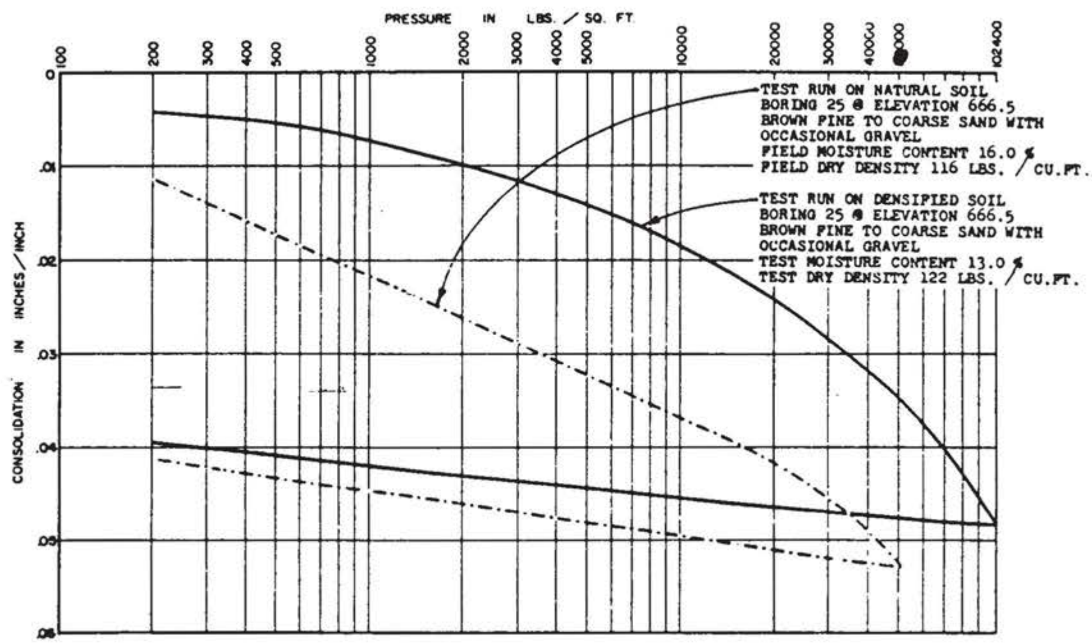
IN TESTING, THE SAMPLE IS RIGIDLY CONFINED LATERALLY BY THE BRASS RING. AXIAL LOADS ARE TRANSMITTED TO THE ENDS OF THE SAMPLE BY POROUS DISKS. THE DISKS ALLOW DRAINAGE OF THE LOADED SAMPLE. THE AXIAL COMPRESSION OR EXPANSION OF THE SAMPLE IS MEASURED BY A MICROMETER DIAL INDICATOR AT APPROPRIATE TIME INTERVALS AFTER EACH LOAD INCREMENT IS APPLIED. EACH LOAD IS ORDINARILY TWICE THE PRECEDING LOAD. THE INCREMENTS ARE SELECTED TO OBTAIN CONSOLIDATION DATA REPRESENTING THE FIELD LOADING CONDITIONS FOR WHICH THE TEST IS BEING PERFORMED. EACH LOAD INCREMENT IS ALLOWED TO ACT OVER AN INTERVAL OF TIME DEPENDENT ON THE TYPE AND EXTENT OF THE SOIL IN THE FIELD.

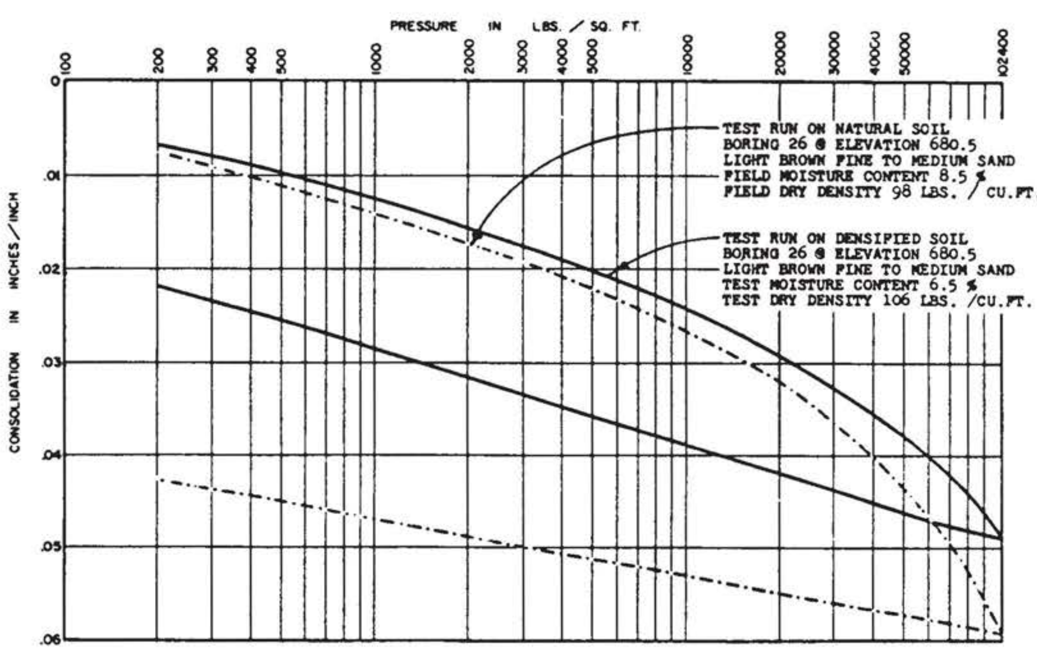
DAMES & MOORE
APPLIED EARTH SCIENCES



STATIC CONSOLIDATION TEST

DAMES & MOORE

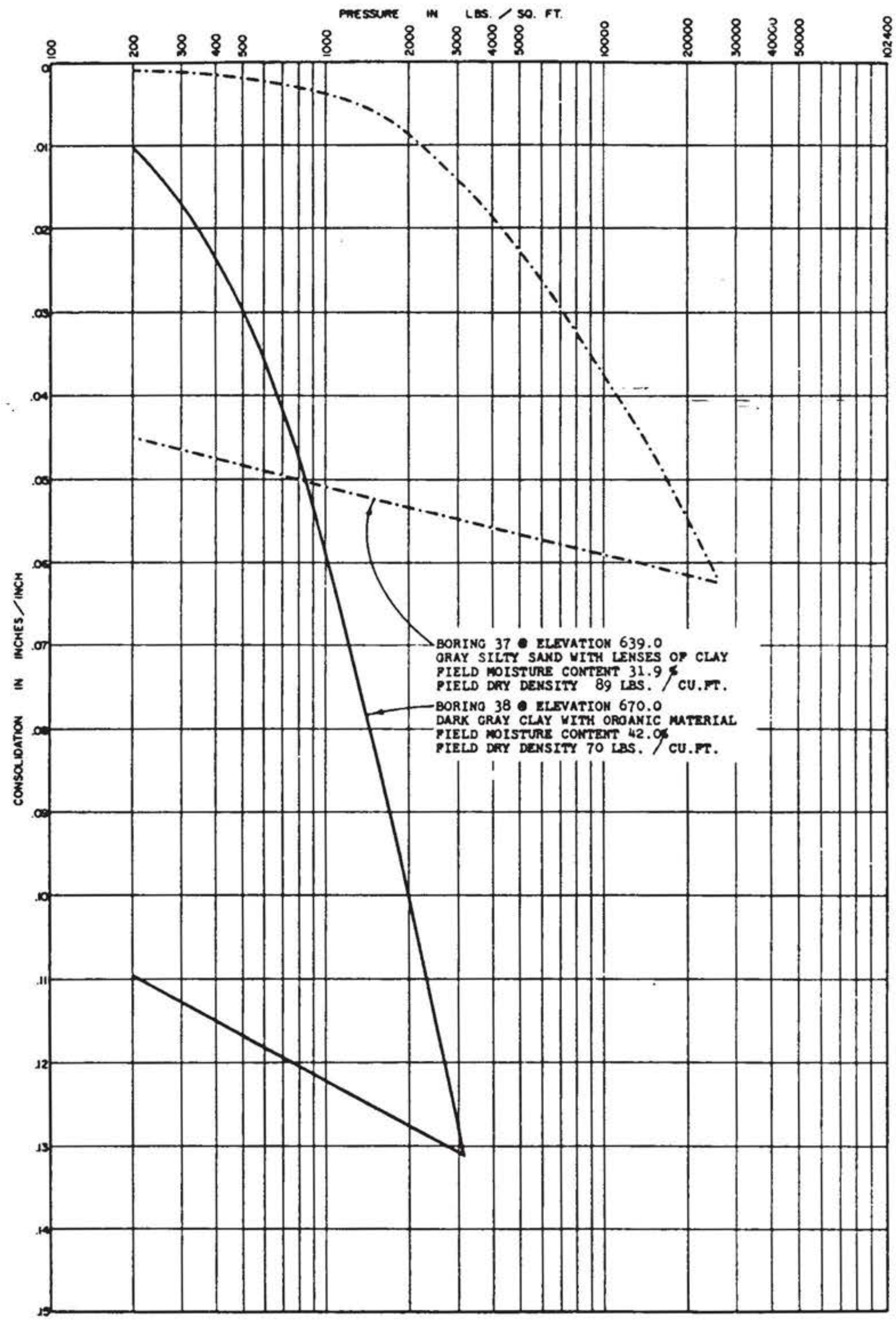




STATIC CONSOLIDATION TEST

DAMES & MOORE

PLATE 6.43



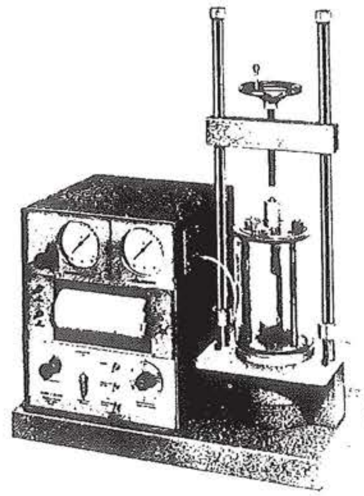
STATIC CONSOLIDATION TEST

DAMES & MOORE

Methods of Performing Unconfined Compression and Triaxial Compression Tests

THE SHEARING STRENGTHS OF SOILS ARE DETERMINED FROM THE RESULTS OF UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS. IN TRIAXIAL COMPRESSION TESTS THE TEST METHOD AND THE MAGNITUDE OF THE CONFINING PRESSURE ARE CHOSEN TO SIMULATE ANTICIPATED FIELD CONDITIONS.

UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS ARE PERFORMED ON UNDISTURBED OR REMOLDED SAMPLES OF SOIL APPROXIMATELY SIX INCHES IN LENGTH AND TWO AND ONE-HALF INCHES IN DIAMETER. THE TESTS ARE RUN EITHER STRAIN-CONTROLLED OR STRESS-CONTROLLED. IN A STRAIN-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO A CONSTANT RATE OF DEFLECTION AND THE RESULTING STRESSES ARE RECORDED. IN A STRESS-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO EQUAL INCREMENTS OF LOAD WITH EACH INCREMENT BEING MAINTAINED UNTIL AN EQUILIBRIUM CONDITION WITH RESPECT TO STRAIN IS ACHIEVED.



TRIAXIAL COMPRESSION TEST UNIT

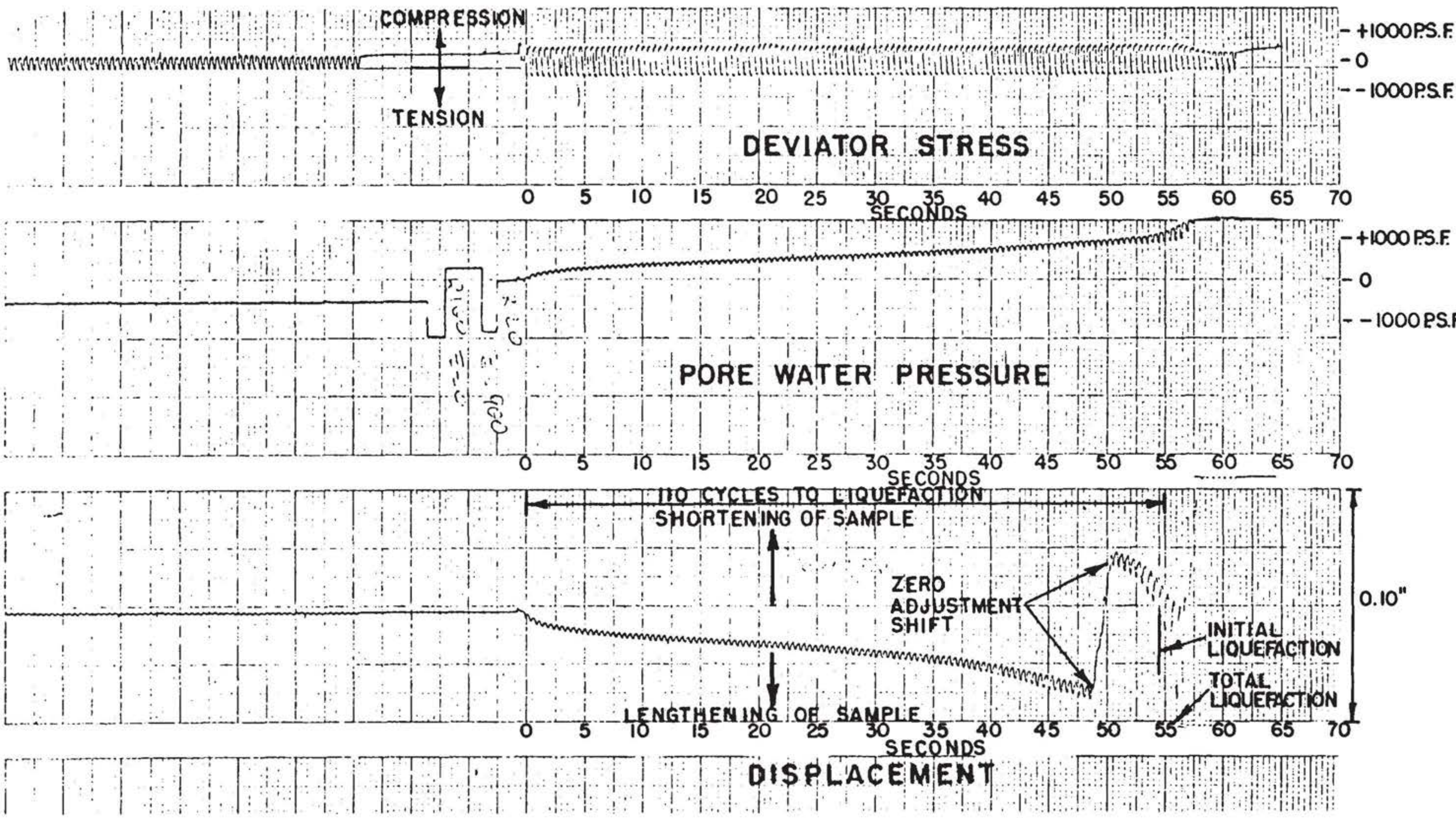
YIELD, PEAK, OR ULTIMATE STRESSES ARE DETERMINED FROM THE STRESS-STRAIN PLOT FOR EACH SAMPLE AND THE PRINCIPAL STRESSES ARE EVALUATED. THE PRINCIPAL STRESSES ARE PLOTTED ON A MOHR'S CIRCLE DIAGRAM TO DETERMINE THE SHEARING STRENGTH OF THE SOIL TYPE BEING TESTED.

UNCONFINED COMPRESSION TESTS CAN BE PERFORMED ONLY ON SAMPLES WITH SUFFICIENT COHESION SO THAT THE SOIL WILL STAND AS AN UNSUPPORTED CYLINDER. THESE TESTS MAY BE RUN AT NATURAL MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SOILS.

IN A TRIAXIAL COMPRESSION TEST THE SAMPLE IS ENCASED IN A RUBBER MEMBRANE, PLACED IN A TEST CHAMBER, AND SUBJECTED TO A CONFINING PRESSURE THROUGHOUT THE DURATION OF THE TEST. NORMALLY, THIS CONFINING PRESSURE IS MAINTAINED AT A CONSTANT LEVEL, ALTHOUGH FOR SPECIAL TESTS IT MAY BE VARIED IN RELATION TO THE MEASURED STRESSES. TRIAXIAL COMPRESSION TESTS MAY BE RUN ON SOILS AT FIELD MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SAMPLES. THE TESTS ARE PERFORMED IN ONE OF THE FOLLOWING WAYS:

> UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.
>
> CONSOLIDATED-UNDRAINED: THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND/OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH ARE MEASURED ARE THE SAME AS FOR THE UNCONSOLIDATED-UNDRAINED TEST.
>
> DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP OF PORE WATER PRESSURES. THE RESULTING STRESSES WHICH ARE MEASURED REPRESENT ONLY THE INTERGRANULAR STRESSES. THESE TESTS ARE USUALLY PERFORMED ON SAMPLES OF GENERALLY NON-COHESIVE SOILS, ALTHOUGH THE TEST PROCEDURE IS APPLICABLE TO COHESIVE SOILS IF A SUFFICIENTLY SLOW TEST RATE IS USED.

AN ALTERNATE MEANS OF OBTAINING THE DATA RESULTING FROM THE DRAINED TEST IS TO PERFORM AN UNDRAINED TEST IN WHICH SPECIAL EQUIPMENT IS USED TO MEASURE THE PORE WATER PRESSURES. THE DIFFERENCES BETWEEN THE TOTAL STRESSES AND THE PORE WATER PRESSURES MEASURED ARE THE INTERGRANULAR STRESSES.

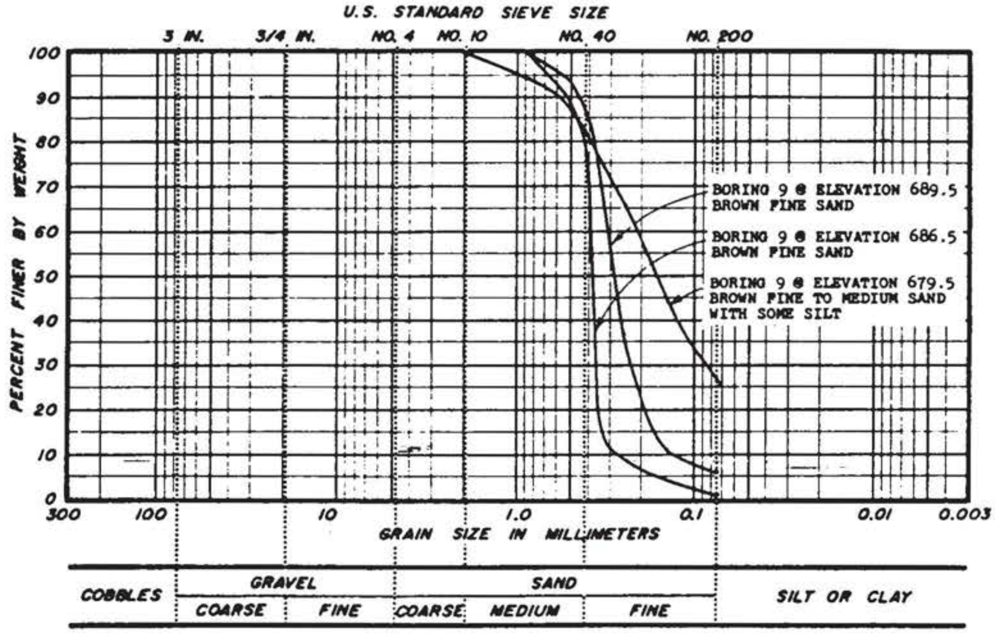


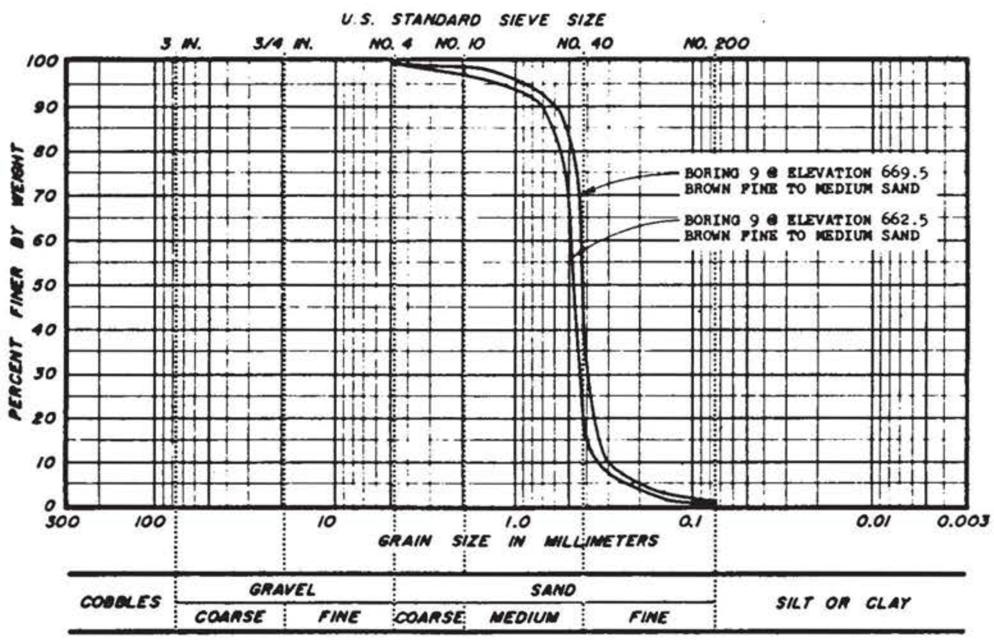
TYPICAL LABORATORY RECORD OF LIQUEFACTION

BORING 9 - SAMPLE AT 26 FEET

DAMES & MOORE
APPLIED EARTH SCIENCES

PLATE 6.46

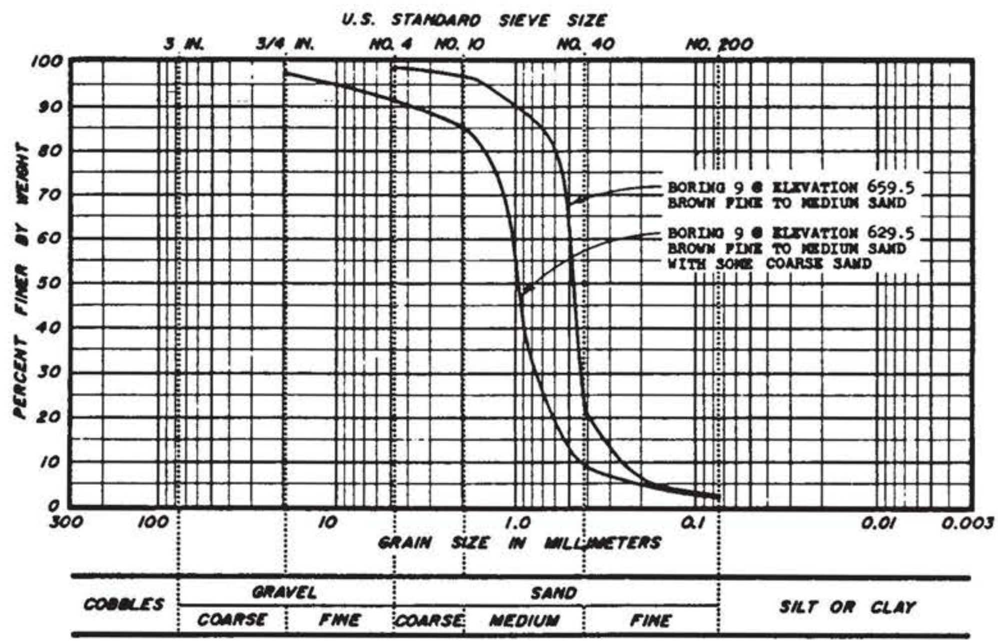




PARTICLE SIZE DISTRIBUTION

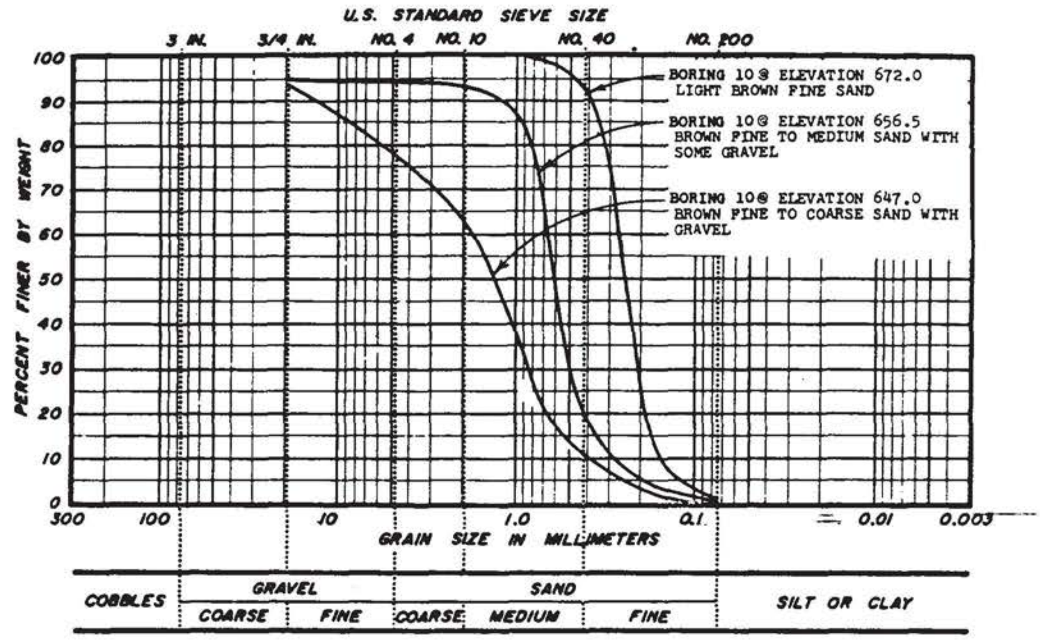
DAMES & MOORE

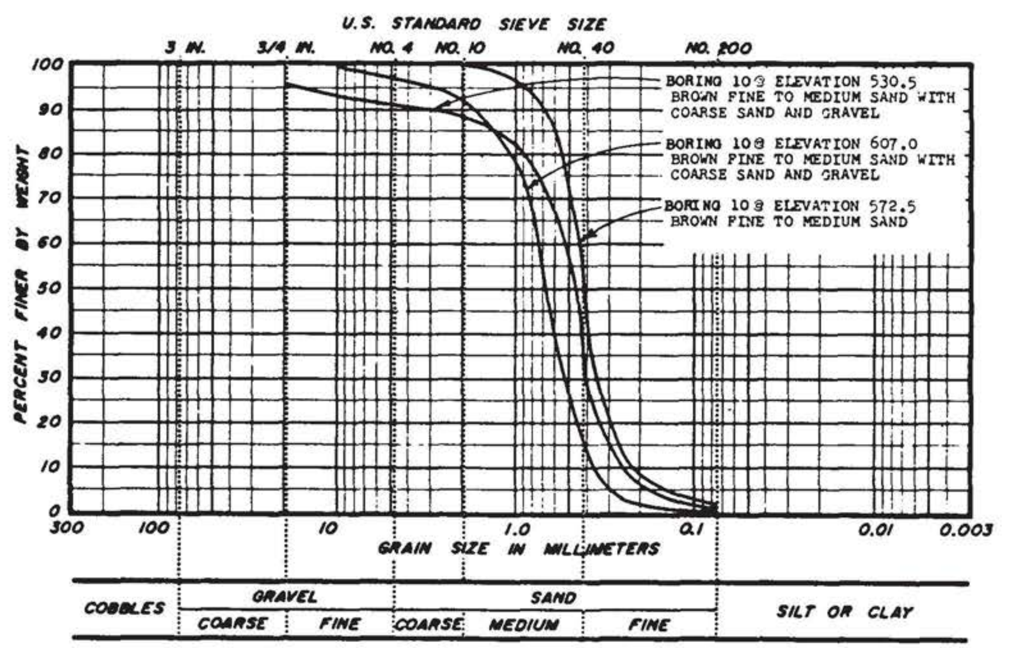
PLATE 6.47



PARTICLE SIZE DISTRIBUTION

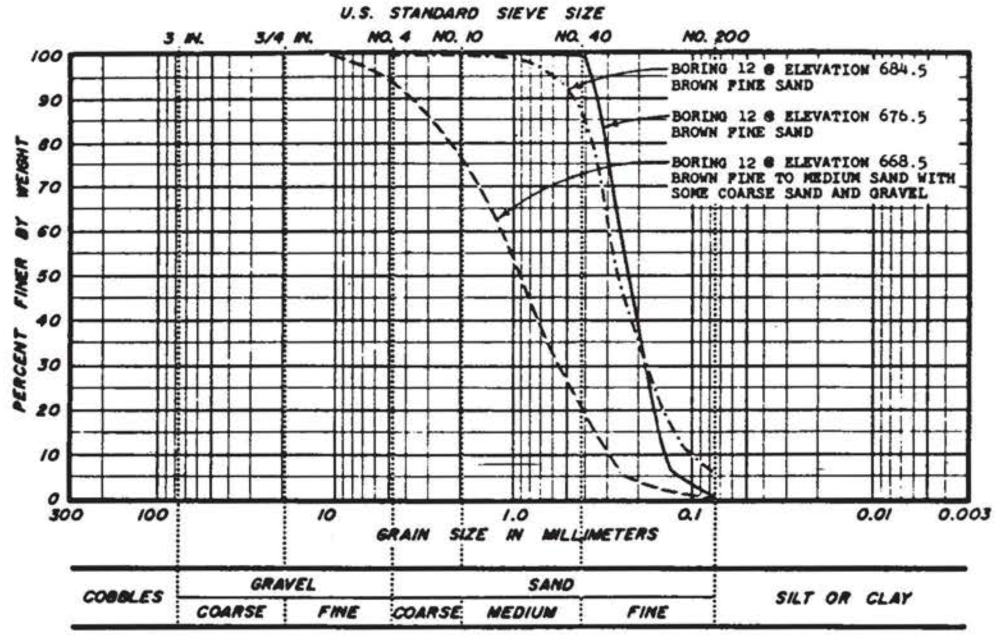
DAMES & MOORE

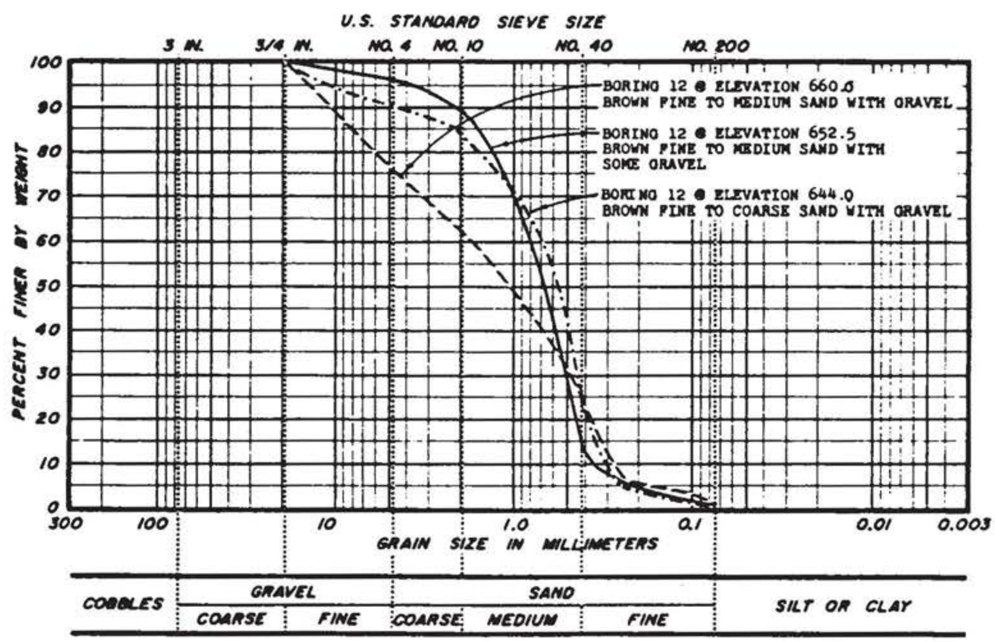




PARTICLE SIZE DISTRIBUTION

DAMES & MOORE

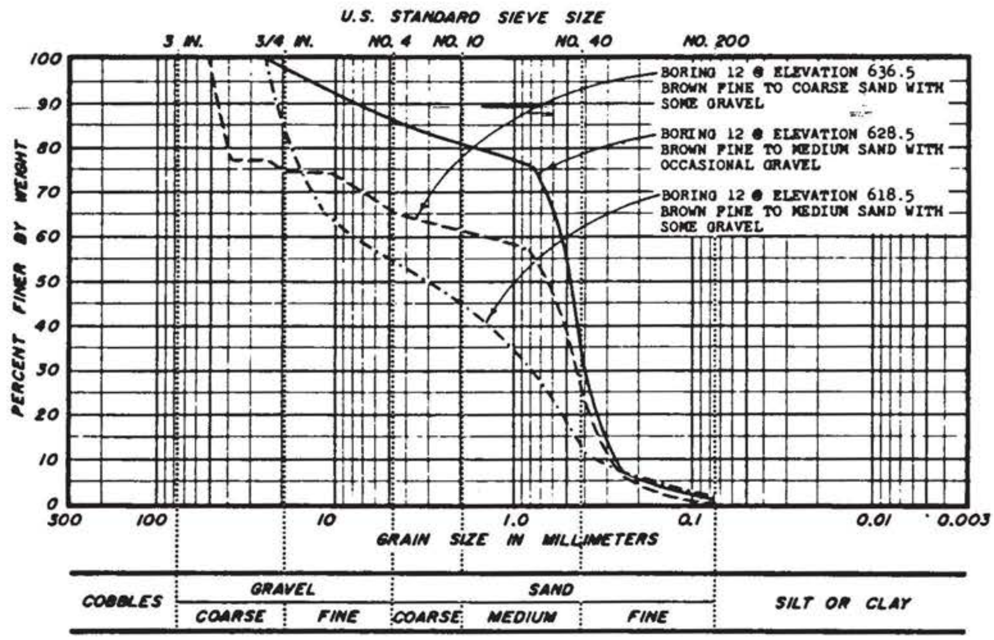




PARTICLE SIZE DISTRIBUTION

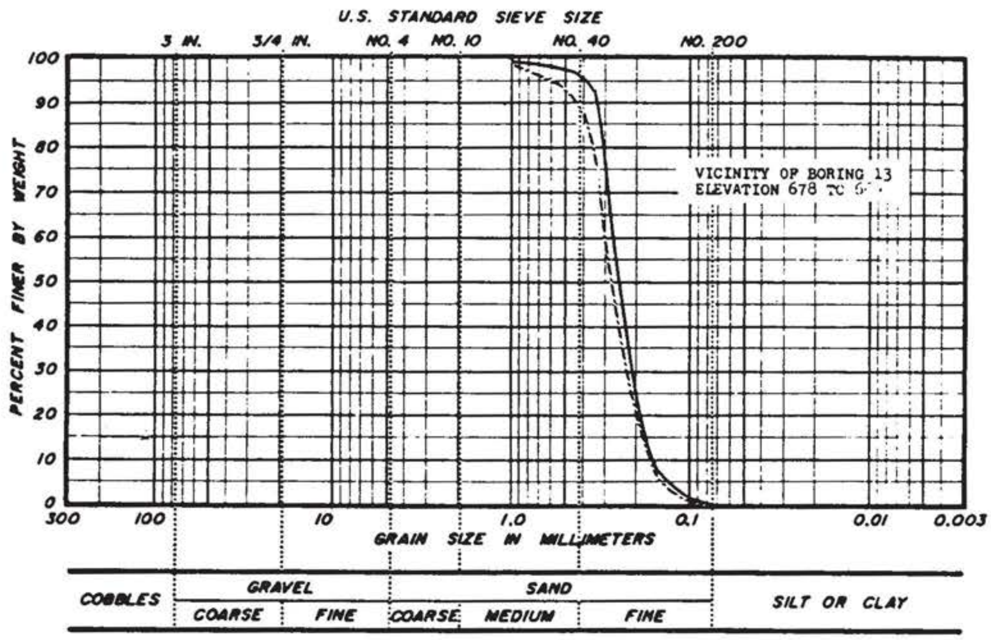
DAMES & MOORE

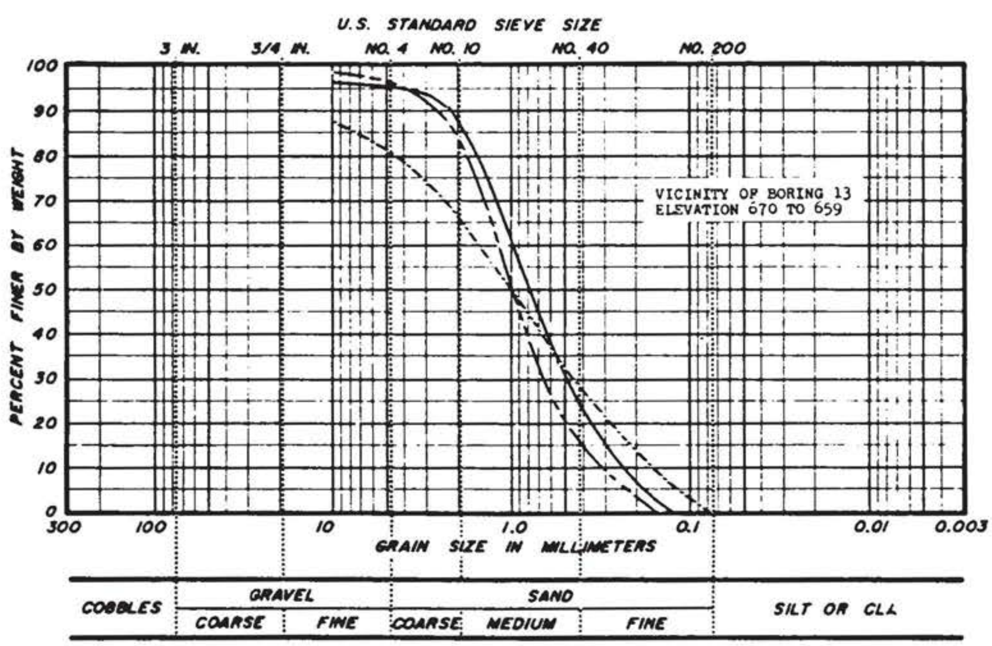
PLATE 6.50



PARTICLE SIZE DISTRIBUTION

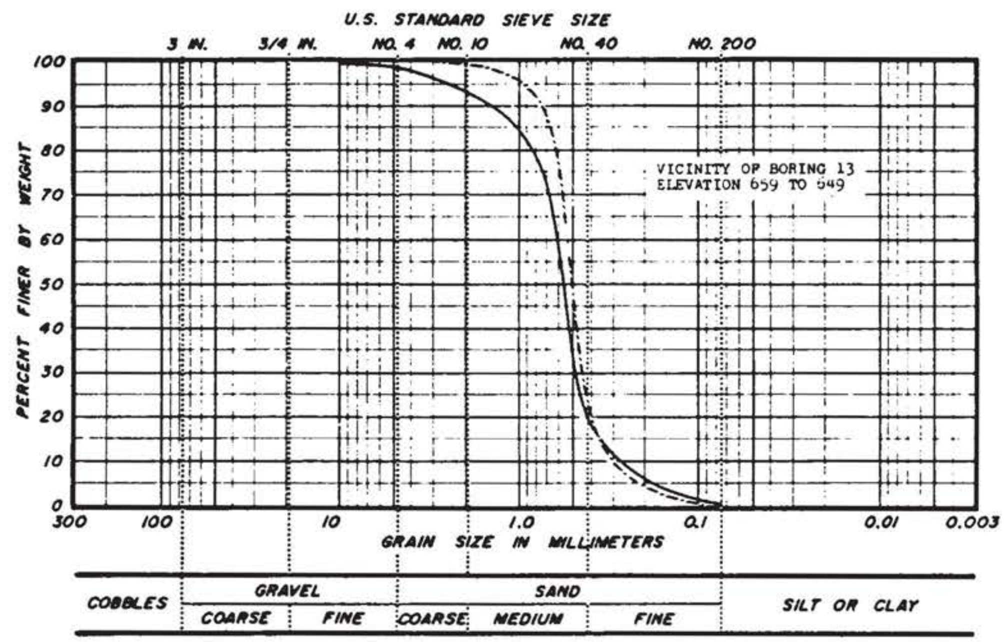
DAMES & MOORE





PARTICLE SIZE DISTRIBUTION

DAMES & MOORE



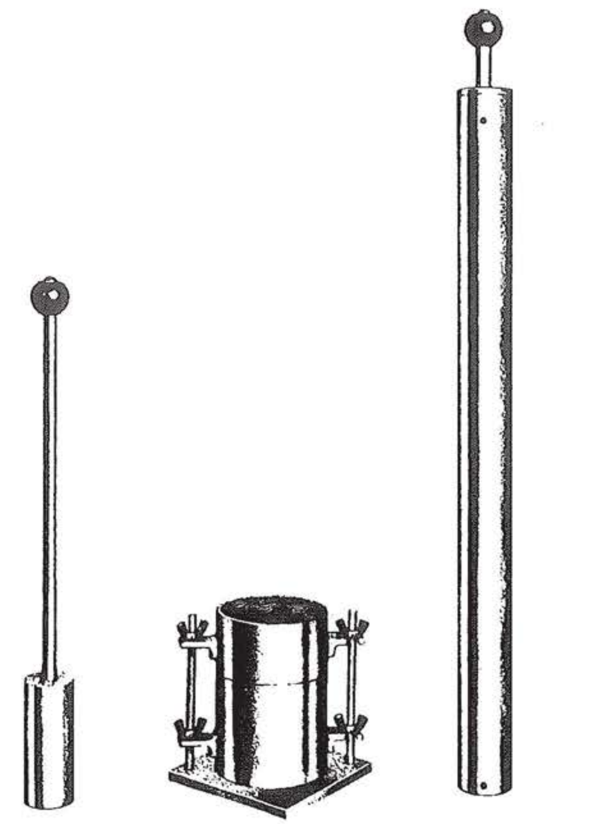
PARTICLE SIZE DISTRIBUTION

DAMES & MOORE

Method Of Performing Compaction Tests
(STANDARD AND MODIFIED A.A.S.H.O. METHODS)

IT HAS BEEN ESTABLISHED THAT WHEN COMPACTING EFFORT IS HELD CONSTANT, THE DENSITY OF A ROLLED EARTH FILL INCREASES WITH ADDED MOISTURE UNTIL A MAXIMUM DRY DENSITY IS OBTAINED AT A MOISTURE CONTENT TERMED THE "OPTIMUM MOISTURE CONTENT," AFTER WHICH THE DRY DENSITY DECREASES. THE COMPACTION CURVE SHOWING THE RELATIONSHIP BETWEEN DENSITY AND MOISTURE CONTENT FOR A SPECIFIC COMPACTING EFFORT IS DETERMINED BY EXPERIMENTAL METHODS. TWO COMMONLY USED METHODS ARE DESCRIBED IN THE FOLLOWING PARAGRAPHS.

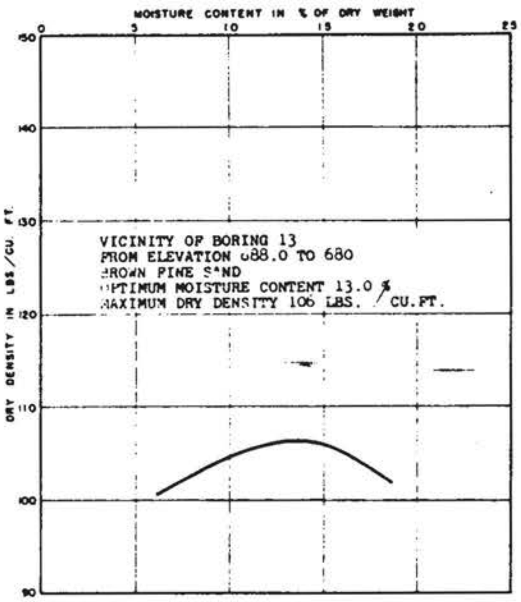
FOR THE "STANDARD A.A.S.H.O." (A.S.T.M. D698-58T & A.A.S.H.O. T99-57) METHOD OF COMPACTION A PORTION OF THE SOIL SAMPLE PASSING THE NO. 4 SIEVE IS COMPACTED AT A SPECIFIC MOISTURE CONTENT IN THREE EQUAL LAYERS IN A STANDARD COMPACTION CYLINDER HAVING A VOLUME OF 1/30 CUBIC FOOT, USING TWENTY-FIVE 12-INCH BLOWS OF A STANDARD 5-1/2 POUND RAMMER TO COMPACT EACH LAYER.

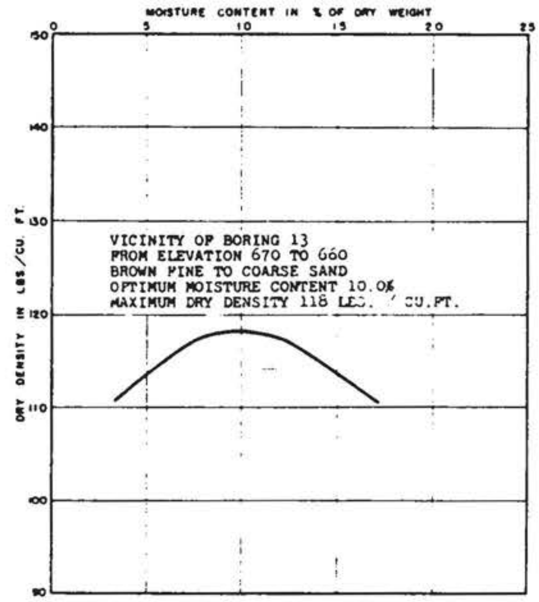


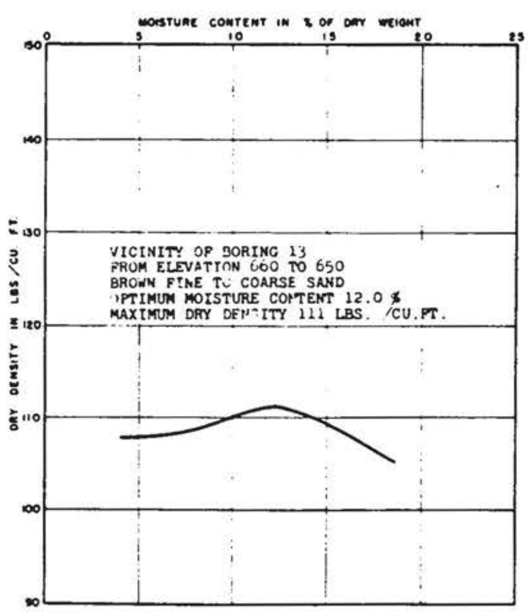
SOME APPARATUS FOR PERFORMING COMPACTION TESTS
Shows, from left to right, 5-1/2 pound rammer (sleeve controlling 12" height of drop removed), 1/30 cubic-foot cylinder with removable collar and base plate, and 10 pound rammer within sleeve.

IN THE "MODIFIED A.A.S.H.O." (A.S.T.M. D-1557-58T & A.A.S.H.O. T 180-57) METHOD OF COMPACTION A PORTION OF THE SOIL SAMPLE PASSING THE NO. 4 SIEVE IS COMPACTED AT A SPECIFIC MOISTURE CONTENT IN FIVE EQUAL LAYERS IN A STANDARD COMPACTION CYLINDER HAVING A VOLUME OF 1/30 CUBIC FOOT, USING TWENTY-FIVE 18-INCH BLOWS OF A 10-POUND RAMMER TO COMPACT EACH LAYER. SEVERAL VARIATIONS OF THESE COMPACTION TESTING METHODS ARE OFTEN USED AND THESE ARE DESCRIBED IN A.A.S.H.O. & A.S.T.M. SPECIFICATIONS.

FOR BOTH METHODS, THE WET DENSITY OF THE COMPACTED SAMPLE IS DETERMINED BY WEIGHING THE KNOWN VOLUME OF SOIL; THE MOISTURE CONTENT, BY MEASURING THE LOSS OF WEIGHT OF A PORTION OF THE SAMPLE WHEN OVEN DRIED; AND THE DRY DENSITY, BY COMPUTING IT FROM THE WET DENSITY AND MOISTURE CONTENT. A SERIES OF SUCH COMPACTIONS IS PERFORMED AT INCREASING MOISTURE CONTENTS UNTIL A SUFFICIENT NUMBER OF POINTS DEFINING THE MOISTURE-DENSITY RELATIONSHIP HAVE BEEN OBTAINED TO PERMIT THE PLOTTING OF THE COMPACTION CURVE. THE MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT FOR THE PARTICULAR COMPACTING EFFORT ARE DETERMINED FROM THE COMPACTION CURVE.







COMPACTION TEST DATA

NOTE:

THESE COMPACTION TESTS WERE PERFORMED
IN ACCORDANCE WITH THE A.A.S.H.O., T 180-57
METHOD A, METHOD OF COMPACTION

DAMES & MOORE

PLATE 6.55