2.0 SITE CHARACTERISTICS

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 SITE LOCATION AND DESCRIPTION

2.1.1.1 Specification of Location

The SHNPP site is located in the extreme southwest corner of Wake County, North Carolina, and the southeast corner of Chatham County, North Carolina. The City of Raleigh, North Carolina, is approximately 16 mi. northeast, and the City of Sanford is about 15 mi. southwest.

Carolina Power & Light Company (now Duke Energy Progress, Inc.) has constructed a dam on Buckhorn Creek about 2.5 mi. north of its confluence with the Cape Fear River. This dam has created an approximately 4000-acre reservoir which will be used for cooling tower makeup requirements. The power block structures are located on the northwest shore of the Main Reservoir about 4.5 mi. north of the Main Dam.

Coordinates of the reactor are:

Latitude	(North)	35° 38’ 00”
Longitude	(West)	78° 57’ 22”
North Carolina	(North)	685,444.524
Plane Coordinates	(East)	2,013,001.262
Universal Transverse	(North)	3,945,013.683
Mercator Coordinates	(East)	685,064.389

The universal transverse Mercator zone number for the SHNPP is 17.

All elevations are referred to the National Geodetic Vertical Addendum of 1929, commonly known as mean sea level (MSL).

2.1.1.2 Site Area Map

Maps of the site area are included as Figures 2.1.1-1 and 2.1.2-1. Indicated are the site boundary line (which is the same as the station property boundary), the principal plant structures, the exclusion area, and the principal transportation routes. The station requires approximately 10,800 ac. Duke Energy Progress, Inc. (DEP) owns all land within the site boundary lines. There are no private, residential, industrial, institutional, or commercial structures (other than those related to plant operation) within this area. However, as recreational usage increases at the Main Reservoir, some recreational structures may be constructed in accordance with DEP’s land use policy.

The minimum distance (± 25 ft.) and direction from the reactor to an exclusion area boundary is 6790 ft. ESE.

U.S. Highway 1 passes north of the site, and several State maintained roads traverse the area, allowing access to the plant and reservoir. The CSX Corp. Railroad passes north of the plant, and the Southern Railroad crosses south of the Main Dam. Railway access to the plant is provided by a DEP rail spur which connects to the CSX Corp. Railroad.

The Cape Fear River lies adjacent to the site. Use of the river near the plant is limited to small, recreational boating activities.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

The protected area, defined for the purpose of controlling access and egress to and from the site, coincides with the plant security fence as reflected in the Security Plan. The plant security fence and the protected area and its relation to the Main and Auxiliary Reservoirs are shown on Figure 2.1.2-1. Access and egress to the protected area is controlled by a security organization on the site. Authority to enter the area is given to plant personnel, authorized contractor personnel, and authorized visitors only. Before entry is authorized to a new employee or visitor, such a person may be subject to radiological safety training and may be issued a dosimetry device for recording personal exposure to radiation. Unauthorized access is prevented by physical barriers, closed-circuit TV cameras, security force patrols, intrusion detection equipment, and access control. These measures are described in greater detail in Section 13.6 and the Security Plan, which is submitted separately. Radiation monitors are also located at the exit to the plant-protected area for radiation protection purposes at egress from the area.

The effluent release limits are established in accordance with 10 CFR 20 and Appendix I to 10 CFR 50 in order to ensure that (1) the concentrations of radionuclides in gaseous effluent at the exclusion boundary do not exceed the limits set forth in Table 2, Column 1 of Appendix B to 10 CFR 20; (2) the annual average concentrations of radionuclides in liquid effluent at the point of discharge do not exceed the limits set forth in Table 2, Column 2 of Appendix B to 10 CFR 20; and (3) the cumulative liquid and gaseous radionuclide releases do not result in exposures to individuals outside the exclusion boundary in excess of the limits set forth in Appendix I to 10 CFR 50.

The liquid radioactive release point (via the cooling tower blowdown discharge line) is shown on Figure 2.4.1-1. Gaseous radioactive release points are shown on Figure 9.4.0-2. This figure identifies gaseous release points (airborne effluent release points) as well as outside air intakes. Figure 9.4.0-2 references Figure 9.4.0-1 for definition of the appropriate symbols for the gaseous release points as opposed to the outside air intake. Figure 9.4.0-2 also references Figure 2.1.2 1 for definition of the distances of the gaseous release points (airborne effluent release points) to the plant exclusion boundary. The Liquid and Gaseous Waste Processing Systems are discussed in Sections 11.2 and 11.3, respectively. These radioactive releases are within the limits set forth in 10 CFR 20 and 10 CFR 50.

2.1.2 EXCLUSION AREA AUTHORITY AND CONTROL

2.1.2.1 Authority

The exclusion area is shown on Figure 2.1.2-1. All lands within the exclusion area are owned by DEP. For the most part, roads which existed within the exclusion area prior to construction have been abandoned by the State and are blocked to prevent public use. The exception to this is Shearon Harris Road (SR1134) which connects with access roads to the plant (Figure 2.1.2-

1). Easements have been granted to the State of North Carolina for maintenance of this road. An easement has also been granted to the area Telephone Service Provider for maintenance of communications lines to the plant.

Duke Energy Progress, Inc. owns and maintains the rail spur which connects the plant with commercial railroad service. Under a commercial side-track agreement, rail carriers have access to the plant over the DEP track.

No mineral rights have been leased within the exclusion area and there are no rights outstanding which could allow production of either surface or sub surface minerals. Furthermore, the potential for commercial exploitation of minerals within the exclusion area is minimal, and leasing of mineral rights by DEP is not anticipated.

The distance from the plant to the exclusion area boundary for each major compass direction is given in Table 2.1.2-1.

2.1.2.2 Control of Activities Unrelated to Plant Operation.

Activities unrelated to plant operations which will be permitted within the exclusion area (aside from transit through the area) are described below:

a) Activities along the State road will generally be limited to highway/utility maintenance. This activity could take place anywhere within the easement granted to the State. In the event of an emergency requiring an exclusion area evacuation, road/utility maintenance personnel will be evacuated in accordance with Security Procedures. Signs are posted along the road at the exclusion area boundary stating that the area is private property and advising persons therein that they are subject to evacuation. It is estimated that the road could be cleared of any maintenance personnel within thirty minutes.

b) Activity on the rail spur will be limited to that which is directly related to delivery of rail cars to the plant. The rail spur is owned by DEP, and necessary maintenance is under the cognizance of the Company. Commercial railroad personnel in the exclusion area involved in the delivery of rail cars could be evacuated within fifteen minutes.

c) Activity along the easement granted to the area Telephone Service Provider consists of construction, modification, repair, and maintenance of telephone lines to the plant. Signs are posted whenever the easement intersects the exclusion area boundary, stating that the area is private property and advising that anyone therein is subject to evacuation.

   It is estimated that no more than ten people will be involved in telephone maintenance/installation operations in the exclusion area at any one time and that they could be evacuated within thirty minutes.

d) Recreational use of the land and main reservoir within the exclusion area, by the general public, is permitted. Warning signs, similar to those described above, are posted at known points of entry on the exclusion area boundary (land) and buoyed in conspicuous locations within and on the boundary of reservoir waters inside the exclusion area. Persons in these areas should be able to clear the exclusion area within one hour of notification.

e) HNP Nuclear Security Firearms Training Range, Cary Police Department Firing Range and Wake County Fire Department Training Facility are located within the Owner Controlled Exclusion Area. It is estimated that normally not more than 75 personnel combined will occupy these areas at one time and could be evacuated within thirty (30) minutes.

2.1.2.3 Arrangements for Traffic Control

If it becomes necessary to control traffic into the exclusion area, the following actions will be initiated:

a) Access control will be established by Plant Security/Local Law Enforcement personnel on Shearon Harris Road (SR1134) where it intersects with the exclusion area boundary in order to limit access to the area to authorized personnel.

b) In a similar manner, the rail spur will be closed to rail traffic. Scheduled rail deliveries may be cancelled or postponed.

c) Telephone service activities along the easement in the exclusion area will be prohibited or postponed. If necessary, warning signs will be posted at the intersection of the easement and the boundary.

d) Control of public access to recreational land and water areas will be exercised by motorized patrols of known or likely points of entry on land and by patrol boats on the reservoirs. Plant Security/Local Law Enforcement will provide traffic control. Assistance will be requested from the county sheriff departments and the North Carolina Highway Patrol.

2.1.3 POPULATION DISTRIBUTION

Estimates of existing population were based on the 2012 Evacuation Time Estimate Study completed by KLD Engineering, which was derived using data from the 2010 U.S. Census. Geographical Information System (GIS) software was used to process the geographic data and associated population counts for census blocks in each of the counties surrounding the plant. The populations were then aggregated over subzones to generate a permanent resident population count.

2.1.3.1 Population Within Ten Miles

A map showing the 10-mile radial area of the site is presented in Figure 2.1.3-1. Concentric circles have been drawn at distances of 1, 2, 3, 4, 5, and 10 miles from the center point of the originally planned four units. The circles have been divided into 22-1/2-degree segments, with each segment centered on one of the 16 compass points. The 2010 estimates of residential population within each of these areas are presented in Figure 2.1.3-5.

2.1.3.2 Population Between Zero and Fifty Miles

The population within a 50-mile radius of the plant site is marked by concentrations of people in and around the cities of Raleigh (16-28 mi. NE; 2010 population of 403,892); Durham (20-30 mi. N; 2010 population of 228,330); Fayetteville (37-43 mi. S; 2010 population of 200,564); and

Cary (13-18 mi. NE; 210 population of 135,234). Several other smaller cities and towns have populations greater than 10,000. Away from these population concentrations, there is a rural-type population distribution with small towns interspersed through the area. A map showing the 50-mile radial area and identifying major cities and towns is presented in Figure 2.1.3-2. Concentric circles have been drawn at distances of 10, 20, 30, 40, and 50 miles, using the center point of the originally planned four units. The circles have been divided into 22-1/2-degree segments, with each segment centered on one of the 16 compass points.

2.1.3.3 Transient Population

Recreational land uses which would attract transient concentrations of people within the 50-mile radius of the site are not extensive and are limited to the Harris County Park (2 mi. SE), Jordan Lake State Recreation Area (5-12 mi. NW), Umstead State Park (20 mi. NE), Raven Rock State Park (13 mi. SSE), Eno River State Park (30 mi. N), and Falls Lake State Recreation Area (30 mi. NNE) (Figure 2.1.3-3). On occasions, there are also high concentrations of people at sporting events and at functions at the various universities in the area. The North Carolina State Fair, a 10-day event held during October of each year in Raleigh, attracted a maximum of 151,467 (single-day attendance) people in 2010 (Reference 2.1.3-1).

Daily transient population concentrates in and around the major industrial areas of the region as a result of commuting patterns of workers. Figure 2.1.3-6 summarizes the transient population within the 10-mile EPZ. Reference 2.1.3-2, Appendix E, lists all public use facilities within the 10-mile Emergency Planning Zone (EPZ).

2.1.3.4 Low Population Zone

As stated in Reference 2.1.3-5, the analysis of the Low Population Zone (LPZ) is reviewed to assure that appropriate protective measures could be taken in this area in the event of an emergency. The SHNPP Emergency Plan is the primary document to provide this assurance.

The low population zone is defined as land within a three-mile radial area as measured from the center point of the originally planned four reactors. The basis for its selection is in conformance to both the definition of "low population zone" specified in 10 CFR 100.3 and the method for determining a low population zone specified in 10 CFR 100.11.

The three-mile radial area scribed from the center point of the plant is conservatively large such that it envelopes the set of low population zones scribed from the center point of each originally planned reactor. Dose calculations which show conformance to the dose limit criteria of 10 CFR 50.67 are described in Section 15.6.5.

The low population zone is shown in Figure 2.1.3-1 and is superimposed on the three-mile concentric circle. Highways, railways, and waterways are identified. The Harris County Park is located 2 mi. SE of the plant. The Harris Energy & Environmental Center is located 2.1 mi. ENE of the plant. Two private nursing homes (Brown’s Family Care Home and James Rest Home) are located between 2 and 3 miles NE. There are no other facilities or institutions such as schools, hospitals, prisons, or beaches, within the low population zone. However, as recreational usage increases at the Harris Reservoir, additional recreational areas may be established within the low population zone in accordance with the Duke Energy Progress, Inc. land use policy.

There are no other facilities or institutions beyond the low population zone and within 5 mi. of the plant that require special consideration when evaluating emergency plans.

The daily transient population within the low population zone (three mile radius) is approximately 2,455 people. This estimate includes employees and recreational transients within the three mile radius.

2.1.3.5 Population Center

The nearest population centers as defined in 10 CFR 100 are Holly Springs, NC (8 mi. E), and Apex, NC (8 mi. NE). The 2010 population of Holly Springs was 24,661. This number reflects a 268% increase over the 2000 census figure of 9,192. The 2010 population of Apex was 37,746. This number reflects a 54% increase over the 2000 census figure of 20,212. Cary, NC (13-18 mi. NE), the previous nearest population center, had a 2010 population of 135,234.

Holly Springs, Apex, and Cary are located at a distance greater than one and one-third times the distance from the reactor to the nearest Low Population Zone boundary.

2.1.3.6 Deleted

2.1.3.7 Evacuation Planning

Evacuation time studies provide Duke Energy Progress, Inc. and state/local governments site-specific information helpful to protective action decision making. The studies provide information on current population within the 10-mile Emergency Planning Zone (EPZ) and time estimates for evacuation of each area. See Table 2.1.3-6 and Figure 2.1.3-4. This information is extracted from a study, Reference 2.1.3-2, prepared by KLD Engineering in December, 2012.

REFERENCES: SECTION 2.1

2.1.3-1 North Carolina State Fair. About Us: Attendance (1986-2013) http://www.ncstatefair.org/2013/About/Attendance.htm Retrieved 11/9/2013.

2.1.3-2 KLD Engineering. Harris Nuclear Plant: Development of Evacuation Time Estimates. December 2012.

2.1.3-3 Deleted

2.1.3-4 Deleted

2.1.3-5 U. S. NRC. Standard Review Plan, NUREG-0800, July 1981, Section 2.1.3.

2.2 NEARBY INDUSTRIAL, TRANSPORTATION AND MILITARY FACILITIES

2.2.1 LOCATION AND ROUTES

An investigation was undertaken to locate all significant manufacturing plants, chemical plants, refineries, storage facilities, mining and quarrying operations, active military bases, transportation facilities, oil and gas pipelines, and underground gas facilities. Airplane high and

low level flights and landing patterns (commercial, general aviation, and military) were also included in the search.

There are no significant military facilities within a 25-mile radius of the plant site. The nearest active military facility is Fort Bragg (35 miles south), a support base for Army training operations.

Industrial activity in the area surrounding the plant is not intensive. Durham, Guilford, Alamance, and Orange counties provide the most concentrated industrial areas within a 50-mile radius of the plant.

There is some light industry at the 5600-acre Research Triangle Park, which is located approximately 20 miles NNE of the plant site. The nearest industrial facilities are near Moncure, about seven miles west of the site. There are no industrial facilities within a five-mile radius of the power block.

The City of Raleigh, and to a lesser extent the City of Durham, serve as rail and highway transportation centers for the area. They are both over 15 miles from the plant site. The highways in and around these cities carry large amounts of traffic. US Highway 1, which passes approximately 6640 ft. north northwest of the plant site (Figure 2.1.1-1), has a range of 9,900 vehicles near the plant to 21,800 vehicles near Apex to 61,700 vehicles near the City of Raleigh.

Rail transportation is principally for freight to and through the major cities. The CSX Railroad passes approximately 8000 feet north of the plant site. The Norfolk Southern Railroad (previously called the Southern Railroad) passed through the Main Dam area and was relocated just south of the Main Reservoir. The Durham branch of the Norfolk-Southern Railroad which ran from Bonsal, North Carolina, passed directly through the location of the plant site. A section of this branch, which ran from north of the plant site to Duncan, North Carolina, was purchased by CP&L. A small section of this purchase is utilized as an access spur to the plant. Use of the remainder of this line was discontinued.

Within a ten-mile radius surrounding the plant, there is one general aviation airport, which is discussed in Section 2.2.2.5. The Cape Fear River runs southwest of the plant site, but this part of the river is not used for commercial traffic (Figure 2.1.3-1).

2.2.2 DESCRIPTIONS

2.2.2.1 Description of Facilities

Tobacco manufacturing and processing is the principal industry in Durham County. Furniture manufacturing is found in Orange, Alamance, and Guilford counties. In Guilford and Alamance counties, textile manufacturing is also a very prevalent industry.

The Research Triangle Park and the Raleigh/Wake County area (including Apex, Fuquay-Varina and Holly Springs) contain light industry such as electronic component manufacturing, electronic research, fiber chemistry research, pharmaceutical research, health statistics studies, and air pollution control studies. This area employs approximately 600,000 people.

There is no industrial development within a five-mile radius of the plant site. However, there is a local concentration of industry which has developed in the vicinity of the Moncure community (seven miles west). Wood products, adhesive resins, and synthetic fibers are manufactured

there by approximately 1500 employees. These facilities and their products are at such a distance from the nuclear reactor that they will pose no safety hazard to the plant site.

2.2.2.2 Description of Products and Materials

Sections 2.2.3 and 9.5.1 describe hazardous materials which are stored at and transported to the SHNPP site. A liquefied propane gas pipeline is located near the site, and is shown on Figure 2.2.3-1 and discussed in Sections 2.2.2.3 and 2.2.3.2. Also, there are two mining operations and five inactive quarries within a ten-mile radius surrounding the plant site. They are listed in Table 2.2.2-1 and shown on Figure 2.2.2-1.

2.2.2.3 Pipelines

A liquified propane pipeline (Figure 2.2.3-1) is operated by Dixie Pipeline Company and has been rerouted outside of the exclusion area north of US Highway 1. This line, 3 ft. underground, will carry approximately 1600 barrels per hour at peak flow at a maximum pressure of 1440 psi. There were three ANSI-600 through-conduit, flanged-end isolation valves installed in the relocated line, one at the midpoint and one at each end. The isolation valve locations are shown on Figure 2.2.3-1. The line will terminate at Apex, North Carolina, where the fuel will be stored and distributed for local use. The pipeline is not used for storage of gas at higher than normal pressures. There are no plans to carry any product other than liquified propane in the pipeline. There are no other petroleum operations within a ten-mile radius of the plant site.

The Dixie Pipeline has easements for the relocated pipeline. The conditions of the easement grant the Dixie Pipeline Company a 50-foot right-of-way for their pipeline. The easement grants Dixie Pipeline Company the right of ingress and egress to this right-of-way, the right to install, maintain, inspect, protect, operate, modify, replace, or remove the pipeline.

2.2.2.4 Waterways

River traffic on the Cape Fear River in the vicinity of the plant is limited to small boats used for pleasure. Barge traffic is not possible in the Cape Fear River upstream of Fayetteville, North Carolina (Figure 2.4.1-4). No provisions are in place to keep small boats out of the vicinity of the intake structure. However, trash screens are used in each bay.

2.2.2.5 Airports

Raleigh Executive Airport is located approximately six miles from the plant site. It provides no commercial services. There are from 5-10 landings per day at this airport.

The nearest major airport is the Raleigh-Durham Airport, 19 miles north northeast of the plant site. There are twelve major airways branching out from this airport. Three pass within ten miles of the site (Figure 2.2.2-1). Raleigh-Durham is the only airport in the area which serves commercial traffic. Actual traffic for 1976, 1977, 1978, 1979, 1980, 1985, 1990, 1995, and 2001, and projected traffic through the year 2025 are listed in Table 2.2.2-2. The traffic is classified into air carrier, general aviation, air taxi, and military types.

The closest aviation related, active military base is Pope Air Force Base, 35 miles south of the plant site and adjacent to Fort Bragg. In addition, a National Guard facility is located at Raleigh-Durham Airport.

2.2.2.6 Projections for Industrial Growth

As reflected by the U. S. Bureau of Economic Analysis' projections on manufacturing, between 1980 and 2020 for the six-county area of Wake, Durham, Orange, Chatham, Lee, and Harnett, industrial development will tend to expand at a rate somewhat higher than the forecast for the State of North Carolina. Using 1980 as the base year, manufacturing is expected to increase approximately 201.8 percent by 2010, and 306.2 percent by 2020.

Major industrial development within a ten-mile radial distance of the plant is limited due to the lack of waste treatment and sewage facilities. Exceptions to this are in the immediate vicinity of Apex, Fuquay-Varina, and Moncure, where industrial sites are available for development.

2.2.3 EVALUATION OF POTENTIAL ACCIDENTS

Shearon Harris Nuclear Power Plant is situated in a non-industrialized area. However, transportation and use of some materials present a potential for explosions, fires, or release of toxic gases. The hazards associated with chemicals transported or stored in quantity in the vicinity of SHNPP were evaluated to assure appropriate design consideration. The spectrum of credible explosive events and missiles generated, as well as the delayed ignition of flammable vapor clouds, are addressed in Sections 2.2.3.1 and 2.2.3.2. The Control Room design, coupled with administrative procedures prevents the incapacitation of control room operators during postulated toxic gas episodes.

The design basis events for SHNPP are discussed below.

2.2.3.1 Design Basis Explosive Events

Review of all combustible materials transported or stored within five miles of SHNPP revealed that the only sources which present a potential hazard are rail and/or truck transportation of high explosives, and three 10,000 gallon underground tanks containing gasoline and/or diesel fuel. In addition, combustible material is piped in the vicinity of the plant, as discussed in Section 2.2.3.2.

2.2.3.1.1 Design Basis Events Arising from Transportation of Explosives

The complete and instantaneous detonation, at the closest point to the plant, of one train car load of TNT (200,000 lb.) was evaluated to determine the blast loadings on critical plant structures. Loadings were calculated by methods set forth in "The Effect of Nuclear Weapons Blast," Dept. of Army Pamphlet No. 39-3, 1962, and "Design of Structures to Resist Nuclear Weapons Effects," ASCE - Manual of Engineering Practice No. 42, 1961. The maximum loads were determined to be 0.4 psi, or less, within 0.062 seconds. This loading, as well as any missiles generated by the explosion, will be satisfactorily resisted by all Seismic Category I structures and critical storage tanks. In addition, other safety related equipment which is not capable of withstanding this pressure pulse is protected against the pressure pulse by location inside an enclosure.

The complete and instantaneous detonation of one truck load of TNT (approximately 50,000 lb.) would result in less severe loadings on critical plant structures.

The explosion of one of the ten thousand gallon buried gasoline tanks is considered equivalent to the detonation of approximately 100 lbs. of TNT. This assumes that the detonable mixture consists of 10 percent gasoline vapor by volume, that the remainder is air, and that the yield per pound of the mixture is the same as that of TNT.

The detonation of one of these tanks is considered highly unlikely. However, if it were to occur, it would not pose a hazard to the plant safety related structures.

2.2.3.2 Nearby Gas Pipeline

An eight in. liquified petroleum gas (LPG) pipeline, operated by the Dixie Pipeline Company, is buried three ft. underground; it carries approximately 1600 barrels per hour at peak flow at a maximum pressure of 1440 psi. It terminates at Apex, North Carolina, where the fuel is stored and distributed for local use. The locations of this pipeline and its isolation valves are shown on Figure 2.2.3-1.

The eight inch line was originally analyzed in this section as a six inch line that carried 1100 barrels per hour. The original analysis has not been changed. Rather, following the same methodology, an engineering evaluation was performed under ESR 9800222 to demonstrate that the effects of the propane gas line rupture described in this section were still acceptable with a pipeline size increase from 6" to 8". The pertinent results of this evaluation are contained in Table 2.2.3-5.

The line passes in excess of 8500 ft. of the closest plant critical structures. The analyses described below form a conservative basis for evaluating plant safety-related structures. The elevation of various points along the new location of the pipeline is given on Figure 2.2.3-1.

The effects on the plant safety-related structures resulting from a break in the LPG line have been evaluated on the basis of the following:

a) Assumptions in Calculations:

1) Double-ended rupture or slot rupture with the slot size equal to twice the flow area of the pipeline. Rupture occurs instantaneously at the closest location to the plant.

2) The released LPG liquid-gas mixture initially escapes from the break at the critical velocity for single phase flow at the design pressure of the line (1440 psi), then drops to two phase critical flow at its saturation pressure, and finally drops to inertial flow from one end and to the flow passed by the pumps at the other end, as the pressure in the pipeline falls.

3) The temperature of the atmosphere is assumed to be 72°F. Higher temperatures would lead to somewhat higher vaporization of escaping LPG fluid, but the initial flowrate would be less due to the higher quality at the exit plane. The fluid-gas cloud size and explosive yield are largely insensitive to the assumed temperature within a temperature variation of ± 30 degrees.

4) Propane disperses toward the plant at 1 m/sec. (Pasquill F. stability condition) as an airborne cloud, or alternatively, it drifts by gravity toward the plant.

5) Regardless of the potential source of ignition, a detonation of the resulting cloud is assumed to occur at selected centroids of the cloud, after the centerline concentration has reached the rich (explosive) limit.

It would take approximately 5 to 10 minutes to detect and isolate a major line leak or rupture during the day, or 30 minutes at night.

b) Calculation of Flowrate Out of the Break

The propane in the line will, upon the instant of the break, decompress isenthalpically to a saturation pressure of 125 psia essentially immediately because of the very large velocity of sound in the liquid. A decompression wave will travel very rapidly away from the break, leaving the fluid behind it at the saturation pressure. Since the propane would issue from the break at 72°F, approximately 1/3 of it would quickly vaporize, cooling the remainder to its boiling point of about -44°F. Hence the process of decompression is described by the throttling process in a pressure-enthalpy diagram. From such a diagram the exit plane quality, x, of the fluid can be estimated from:

$v = v_f + x\, v_{fg}$

where v = 2.4 ft.^{3}/lb., v_f= .0275 ft.^{3}/lb., v_g= 6.6 ft.^{3}/lb.,

$v_{fg} = v_g - v_f$

Hence x = 0.36

The value of specific volume x 2.4 ft^3/lb was determined from the pressure-enthalpy diagram for isenthalpic process to ambient pressure. V is the specific volume of the 2-phase fluid.

To estimate the flow rate out of the break, G_{Crit}, Fauske's equation, (Reference 2.2.3-1) for critical two-phase mass velocity is used:

$$G_{Crit} = \left[\frac{-\sigma g_c}{k_1 \frac{dv_g}{dp} + k_2 \frac{dx}{dp} + k_3 \frac{dv_f}{dp}} \right]^{1/2}$$

where:

	g_c	=	32.2 lbm ft./lbf sec^2
	k_1	=	$(1 - x + \sigma x)x$
	k_2	=	$v_g(1+2\sigma x-2x)+v_f(2\sigma x-2\sigma^2 x+\sigma^2)$
	k_3	=	$[1+x(\sigma-2)-x^2(\sigma-1)]\sigma$
and	σ	=	$(v_g/v_f)^{1/2}$

Since the derivatives are not known, they are approximated by

$dv_g/dp \equiv \Delta v_g/\Delta p$; $dv_f/dp \equiv \Delta v_f/\Delta p$; $dx/dp = (-1/h_{fg})(dh_f/dp + x\,dh_{fg}/dp) \equiv$

$(-1/h_{fg})\ (\Delta h_f/\Delta p + x\Delta h_{fg}/\Delta p)$

where the differences are evaluated about the saturation pressure point

($\equiv$ 125 psi)

$\sigma = (.852/.032)^{1/2} = 5.16$

$k_1 = [1 - .36 + .36\ (5.16)]\ (.36) = 0.9$

$k_2 = .852\ [1 + 10.32\ (.36) - 72] + .032\ [10.32\ (.36) - 10.32 - .72\ (26.6) + 26.6] = 3.44\ ft.^3/lb_m$

$\Delta vg\ /\Delta p = -\ 4.4 \times 10^{-5}\ \ ft.^5/lb_m - lb_f$

$k_1 dv_g/dp = -3.96 \times 10^{-5}\ ft.^5/lb_m - lb_f$

$\Delta v_f/\Delta p = 0$

$h_{fg} = h_g - h_f = 291.4 - 143.4 = 148$ Btu/lb_m

$\Delta h_f/\Delta p = 2.33 \times 10^{-3}\ \frac{Btu\ ft.^2}{lb_m - lb_f}$

$\Delta hfg/\Delta p = -1.54 \times 10^{-3}\ \frac{Btu\ ft.^2}{lb_m - lb_f}$

$dx/dp = \frac{1}{(-148)}\left[2.33\ \times 10^{-3} +\ .36\ (-154\ \times\ 10^{-3)}\right] = -1.2\ \times\ 10^{-5}\ ft.^2\ lb_f$

$k_2 dx/dp = -4.13 \times 10^{-5}\ \ ft.^5/lb_f - lb_m$

$G_{crit} = 1433$ lb./ft.^{2} sec.

Hence, for the given pipe area, .196 ft.^{2}, the flowrate out of the break is at most 560 lbm/sec., assuming that it flows out of both ends of the broken pipeline.

While it may appear that such flowrates are low for upstream stagnation pressures of 1440 psi, upstream pressure decays extremely rapidly and thus the effective stagnation pressure for the line is nearly 125 psi, except for the first fraction of a second during which the flow progresses from single phase, subcooled fluid flow to the steady two-phase flow. The 125 psi represents the static pressure at the exit, which is taken to be at saturation.

During the initial phase of the blowdown of the pipe (single phase flow) the maximum flowrate can be estimated from

$G_{max} = \left[-g_c\ \frac{dp}{sv}\ s_o\ \right]^{1/2}$

Since the constant entropy and constant enthalpy lines essentially coincide from the initial condition of p = 1440 psi and v_f = .0308 ft.^{3}/lbm to the saturated state of p = 125 psi, v_f = .0319 ft.^{3}/lbm, the partial derivative can be evaluated as the ratio of the differences

$\left(\frac{\Delta P}{\Delta v}\right)_{S_O}$ =1.06 lbf - lbm/ft.^{5}

where the initial v_f is taken along the isentrope at the initial pressure, v_f = 0.031 ft.^{3}/lbm.

The peak flowrate at t=0 would be 87,000 lbm/ft.^{2}-sec. of liquid at a specific volume of 0.031 ft.^{3}/lbm escaping with sonic velocity, i.e., 2570 ft./sec. This discharge would last a fraction of a second; i.e., the time for the wave to travel back to a location where the frictional pressure drop would equal the pressure drop to saturation. Thereafter, steady blowdown would proceed at the conservative rate of 560 lb./sec until inertial flow would be established.

At a subsequent time, as the pressure in the line falls, the flow becomes inertial or two-phase critical.

The portion of the piping not pump pressurized would drop rapidly to saturation pressure. The saturated liquid inertial flow, which is higher than the two phase critical flow for the same pressure, is approximately 30 lb./sec. Hence one can conservatively assume that a maximum of 30 lb./sec. would issue continuously from the unpressurized side of the break after the initial period of time required to depressurize the line to near saturation pressure. This time, for a line length of approximately seven miles, is about 40 seconds.

The pump pressurized side would behave differently since the pump at peak flow can deliver approximately 70 lb./sec. This quantity is sufficient to maintain the pressure in the line at nearly 600 psi. Hence, from the pump side of the break a continuous flow of approximately 70 lb./sec. can be expected.

The total long term escape flow from the break can thus be taken as approximately 100 lbm/sec.

Since about 1/3 of this would vaporize, the vapor flow would correspond to a uniform source of propane vapor of 36 lbm/sec. or 324 scfs which is available for atmospheric dispersion.

This vapor flowrate is of the same order of magnitude as that observed by Burgess & Zabetakis (Reference 2.2.3-2) in their investigation of a propane line break in Port Hudson, Missouri.

A check on the validity of the model can be obtained by comparison of predicted and observed total quantities of propane that have escaped from actual breaks. For instance, for the Port Hudson, Missouri, break, the total release of liquid propane in barrels, estimated by Burgess & Zabetakis, during the first 24 minutes, was 750 barrels.

For 100 lbm/sec. and the specific volume of saturated liquid propane (0.031 ft.^{3}/lbm), the release during the first 24 minutes is estimated to be 795 barrels. This agreement confirms that the contribution from the subcooled or two-phase critical blowdown portion is negligible.

The same methodology applied to the events at Ruff Creek (Reference 2.2.3-3) and Austin (Reference 2.2.3-4) produced the following comparison.

At Ruff Creek, a 0.0174 ft.^{2} break is computed to have a mass flowrate of 20.5 lb./sec. during the first phase of the accident, lasting approximately 1.5 hours, and then a 5.5 lb./sec. release rate for the remaining 14.5 hours.

The predicted release quantity of propane is 1691 barrels, while approximately 1800 barrels were observed to have been released. In Austin, a 0.163 $ft.^2$ break is computed to have released an average of 117 lb./sec. during the 1.17 hr. required to isolate the broken section. The contents of the isolated section is discharged thereafter. In this case, 6307 barrels are predicted to have been discharged versus the observed 6640.

Two situations can be imagined for the ensuing clouds of propane-air mixtures that form after the break. The first one envisions a propane cloud transported by atmospheric dispersion toward the plant site. The second one assumes that the propane would form a very tenacious layer close to the ground, wherein air entrainment would occur only at the air surface of the layer. This fog-like ground-hugging cloud would advance toward the plant under its own gravity (cloud slumping), because of sloping ground.

c) Calculation of Detonable Cloud Size, Assuming Atmospheric Dispersion

The dimensions of the detonable plume downwind of a 324 $ft.^3$/sec. propane source, for Category F stability and a constant invariant wind speed of 3.3 ft./sec. are given in Table 2.2.3-1.

The centerline (directly downwind) concentration, χ_{Cl}, of propane is determined by

$$\chi_{cl} = \frac{\dot{Q}}{\pi \sigma_y \sigma_z u}$$

where $\dot{Q}$ = 324 $ft.^3$(STP) of propane per second

u = 3.3 ft./sec. (1.0 m/sec.) wind velocity

and σ_y, σ_z are the plume dispersion standard deviations obtained from Reference 2.2.3-5.

The off-centerline concentrations, χ are determined by

$$\chi = \chi_{cl} \exp \{ -1/2 [(y/\sigma_y)^2 + (z/\sigma_z)^2]\}$$

The proper dispersion standard deviations are the continuous release standard deviations.

The potential cloud configuration is plotted on Figure 2.2.3-2. The dashed portion represents the fraction of the cloud which falls within the detonable limits. These limits are reported in Reference 2.2.3-6 to be 2.8 percent and 7.0 percent, respectively, of propane in the propane-air mixture.

The volume of the detonable cloud is calculated to be that of the difference between the ellipsoid enclosing all gas above the 2.8 percent mixture and the ellipsoid engulfing the gas above the 7.0 percent mixture level.

Volume of ellipsoid (2.8 percent) = 1.19×10^6 $ft.^3$

Volume of ellipsoid (7 percent) = 2.94×10^5 $ft.^3$

Volume of detonable cloud = 9.0×10^5 $ft.^3$

During the 1/2 hour period required to close the valves, the major portion of the propane vapor (90 percent) is dispersed past the lower detonable limit.

d) Calculation of Effects of Detonation

From an enthalpy of detonation release of 260 K cal./lb. of propane-air mixture of 4.9 percent (enthalpy of detonation is insensitive to mixture ratios between 4 and 5 percent), and a volume of mixture of 900,000 ft.^{3}, it is possible to compute the total energy released in a hypothetical detonation of the entire detonable cloud, assuming that the whole cloud is a mixture averaging 4.9 percent of propane by volume.

The total volume of propane in the detonable cloud is 4.42 x 10^4 ft.^{3} and the volume of air is 8.6 x 10^5 ft.^{3}. The total weight of the mixture is approximately

$$\frac{9.0 \times 10^5 ft.^3}{[(13.3)(.951)+(.049)(9.43)] ft.^3/lb.} = 68{,}600\ lb.$$

Total enthalpy released = 1.78 x 10^7 K cal.

Equivalent TNT = $\frac{1.78 \times 10^7 K\ cal.}{500\ K\ cal./lb.\ TNT} = 17.8\ tons$

Comparison of this value with the occurrence at Port Hudson, Missouri, (Reference 2.2.3-2) shows that this estimate might really be the minimum hazard expected from the break, and that ground and atmospheric conditions may result in more propane being trapped in a detonable cloud.

The absolute upper limit of the size of the detonable cloud would be a cloud where all the liquid propane which escapes from the break eventually vaporizes and mixes with air in a detonable mixture.

Assuming that the heat to vaporize the liquid propane comes from the air, then it would require approximately 4.15 lb. of air at 72°F to vaporize 1 lb. of propane at -44°F.

The resulting cold mixture would be 19.4 percent by weight, or 11.4 percent by volume, of propane, and it would be denser than the ambient air. Thus, it is possible to visualize a ground layer of propane-air mixture above the detonable limit, which would contribute appreciably more propane to the detonable cloud as its temperature increases than that calculated by the dispersion technique. This ground layer could eventually become dispersed in the atmosphere as its temperature increases, or it could move by gravity. For lower atmospheric temperatures, proportionately more air would be required to vaporize the propane. At near freezing temperature (≅ 32°F), vaporization would result in a mixture of 7 percent by volume of propane, which is the upper detonable limit.

A portion of the liquid ejected from the break would be expected to be vaporized while in the form of droplets in the jet escaping from the break; the remainder of the propane would eventually evaporate as the soil and air provide the necessary heat. The rate at which the vapor is injected in the air is determined by:

1) The amount vaporized in the jet and in the immediate vicinity. This fraction would form a heavier than air mixture which would at the same time disperse in the air and also tend to settle into a "fog-like" ground layer. Since the instantly vaporized propane (amount flashing at break) would exhibit similar behavior, the fraction of propane liquid vaporized in the jet and in the immediate vicinity can be treated, at equilibrium, as an additive to the constant vapor flow assumed for the initial dispersion calculation.

2) The amount of propane which remains liquid and experiences delayed evaporation at the ground surface.

The rate of evaporation of the liquid in contact with the ground depends on the area and depth of the liquid. Since this in turn is determined by terrain, assessment of this rate is difficult. It is reasonable to assume that an equilibrium condition can be established whereby a like amount of liquid evaporates as that which exits from the break. Less evaporation would result in an ever increasing area (or depth) of the liquid. An estimate of atmospheric clouds that could result from delayed evaporation is presented in Section 2.2.3.2e).

Conservatively, however, it is possible to assume that all of the escaping propane would be atmospherically dispersed. Under this assumption, the resulting maximum detonable cloud would be that established by a constant escape of approximately 1000 $ft.^3$/sec. of propane.

In the previous calculation, the heavier than air density of the propane mixture had not been considered, but the limit in vertical dispersion was implied in the choice of $\sigma_z \cong 0.5\ \sigma_y$ (characteristic of F stability conditions).

Reference 2.2.3-2 recommends values of $\sigma_z \cong 0.2\ \sigma_y$. Table 2.2.3-2 and Figure 2.2.3-3 show the dimensions of the propane plume downwind of the assumed 1000 $ft.^3$/sec. source, with $\sigma_z \cong 0.2\ \sigma_y$.

This detonation would then be equivalent to the detonation of 4.06×10^5 lb. = 203 tons of TNT (assuming 100 percent yield).

In actuality the yield will not be 100 percent. Reference 2.2.3-1 cites a yield of 7.5 percent. Work by Iotti, et al., (Reference 2.2.3-7) shows that the yield of a gaseous detonation is lower than that of TNT; Reference 2.2.3-7 compares overpressures calculated by assuming gaseous point sources (Reference 2.2.3-8) to overpressures obtained by Kingery (Reference 2.2.3-9) for the same yield, and those measured by Kogarko et al. (Reference 2.2.3-10), for gaseous detonations. This comparison shows that Kingery's result would have been comparable to those of References 2.2.3-8 and 2.2.3-10, if a TNT yield of 50 percent had been employed. Thus, a conservative estimate of the TNT equivalent to the detonation of the entire cloud can be obtained by using 50 percent yield and Kingery charts, see Figure 2.2.3-4, and the hypothetical detonation of the entire propane discharged by the line would result in consequences similar to the detonation of 100 tons of TNT. As explained above, this detonation cannot occur, and more realistically the detonation will be somewhere between the 8.9 tons and the 100 tons of TNT, but closer to the 8.9 tons.

Table 2.2.3-3 lists the pertinent shock wave parameters for the two yields. These parameters are obtained from Reference 2.2.3-9, assuming that the center of the detonation is in excess of

7500 ft. from the plant critical structure for the 8.9 ton detonation, and 5000 ft. from the plant critical structure for the 100 ton detonation.

Table 2.2.3-3 also lists the seismic parameters at the plant site due to air blast induced ground motions. These parameters are obtained from equations in Reference 2.2.3-7.

Critical plant structures are designed so that they are able to withstand the overpressures and ground motions listed in Table 2.2.3-3, hence it is concluded that a detonation of propane that has escaped from a break in the 8 in. LPG line will not result in unacceptable consequences.

e) Calculation of Flammable (Detonable) Cloud Size - Ground Layer Formation and Dispersion Model

The dispersion model considers the formation of a cloud by assuming that all of the propane issues from the break as a liquid, then evaporates and forms a cloud denser than air which travels by gravity until it becomes neutrally buoyant, i.e., its travel velocity is equal to or less than the prevailing wind.

For a continuous release rate of liquid propane, W, the maximum pool radius, r_{max} is determined from the liquid pool area required to vaporize the LPG at a rate equal to the release rate, hence:

$$r_{max} = \left[\frac{\rho_f\, W(h_g h_f)}{\pi B}\right]^{1/2} \quad (1)$$

where B is the heat flux determinable from the following equation (Reference 2.2.3-11), which is in metric units:

$$\left[\frac{h}{k}\right]\left[\frac{\sigma}{\rho_f}\right]^{1/2} = 30\left[\frac{B}{\rho_g \lambda w_b}\right]^{0.8} \quad (2)$$

In equation (2), σ, k, ρ_f, ρ_g and λ are the surface tension, thermal conductivity, saturated liquid density, and saturated vapor density of propane at -44°F and latent heat of vaporization; W_b is an empirical constant (equal to 280 m/hr) which is the product of the diameter of a sphere that has the same volume as a bubble of average size and the frequency of bubble formation.

The ground to liquid heat transfer coefficient, h, can be determined from the equation for natural convection of flat plates (Reference 2.2.3-11).

$$\frac{h}{k\ell} = 0.14 \left[\frac{\rho_\ell{}^2 g\, \beta_\ell\, \Delta t\, C\rho_\ell}{\mu_\ell\, k_\ell}\right]^{1/3} \quad (3)$$

In equation (3) the subscript ℓ refers to the film properties, and k_ℓ, ρ_ℓ, μ_ℓ, $C_{p\ell}$ are the thermal conductivity, liquid density, viscosity, and heat capacity of propane at $t_\ell = \frac{t_g - 44°F}{2}$ with t_g equal to the ambient temperature.

β_ℓ is the volumetric expansion coefficient given by

$$\beta_\ell = \frac{\rho_f - \rho_\ell}{\rho_\ell\,(t_\ell - t_f)} \quad (4)$$

and $\Delta t = t_g - t_\ell$

Inserting the appropriate values and converting to English units, the ambient temperature is 532 R and the heat flux B, is 33,800 BTU/ft.^{2}/hr.

This value compares well with values which can be extrapolated from the Jacob experiments (Reference 2.2.3-11) on vaporization of propane from clean surfaces, and also with values reported in LNG studies (Reference 2.2.3-12).

The initial height of the vapor cloud, which is input to the gravity spreading analysis from an initial radius, r_{max}, is computed by determining the amount of vapor injected in the air during the time it takes the prevailing wind to traverse the pool radius, if wind is present, hence

u = wind velocity $$h_{in} = \frac{Br_{max}}{\rho_g u\,(h_g - h_f)} \qquad (5)$$

In case of no wind, the initial height, h_{in}, is computed by determining the equivalent instantaneous volume of a spill which would give rise to the same r_{max} as computed in equation (1).

$$V_{inst} = \frac{\pi r^2{}_{max}\, h_{\cdot in}}{240} \qquad (6)$$

where the 240 is the volume ratio between liquid and vapor at -44°F, at which the propane issues from the break. For an instantaneous spill, several expressions for r_{max} are available. The one chosen here is from Reference 2.2.3-12.

$$r_{max_{inst}} = 1.23\left(\frac{V^3 inst}{Q^2}\right)^{1/3} \qquad (7)$$

with all quantities in metric units. The liquid regression rate, Q, is given by

$$Q = \frac{B}{\rho_f\,(h_g - h_f)} \qquad (8)$$

The gravity spread of the cloud can be computed by

$$\frac{dR(t)}{dt} = \left\{2g\left[\left(\frac{\rho_c\,(t) - \rho_a}{\rho_a}\right)\ h(t) + S\Delta R\right]\right\}^{1/2} \qquad (9)$$

wherein ρ_c (t) is the time varying cloud density (assuming a homogenized cloud at each instant of time), ρ_a is the air density at ambient temperature, g the acceleration of gravity, S the slope of the ground in ft./ft., h(t) is the time varying cloud height, and R(t) is either the radius of the cloud front at time t (for the case of flat terrain), or the distance travelled by the advancing cloud front which is confined by banks to travel along a channel of width 2W. ΔR is the change in radius of the cloud front.

As the cloud spreads, it will entrain air in an amount given by either

$$dQ(t) = \alpha V_c\, 2\,\pi\, rdr \qquad (10a)$$

when the cloud is expanding cylindrically or,

$$dQ(t) = \alpha V_c \, W \, dr \qquad (10b)$$

as the cloud spreads confined in a channel of width 2 W. In equation (10), α is the entrainment coefficient, and V_c is the local air entrainment velocity, which is defined as:

$$V_c = \frac{r}{R}\frac{dR}{dt} \qquad \text{for (10a)} \qquad (11a)$$

$$V_c = \frac{dR}{dt} - u \qquad \text{for (10b)} \qquad (11b)$$

Integrating from r = O to r = R yields the total volume of air entrained by the cloud.

$$Q(t) = \frac{2\pi\alpha}{3} R^2(t) \frac{dR(t)}{dt} \qquad \text{for radial spreading} \qquad (12a)$$

or

$$Q(t) = \left\{\frac{\pi}{3} W^2 + \gamma W \left[R(t) - W\right]\right\} \alpha_{\left[\frac{dR}{dt} - u\right]} \qquad (12b)$$

for initial radial spreading to W, followed by axial spreading; γ is a parameter which equals unity for triangularly shaped channels and two for channels with a rectangular cross section.

The wind velocity, u, is additive to the radial spreading velocity upwind of the break, but subtractive downwind of it. Thus its net effect is ignored in equation (12a).

The choice of the entrainment coefficient α is prompted by data reported for plumes (References 2.2.3-13 and 2.2.3-14) and work by Lofquist (Reference 2.2.3-15). In Lofquist's work, the important dimensionless parameters are the Reynolds number and the densimetric Froude number, which he defines as

$$F = \frac{2\frac{dR}{dt}}{\left[2gh\left(\frac{\rho - \rho_a}{\rho}\right)\right]^{1/2}} \qquad (13)$$

He finds that even for very low Reynolds numbers ($<10^5$), the entrainment is simply related to the F number and is independent of the Reynolds number. Figure 2.2.3-5 shows the values of α derived from Lofquist's work. Examination of equations (9) and (13) shows that, in this case, wherein ρ = ρc, F ≥ 2 since ρc > ρa and S is positive.

From Figure 2.2.3-5, α is taken as 0.1 or larger. Values reported by others are also in the range of 0.1 to 0.12. Hence a value of 0.1 is conservative (less dispersion for lower propane concentrations).

Mass conservation requires that

$$\frac{dM_c(t)}{dt} = \rho_a Q(t) + \frac{B}{h_g - h_f} \pi r_{max}^2 \qquad (14)$$

where M_c (t) is the mass of the cloud at time t

Energy conservation further requires that:

$$\frac{d(CM_cT)}{dt} = C_a\,\rho_a Q(t)T_a + Q_g + Q_g + Q_w + Q_p \qquad (15)$$

In equation (15), C and T are the cloud heat capacity and temperature at time t, C_a and T_a are the air heat capacity and ambient temperature in R. Q_w is the heat of condensation or freezing of water vapor, which is set equal to zero in the interest of maximizing cloud travel. Q_p, the rate of change of the heat content of the cloud due to the mass transfer from the pools, is computed from Q_p = 98 m_p; m_p is the effluent rate in lb./sec., and 98 (BTU/lb. m) is the heat content of propane vapor at -44°F (the reference point is zero heat content at 0 R).

Q_g is the heat transfer rate from the ground to the cloud. To minimize heat transfer, it can be assumed that this heat is transferred in the natural convection regime, where:

$$Q_g = h\pi\, R(t)^2\left[T_g - T(t)\right] \qquad (16)$$

with h given by equation (3).

Equations (9), (14), and (15) are solved numerically given an initial cloud radius, r_{max} [equation (1)], height, h_{in} [equation (5)], and temperature, -44°F, to yield the configuration of a homogeneous cloud as a function of time.

The progress of the cloud is either followed until the cloud concentration falls below the lower flammable limit (2.4 percent), assuming no atmospheric dispersion, or until it is stopped when the cloud velocity falls below the prevailing wind velocity, at which point atmospheric dispersion would be assumed to commence.

With this approach, only average (time and space) cloud properties can be obtained.

Figure 2.2.3-6 shows downwind and downslope distances achievable by a cloud of 2.4 percent concentration of propane for various wind velocities. Also shown is the final cloud height at that concentration. These are derived for the topography between the LPG line and the SHNPP, as shown on Figure 2.2.3-1.

For the existing topography, low lying clouds resulting from a break in a portion of the line west of the plant could travel by gravity toward the plant until intercepted by a portion of the Auxiliary Reservoir, if the break occurred between STA 450 and 500 (shown on Figure 2.2.3-1). Otherwise, gravity would propel the cloud away from the plant, i.e., in a westerly direction.

After the cloud reached the Auxiliary Reservoir, the cloud would disperse so that at the point of closest approach the fringe of the cloud would be more than 2200 ft. from safety-related structures.

The cloud itself would initially have to travel a minimum of 500 ft. to reach the reservoir, after which an average channel width in excess of 300 ft. would be available for spreading.

Breaks near STA 400 could result in clouds which also could roll toward the Auxiliary Reservoir, where again channel widths of 300 ft. or more would be available.

Between STA 400 and STA 340, clouds resulting from breaks could roll either northward, away from the plant, or westward into the Auxiliary Reservoir, or eastward into the depression leading to Thomas Creek. This relatively large depression is cut in half by U S Highway No. 1, so that the cloud would have to either roll over the highway or pass toward the lower elevation through three culverts underneath the highway. The average channel width of the lower depression leading to the Thomas Creek is in excess of 500 ft. at the Elevation 250 ft. contour, and 300 ft. at the Elevation 220 ft. contour.

Breaks between STA 340 and approximately STA 300 could also result in clouds rolling toward Thomas Creek. East of STA 300, any break could result in clouds travelling by gravity either away from the plant or toward Little White Oak Creek. Because of the much greater distances involved, and also the somewhat larger channel widths, breaks east of STA 300 present significantly less hazard to the plant than breaks west of STA 300 and are therefore not analyzed.

Figure 2.2.3-6 which involves an average ground slope of less than one percent, shows that, under extraordinary conditions in which the wind velocity closely matches the cloud velocity, the cloud can travel long distances. Although molecular diffusion, which will take place between the air and the cloud, has been ignored in the development of the previous equations, its contribution to decreasing the distances travelled by the cloud is not significant, except for long distance clouds where it becomes the predominant effect.

In most instances however, the cloud will travel a distance of six to seven thousand ft. in relatively narrow channels, and less as the channel width increases.

The final cloud height is relatively insensitive to the wind velocity (under the assumption of no atmospheric dispersion). The final height is determined by the break outflow; however, it is affected by channel width.

An analysis of the effect of ground slope on cloud slumping concluded that ground slope is not an important factor so long as it is limited to less than one or two percent. Much higher slopes would affect the results by increasing the cloud's equilibrium speed.

Except for the analyses performed for low wind velocities, it was shown that the cloud velocity soon falls below the wind speed. From that point on, clouds travelling long distances can only be sustained by assuming that atmospheric dispersion does not occur. This would of course happen only if the air flow is laminar (i.e. no disturbance), a condition which is extremely unlikely.

At the point where the cloud velocity approaches the level of the wind speed, the more realistic assumption can be made that atmospheric dispersion begins. The concentration downwind can then be obtained from the equation for a continuous line source:

$$\chi = \sqrt{\frac{2}{\pi}}\,\frac{\dot{Q}}{Lu\sigma_z}\exp\left(\frac{-z^2}{2\sigma_z}\right)\left[\operatorname{erf}\left(\frac{\frac{L}{2}-y}{\sqrt{2}\,\sigma_y}\right)+erf\left(\frac{\frac{L}{2}+y}{\sqrt{2}\,\sigma_y}\right)\right] \qquad (17)$$

in which Q is the source strength in ft.^{3}/sec., u is the wind speed in fps, and L is the width of the source (which is taken to be that of the cloud spread across the wind at the point in time at which the cloud velocity equals the wind velocity).

For points along the wind direction, y = 0, equation (17) for ground levels reduces to

$$\chi = 2\sqrt{\frac{2}{\pi}}\ \frac{\dot{Q}}{Lu\sigma_z}\ erf\left(\frac{\frac{L}{2}}{\sqrt{2}\,\sigma_y}\right)^2 \qquad (18)$$

The virtual location of the line source is chosen upwind of the point at which the cloud and wind speed become equal, by solving the following equation

$$\chi_u = 2\sqrt{\frac{2}{\pi}}\ \frac{Q}{u\,L_u\sigma_z}\ erf\left(\frac{\frac{L}{2}}{\sqrt{2}\,\sigma_y}\right) \qquad (19)$$

where χ_u is the computed concentration, in volume fraction, at the time when the wind velocity, u, equals the cloud velocity, $\frac{dR}{dt}$. In equations (17), (18), and (19), the downwind distance does not appear explicitly, but is found by iteration for σ_z and σ_y, the dispersion parameters which are functions of that distance. These are the same as previously defined.

Equation (18) was used to calculate the width of the source and its distance upwind that would produce concentrations of 2.4 percent of propane at a point on the ground. The results are shown in Figure 2.2.3-7 for both cases examined, i.e., $\sigma_y \simeq 2\ \sigma_y \simeq 5\ \sigma_z$.

The results shown in Figure 2.2.3-7 have been derived by assuming that the line source is pure propane. In reality, the propane concentration of the cloud at the point where atmospheric dispersion is assumed to commence, as computed from equation (19), is generally significantly lower, as a result of air entrainment accompanying gravity slumping. Thus Figure 2.2.3-7 is a very conservative upper limit of the distance downwind from the point at which gravity slumping ceases and atmospheric dispersion takes over for which the cloud is still flammable. Since it has been found that the virtual location of the line source, determined from equation 19, is upwind of the break for the channel widths and wind velocities of interest, the actual dimension downwind from the pipe break of the flammable cloud resulting from a combined gravity slumping and atmospheric dispersion is less than that predicted by Figure 2.2.3-7, which in fact considers atmospheric dispersion only. This result indicates that air entrainment during gravity slumping is more effective in reducing the propane concentration in the cloud than atmospheric dispersion. This is further evident in Figure 2.2.3-6. It is seen from this figure that very long clouds can only result from gravity slumping in those instances where the combination of channel width and mass flow produces a cloud advancing front velocity equal to the wind velocity, and when atmospheric dispersion is ignored. Otherwise, air entrainment rapidly drops the cloud bulk concentration below the flammable limit.

For a channel width of 300 ft. and a wind velocity of 3.3 fps, the gravity slumping model predicts a flammable cloud length of 2500 ft. for a propane flow out of the break of 100 lbm/sec. If the channel width is increased to 800 ft., the cloud length is reduced to 2000 ft. The theory predicts that for a particular wind velocity there is a constant channel width which would result in very long clouds; two things, however, prevent this from occurring. First, the topography of the SHNPP site shows a variable channel width growing from a small dimension in the pipeline

vicinity to a much larger dimension at the reservoirs. Second, atmospheric dispersion does take over as the cloud velocity approaches the wind velocity.

From examination of Figure 2.2.3-6, cloud lengths to be expected for variable channel widths and constant wind velocity at 3.3 fps should not exceed 5000 to 6000 ft., whereas from Figure 2.2.3-7, cloud lengths in excess of 7500 ft. are not expected. Hence the credible largest cloud downwind (or down slope) dimension is between 5000 and 7500 ft.

This distance is enough for the cloud to settle in either the Auxiliary Reservoir, or perhaps reach the Thomas Creek branch of the Main Reservoir depending on where the break occurs.

That these are the maximum credible distances is further confirmed by the fact that the time required for the formation of such clouds is roughly comparable to the time required to isolate the break, which is approximately 1/2 hour. To generate a much larger cloud, the flow of propane from a break would have to continue uninterrupted at the assumed rate for a much longer time.

For these distances, none of the flammable cloud would reach any part of the plant site. Hence, on the basis of this analysis, it is concluded that fires from such clouds would pose no hazard to the SHNPP.

Similarly, the detonation of such clouds has been evaluated to present no hazard; this assessment is presented hereafter.

The preceding is a simplistic analysis. The actual ground configuration is not accounted for, not even in the time varying width of the channel; and as the cloud progresses, the cloud is homogenized in space and time, when in fact this will not be the case, and concentration gradients will exist within it which will affect both cloud height and velocity, hence air entrainment.

Therefore another approach is used to confirm the reasonableness of the model just described. In this approach, the maximum flammable cloud extent is computed by equating the flow of propane out of the break to the quantity of propane transferred to the atmosphere across the cloud/atmosphere interface when the interface propane concentration reaches the lower flammable limit.

The propane transfer rate across the cloud surface is given by

$$N - x\,(N + N_a) = k_m\,A(x - x_b) \qquad (20)$$

where:

N = moles of propane crossing the cloud surface per unit time

Na = moles of air crossing the surface per unit time

A = cloud surface area

k_m = mass transfer coefficient

x_b = average propane concentration in the full atmosphere (x_b = 0 in this case)

x = propane cloud concentration (assumed to be the average mole fraction of a cloud extending to either 2.4 percent (flammable limit) or 2.8 percent (detonable limit) concentration).

The mass transfer coefficient, km, is given by:

$$\frac{k_m L\prime}{c D_{AB}} = 0.036 \left(\frac{\mu}{\rho D_{AB}}\right)^{1/3} \left(\frac{\rho u_i L\prime}{\mu}\right)^{0.8} \qquad (21)$$

where: L' =distance from break to farthest reach of cloud (for continuously emitting break)

ui =interface velocity at the surface (fps)

μ =air viscostiy

c =molar concentration of air

ρ =air density

DAB =Coefficient of molecular diffusivity between two gases.

$$D_{AB} = 2.745 \times 10^{-4} \left(\frac{T}{\sqrt{T_{CA}\, T_{CB}}}\right)^{1.823} (P_{CA}\, P_{CB})^{1/3} (T_{CA}\, T_{CB})^{5/12} \left(\frac{1}{M_A} + \frac{1}{M_B}\right)^{1/2} \qquad (22)$$

Therein M_A, M_B are the molecular weights of air and of the propane cloud at the given concentration

P_{CA}, P_{CB} are the critical pressures of air and of the propane cloud, and

T_{CA}, T_{CB} are the critical temperatures of the same.

Values of k_m are a function of the wind velocity (interfacial velocity) and the cloud dimension along the wind, L', and are given by

$$k_m = 1.997 \times 10^{-5} \frac{(u_i)^{0.8}}{(L')^{0.2}} \qquad (23)$$

The solution to equation (20) requires knowledge of the average mole fraction of propane in the cloud. This mole fraction varies from the vicinity of the break, where it is nearly unity, to the location where it reaches the lower flammable or detonable limit. For example, for a given k_m and a constant mass transfer, the concentration varies inversely with the cloud surface area. An average mole fraction of propane is computed to be about 12 to 13 percent of the mixture.

Flammable or detonable cloud areas for the average widths of the cloud, ranging from 300 ft. upward, and wind velocities equal to 3.3 fps, will be below 1.8×10^6 ft.^{2}. Typical lengths of the clouds are about 6000 ft.

On the basis of a cloud height of approximately 30 ft., chosen conservatively from the previous model, the total volume of detonable cloud would be 5.4×10^7 ft.^{3}.

The average mole fraction of propane of 12 to 13 percent corresponds to an average volume percentage of approximately 15 percent. Thus, to obtain such a cloud, approximately 8.1 x 10^6 $ft.^3$ propane would have to escape. At a rate of 1000 $ft.^3$/second, it would take 2.25 hours to release this quantity of propane. Since it is estimated that only about 30 minutes would be required to detect and isolate a break in the line, clouds of this size are not expected, and the largest credible cloud will be approximately 1.2 x 10^7 $ft.^3$. Only a fraction of this cloud will be within the detonable limits. If 50 percent is assumed, the volume that could detonate would be equivalent to the detonation of 118 tons of TNT. The closest approach of a cloud lying on the Auxiliary Reservoir or Thomas Creek to the critical plant structure is approximately 2000 ft. The effective center of a detonation cannot be predicted, but it could range from 2200 ft. to larger distances. Even when the center of the detonation is chosen at essentially the closest point to the Seismic Category I structures, the resulting over pressure is approximately the one psi. The plant can withstand the blast parameter values listed in Table 2.2.3-3 for the detonation. Thus, it is concluded that the detonation of flammable clouds from a LPG pipeline break present no hazards to the SHNPP.

The following paragraphs discuss more fully the conclusion that no hazards are posed to the plant by either fires or detonations by a LPG pipeline break.

The flammable lower limit (and detonable limit) of the cloud are reached within approximately 7000 ft. of the break when one end of the cloud is continually fed by escaping propane. This is predicted by the atmospheric dispersion model, the gravity slumping model, the combined gravity slumping and atmospheric dispersion model, and lastly by the mass transfer balance model. The mass transfer balance model, however, gives no information regarding cloud height.

The atmospheric dispersion, as well as the gravity slumping, models predict cloud heights of approximately 30 ft. for the channel widths of approximately 300 ft. that exist in the vicinity of the postulated break.

The average concentration of propane in a cloud that continues slumping toward the plant should fall below the lower flammable limit. However, the simplicity of the models gives no absolute assurance that this will occur.

It is, therefore, conceivable that a cloud of flammable or detonable concentration could spread over the surface of either the Thomas Creek branch of the Main Reservoir or the Auxiliary Reservoir.

The width available for gravity slumping in the Thomas Creek branch of the Main Reservoir is a reasonably constant 600 ft. in the immediate vicinity of the plant at the Elevation 220 ft. contour. The width of the Auxiliary Reservoir is more variable, but can be taken as an average of 1000 to 2000 ft. at the Elevation 250 ft. contour.

Since the cloud height, derived on the basis of a continuously supplied, homogeneous propane air cloud slumping by gravity with zero ground slope, is a conservative estimate of the actual cloud height, particularly since the propane flow should be interrupted after approximately 30 minutes, an estimate of the maximum height of the cloud in the Thomas Creek branch of the Main Reservoir and the Auxiliary Reservoir can be obtained from Figure 2.2.3-6.

For a 400 to 600 ft. channel width, the maximum cloud height is about 15 ft. The Thomas Creek level is Elevation 220 ft. while the plant site is located at Elevation 260 ft. Hence, no portion of a flammable cloud lying in the Thomas Creek Branch of the Main Reservoir could encroach on the plant site, and therefore there is no fire hazard. The effect of detonation of such a cloud has already been addressed and found to be of no consequence.

The difference in elevation between the plant site (Elevation 260 ft.) and the normal level of the Auxiliary Reservoir (Elevation 252 ft.) is eight ft. The maximum cloud height predicted for a channel width of 1200 ft. is about 9.5 ft. for a continuously fed cloud. In fact, the average width of the Auxiliary Reservoir taken at a level halfway between the Elevation 252 ft. contour (normal water level) and the plant site Elevation 260 ft., is closer to 1600 ft.; therefore, the expected maximum cloud height in the Auxiliary Reservoir would be less than eight ft.; there would be no direct fire hazard to the plant from such a cloud.

f) Potential Missiles From Detonation

It is difficult to assess the potential hazard to critical plant structures which could result from missiles generated by a detonation of LPG, either at the initial crater or propelled by the blast wave. Since the LPG line crosses the plant vicinity in an open area, there is little likelihood that substantial missiles would be generated other than from the place where the detonation is postulated to occur.

There are three possible ways in which the detonation of the propane cloud could be initiated:

1) At the pipeline break, either by shocks from the high pressure jet of propane, static electricity, or frictional effects.

2) At structures which have operating electrical equipment that could provide initial ignition to the cloud, followed by shock propagation to a detonation.

3) Lightning.

The first and second cases are most likely to generate missiles.

Work by Ahlers (Reference 2.2.3-16) on observed maximum debris distance and equivalent yield, which is reproduced on Figure 2.2.3-8, shows that the maximum range of missiles from the 8.9 tons detonation could be in excess of 7,500 ft., but that most missiles would not reach critical plant structures. Detonations of larger amounts of propane, however, could result in many missiles reaching the plant. For instance, for a 100 ton detonation, the maximum range could be in excess of 10,000 ft.

Missiles which travel the longer distances, however, are expected to be smaller, since air drag will affect larger missiles proportionally more. Studies on several detonations (References 2.2.3-17 and 2.2.3-18) have shown that the mass density of missiles follows an exponential law.

$$\delta = (K)\,(r^{-3.5})$$

where δ is the areal density, and r the distance from the detonation center.

To ascertain the probability of a missile from even the maximum credible detonation of the propane (100 ton) hitting critical plant structures, it is assumed that the total weight of the missiles is proportional to the volume of the crater which would be created by the detonation, if it had been a TNT detonation near the surface. The crater, in turn, is proportional to the detonation yield. The volume of the crater can be estimated by the scaling law:

Diameter (depth) of crater = (Diameter (depth) of a crater for a 1 kt explosion) x $W^{1/3}$

and the knowledge that a one kt surface detonation in dry soil results in diameters of 180 ft. and depths of 35 ft.

Hence the total mass of missiles at an assumed missile density of 95 lb./ft.^{3} will be equal to 5.63 x 10^6 lb.

Assuming that none of the mass falls back within the crater, then the total mass is given by

$$M = \int_{41.7}^{\infty} 2\pi\, r\, \delta\, dr = \int_{41.7}^{\infty} 2\pi K r^{-2.5}\, dr = .0155\, K$$

$$K = 3.63 \times 10^8 \text{ lb.-ft.}^{3/2}$$

Hence the areal density 5000 ft. away from the center of the hypothetical (considered improbable) 0.1 kt detonation is

$$\delta = \frac{3.63 \times 10^8\ lb{-}ft.^{3/2}}{8.85 \times 10^{12}\ ft.^{7/2}} = 4.10 \times 10^{-5}\ lb./ft.^2$$

Since the area of critical structures is approximately 10^5 ft.^{2}, the total weight of missiles hitting this area for the hypothetical maximum detonation would be only 4.1 pounds.

Assuming that all of this mass is concentrated in one missile, and that the missile travels at the maximum air particle velocity, u_p, given by

$$u_p = \frac{5PC_o}{7P_o\,(1+6_p/7P_o)} = 26.7\ fps$$

where p is the peak overpressure at the critical structure (≃ .5psi), P_o is the ambient pressure (14.7 psi), and C_o is the ambient sonic speed (taken as 1130 ft./sec.), then the impact energy of this missile would be 45 ft.-lb., which is considerably below the energy required for penetration of the structures.

Therefore, it is concluded that missiles from a propane cloud detonation would present no hazards to the SHNPP, and that no plant damage due to a LPG detonation at the plant site could occur.

g) Summary and Conclusion

To summarize the preceding sections, the possibility of transport of a propane cloud toward the SHNPP site has been investigated by an atmospheric dispersion model, a gravity slumping model, and a combination atmospheric dispersion-gravity slumping model.

Although the analyses demonstrate that it is possible for a flammable cloud to form in the vicinity of the plant site, fires or a detonation of the cloud will pose no hazard to the plant.

2.2.3.3 Design Basis Toxic Chemicals

A summary analysis of off-site and on-site toxic chemical hazards that may impact control room habitability is contained in Calculation 9-CRH. There are no significant off-site stationary sources of toxic chemicals within a five-mile radius of SHNPP.

2.2.3.3.1 Releases of Toxic Materials Due to A Railroad Accident Other Than Chlorine

The accidental release of toxic chemicals resulting from a railroad accident in the vicinity of the site has been calculated to be an event with a probability of occurrence of less than 10^{-7} per year, thus, the release of a hazardous material due to a railroad accident is not a design basis event.

Three railroad segments come within five miles of the SHNPP site, which carry scheduled railroad traffic. The three segments are:

a) The Bonsal-Durham segment which is 2.5 miles northwest of the plant.

b) The Fuquay-Varina-Brickhaven segment which is 4.3 miles south of the plant.

c) The Raleigh-Moncure segment which is 1.9 miles northwest of the plant.

Only the Raleigh-Moncure segment of railroad traffic within the vicinity of the SHNPP site carries hazardous materials on a regular basis. The data used in the analysis was supplied by the Seaboard System Railroad. There was no exact break-down as to type of material transported, thus an assumption was made that of the top 25 hazardous materials shipped by rail, the distribution of shipment is even. Of these 25 materials, two are considered in the evaluation (see Table 2.2.3-4 for justifications). However, the probability of occurrence of an accidental release that could result in a release of these two materials (anhydrous ammonia and vinyl chloride), as hereafter calculated, is less than the probability for design basis events; thus, a dose analysis is not required.

From the NUREG/CR 2650, SAN-82-0774 document, the statistics on number of events involving hazardous materials per mile of rail travel per year is given to be 2.2E-08. This figure is a national average, but is believed to be acceptable since no known railcar accidents have ever occurred on this segment of the railroad.

The probability of such an event is given by the following equation:

$$AP_{il} = P \times N \times M_i \times \sum_{j=1}^{N} D_j \times F_{ji}$$

where:

AP_{il} = Annual probability of design basis event under atmospheric class "l" involving the chemical of concern.

P = The national average probability of a tank car containing the chemical of concern will have an accident from a mobile source per unit length of travel.

N = Annual number of trips involving the chemical of concern.

M_i = Annual probability of an atmospheric stability class.

D_j = The length of railroad segment, in the direction of concern.

F_{ji} = Wind frequency from the sector "j" to outside air intake of the control room stability class "i".

n = Number of wind direction sectors affecting the plant.

The following have been determined to be the length of railroad track within 5 miles of the SHNPP site on the Raleigh-Moncure track segment:

West track segment	2.0 miles
West northwest segment	1.0 miles
Northwest segment	0.8 miles
North northwest segment	0.7 miles
North segment	1.2 miles
North northeast segment	2.5 miles

If a train accident occurred on these segments of railroad track and the wind was blowing from that direction, an airborne cloud of hazardous materials might be transported to the SHNPP plant control room intake location.

The meteorological data used in the analysis is from the SHNPP FSAR, Table 2.3.3-13, pages 2.3.3-73 and 2.3.3-74. This data is from the lowest wind measuring level on the SHNPP meteorological tower and represents a three year period of record which is representative of onsite meteorological conditions. The Pasquill stability class types "F" and "G" were the only atmospheric stability conditions used in the analysis, since under these conditions, the plume of released material would remain sufficiently concentrated so that by plume travel time over the distance from the railroad to the intake structure at the SHNPP plant, a sufficient concentration would remain so as to pose a potential concern. The frequency of calm wind conditions has been subtracted from the total wind direction frequency since under calm winds, the plume would either meander so dramatically that no concentration would be received onsite or the plume would "puddle" around the location of the release to spread in a uniform manner in all directions.

The probability of an event under Pasquill stability type "F" and the probability of an event under Pasquill stability type "G" for the entire hazardous travel distance of 8.2 miles is the sum of the values calculated for each sector, resulting in a combined annual probability less than 10^{-7} per

year; thus the release of a hazardous material due to a railroad accident is not a design basis event.

The above railroad accident analysis was reviewed and updated in 2006 (and documented in Calculation 9-CRH). The results confirmed the overall conclusion that a railroad accident is not a credible design basis event for HNP.

2.2.3.3.2 Accidental Releases of Chlorine

The analyses for accidental off-site releases of chlorine are contained in Calculations CPL-XI-4 and 9-CRH. Two design basis accidents were selected: the complete loss of loading of a 20-ton tank truck on US 1 and of a 90-ton tank car on the Seaboard Railroad, both at points of nearest approach to SHNPP. The analysis found that the probability of an accident involving transient sources of chlorine was less than is required for consideration. Therefore, an accident involving the transportation of chlorine in the vicinity of Shearon Harris Plant affecting the Control Room personnel is not considered to be a credible event and need not be considered in the safety evaluation of the plant, provided that Control Room personnel have access to breathing apparatus and are trained to recognize chlorine by its odor.

Positive pressure, full-face, self-contained, breathing apparatus are stored in the Control Room.

2.2.3.4 Fires

The only fire hazard in the vicinity of the SHNPP is the potential delayed ignition of flammable vapor clouds associated with a propane line break. This information is discussed in Section 2.2.3.2.

2.2.3.5 Collision with the Intake Structure

The SHNPP site is not located on a navigable waterway; therefore this section is not applicable.

2.2.3.6 Liquid Spills

There are no storage facilities for oil or liquids which may be corrosive, cryogenic, or coagulant, located where failure of the facility would allow these materials to be drawn into the intake structures and affect the plant's safe operation.

2.2.3.7 Aircraft Operations Evaluation

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2.3 METEOROLOGY

2.3.1 REGIONAL CLIMATOLOGY

2.3.1.1 General Climate

The SHNPP site lies in the transition zone delineating the Coastal Plain Region and the Piedmont Region of North Carolina. The climatology of North Carolina largely depends on elevation above sea level and distance from the Atlantic Ocean. At an elevation of about 260 ft. above mean sea level and a distance of about 115 mi. from the nearest Atlantic coastline, the site area has a temperate climatic regime. Stations representing the regional climatology, their locations with respect to the site area, and their elevations above mean sea level are presented in Table 2.3.1-1. Information and data provided is based upon information available prior to the issuance of the Harris Plant's operating license.

The summer months of June, July, and August are characterized by a southwesterly air flow resulting from the extension of the Azores-Bermuda high pressure system. This Gulf of Mexico and occasionally Atlantic moisture laden air produces the bulk of precipitation for these months in the form of afternoon and evening thundershowers. During this three-month period, an average of 39 days reach 90°F or hotter as reported by the Raleigh-Durham Weather Service, the nearest first-order reporting station to the site area. July is the hottest month at all stations within the site area. These months can be quite oppressive with dewpoints averaging between 66 and 67°F (Reference 2.3.1-1).

The autumn months of September, October, and November show a gradual decrease of average temperature of about 10°F per month. As the sun moves south, days become shorter and correspondingly, nights become longer. The combination of residual summer moisture and increased radiational cooling due to longer nights makes this the season of highest fog frequency. Although precipitation is distributed rather uniformly on an annual basis, the autumn months tend to be the driest. Though moisture is usually available, daytime heating is not sufficiently intense to produce convective activity, and the general north-south temperature gradient does not substantially materialize to generate strong frontal precipitation. Winds tend to be from the northeast during the autumn, reflecting a change in the pressure distribution. The summer wind flow configuration of a high pressure system offshore and a low pressure system over the continent is replaced by the northerly wind flow configuration of a continental high pressure system with a low pressure system offshore. The land-sea temperature contrast favors higher pressure over the ocean in spring and summer and higher pressure over the continent in autumn and winter, thus providing the seasonal reversal of wind. The higher autumnal northeastern frequency, when compared to the winter frequency, is the result of the slower moving continental high pressure systems of autumn that tend to prolong the associated northeasterly wind flow.

The winter months of December, January, and February show a shift of the wind direction frequency into the southwesterly and northwesterly quadrants responding to a westerly component added to the predominant southwest northeast bimodal distribution. January is the coldest month, averaging 18 days with a minimum temperature below 32°F at the Raleigh-Durham Weather Service (Reference 2.3.1-1). However, cold air outbreaks are either blocked

or significantly modified by the Appalachian Mountain chain located some 150 miles to the west and northwest of the site. Most sustained winter precipitation is the result of two storm tracks. One track originates in the warm waters of the western Gulf of Mexico, crosses Florida, then skirts the Atlantic Coast as it moves northward. The second track is called the "Cape Hatteras Low," so named because the temperature contrast of the offshore Gulf Stream and the shape of the coastline just south of Cape Hatteras, North Carolina, provide excellent breeding conditions for cyclonic circulations. These two storm tracks are responsible for virtually all of the snowfall in the site area; January has the greatest average snowfall totals.

The spring months of March, April, and May are characterized by consistently rising temperatures on the order of 9°F per month. Precipitation occurs in a mixed mode of frontal and convective forms. This transitional season generally has more winter than summer characteristics. The mean date of the last 32°F temperature for the area is around the first week in April (Reference 2.3.1-1). Maximum wind speeds are generally observed in this season due to the peak of the general north-south temperature gradient.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Bases

2.3.1.2.1 Tornadoes

The SHNPP site lies within Region I for determining the Design Basis Tornado (Reference 2.3.1-2). The Region I associated Design Basis Tornado parameters are as follows:

Maximum wind speed	360 mph
Rotational wind speed	290 mph
Translational speed	70 mph maximum; 5 mph minimum
Radius of maximum rotational speed	150 ft.
Pressure drop	3.0 psi
Rate of pressure drop	2.0 psi/sec.

Calculation of the tornado strike probability is accomplished by the following equation:

$$P_{S=\bar{n}\,(a/A)} \tag{1}$$

where:

P_s = Probability that a tornado will strike a particular location during a one-year interval

$\bar{n}$ = Average number of tornadoes per year, equal to 1.46 for the SHNPP site area (Reference 2.3.1-3).

a = Average individual tornado area, equal to 2.82 sq. mi. for the SHNPP site area (Reference 2.3.1-3).

A = Total area of concern (e.g., 1 square with 35° 30' mid/latitude) equal to 3891.15 sq. mi.

Using these parameters, P_s is equal to .00106 or stated conversely, a return period of 944 years. Consequently, one would expect a tornado strike every 944 years. Since no large body of water capable of sustaining waterspouts is located near the site, these need not be considered.

2.3.1.2.2 Thunderstorms

Charlotte, Greensboro, and Raleigh-Durham have a mean total of 42, 47, and 46 thunderstorm days per year, respectively (References 2.3.1-4 through 2.3.1-6). The distribution by month is presented in Table 2.3.1-2. July has the most thunderstorms; these occur mainly in the afternoon and evening, and most frequently they are the scattered, air-mass type. This provides variable precipitation patterns; on the average, there is one thunderstorm every third day in July. Thunderstorms which occur in the autumn through spring period are usually the result of frontal activity, rather than the result of the convective heating process that prevails during the summer months.

2.3.1.2.3 Lightning

Three kinds of lightning occur in thunderstorms: cloud-to-cloud, in-cloud, and cloud-to-ground. Although cloud-to-cloud and in-cloud strokes outnumber cloud-to-ground strokes by about two to one (Reference 2.3.1-7), cloud-to-ground strokes present the only hazard to nuclear power plant safety. Table 2.3.1-3 presents the annual and seasonal frequencies of cloud-to-ground flashes for Charlotte, Greensboro, and Raleigh-Durham (latitude variations excluded). These frequencies are calculated with a technique outlined by Marshall (Reference 2.3.1-8) by using the following equation:

$$NE = (0.1 + 0.35 \sin d)(0.40 \pm 0.20) \quad (2)$$

where:

NE = Number of flashes to earth per thunderstorm day per sq km.

d = Geographical latitude of the SHNPP, equal to 35° 35'

Using the conservative estimate of 0.40 + 0.20 (equaling to 0.60) in the above equation, the site area NE equals to 0.182 flashes per thunderstorm day per sq. km.

2.3.1.2.4 Hail

Hail is an indication of strong vertical velocities that occur in severe thunderstorms. Storms reaching hail severity stage are mostly associated with strong frontal zones during late spring to late summer; they are infrequent in the plant site area.

The Raleigh-Durham Weather Service observed hail a maximum of three times in any one year during the period 1951-1978, and has never experienced hail more frequently than once in any one month. Hail occurred most frequently in May and only during the months of March through August, except for one instance in November and one instance in February (Reference 2.3.1-6).

2.3.1.2.5 Ice Storms (freezing rain)

The U. S. Weather Service defines glaze as ". . . homogeneous, transparent ice layers which are built upon horizontal, as well as on vertical surfaces either from supercooled rain or drizzle, or from rain or drizzle when the surfaces are at a temperature of 32°F or lower". Although glaze may occur at air temperatures far below 32°F, the majority of ice storms occur with air temperature between 25°F and 32°F. Ice storms at the SHNPP site are caused primarily by polar front waves that have their origin in the Gulf of Mexico. Below-freezing temperatures seldom last in this area more than a few hours after ice storms. Consequently, the mean duration of ice on utility wires during the period 1928-1937 was 21 hours. The greatest radial thickness on utility wires observed during the nine winters of the period between 1928 and 1937 was .74 in. During the 10-year period of 1939-48, a total of 40 freezing precipitation days were observed at the Raleigh-Durham Weather Service; 17 of these days occurred in January. Therefore, an annual average of four freezing precipitation days per year was experienced (Reference 2.3.1-9).

2.3.1.2.6 Hurricanes

Sustained hurricane force winds (>74 mph) have never been recorded by the Raleigh-Durham Weather Service, although they have been observed in coastal areas of the State. Hurricanes deteriorate rapidly as they move onshore because of increased frictional drag and a loss of the energy source (water through release of latent heat of evaporation and condensation). Once onshore, the increased frictional effects have a tendency to turn the winds inward towards the hurricane's center; this yields greater vertical velocities which are capable of producing intense rainfall. Since the SHNPP site lies approximately 115 miles from the nearest coastline, the major effect on the Raleigh area due to hurricanes is heavy precipitation. A list of hurricanes that have affected the Raleigh area are listed in Table 2.3.1-4. The maximum 24-hour precipitation of 5.20 in. at the Raleigh-Durham Weather Service was the result of Hurricane Diane in August 1955. The fastest one-minute wind observed at the Raleigh-Durham Weather Service was associated with Hurricane Hazel in October 1954 (Reference 2.3.1-6). As would be expected, the intensities of wind and precipitation produced by hurricanes at the plant site are generally no greater than those produced by severe thunderstorms in the area.

During the period 1901 to 1955, destruction due to tropical storms occurred about ten times in the plant site area. Consequently, one would expect a return period of a destructive tropical storm of 5.5 years (Reference 2.3.1-10).

2.3.1.2.7 Extreme Winds

Using a Fisher Tippett Type II extreme value distribution, Thom (Reference 2.3.1-11) has calculated and plotted the annual extreme-mile 30 ft. level, 100-year mean recurrence interval winds for the United States. From this publication, the SHNPP site extreme-mile 100-year recurrence period wind speed is 90 mph. The vertical distribution of velocity is presented in Figure 2.3.1-1, computed from the l/7 power law and the 30 ft. level, 90 mph value. Other return periods are shown by Figure 2.3.1-2.

The extreme-mile wind is defined as the one-mile passage of wind with the highest speed. This includes all meteorological phenomena except tornadoes, which are dealt with separately. The extreme-mile wind does not reflect gustiness occurring during a short time interval. As an adjustment, Huss (Reference 2.3.1-12) suggests that a gust factor of 1.3 be applied to the 30 ft.

level extreme-mile wind. Therefore, an instantaneous gust of 117 mph would be expected to occur at the 30 ft. level once within a 100-year period.

The highest observed wind speed recorded at the Raleigh-Durham Weather Service is a 79 mph wind in September 1996, at Greensboro a 63 mph wind from the north in July 1932, and at Charlotte a 59 mph wind from the southwest in July 1962 (References 2.3.1-6, 2.3.1-5, and 2.3.1-4). The 90 mph extreme-mile Thom value is greater than the highest observed wind speed at Raleigh-Durham and therefore, can be used as a conservative value in additional analysis.

2.3.1.2.8 Precipitation extremes

Table 2.3.1-5 lists precipitation extremes along with other parameter extremes for the Charlotte, Greensboro, Pinehurst, Asheboro, Moncure and Raleigh-Durham stations (References 2.3.1-4, 2.3.1-5, 2.3.1-6, and 2.3.1-13). The maximum monthly precipitation total observed in the plant site area was 13.88 in. at Pinehurst in July, 1959. The maximum 24-hour precipitation recorded in the site area was 8.96 in. at Asheboro in August, 1966. Conversely, the minimum monthly precipitation recorded in the site area was a trace at Charlotte in October, 1953. The maximum monthly snowfall for the region was a 22.9 in. total which fell in Greensboro during January, 1966. The maximum 24-hour snowfall total for the site area was 14.3 in. which fell in Greensboro in December, 1930.

The site area on ground snow load, 100-year mean recurrence interval is 15 lbs. per sq. ft. (Reference 2.3.1-14). The minimum design roof snow load is determined by multiplying the on-ground snow load of 15 lbs. per sq. ft. by the basic snow load coefficient of 0.8. This gives a value of 12 lbs. per sq. ft. as the 100-year recurrence interval roof snow load. Additionally, the 48-hour probable maximum winter precipitation occurs in December and is approximately 19.2 in. (water equivalent) using a 200 sq. mi. reference area (Reference 2.3.1-15). The month of December averages only 1 in. of snowfall, therefore, it is very improbable that the probable maximum winter precipitation would fall in the form of snow. The floors of the unroofed areas of the Tank Building and the roofs of other safety-related structures where water due to probable maximum winter precipitation (PMWP) can accumulate, are structurally capable of withstanding the dead-load combination due to the PMWP and 12 lb./ft.^{2} (snowpack load), in combination with the dead-loads defined in Sections 3.8.1.3.1 and 3.8.4.3.1.

2.3.1.2.9 Atmospheric conditions

The extent of vertical mixing is a major factor in determining atmospheric diffusion characteristics. As a rule, mixing depths are characterized by a diurnal cycle of a nighttime minimum and a daytime maximum. The nighttime minimum is the result of surface radiational cooling which produces stable conditions, frequently coupled with low level temperature inversions or isothermal layers.

The mid-afternoon maximum is attributable to surface heating which produces instability and convective overturning through a larger portion of the atmosphere. Mean mixing depths also show a seasonal cycle of a winter season minimum and a summer season maximum. Holzworth has shown this (Reference 2.3.1-16) by listing monthly mean maximum mixing depths. Table 2.3.1-6 lists these results for Greensboro (nearest data point to the plant site). The lowest mean maximum mixing depth occurs in January (390m), and the greatest mean maximum depth occurs in June (1790m).

Low level temperature inversions also inhibit vertical mixing. Hosler (Reference 2.3.1-17) has compiled frequencies based on the percent of total hours of occurrence of an inversion or isothermal layer based below 500 ft.; the frequency of low level temperature inversions for Greensboro are presented in Table 2.3.1-7. The summer season averages inversions about 33 percent of all hours. Comparatively, the winter season averages inversions approximately 43 percent of all hours.

Cases of high air pollution potential occur during periods of stagnating anticyclones which exhibit low surface winds, no precipitation, and shallow mixing depths that result from a subsidence inversion. These conditions occur most frequently at the plant site during the fall months, particularly October. According to Korshover (Reference 2.3.1-18), about 32 cases of autumnal atmospheric stagnation that lasted four days or more occurred during the 35-year period from 1936 to 1970. A total of four cases that lasted seven days or more were recorded during the same 35-year period.

2.3.1.2.10 Ultimate heat sink

The meteorological data that was used for evaluating the performance of the ultimate heat sink (Main or Auxiliary Reservoir) are discussed in Section 2.4.11.7.

2.3.2 LOCAL METEOROLOGY

2.3.2.1 Normal and Extreme Values of Meteorological Parameters.

The local meteorology is based upon SHNPP on site data collected from January 14, 1976 through December 31, 1978, and offsite data from Charlotte, Greensboro, Raleigh-Durham, Moncure, Asheboro, and Pinehurst. Normal and mean data for the National Weather Service stations of Charlotte, Greensboro, and Raleigh-Durham are based on the 1941-1970 recording period; Moncure, Asheboro, and Pinehurst normal temperature data are based on the 1951-1973 recording period, normal precipitation data on the 1941-1970 period. Data provided is based on information available prior to issuance of the Harris Plant's operating license.

2.3.2.1.1 Wind

Wind direction and speed distributions are essential parameters for determining site characteristic diffusion climatology. Onsite joint frequency distributions of direction and speed by stability class and a summary of all winds, as outlined by Regulatory Guide 1.23 (Rev. 0) (Reference 2.3.2-1), for the period January 1976 through December 1978 are given by Tables 2.3.3-12 and 2.3.3-13. Tables 2.3.3-16 and 2.3.3-17 show joint frequencies for the lower and upper level wind direction and speed, respectively, by atmospheric stability class. Annual and seasonal wind roses for Raleigh (Reference 2.3.2 2), Greensboro (Reference 2.3.2-3), and Charlotte (Reference 2.3.2-4) are illustrated by Figures 2.3.2-1 through 2.3.2-6, respectively.

The Raleigh-Durham Weather Service (1955-1964) joint frequency distribution of wind direction and speed by Pasquill stability classes is given in Table 2.3.2-1. Pasquill stability classes were determined by the STAR method. Stability classes F and G were combined into F stability. Observations of wind speeds less than 3 knots were directionally distributed according to the frequency of occurrence of speeds from 3 to 6 knots in each direction category.

Despite differing techniques used to determine atmospheric stability (delta temperature method for on-site data and the STAR method for Raleigh data), the on-site joint wind frequencies for the SHNPP site (Tables 2.3.3-12 and 2.3.3-13) compare favorably to those compiled for Raleigh. Neutral (D) and slightly stable (E) stability classes occur more frequently at both stations. However, stable (F) and extremely stable (G) stability classes are more frequent at the on-site meteorological station. This is due in part to some nighttime cold air drainage into the broad, shallow basin in which the site is located (see Section 2.3.2.2.2).

The characteristic northeast-southwest bimodal frequency distribution is evident at all locations and is depicted by the on-site wind rose given in Figure 2.3.2-7. Average wind speeds from the area offsite stations are rather uniform, ranging from 6.9 mph at Charlotte to 7.9 mph at Raleigh-Durham.

The on-site lower level (12.5m) mean wind speed based on 1976-1978 data is 4.6 mph. This on site value is about 35 percent lower than the 7.9 mph value observed at the Raleigh-Durham Weather Service. Differing time periods, averaging methods, and instrumentation account, in part, for the lower on site wind speed value.

However, topography probably is the single most influential factor determining the lower average on-site wind speed. The SHNPP site lies in a broad, shallow 200 ft. deep basin that extends about 10 miles in directions west through south of the site (See Section 2.3.2.2.2). Hypothetically, the basin has a discoupling effect on the wind flow in the site area, particularly during nighttime hours when cold air drainage into the basin can be expected. This colder air is denser than the surrounding environment and hence difficult to displace. Therefore, turbulence and mean wind speed tend to be suppressed.

From the seasonal wind roses, the southwesterly component is more evident in the spring, summer, and winter seasons. The northeasterly fall season distribution is the result of the combination of a trend toward continental high pressure systems introducing a northerly wind flow and the relatively slow movement of these synoptic systems due to weak upper steering currents prevailing at this time of year. Winds from the southeastern quadrant are rare and for the most part precede warm frontal passages.

Wind persistence is defined here as the number of consecutive hours during which the wind direction was from the same 22.5° direction sector. Table 2.3.2-2 shows the number of occurrences of persisting wind directions by stability class as recorded at both upper and lower on-site levels of operation at the SHNPP site for January 1976 through December 1978. The maximum persistent wind for the upper level was from the south-southwest and lasted 37 hours ending at 1:00 p.m. on December 9, 1978. The same synoptic pattern produced the maximum persistent wind direction at the lower level, which lasted 30 hours. Maximum persisting winds at both levels were of the same direction and end time. Figure 2.3.2-8 is a graph of the number of persisting wind direction hours for all directions versus cumulative probability of wind persistence occurrence for the same period of record as Table 2.3.2-2. An estimate of the percentage of the total time a known number of wind persistence hours occurs can be taken directly from Figure 2.3.2-8. For example, 10-hour wind direction persistence from any one of the 16 compass directions occurred about 2 percent of the total hours.

Sustained winds greater than 50 knots have occurred only twice in the past 24 years as recorded by the Raleigh-Durham Weather Service. A one-minute average 69 mph wind from the southwest was recorded during a thunderstorm on July 21, 1962. The maximum site area

one-minute average wind of 73 mph from the west northwest was recorded during Hurricane Hazel on October 15, 1954 (Reference 2.3.2-6). A complete list of hurricanes affecting the site area, the amount of precipitation, and fastest-mile wind associated with each is given by Table 2.3.2-3. As would be expected, the intensities of wind and precipitation produced by hurricanes at the plant site are generally no greater than those produced by severe thunderstorms in the area.

2.3.2.1.2 Temperature

Monthly and annual summaries of climatological normal maximum, minimum, and average temperatures for Raleigh-Durham (Reference 2.3.2-6), Greensboro (Reference 2.3.2-7), Charlotte (Reference 2.3.2-5), Moncure (Reference 2.3.2-8), Pinehurst (Reference 2.3.2-8), and Asheboro (Reference 2.3.2-8) are given in Tables 2.3.2-4 through 2.3.2-9. Monthly and annual on-site average temperature data for January 1976 through December 1978 is presented in

Table 2.3.2-10. The mean maximum and minimum temperature data from the onsite meteorological station is shown in Table 2.3.2-11. The site area diurnal temperature range spans from about 20°F in the winter and summer seasons to around 25°F in the transitional autumn and spring months (Reference 2.3.2-9). Measured maximum and minimum temperature extremes for the offsite stations are summarized in Table 2.3.2-12. The lowest temperature recorded was -9°F in January 1985 at Raleigh Durham and the highest recorded temperature was 107°F at Moncure in July 1952 (Reference 2.3.2-5 through 2.3.2-8).

2.3.2.1.3 Water Vapor

Mean monthly and annual dew point temperatures and corresponding absolute humidity values for Raleigh-Durham, Charlotte, and Greensboro are given in Table 2.3.2-13 (Reference 2.3.2-9). Monthly and annual on-site dew point temperatures for the period January 1976 through December 1978 are given in Table 2.3.2-14. The on-site average dew point of 47.4°F compares very well to the 48°F average dew point observed at Raleigh-Durham. On site winter dew point temperatures tend to be lower and summer values a little higher. The maximum persisting 12-hour surface dew point temperature for the period of record for the site area is approximately 77°F; it occurred during a period of extended air flow trajectories from the Gulf of Mexico (Reference 2.3.2-9).

Diurnal variations of relative humidity for Charlotte, Greensboro, and Raleigh-Durham are given in Tables 2.3.2-15 through 2.3.2-17 respectively for local standard times of 1:00 a.m., 7:00 a.m., 1:00 p.m., and 7:00 p.m. (Reference 2.3.2-5 through 2.3.2-7). The 7:00 a.m. and 1:00 p.m. times correspond to the general maximum and minimum respective values of the diurnal relative humidity cycle, with 1:00 a.m. and 7:00 p.m. providing approximate midrange values. The late summer to early fall maximum of early morning (7:00 a.m.) relative humidity values also results in the same seasonal maximum of radiational fog frequency.

2.3.2.1.4 Precipitation

Precipitation is rather uniformly distributed on an annual basis in the site region. Tables 2.3.2-4 through 2.3.2-9 give climatological normal monthly and annual precipitation amounts for area recording stations (Reference 2.3.2-5 through 2.3.2-7). On-site precipitation totals are summarized in Table 2.3.2 18. Climatologically, July has a tendency to be the wettest month, October the driest, but the variance is small such that the region does not possess a "wet" and

"dry" season. Extreme precipitation amounts for area recording stations are listed in Table 2.3.2-12 (Reference 2.3.2-5 through 2.3.2-8). The extreme rainfall rates summary for the on-site facility for the January 1976 through December 1978 period is shown in Table 2.3.2-19. The on-site extreme rainfall rates for all time periods included in the table occurred on the same date, March 21, 1976, with a maximum 24-hour precipitation total of 4.41 in.

Yearly, the site area receives precipitation one day in three. Table 2.3.2-20 displays precipitation statistics for the site area stations of Raleigh-Durham, Greensboro, and Charlotte (Reference 2.3.2-10). These statistics are presented for the months of January, April, June, and October which are considered representative of the four seasons. Table 2.3.2-20 indicates that, on the average, precipitation intensities of July are about double those of January for the site region. Table 2.3.2-20 further characterizes the more convective nature of higher intensity, shorter duration July precipitation versus lower intensity, longer duration January precipitation. Generally, winter precipitation duration is about twice as long as that of July. However, daily rain totals are generally smaller in winter due to the low precipitation rates that offset the longer winter durations. The transitional April and October months seem to fit the winter regime better, partly due to precipitation dependence on slower moving rain systems in the transitional seasons; however, in mid-winter the systems are larger in area and of more uniform intensities than in the transitional period. On-site data showing the number of hours with measurable precipitation by month and year, including the overall average for the January 1976 through December 1978 period, is depicted in Table 2.3.2-21.

Seasonal and annual precipitation wind roses from the Raleigh-Durham Weather Service (Reference 2.3.2-2) are illustrated by Figures 2.3.2-9 and 2.3.2-10. On-site precipitation wind roses for the period January 1976 through December 1978 are presented on Figure 2.3.2-11. A northeast-southwest wind frequency distribution is the dominant flow regime during precipitation periods for both stations. Extreme precipitation totals for nearby representative stations are shown by Table 2.3.2-12 along with measured extreme snowfall totals (References 2.3.2-5 through 2.3.2-8).

Table 2.3.2-25 gives joint frequencies for the lower wind direction (10m) and lower wind speed by precipitation rate classes. The precipitation rate classes are divided into 0.2 inches per hour increments.

Table 2.3.2-26 gives joint frequencies for the upper level (60m) wind direction and upper wind speed by precipitation rate classes. These precipitation rate classes are also delineated into 0.2 inches per hour increments.

Wind directions from the northeast quadrant predominate with lower precipitation rates associated with the more common synoptic scale storms that produce general rainfall of lighter intensities.

Higher precipitation rates are coupled with a southwesterly wind flow associated with the less frequent mesoscale thunderstorms that produce high intensity rainfalls for short durations.

2.3.2.1.5 Fog

For the period 1950-1977, heavy fog (visibility ≤ 1/4 miles) occurred at Raleigh-Durham on an average of 36 days per year, with the fall and winter months showing the greater number of days of nearly 4 per month (Reference 2.3.2-6). The most common type of fog occurring in the

SHNPP area is ground fog as a result of nighttime radiational cooling. For this purpose, fog may be defined as a stratus cloud occurring at the surface with its top at the base of the radiationally induced temperature inversion. Ordinarily ground fog occurs more frequently in the early morning hours near sunrise when the daily minimum surface temperature is reached. It is usually shallow and disappears shortly after sunrise (Reference 2.3.2-11). Seasonally, the greater frequencies of occurrence are the fall and winter as a result of the combination of relatively moist air at the surface due to summer vegetation and the increasingly longer nights in the early fall. Also, the continental anticyclone which develops in late summer results in low surface winds, especially near sunrise, to give a consistently high frequency of fog occurrences during August and September; however, the fog persists for only about 5 hours. Strong nightly radiational cooling coupled with a predominance of anticyclonic circulation, which results in marked stability in the lower atmosphere, produces the greater fog frequencies.

2.3.2.1.6 Atmospheric Stability

Table 2.3.2-22 gives onsite frequencies of Pasquill Stability categories for the 1976-1978 period. Temporal variations of frequencies within the individual stability classes are small. Almost 50 percent of all hours fall into either neutral (D) or slightly stable (E) stability categories. Nearly 20 percent of all hours fall into the extremely stable (G) stability category. Extremely unstable (A), moderately unstable (B), and slightly unstable (C) stability categories combined occur only approximately 16 percent of the total hours.

Stability persistence data for the on-site meteorological station is given in Table 2.3.2-23. The longest stability persistence period recorded during the 1976-1978 period occurred in the neutral (D) stability category; it lasted 62 hours ending at 12 a.m. on January 2, 1978. Four periods of neutral (D) stability persisted between 49 and 72 hours during the period 1976-1978. Neutral stability primarily occurs during day-night transitions; extended periods occur under cloudy skies with moderate winds in both day and night. Over 50 percent of neutral stability occurrences persisted two hours or less; the fewer, longer persistence periods occurred during cloudy sky, moderate wind conditions. The many occurrences of short duration neutral stability persistence offset the fewer number of longer persistence periods resulting in an average duration of 4.2 hours.

Extremely stable (G) stability is prevalent under nighttime conditions of light surface winds and clear skies which allow maximum surface radiational heat loss. These restrictions tend to limit the maximum persisting hours of the extremely stable stability to shortly before sunset to shortly after sunrise, a maximum of approximately 16 or 17 hours. Consequently, five periods of very stable (G) stability lasting 16-19 hours were recorded at the on-site meteorological station from 1976-1978. Once G stability is set up, it is likely that the initial conditions will persist throughout the night resulting in the on-site average G stability persistence duration of 7.6 hours.

2.3.2.1.7 Monthly Mixing Heights

Mixing height data is presented in Section 2.3.1.2.9.

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

2.3.2.2.1 Cooling tower impact on local meteorology

Data used to evaluate the potential impact of the cooling tower on local meteorology is based upon information available prior to the issuance of the Harris Plant's operating license. The natural draft cooling tower that is used to dissipate waste heat to the atmosphere is not expected to have a significant influence on local meteorology. This is due primarily to the height of discharge (approximately 520 ft. above plant grade). After leaving the tower, the plume may rise another 1000 to 3000 ft., depending on wind speed and atmospheric temperature conditions. At these elevations, the additional water and heat added to the atmosphere will not significantly affect conditions at ground level.

At full power, between 8,000 and 12,000 gpm of water (depending on weather conditions) will be evaporated and discharged to the atmosphere by the tower. Under most meteorological conditions, the discharge will condense upon leaving the tower and will be visible (as condensed water vapor) until it is evaporated to invisibility after mixing with the drier (unsaturated) air in the atmosphere. The length of the visible plume depends on the temperature and humidity of the atmosphere. Colder and more humid weather is conducive to longer plumes. Most of the time, the visible plume will extend only a short distance from the tower and then disappear by evaporation. This was shown in a study at Keystone, where 97.3 percent of the time the plume length was less than 5000 ft. (Reference 2.3.2-12). On very humid days, when longer plumes are expected, there may be a naturally occurring overcast. On such occasions it is difficult to distinguish the cooling tower plume from the overcast.

After a quick increase of the invisible plume radius after it leaves the tower, entrainment or mixing with ambient air keeps the visible plume radius constant or decreases it. Long persistent visible plumes occur with stable air, hence, vertical mixing is very limited and plumes tend to flatten after injection into a layer due to initial plume rise and then maintain a constant horizontal dimension normal to the wind due to lateral mixing. The Keystone photographs show this to be the case (Reference 2.3.2-12).

The plume lengths and orientation with respect to the plant were determined for all hours with visibilities greater than one-half mile during the three-year period. Plume frequencies were calculated in 820-foot (250 meter) plume length intervals. Plume characteristics were categorized by season and annual average.

Figure 2.3.2-12 shows the annual cumulative frequency of plume lengths from the tower. A large proportion of the plumes (about 85 percent) will be confined to plant property; 97 percent of the plumes will have lengths less than 1.5 km, and 99.6 percent less than 2.5 km. The maximum plume length expected is 3.5 km, occurring on average once every three years; plumes 3.0 km in length can be expected about one hour per year, and 2.0 km plumes about 10 hours per year.

The nearest airport to the plant is the Raleigh-Durham Airport, located 29 km northeast of the plant. There will be no safety hazard created to air traffic.

Table 2.3.2-24 shows seasonal frequencies of 1, 2, and 3 km plumes associated with wind direction. The greatest frequency of the plumes occurs during winter and fall months. The largest plumes are expected during the winter; this is due to the fact that condensation is

enhanced and plume lengths increase with increasing ambient moisture content and decreasing temperature. The greatest frequency of plumes is associated with north to northeast and south to southwest winds, which indicates the importance of colder temperatures (winds with northerly component) and greater moisture (winds with southerly component) in producing plumes.

Figure 2.3.2-13 shows the annual frequency of plumes associated with the operation of the natural draft tower. Results are similar to those given for Table 2.3.2-24. The greatest frequency of plumes is expected north of the plant, and the longest plumes are expected southwest of the plant.

Ground fogging could occur if ground elevations in the plant vicinity were comparable to plume heights; however, the release elevation at the plant is approximately 780 ft. msl and the highest ground elevations in the vicinity are 430 ft. msl (8 km southeast of the site), and 400 ft. msl (10 km west of the site). Plumes will easily clear these areas without considering the rise of the plume above the release elevation.

Extended visible plumes will probably occur during periods of high humidity when restricted visibility occurs naturally. An average frequency of 845 hours per year of naturally occurring fog was reported during the three year period 1955-1957; visibilities were less than one-half mile for 124 hours per year during this period and less than three-eighths of a mile for 90 hours per year. The tower will therefore only slightly increase the severity of the condition.

Ice formation on structures is not expected to occur if the structure is lower than half the cooling tower height. From a 520 ft. tower and a plume rise that extends 1,000 ft. above the tower in the most stable case, the plume will not ordinarily pass across any structure having a height less than the cooling tower. The only exception to this will be at high-wind speeds. While plumes will be very short when winds are strong, occasionally the wake effect of the tower will cause the plume to curl below the lip. Flow around the cylindrical natural draft tower quickly removes the downwash, and it either ascends or evaporates. The downwash at Keystone was not observed to go more than approximately 75 ft. below the lip of the tower.

There are no large safety-related plant structures or other nearby structures which could be affected by icing from this cause. During times of naturally occurring snowfall, it is conceivable that snow conditions could be more intense under the plume and cause greater accumulation on the surrounding area and roadways. This should not create any greater hazard, since normal precautions taken by travelers in such circumstances would be adequate. Such an effect is expected to be very local if it occurs.

A theoretical study showed that collection of water by rain falling through the plume contributes less than one percent at rainfall rates greater than 1 mm/hr. Therefore, significant increases in rainfall are not expected in the area surrounding the tower.

No synergistic effects of cooling tower operation at the site location have been identified. Gaseous effluents will be released from the plant from rooftop vents approximately 120 ft. above grade. None of these vents release significant amounts of heat; therefore, there will be only a small plume rise. As stated previously, the cooling tower plume will be at a much higher elevation. If the two plumes eventually mix, it would be well downwind where any water droplets in the cooling tower plume would have evaporated, and the gas concentrations in the plant effluent plume would be well diluted.

A very small fraction of the water circulating through the cooling towers will be carried as small droplets in the rising air which leaves the tower top. This drift rate fraction (defined as kg of water per second leaving the tower top divided by the kg of water per second circulating through the tower heat exchange section) will average about 2×10^{-5} (or 0.002 percent).

2.3.2.2.2 Topographic Features

The SHNPP site lies within a shallow basin, as depicted by Figures 2.3.2-14 through 2.3.2-17 which give plots of elevation versus distance from the plant center by directional sectors. Generally, within 10 miles of the site, elevations above mean sea level gradually increase from the plant grade of 260 ft. to around 400 ft. in all but the west-southwest, southwest, north-northwest, north, and north-northeast sectors.

Topographic features within a 5-mile radius (as modified by the plant) are shown on Figure 2.3.2-18. Filling of the main reservoir south and southeast of the plant adds an additional heat and moisture source to the area. As a result, a slight increase of speed and frequency of southerly winds are expected. Additionally, the heat source will tend to destabilize the nighttime surface inversion, thereby reducing the frequency of occurrence of Pasquill Class G stability.

Topographic features within a 50-mile radius are shown on Figure 2.3.2-19. In general, the terrain slopes upward northwest of the site area, averaging about 10 ft. per mile, to an elevation of about 800 ft. at 50 miles from the plant site. The terrain through the north through west sectors is gently rolling, ranging from about 100 ft. to 500 ft. above mean sea level.

2.3.2.3 Local Meteorological Conditions for Design and Operating Bases

Local meteorological data have not been used for design and operating basis considerations other than those conditions referred to in Sections 2.3.4 and 2.3.5. However, local meteorological data are conservative with respect to the design and operating parameters, and are discussed in the sections outlined below:

Ultimate Heat Sink	9.2.5
Tornadoes	3.3.2
Winds	3.3.1
Flooding	3.4

2.3.3 ON-SITE METEOROLOGICAL PROGRAM

2.3.3.1 On-Site Operational Program

Collection of SHNPP onsite meteorological data began in March 1973. A 200-ft. guyed, open-latticed tower supports the lower and upper levels of instrumentation. Wind direction, wind speed, wind variance (sigma theta), and two ambient temperatures are monitored at both the lower and upper levels. Two channels of differential temperature between the upper and lower levels are monitored simultaneously. Solar radiation and precipitation are collected near ground level. The wind sensors are mounted on 12-foot fiberglass booms oriented perpendicular to the general NE-SW prevailing wind flow to minimize tower shadow effects. The temperature probes

and relative humidity sensor are housed in aspirated shields mounted on 8-foot fiberglass booms. Current operational sensor elevations are displayed in Table 2.3.3-3.

The meteorological tower is located approximately 1.1 miles northeast of the reactor complex. The base of the tower is at approximate plant grade level of 260 ft. above mean sea level. A topographical map showing the meteorological tower with respect to the reactor complex is given on Figure 2.1.2-1.

An environmentally controlled shelter, which houses the meteorological system datalogger and remote data access equipment, is located about 40 ft. northwest of the tower, perpendicular to the prevailing wind flow to minimize air trajectory deviations. A complete illustration of the meteorological facility layout is presented in Figure 2.3.3-1.

The datalogger acquires signals from the meteorological sensors and converts those signals into engineering units. 15-minute averaged values for each parameter are stored internally within the datalogger memory. The memory is battery-backed to prevent loss of stored data during power outages. A date/time stamp on each 15-minute average recorded data set corresponds to the end time of the data interval. The data is transmitted to the plant computer system for display and archiving. The Company's offsite meteorological consultant will be permitted access to the stored data for download.

2.3.3.2 Data Reduction

A host computer located in the office of the Company's meteorological consultant retrieves the meteorological data from the HNP datalogger system daily (except weekends and Holidays). This data is reviewed for potential immediate data problems by meteorological personnel. The datalogger data is then rigorously checked for consistency with the Raleigh-Durham National Weather Service data. Erroneous data is then discarded prior to insertion into the historical data base. The edited 15-minute averaged data is then stored on magnetic history tapes. Routine computer outputs include:

1. Monthly Data Summaries listing maximum temperature, minimum temperature, average temperature, barometric pressure, precipitation, solar radiation, and dew point temperature as a daily average and monthly average.

2. Hourly averages of precipitation, barometric pressure, ambient temperature, differential temperature, dew point temperature, upper and lower level wind direction and wind speed, upper and lower level wind direction variance (sigma theta), Pasquill stability classes (as outlined in Regulatory Guide 1.23) computed from the average of the two delta temperature systems, and accumulated solar radiation (langlies/minute).

3. The 15-minute averages of all parameters, except precipitation which is displayed as a 15-minute total value.

4. Joint wind frequency distributions (as outlined in Regulatory Guide 1.23) for both upper and lower levels showing average wind speeds and number of unrecovered data hours.

2.3.3.3 Maintenance and Calibration

System reliability is supported by performance of the following maintenance:

1. The datalogger input channels are calibrated semi-annually.

2. The wind sensors are calibrated or replaced with NIST-traceable calibrated sensors semi-annually.

3. The precipitation collector is calibrated semi-annually.

4. The barometric pressure, relative humidity and solar radiation channel sensors are calibrated or replaced with NIST-traceable calibrated sensors annually.

5. The temperature sensors are thermistors purchased with NIST-traceable calibration documentation. Since thermistors are inherently stable (100 month drift <0.01°C), routine sensor calibration or replacement is not performed. Deviation between the two ambient/differential temperature channel indications provides early warning of a problem with one of these channels.

Routine analysis of the accumulated system data, including comparison to appropriate alternate weather data sources, provides an opportunity to screen for inconsistent of erratic data.

2.3.3.4 On Site Data

Westinghouse System on-site joint wind frequency distributions (compiled per Regulatory Guide 1.23) for both upper and lower sensor elevations for the period January 14, 1976, at 4:00 p.m. EST through December 31, 1978, at 11:00 p.m. EST is presented in Tables 2.3.3-6 through 2.3.3-13. Upper and lower level annual joint wind frequency distributions for January 14, 1976, at 4:00 p.m. EST to December 31, 1976, at 11:00 p.m. are displayed in Tables 2.3.3-6 and 2.3.3-7, respectively. Data recovery percentages for this period are 96.3 percent for the lower level and 95.8 percent for the upper level.

Tables 2.3.3-8 and 2.3.3-9 depict the annual joint wind frequency distributions for the period January 1, 1977, at 12:00 a.m. EST through December 31, 1977, at 11:00 p.m. EST for the upper and lower levels, respectively. Data recovery percentages for this period are 93.6 percent for the lower level and 91.8 percent for the upper level.

Annual joint wind frequency distributions for both upper and lower levels for January 1, 1978, at 12:00 a.m. EST through December 31, 1978, at 11:00 p.m. are given by Tables 2.3.3 10 and 2.3.3 11, respectively. Data recovery percentages for this period are 97.9 percent and 98.6 percent for the upper and lower levels, respectively.

Both upper and lower level distributions for the entire three-year period are given by Tables 2.3.3-12 and 2.3.3-13, respectively. Data recovery percentages for this period are 95.2 percent for the upper level and 96.2 percent for the lower level.

All on-site joint wind frequency distributions were compiled by using the delta temperature stability classifications as outlined by Regulatory Guide 1.23.

Average on site wind speeds for the total three year period at the lower and upper levels are 4.6 mph and 8.9 mph, respectively. Representation of the data to long-term, climatological averages is discussed in Section 2.3.1 and 2.3.2.

Joint wind frequency distributions by stability class for data recorded via the data logger sensor system for the period February 1, 1979, through January 31, 1980, are given in Tables 2.3.3-14 and 2.3.3-15.

Table 2.3.3-16 provides joint frequencies for the lower level (10mm) wind direction and lower wind speed by atmospheric stability class (per Regulatory Guide 1.23). These frequencies represent three year monthly totals for the period 1976-1978.

Table 2.3.3-17 provides joint frequencies for the upper level (60m) wind direction and upper wind speed by atmospheric stability class (per Regulatory Guide 1.23). The frequencies represent the three year monthly totals for the period 1976-1978.

2.3.3.5 Regional Air Flow Trajectory Considerations.

Meteorological data and analysis of the preceding sections have included onsite and representative offsite stations both within and outside an 80 km radius of the plant. Because of the homogeneous nature of the topography and climatology of the parameters that govern atmospheric transport processes, the analysis presented in the preceding sections is also sufficient to characterize transport processes to within an 80 km radius.

All wind data used in analysis were from the first order Weather Service Stations, Raleigh-Durham, Greensboro, and Charlotte. All have exposures typical of airport locations. Table 2.3.3-18 shows wind instrument height above ground level for each of these stations.

2.3.4 SHORT-TERM (ACCIDENT) DIFFUSION ESTIMATES

2.3.4.1 Objective

On site data from SHNPP for the period of January 1976 through December 1978 (Table 2.3.3-13) has been used to evaluate the design basis accident meteorology for the SHNPP site. The design basis accidents are postulated (and evaluated in Chapter 15) to characterize highly unlikely physical events, upper limit radioactivity concentrations, and doses at on-site and off-site locations. Among the basic inputs to the accident analyses are the meteorological parameters which determine the dilution capacity of the atmosphere.

2.3.4.2 Calculations

Diffusion calculations for accidental or short-term releases of radionuclides were performed in accordance with the criteria provided in Draft NRC Regulatory Guide 1.145, "Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants," September 30, 1977. It was assumed that the releases emanate from a point source at ground level; no advantage is claimed from effluent emissions at elevated release points. The effluent plume is assumed to spread according to a Gaussian dispersion model, except during atmospheric conditions exhibiting neutral (D type) or stable thermal structure (E, F and G type) conditions as defined by Regulatory Guide 1.23 accompanied by light wind speeds less than 6

m/sec. During the expected case, atmospheric dispersion includes the consideration of the "plume meander" using a method described in Draft Regulatory Guide 1.145.

2.3.4.2.1 The diffusion model for eight hours or less. Design basis accident χ/Qs are calculated using one of the following three formulae:

$$\chi/Q = \frac{1}{\bar{u}_{10}\,\pi\,\Sigma_y\,\sigma_z} \quad (1)$$

$$\chi/Q = \frac{1}{\bar{u}_{10}(\pi\sigma_y o_z + A/2)} \quad (2)$$

or

$$\chi/Q = \frac{1}{\bar{u}_{10}(3\pi\sigma_y o_z)} \quad (3)$$

where:

χ/Q = is the relative concentration (sec/m^3) at ground level

π = is 3.14159

$\bar{u}_{10}$ = is the wind speed (m/sec) at ten meters above ground level

Σ_y = is the lateral plume spread (m), as a function of atmospheric stability, wind speed ($\bar{u}_{10}$) and downwind distance from release. For distances to 800 meters, Σ_y = Mσ_y: M being a function of atmospheric stability and wind speed (see Figure 2.3.4-1). For distances greater than 800 meters, Σ_y = (M-1)σ_y 800 m + σ_y.

σ_y = is the lateral plume spread (m), as a function of atmospheric stability and distance, (Figure 2.3.4-2).

σ_z = is the vertical plume spread (m), as a function of atmospheric stability and distance (Figure 2.3.4-3).

A = is the smallest vertical plane, cross-sectional area (m^2) of the building from which the effluent is released.

During conditions of neutral (D) and stable (E, F, and G) stability when the wind speed at the 10-meter level is less than 6 m/sec, credit for horizontal plume meander was considered by determining χ/Q using equation (1). This value was used if it was less than the higher of the χ/Q values obtained using equations (2) or (3). Otherwise the χ/Q value used was the greater value calculated from either Equation (2) or (3).

Wind velocities were grouped as indicated in Table 2.3.4-1. χ/Q values were computed for each stability class defined by Regulatory Guide 1.23 using the corresponding wind group for each of the 16 cardinal directions. The critical values are summarized in Table 2.3.4-5.

2.3.4.2.2 Selection of Downwind Distances

In order to allow for changes in the airflow trajectory, plume segmentation (particularly in the light wind, stable condition), wind speed and direction frequency variations from year to year, the following procedure was used to determine the distance at which the calculations of atmospheric dilution (χ/Q) were made.

For each of the 16 cardinal wind direction sectors, the distance used for χ/Q computations at the minimum exclusion area boundary was the minimum distance from the original SHNPP plant center to the nearest point of the exclusion area boundary within a 45-degree sector centered on the compass direction of interest.

2.3.4.2.3 Choice of Dilution Factor (χ/Q)

To choose the appropriate χ/Q value for the dose assessment analysis, cumulative probability distributions of the χ/Q values, as determined from the method described in Section 2.3.4.2.1 at a specified distance from the plant center were constructed for each of the 16 cardinal compass point directions (22-1/2 degree direction sectors). Each directional probability distribution was normalized to 100 percent. Since the joint frequency table data was used to calculate the χ/Q values, the cumulative probability distribution function was computed such as to envelop the data points.

The effective probability level (P_e) for the selection of the χ/Q value in each direction sector was defined by the following equation:

$$P_e = \frac{P\ (N/n)}{S} \qquad (4)$$

Where:

P = probability level.

N = total number of hours having wind and stability data in the meteorological data record.

n = total number of hours having wind flow in the direction of interest.

S = total number of Sectors (16).

For the realistic accident assessment χ/Q determination, P should be selected as 50 percent. Note that P_e can exceed 100 percent if n is sufficiently small. In those directions, the selection of a χ/Q value may be ignored unless the χ/Q values for that sector are very high when compared with χ/Q values of P_e in other direction sectors.

For each effective probability level (P_e = 5% or P_e = 50%), the χ/Q values for each of the cardinal directions are compared to each other and the highest value is utilized as the atmospheric dilution factor to be assumed in the accident analysis.

2.3.4.2.4 Results of dilution computations

Using the described procedure and the joint frequency data compiled from on-site meteorological data, χ/Q values were calculated for the SHNPP site. Table 2.3.4-2 presents the distances to the nearest SHNPP site boundary for the 16 cardinal directions. Table 2.3.4-3 summarizes the minimum distances from the original SHNPP plant center to the exclusion boundary and the distances used in the accident dilution assessment as prescribed by NRC Draft Regulatory Guide 1.145, September 1977. Table 2.3.4-4 presents a summary of the zero- to two-hour, 5 and 50 percentile χ/Q values at the exclusion boundary for each direction. The dilution factor which is exceeded 5 percent of the time is 6.17×10^{-4} seconds/meter^3 in the south sector at the minimum exclusion boundary. The dilution factor of 5.1×10^{-4} seconds/meter^3 was calculated for west sector at the actual site boundary.

Values of χ/Q for other points of interest are presented in Table 2.3.4-5.

The dilution factors for the outer boundary of the low population zone (LPZ) have been calculated for 0-8 hours, 8-24 hours, 1-4 days, and 4-30 days for each wind-direction sector. (Minimum distance to the outer boundary of the LPZ is three miles). The χ/Q value was determined for the appropriate time period at the distance of interest in each direction sector by using a logarithmic interpolation between the calculated value that was selected in that sector of interest for the 0-2 hour period and the annual average (8760 hour) value at the distance of interest in that direction sector. The computation of the annual average value is described in Section 2.3.5. The appropriate time period was selected from the interpolation and the highest sector χ/Q value was selected.

Table 2.3.4-5 is a summary of the worst, 5 and 50 percentile values of χ/Q for all time periods and distances of interest.

As discussed in Section 2.3.2, the on-site data sample was considered to be conservatively representative of meteorological conditions at the site. Observed differences between the three year on-site data period (Section 2.3.3.4) and ten year joint frequency distribution from the Raleigh Durham Airport (Section 2.3.2.1.1) reflect the bimodal wind direction distribution characteristic of the area. Although the percentage of calm winds annually at the site are about one-half of those recorded at the airport (probably due to differences in instrumentation) the percentage of stable atmosphere conditions are about 30 percent above those of the Raleigh-Durham Airport (probably due to differences in Raleigh-Durham "STAR" Stability Classification and the Regulatory Guide 1.23 Stability Classification System).

Topographic influences at the site are discussed in Section 2.3.2.2. The area of gently rolling hills will have only slight effect on short-term diffusion estimates. It is anticipated that after creation of the SHNPP reservoir system, a slight improvement in diffusion conditions will be observed by alteration of the area's micrometeorological regime.

2.3.5 LONG-TERM (ROUTINE) DIFFUSION ESTIMATES

2.3.5.1 Objective

The on-site meteorological record (January 1976 through December 1978) has been used to provide estimates of annual average atmospheric dilution factors (χ/Q) for locations at selected distances out to 50 miles from the plant. These χ/Qs are used to evaluate the dispersion of

radionuclides released during routine plant operations through air pathways. The on-site meteorological data recorded on magnetic tape, has been provided to the NRC separately.

2.3.5.2 Calculations

On-site annual joint frequencies of wind direction, wind velocity, and stability class were determined from hourly averages. The temperature differences were measured between the 11.03 meter and 59.85 meter temperature sensing levels. These parameters were used as input to a computerized Gaussian model, which calculates annual average χ/Q values for distances out to 50 miles from the Shearon Harris Nuclear Power Plant. The basic equation used in the diffusion model is:

$$\frac{\bar{X}(xk)}{Q} = \frac{2.032}{x}\ RF_k(x) \sum_{ij} DEPL_{ijk}\,(x) \cdot DEC_i\,(x) \cdot f_{ijk}\ \left[\bar{u}_i \left(\sigma_{zj}^2(x) + D_{\frac{z}{\pi}}^2\right)^{1/2}\right]^{-1} \qquad (5)$$

$$\frac{\bar{X}(xk)}{Q} = \frac{2.032}{x}\ RF_k(x) \sum_{ij} DEPL_{ijk}\,(x) \cdot DEC_i\,(x) \cdot f_{ijk}\ \left(\sqrt{3}\bar{u}_i \sigma_{zj}(x)\right)^{-1} \qquad (6)$$

Where:

$\frac{\bar{X}(xk)}{Q}$	=	average effluent concentration normalized by source strength at distance x and direction k;
$\bar{u}_i$	=	mid-point values of the ith wind speed class;
$\sigma_{zj}(x)$	=	vertical (z) spread of the effluent at distance x for jth stability class;
f_{ijk}	=	joint probability of the ith wind speed class, jth stability class, and kth wind direction.
x	=	downwind distance from release point or building;
$DEC_i\,(x)$	=	reduction factor due to radioactive decay at distance x for the ith wind speed class;
$DEPL_{ijk}\,(x)$	=	reduction factor due to plume depletion at distance x for the ith wind speed class, jth stability class, and kth wind direction;
$RF_k(x)$	=	correction factor for air recirculation and stagnation at distance x and kth wind direction; and
D_z	=	the building height which is used to describe the dilution due to the building wake.

Equation (5) yields the χ/Q using the maximum building wake dilution allowed by the NRC; the computer code uses the higher value of (χ/Q) calculated from Equation (5) and (6).

The computer code used to generate the annual long-term values is described in NUREG-0324, "XOQDOQ Program For the Meteorological Evaluation of Routine Effluent Releases at Nuclear Power Plants", September 1977. The recirculation factors for an inland location are specified as input, along with the exclusion boundary distances and the special points of interest.

The annual average χ/Q values for the exclusion boundary, site boundary and low population zone boundary are presented in Tables 2.3.5-1, 2.3.5-2, and 2.3.5-3 respectively. Values for annual average χ/Q no decay, undepleted; annual average χ/Q 2.260 day decay, undepleted; annual average χ/Q 8.0 day decay, depleted; and annual average D/Q relative deposition for incremental distances to 50 miles are presented in Tables 2.3.5-4, 2.3.5-5, 2.3.5-6, and 2.3.5-7, respectively.

The annual average dilution factors given in this section are likely to be quite conservative for the reasons given in Section 2.3.4.2.5.

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2.4 HYDROLOGIC ENGINEERING

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

The Shearon Harris Nuclear Power Plant is located between Tom Jack Creek and Thomas Creek. These creeks are two of the tributaries of Whiteoak Creek; Whiteoak Creek is a tributary of Buckhorn Creek. A complete description of the site area is presented in Section 2.1. The location of the site relative to the various tributaries of Buckhorn Creek and the Cape Fear River is shown on Figure 2.4.1-1.

The Seismic Category I structures which should be considered from the hydrologic standpoint include the Main Dam, the Auxiliary Dam, their respective spillways, and those safety related structures located on the plant island (see Section 3.2). The top of the Main Dam, the top of the Auxiliary Dam, and nominal plant grade are all at Elevation 260 ft., while the maximum water level at the corresponding locations resulting from the probable maximum flood coincident with the corresponding designed wind velocity are 243.1 ft., 258.0 ft., and 257.7 ft., MSL respectively (see Tables 2.4.5-1, -2).

Emergency service water and cooling tower makeup water for the power plant are supplied through the Emergency Service Water and Cooling Tower Makeup Water Intake Structure. The preferred source of emergency service water is the Auxiliary Reservoir. Water from the Auxiliary Reservoir passes through the Emergency Service Water Intake Channel and the

associated intake screening structure to the Emergency Service Water and Cooling Tower Makeup Water Intake Structure. The cooling tower makeup water source and the secondary source of emergency service water is the Main Reservoir.

Water from the Main Reservoir passes through the Cooling Tower Makeup Water Intake Channel to the Emergency Service Water and Cooling Tower Makeup Water Intake Structure. The Emergency Service Water is discharged from the Emergency Service Water Discharge Structure into the Auxiliary Reservoir. The decks of the Emergency Service Water and Cooling Tower Makeup Water Intake Structure, the Emergency Service Water Intake Screening Structure, and the Emergency Service Water Discharge Structure are all at Elevation 262 ft. MSL. This elevation is 2.0 ft. above the plant grade, 4.0 ft. above the probable maximum water in the Auxiliary Reservoir, and 18.9 ft. above the probable maximum water in the Main Reservoir. All three of these structures are Seismic Category I reinforced concrete structures and are capable of withstanding wave action during the probable maximum flood and the probable maximum hurricane.

As shown on Figure 2.4.1-2, the southern half of plant grade is above the existing ground, while the northern half extends to the cutting region of the pre-existing ground contours. Filling and cutting of the pre-existing ground has altered the existing local drainage pattern. A description of on-site water bodies and drainage patterns is presented in Section 2.4.1.2.1.5. Most of the runoff from the plant grade is drained by means of a graded ground surface with inlet structures and associated underground reinforced concrete pipe. Along the peripheral areas of the plant island, the drainage system consists of open ditches and underground reinforced concrete pipe. The reinforced concrete pipe drain to the Main Reservoir or to the sides of the Emergency Service Water Intake and Discharge Channels.

The plant drainage system is designed for a storm of five in. per hour rainfall intensity. The maximum accumulation of rainwater on the plant island during the local probable maximum precipitation storm is 14.8 in. (see Section 2.4.2.3). For protection of Seismic Category I structures and safety-related systems, see Section 3.4.1.1. Storm runoff will flow into the Main Reservoir through the underground reinforced concrete pipe and into the Auxiliary Reservoir via the Emergency Service Water Intake and Discharge Channels. Should the flow through the drainage system for the plant island area become blocked during a period of such rainfall intensity, the plant island is capable of being drained by overland flow on the open roads and ground surface directly to the Main Reservoir or the Emergency Service Water Intake and Discharge Channels. Sediment buildup in the Emergency Service Water Channels and Auxiliary Reservoir is monitored in accordance with the requirements of Reg. Guide 1.127, Rev. 1.

2.4.1.2 Hydrosphere

2.4.1.2.1 Surface and Groundwater Hydrology

The principle source of water for the SHNPP is a storage reservoir system, which consists of two reservoirs. The Main Reservoir, situated on Buckhorn Creek, is impounded by an earthen dam located just below the confluence of Whiteoak Creek and Buckhorn Creek, while the Auxiliary Reservoir, located on Tom Jack Creek, is formed by an earthen dam situated to the west of the plant island. There are two creeks adjacent to the plant site; Tom Jack Creek to the west and Thomas Creek to the east. No pre-existing ponds or impoundments were located within the boundary of the plant island.

The location of the Cape Fear River basin is shown on Figure 2.4.1-3. Details of the Cape Fear River drainage basin and its relation to Buckhorn Creek are shown on Figure 2.4.1-4.

2.4.1.2.1.1 Buckhorn Creek

Buckhorn Creek has its headwaters in the vicinity of Holly Springs and Apex, North Carolina, and flows along a southwesterly course to its confluence with the Cape Fear River about 12 miles northwest of the town of Lillington. As shown on Figure 2.4.1-1, Buckhorn Creek has five tributaries above the Main Dam: Tom Jack Creek, Thomas Creek, Little Whiteoak Creek, Whiteoak Creek, and Cary Creek. These five creeks, together with the remainder of Buckhorn Creek's basin, drain a watershed area of approximately 79.5 sq. mi. The entire drainage basin lies near the eastern edge of the Piedmont Plateau, with elevations from 450 ft. to 150 ft. MSL. There is no flow record of the Buckhorn Creek drainage basin before 1972, except for a total of 13 days of low flow records taken at a station near Holly Springs, North Carolina, during isolated periods of drought in the general area. However, the Middle Creek basin lies adjacent to the eastern border of the Buckhorn Creek basin and also has its headwaters in the vicinity of Apex. A permanent United States Geological Survey stream gaging station on

Middle Creek near Clayton, North Carolina, gages the runoff from 80.7 sq. mi. Records have been maintained at this gaging station since October 1939, and the overall average annual runoff for the period 1939 through 1978 is 15.53 in. Due to the immediate proximity and similar size of these basins, the overall average flow of Buckhorn Creek correlates reasonably well with that of Middle Creek. Therefore, flow data of Buckhorn Creek from October 1939 through May 1972 were derived by utilizing the records of flow of Middle Creek modified by multiplying the ratio of their drainage areas. In order to synthesize flow data for Buckhorn Creek prior to this period, six other streams with long term flow records and comparable drainage areas which are in the same general area were analyzed and correlated with those of Middle Creek for the overlapping period of record. These include the following:

USGS Gaging Station	Records	Drainage Areas (sq. mi.)
a) Little River near Princeton	1930-1969	229
b) Deep River at Ramseur	1922-1969	346
c) Deep River at Randleman	1928-1969	120
d) Reedy Fork near Gibbonsville	1928-1969	133
e) Flat River at Bahama	1925-1969	150
f) Eno River at Hillsboro	1927-1969	67

Coincident flow data of Middle Creek and the six streams above were programmed and correlation equations were developed for each stream. Flow data of Little River near Princeton, North Carolina, has the best correlation with that of Middle Creek; the correlation coefficient is 0.93. Flow data from 1930 through 1939 was therefore synthesized for Middle Creek by utilizing its correlation with Little River.

The correlation equation is:

$y = 1.92 + 0.415 x - 0.000118 x^2$

where y is the monthly flow in cfs of Middle Creek, and x is the corresponding monthly flow in cfs of Little River.

The flow of the Deep River at Ramseur, North Carolina, and Randleman, North Carolina shows good correlation with the data of Middle Creek. Their correlation coefficients are 0.65 and 0.63, respectively. Therefore, flow data were synthesized for Middle Creek for the period from 1924 to January 1930 by utilizing the correlation equations developed for the Deep River at both stations:

Deep River at Ramseur:

$y = 0.49 + 0.329 x - 0.000282 x^2$

and

Deep River at Randleman:

$y = 4.36 + 0.744 x - 0.001383 x^2$

where y is the monthly flow in cfs of Middle Creek, and x is the corresponding monthly flow in cfs of Deep River. The average value of data from those two stations was used when coincident records were available.

With the synthesized data (1924-1939) and the observed data (1939-1981) of Middle Creek, the monthly flow data of Buckhorn Creek for the period 1924-1981 were obtained by drainage area direct ratio relationship, and they are shown in Table 2.4.1-1.

The average values of Middle Creek at the Clayton, North Carolina, gage station (D.A. = 80.7 sq. mi.) for the 42-year period (1939-1981) and the 58-year period (1924-1981) are 90.8 cfs and 88.5 cfs, respectively, while the corresponding synthesized values of Buckhorn Creek (D.A. = 79.5 sq. mi.) are 89.4 cfs and 87.2 cfs, respectively.

Subsequent to the above analysis, a review was made of the flows in the New Hope River near Pittsboro, North Carolina, which has flow records dating back to January 1949. This river has a basin area of 285 sq. mi. and is located about seven miles from Buckhorn Creek basin at the closest proximity. The overall average runoff of this river is 12.85 in. With an intention to synthesize Buckhorn Creek flow data from the data of the New Hope River, it was found that since 1955 there has been some flow diverted into the river basin above the station by the city of Durham; therefore, the flow data at the station has been distorted. If the flow data of the New Hope River were used to synthesize flows of Buckhorn Creek, lower values would be obtained during drought periods than those obtained by the correlation with Middle Creek. However, the effect on storage use would be minor.

Since June 1972, a stream-gaging station has been installed on Buckhorn Creek in the vicinity of the Main Dam site in order to accumulate actual flow data prior to the impoundment of water in the Main Reservoir. The US Geological Survey installed and operates the Buckhorn Creek gaging station. The drainage area at this station is 74.2 sq. mi. Table 2.4.1-2 shows the actual observed monthly flow data and the calculated values derived from the drainage area ratio in

relation to that of Middle Creek (D.A. equal to 80.7 sq. mi.). A correlation analysis has been made between the data from the observed and derived flow of Buckhorn Creek during the period of the stream-gaging station operation. It was found that the correlation coefficient is 0.95 during this period of time. The result of the analysis is shown on Figure 2.4.1-5.

From the above correlation analysis, it can be concluded that the synthesized flow data of Buckhorn Creek from 1924 to 1981 can be utilized in hydrological studies.

2.4.1.2.1.2 Cape Fear River

The Cape Fear River basin is oblong in shape; its greatest width is about 60 miles and its length is about 200 miles. The Cape Fear River is formed by the confluence of the Deep and Haw Rivers. It flows generally southeast 198 miles and empties into the Atlantic Ocean at Cape Fear, 28 miles below Wilmington, North Carolina. The basin has a total area of 9136 sq. mi. of which 3127 sq. mi. are located above the confluence of the Deep and Haw Rivers.

The Cape Fear River is an estuary with the tidal reach extending to Lock and Dam No. 1, about 39 miles above Wilmington. The river is navigable to Fayetteville, with a channel width of generally 400 ft. and a depth ranging from 30 to 35 ft. from the Atlantic Ocean to Wilmington; thence a 200-foot width and 25 ft. depth from Wilmington to Navassa; and a depth of eight feet with varying widths for the remaining distance to Fayetteville.

The bankfull flood flow at Fayetteville is about 35,000 cfs, and at Lock No. 2 (River mile point 99, see Figure 2.4.1-4), it is about 20,000 cfs. The average width of the flood plain is approximately 2.2 miles. The difference between high and low stages is 69 ft. at Fayetteville and 44 ft. at Lock No. 2. The maximum flood flow of 150,000 cfs occurred on September 19, 1945, at Lillington and the minimum flow of 11 cfs occurred on October 14 and 15, 1954, at the same location. The monthly average flow data at Buckhorn Dam, shown in Table 2.4.1-3, were obtained from the records at Lillington by a drainage area ratio of 3196 sq. mi. at Buckhorn Dam and 3440 sq. mi. at Lillington.

2.4.1.2.1.3 Tributaries

The Cape Fear River has two major tributaries above the Buckhorn Dam (which is located nearby the confluence of the Buckhorn Creek and the Cape Fear River), the Haw and Deep Rivers, both of which originate in Forsyth County, North Carolina. The Deep River has a total length of 116 miles and a drainage area of 1422 sq. mi. The Haw River is about 90 miles in length and drains approximately 1705 sq. mi. Both rivers originate at elevations of about 1000 ft. MSL and have numerous falls and rapids, with the Haw River having the steepest gradient. The water surface elevation of the junction of the two rivers is about 158 ft.

Other major tributaries downstream of the withdrawal point include the Black River and the Northeast Cape Fear River. The former has a drainage area of 1563 sq. mi. and joins the Cape Fear River at river mile point 44. The latter drains a basin of 1738 sq. mi. and enters the Cape Fear River at Wilmington.

There are numerous minor tributaries including Upper Little River, Little River, Rockfish Creek, and Buckhorn Creek.

2.4.1.2.1.4 Dams, Reservoirs, and Locks

There are a number of regulating structures and reservoirs on the Cape Fear River. The locations of these structures and reservoirs are shown on Figure 2.4.1-4. Lock and Dam Numbers 1, 2 and 3 are located at river mile points 67, 99, and 123 respectively. Buckhorn Dam is at river mile point 192, and its spillway crest is at Elevation 158.18 ft.

In addition to the existing Lockville Dam and Carbonton Dam on the lower reach of the Deep River, the Corps of Engineers has studied additional development of water resources for the Cape Fear River Basin. A summary of this additional development is shown in Table 2.4.1-4. The completion of the proposed plan will furnish a minimum continuous flow of 600 cfs at Lillington.

2.4.1.2.1.5 On-site Waterbodies

The Buckhorn Creek drainage system involves five tributaries (Tom Jack Creek, Thomas Creek, Little Whiteoak Creek, Whiteoak Creek, and Cary Creek). These five creeks together with Buckhorn Creek are affected by the impoundment of the Main Dam. Furthermore, the Auxiliary Dam creates the Auxiliary Reservoir on Tom Jack Creek near the plant site. The plant site is bounded by Thomas Creek on the east side and extends westward to Tom Jack Creek. A nameless tributary of Little Whiteoak Creek had been located on the western half of the plant island where grading has now raised the elevation to 260 ft. MSL.

A drawing of the topography near the plant site is presented on Figure 2.4.1-6. This figure shows the small drainage areas and their divides before construction of the project. The drainage pattern can easily be visualized from this figure, which in comparison with Figure 2.4.1-2 of the plant site and drainage shows that construction of the project did not materially change the drainage pattern.

No existing ponds or impoundments are located within the boundary of the plant island area; however, there are numerous small farm ponds located in the reservoir area, some of which have been inundated by filling the reservoirs. They are shown on Figure 2.4.1-7.

2.4.1.2.1.6 Other Waterbodies

There are other reservoirs, lakes, and ponds in addition to those described in Sections 2.4.1.2.1.4 and 2.4.1.2.1.5 in the vicinity of the plant site.

The rivers, creeks, lakes, reservoirs, and ponds existing, prior to filling of the reservoirs, within 5 miles and 25 miles radius of the plant site are shown on Figures 2.4.1-7 and 2.4.1-8, respectively.

2.4.1.2.1.7 Groundwater

The sources of groundwater in the vicinity of the site are the bedrock units of the Sanford Formation of the Newark Group (Triassic). They consist of claystone, shale, siltstone, sandstone, conglomerate, and fanglomerate. Exceptions to this lithology are the thin diabase dikes in the bedrock. The Triassic rocks are overlain by a thin layer of dense clayey soils and saprolite.

The primary permeability of the Triassic aquifer is very low. However, the aquifer has secondary permeability due to the fractures which are filled with water below the water table. The fractures are common to depths of 100 ft, but become less prevalent and tight below that depth. Below about 400 ft, the fractures are closed and sealed to water flow. For a discussion of groundwater characteristics and usage in the plant vicinity, see Section 2.4.13.

2.4.1.2.2 Effects of Normal or Accidental Release of Contaminants on Water Users

2.4.1.2.2.1 Contaminants

Normal releases of contaminants into the hydrosphere will have negligible effects on surface and groundwater users (see Section 2.4.12.1). Should an accidental release of contaminants occur, adverse effects, if any, will be restricted to the area within the plant island. The only water user within the plant island is the plant itself. The use of water includes makeup to the cooling tower basin, the intake for the Emergency Service Water System, intermittent HVAC cooling water makeup, and fire protection.

Dilution of contaminants (refer to Section 2.4.12), should they enter the reservoirs, will be great enough to reduce concentrations below the limits of 10 CFR 20. The effect of accidental spills into the groundwater is discussed in Section 2.4.13.3.

2.4.1.2.2.2 Water Users

There are no known domestic surface water users of Buckhorn Creek within the proposed reservoir area. There are no surface water users of Buckhorn Creek downstream of the SHNPP project. There are no known domestic potable water supply intakes on the Cape Fear River between Buckhorn Creek and Lillington, North Carolina. The nearest source of potable water supply downstream of the site is at Lillington, North Carolina, approximately 12 miles downstream on the Cape Fear River.

Industrial and municipal surface water uses of the Cape Fear River downstream of Buckhorn Dam are shown in the Tables 2.4.1-5 and 2.4.1-6, respectively. River drainage basin areas at the points of withdrawal are included to indicate the additional flow available, as compared with the drainage area of 3196 sq. mi. at Buckhorn Dam. Most of the water withdrawn is returned to the Cape Fear River.

Carolina Power & Light Company's Brunswick Plant, located 19 miles south of Wilmington at Southport, N.C., nominally withdraws cooling water from the Cape Fear River. However, this user is not included on Table 2.4.1-5 since the withdrawal is within the tidal reaches of the river and does not constitute a consumptive use of river flow. The outfall of the Brunswick Plant is located on the Atlantic Ocean. The drainage area at the plant is 9090 square miles, and the withdrawal and discharge are both 1900 mgd.

Discussions with North Carolina State University's agricultural staff, U.S. Department of Agriculture, and county extension chairmen indicate that there are no known withdrawals for irrigation from the Cape Fear River. The principal economic crop in the Cape Fear basin is tobacco; however, the land along the Cape Fear River is not generally suited to production of tobacco. Tobacco is grown in the uplands and irrigation water, if used, is taken from farm ponds or wells. The lands along the Cape Fear River are either wooded, pasture or used for crops that are not generally irrigated in North Carolina.

2.4.2 FLOOD

2.4.2.1 Flood History

Before 1972, there were no flood records available for Buckhorn Creek. Records of flood flows since November of 1939 are available for the Middle Creek basin. Based on the ratio of drainage areas of Middle Creek (D.A. = 80.7 sq. mi.) and Buckhorn Creek (D.A. = 79.5 sq. mi.), the corresponding maximum historical peak flows for Buckhorn Creek near its confluence with the Cape Fear River are listed in Table 2.4.2-1 and the frequency analysis (Reference 2.4.2-1) of the data is shown in Figure 2.4.2-1.

Table 2.4.2-2 shows the recorded data after 1972 at the USGS gage station on Buckhorn Creek near Corinth (D.A. = 74.2 sq. mi.) together with the corresponding estimated values based on the Middle Creek data from the drainage area relationship. The maximum measured flood flow of Buckhorn Creek of 6920 cfs correlates very well with the estimated value of 7820 cfs, as the recorded data of Middle Creek indicates that a dam failure occurred in this flood.

The Cape Fear River has flow records at Lillington dating back to December 1923. The maximum flood flows at Buckhorn Dam are derived from the data at the Lillington gage by an adjustment by drainage area ratio (D.A. = 3440 sq. mi. at Lillington, D.A. = 3196 sq. mi. at Buckhorn Dam). These flows are shown in Table 2.4.2-3. The frequency analysis (Reference 2.4.2-1) of these data is presented on Figure 2.4.2-2. The maximum flood flow of 139,370 cfs occurred on September 19, 1945.

2.4.2.2 Flood Design Considerations

The SHNPP safety related structures and facilities are protected against all floods and flood waves caused by probable maximum events, such as the probable maximum flood (PMF), and the probable maximum hurricane (PMH). The tops of the Main Dam and the Auxiliary Dam are above the probable maximum flood water level in the reservoirs. The top of the Auxiliary Reservoir Separating Dike is allowed to be below the PMF stillwater level in the Auxiliary Reservoir, and the dike is designed to withstand the wave action under the PMF condition. The Emergency Service Water Intake channel is protected by sacrificial spoil fill along the shore bank of the Main Reservoir. The sacrificial spoil fill is allowed to be eroded during probable maximum events. After the event, the eroded portion will be inspected, restored, and stabilized where required. The design of this fill is discussed in Section 2.4-10. The plant island, where many safety related facilities are located, has a grade elevation above the PMF water level in both reservoirs.

The PMF in the Cape Fear River is not considered because of the large difference in elevation between the river bank (approximate elevation 160 ft. MSL) and the top of the Main Dam (elevation 260 ft. MSL) the nearest Seismic Category I structure to the river. The backwater effect of the PMF in the river through Buckhorn Creek is comparatively small.

The downstream face of the Main Dam is protected by a layer of oversized rock as indicated on Figure 2.5.6-2 for possible wind wave action whenever the backwater reaches the Main Dam. No specific design basis exists for downstream slope protection of the Main Dam. The rockfill shell does not require special slope protection because the Cape Fear River 500-year-flood backwater effect on Buckhorn Creek near the downstream face of the Main Dam is not expected to result in wave action on the dam. This is due to protection afforded by a small

downstream fetch which severely limits the size of wind-generated waves. The oversize rock zone on the downstream face is primarily a construction-related feature. During construction of the Main Dam, oversize rocks were plucked from each of the rockfill lifts in order to meet specifications. Where the oversize rocks were within practical limits (20 to 30 inches) they were placed near the downstream face in order to reduce handling of oversize material and provide additional protection to the downstream face.

Since the drainage area of Buckhorn Creek is small in comparison with that of the Cape Fear River at Buckhorn Dam, the construction of the Main Dam and the Auxiliary Dam of the project will have no significant effect on the 100-yr. and 500-yr. flood levels in the Cape Fear River. Consequently, the floor level shown on Figure 2.4.2-3 represents both the pre-construction and post-construction conditions.

The 100-yr. and 500-yr. floodplains adjoining the Cape Fear River in the vicinity of Buckhorn Creek are shown in Figure 2.4.2-3. The corresponding plains for Buckhorn Creek and the SHNPP reservoirs adjacent to the plant island are shown in Figure 2.4.2-4.

The flood profiles in the Cape Fear River are based on the following data provided by the U.S. Army Corps of Engineers (Reference 2.4.2-2):

Location	100-yr. Flood Water Level (Ft. above MSL)	Standard Project (approx. 500-yr.) Flood Water Level (Ft. above MSL)
10,000 ft. Upstream of Buckhorn Dam	168.5	186.5
Upstream Side of Buckhorn Dam	165.5	182.0
Downstream Side of Buckhorn Dam	159.5	182.0
4 miles Downstream of Buckhorn Dam	147.0	172.0

The flood water level profile slopes uniformly between the two locations upstream of the Buckhorn Dam as well as between the two locations downstream of the Buckhorn Dam.

The pre-construction flood profiles of Buckhorn Creek for the 100-yr. and 500 yr. floods were calculated using the HEC-2 computer program (Ref. 2.4.2-3). The 100-yr. and 500-yr. flood flows in Buckhorn Creek before plant construction were obtained from Figure 2.4.2-1 as 9,900 cfs and 16,000 cfs, respectively, at its confluence with the Cape Fear River. Based on these flows, the corresponding flows in the tributaries of Buckhorn Creek were estimated according to their drainage area ratios. Since the normal creek channel is rather shallow, the creek cross-sections for the flood flows were principally scaled from a 1/12000 scale map at 1000 to 2000 feet intervals. In addition, available project construction maps for the area below the Main Dam and the USGS 1/24000 map of the area adjacent to the Cape Fear River were also used. Manning's n-values of 0.04 and 0.045 were selected for the main and flood channels, respectively, in the flood profiles computation.

The floodplains adjoining Buckhorn Creek and its tributaries were delineated from the 1/12000 contour map as shown in Figure 2.4.2-4.

The construction of the Main Dam and Auxiliary Dam of the plant will reduce the magnitude of the flood flows downstream of the plant because of the storage capacity of the two reservoirs created by the dams. Again, based on the drainage area ratio between that at each dam location and that of the entire Buckhorn Creek, the 100-yr. and 500-yr. floods adopted for the floodplain delineation are:

Flood	At Main Dam	At Auxiliary Dam
100-yr	8850 cfs	215 cfs
500-yr	14300 cfs	350 cfs

Both the Main Dam and Auxiliary Dam have uncontrolled spillways to release floods. The spillway rating curves for these dams are shown in Figures 2.4.3-3 and 2.4.3-4. The corresponding flood level in each reservoir was determined by applying the flood flows to the appropriate rating curve. Since the reservoirs are rather small, no backwater effect in the reservoirs was taken into consideration when the floodplains adjoining the reservoirs were delineated.

The floodplains adjoining the reach of Buckhorn Creek between the Main Dam and Cape Fear River after the construction of the Main Dam were not studied since the flood levels will be less than before construction.

The construction of the plant will increase the extent of the floodplains above the Main and Auxiliary Dams in Buckhorn Creek and reduce the flood magnitude below the Main Dam. The water level (WL) and storage capacity (SC) of both reservoirs at 100-yr. and 500-yr. flood are:

	Main Reservoir		Auxiliary Reservoir	
Flood	WL (ft) MSL	SC (Ac ft)	WL (ft) MSL	SC (Ac ft)
100-yr.	234.0	142×10^3	252.5	5.25×10^3
500-yr.	239.0	174×10^3	252.8	5.35×10^3

The storage capacities are obtained from Figure 2.4.3-5 and 2.4.3-6, the reservoir area and capacity curves, using the calculated water levels.

The pre-construction and post-construction floodplains for the portion of Buckhorn Creek that is influenced by the plant construction are entirely within the site boundary. There are no existing structures within these floodplains other than those constructed for plant use. These structures were designed to preclude adverse effects due to the probable maximum flood. Additional structures may be constructed to support the recreational use of the Main Reservoir. It is expected that the effect of floods will be considered in the design of these structures based on a cost/risk assessment.

Since the Cape Fear River floodplains are not increased due to plant construction, any pre-existing structures in these areas are not subject to increased risk of flood damage due to plant construction.

In determining the probable maximum water level in the reservoirs, the precipitation produced floods in Buckhorn Creek were determined and the water levels in the reservoirs at the beginning of the PMF or during the PMH were considered under the normal operation condition. For compliance with NRC Regulatory Guide 1.59 (Design Basis Floods for Nuclear Power Plants), see Section 1.8. The events considered are summarized below:

a) PMF in Buckhorn Creek and a designed wind wave activity in the reservoirs when the resulting PMF water levels in the reservoir are taken into consideration.

b) PMH wind wave activity when the water levels in the reservoirs are at the normal operation level.

The probable maximum water levels are 243.1 ft., 258.0 ft., and 257.7 ft. at the Main Dam, Auxiliary Dam, and around the plant island, respectively (see Section 2.4.3.6.2). Flood protection design of safety related structures and facilities for flood waters for those heights include water-proofing, riprap, and sacrificial fill. The detailed flood design basis is addressed in Section 2.4.10. Tsunamis and seiches are events not applicable to the SHNPP.

See Section 2.4.4 for additional discussion of dam failures.

2.4.2.3 Effects of Local Intense Precipitation

The general area of the site is subject to local intense precipitation. The plant island faces the Main Reservoir, and its grade is 40 ft. above the normal water level of the Main Reservoir and 8 ft. above the normal water level of the Auxiliary Reservoir. The site is not expected to experience any long term accumulation of ice and snow; thus ice or snow melt is not considered for flooding effects. As described in Section 2.4.1.1, most of the site is drained to the Main Reservoir, while a small portion of the run-off is drained to the Auxiliary Reservoir via the Emergency Service Water Intake and Discharge Channels. Therefore, the site drainage does not pose any potential problems. The local probable maximum precipitation (PMP) is determined by the method described in Section 2.4.3.1 and is assumed to be the same as the PMP for one sq. mi. which is the smallest area considered in the method of determining the PMP. Table 2.4.2-4 presents the time distribution of this PMP. Since losses in unpaved areas are not considered during the PMP, the capacity for the plant site drainage for run-off is four in. per hour. As a result, the accumulated water depth during the PMP considered is approximately 14.8 in. as shown in Table 2.4.2-4. The protections of Seismic Category I structures and safety related systems on the plant island against local flood are discussed in Section 3.4.1.1.

All safety related buildings other than the Emergency Service Water Intake Structure, Screen Structure and Discharge Structure have structural features surrounding their roofs that would impound rainwater on the roofs assuming that the roof drains are plugged. In general, the ponding is caused by curbing whose height varies depending on the roof but is a maximum of one (1) foot above the high point of the surrounded roof. In addition to curbing around roof edges, the portions of the Reactor Auxiliary Building roofs which wrap around the west side of the containment building is partially surrounded by taller structures. Also, the tank building has two areas without roofs where walls enclose the tanks. The roof plans of all safety related buildings where ponding can occur are shown in Figure 2.4.2-5. Top elevation of the curbs and high points of each roof are also indicated in the figure.

No scuppers or openings have been provided in the curbs. If the regular roof drains are assumed to be plugged during a local intense PMP event, the storm water will pond on the roof and overflow the curbs. For the local intense PMP event as given in Table 2.4.2-4, the water level on all roofs will exceed the top of the surrounding curb by less than three (3) inches except for some areas of the Reactor Auxiliary Building roof which are surrounded by higher walls. In these areas the accumulated water depth will exceed the top elevation of the curb by a maximum of 1 1/2 feet. The maximum water levels, including the cascade flow from higher roof levels, are indicated on Figure 2.4.2-5.

The open areas of the Tank Building, which are surrounded by 25 foot high walls (see Figure 1.2.2-84), do not overflow, however, rainwater will accumulate to a depth of 23.36 feet.

The floor of the unroofed areas of the Tank Building and the roofs of all safety related buildings where water accumulates are strong enough to withstand the ponding loads in addition to other dead and live loads that can reasonably be expected to occur coincident with the PMP. The varying depths of water on a given roof due to the slope of the roof were accounted for in determining the structural adequacy.

2.4.3 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS (PMF)

The probable maximum flood (PMF) has been defined as an estimate of the hypothetical flood characteristics that are considered to be the most severe "reasonably possible" at a particular location based on comprehensive hydrometeorological analysis of critical runoff-producing precipitation and hydrologic factors favorable for maximum flood runoff (Reference 2.4.3-1). The objective of this study is to obtain a flood estimate that has a probability of occurrence near zero or a return period of near infinity.

Using the above definition as a guide, the PMF's for the SHNPP were developed as follows:

a) The Buckhorn Creek drainage basin was first analyzed under its natural, pre-construction condition. A unit hydrograph was developed for the entire drainage basin.

b) After construction of the Main Dam, the drainage basin above the dam is 71.0 sq. mi., wherein the area inundated is about 8.7 sq. mi., or about 12 percent of the entire basin. As a result, the inflow hydrograph to the Main Reservoir above the Main Dam is considerably altered from its natural condition; first, because of the reduced distance from the ridge line to the Main Reservoir surface; and secondly, because of the effect of direct rainfall on the reservoir area caused by the impoundment. In order to have a detailed estimate of the PMF, the drainage basin was divided into ten sub-basins, two of which (Sub-basins I and II) are located below the Main Dam site. Unit hydrographs were then developed for each sub-basin.

c) The probable maximum precipitation (PMP) was applied to the unit hydrograph with the appropriate infiltration losses to develop the estimated flood hydrograph for each sub-basin, as well as for the entire drainage basin.

d) An antecedent precipitation, which has an intensity of 1/2 PMP, and the PMP were also applied to the unit hydrograph with appropriate infiltration losses to develop the estimated flood hydrograph for each sub-basin in order to have a more conservative estimate of the PMF still water level in the Main and Auxiliary Reservoirs.

e) The total inflow into the Main Reservoir behind the Main Dam is the summation of the outflow from all the sub-basins located above the Main Dam.

f) After obtaining the inflow hydrograph, the PMF was then routed through the reservoirs to estimate the PMF still water level in the reservoirs.

The following discussions are based on the guidance presented in Regulatory Guide 1.59, Revision 2, Appendix A.

2.4.3.1 Probable Maximum Precipitation (PMP)

The PMP is the theoretical greatest precipitation over the applicable drainage area that would produce flood flows that have virtually no risk of being exceeded. The PMP depths used in this study were developed from the U.S. Weather Bureau's "Hydrometeorological Report No. 33" (Reference 2.4.3-2).

Their values for durations of 6, 12, 24, and 48 hours for the drainage basin of Buckhorn Creek (D.A. = 79.5 sq. mi.) at its confluence with the Cape Fear River at the Main Dam (D.A. = 71.0 sq. mi.), and at the Auxiliary Dam (D.A. = 2.43 sq. mi.) are shown in Table 2.4.3-1. Since the smallest drainage area considered in Reference 2.4.3-2 is 10 sq. mi., the intensity of the PMP of 10 sq. mi. was adopted for the PMP of the area of 2.43 sq. mi. For the PMP of the local drainage study, Hydrometeorological Report Nos. 51 and 52 of NOAA and Corps of Engineers were used (see Table 2.4.3-4). To allow for basin shape and the improbability of exact center of storms occurring over a particular drainage area, the PMP depths were reduced by a factor of 10 percent for the drainage areas of 79.5 sq. mi. and 71.0 sq. mi. as recommended by the Army Corps of Engineers (Reference 2.4.3-3), while those values for the drainage area of 2.43 sq. mi. and for the local drainage study were not reduced.

To facilitate the application of the unit hydrograph to compute the probable maximum flood (PMF), unit rainfall periods of l-hour increments were required. These increments were derived from a 6-hour rainfall distribution curve applicable to the North Carolina region, as shown on Figure 18 of "Design of Small Dams" (Reference 2.4.3-4). Furthermore, these increments were rearranged in accordance with the criteria recommended in Hydrometeorological Reports Nos. 33 (Reference 2.4.3-2) and 40 (Reference 2.4.3-5) for generating the PMF. The time distributions of PMP used are shown in Table 2.4.3-2. In view of the small areas of the drainage basins, the spatial distribution of the PMP was not considered.

2.4.3.2 Precipitation Losses

The HEC-1 computer program (Reference 2.4.3-6) was used to determine the precipitation losses. This program yields the optimized parameters of the unit hydrograph and loss rates for a stream basin by best reconstitution of an observed hydrograph. The data used in this optimization procedure are rainfall, drainage area, antecedent flow, and recession flow characteristics. The best reconstitution is considered to be that for which the weighted squared deviations between the observed and reconstituted hydrographs are a minimum. The univariate gradient search method was used in the optimization procedure.

Since the establishment of a gaging station on Buckhorn Creek near Corinth in 1972, several floods have occurred in Buckhorn Creek (Table 2.4.2-2). Among these floods, that which occurred on February 2, 1973 is the most severe, even when using the flood records of the adjacent Middle Creek basin (which date back to 1940), to estimate Buckhorn Creek flows. Therefore, the reconstitution of the flood hydrograph for the storm of February 1973 was performed using recording (showing hourly values during the storm) and non-recording (showing the total value of the entire storm) precipitation data and recorded flow rates.

Parameters such as precipitation loss and initial loss rate were determined as part of the optimization processes using the HEC-1 computer program. The output from the optimization, showing the observed and reconstituted hydrographs of Buckhorn Creek near Corinth, N.C.,

together with the recording and non-recording precipitation data at various stations, is shown in Figure 2.4.3-1.

Other output from the optimization are the HEC-1 loss parameters (STRKR, DLTKR, RTIOL, and ERAIN, see Reference 2.4.3-6 for their definitions) which are presented in Table 2.4.3-3. They were used in computing the PMF's for the entire Buckhorn Creek basin and its sub-basins.

2.4.3.3 Runoff Model

The unit hydrograph parameters for Buckhorn Creek near Corinth, along with the loss rate parameters, were obtained by using the HEC-1 computer program as mentioned in Section 2.4.3.2. The lag time (t_p) and the basin constant (C_T) thus obtained were utilized to develop unit hydrographs for the entire Buckhorn Creek basin and its sub-basins.

By application of Snyder's synthetic unit hydrograph relationships (References 2.4.3-4 and 2.4.3-7) and the basin geometric parameters, L (the length of the longest water-course from the point of interest) and L_{ca} (the length of water course from the point of interest to the intersection of the perpendicular from the center of gravity of the basin to the stream alignment) of the Buckhorn Creek drainage basin above Corinth, the basin constant CT can be derived from the known lag time, tp. Consequently, lag times for the entire Buckhorn Creek basin and its sub-basins can be obtained from known basin geometric parameters. The basin constant (C_p) and lag time (t_p) for each basin are the input required for the HEC-1 computer program to generate each sub-basin's unit hydrograph.

The unit hydrograph for the whole basin was used for predicting the PMF for the natural preconstruction condition, while those for the sub-basins were used for studying the PMF after completion of the project. Figure 2.4.3-2 shows a map of the entire Buckhorn Creek drainage basin area and the sub-basin areas for the Auxiliary Reservoir and Main Reservoir, and the area between the Main Reservoir and just above its confluence with the Cape Fear River.

Table 2.4.3-3 shows the Snyder and loss parameters of the whole Buckhorn Creek basin and each sub-basin area.

The sub-basin area identified as IX in Table 2.4.3-3 is that area north of the dashed line shown above the Auxiliary Dam on Figure 2.4.3-2. All parameters of the unit hydrograph were determined based only on this area. The drainage area between the dam and the dashed line was divided into lake area (0.63 sq. mi.) and land area (0.07 sq. mi.) and treated separately for inflow into the Auxiliary Reservoir. The total inflow into the Auxiliary Reservoir is comprised of three separate inflows: overland inflow from the 1.73 sq. mi. drainage area, direct rainfall on the 0.63 sq. mi. lake area, and direct runoff from the 0.07 sq. mi. residual land area. This accounts for the entire 2.43 sq. mi. drainage area behind the Auxiliary Dam.

As a result of the construction of the Main Dam and the Auxiliary Dam, two reservoirs were formed. There is a spillway associated with each dam. The spillway crest at the Main Dam has a net length of 50 ft. with a pier at its mid-length, while the spillway crest at the Auxiliary Dam has a length of 170 ft. Both spillways are hydraulically designed for an ogee shape with a design head (H_O) and an upstream dam height (P) of 30 ft. and 10 ft., respectively, for the Main Dam Spillway, while the corresponding values for the Auxiliary Dam Spillway are 5 ft. and 7 ft., respectively. The equation of discharge is:

$Q = C_s L\ H_e^{3/2}$

where Q is the discharge in cfs, L is the effective length of the spillway crest in feet and H_e is the total head on the crest of the spillway in feet including the velocity of approach head. The coefficient of discharge (C_s) is obtained from the curves shown on Figures 249 and 250 of "Design of Small Dams" (Reference 2.4.3-4) with known values of H_E/H_o and $P/_{Ho}$. When the values of $H_e/_{Ho}$ reduce from 1 to 0.5, C_s varies from 3.95 to 3.64 for the Main Dam Spillway, and from 3.85 to 3.54 for the Auxiliary Dam Spillway.

The effective length of the Main Dam Spillway crest is determined from the formula (Reference 2.4.3-4).

$L = L' -2\ (NK_p + K_a)\ H_e$

where L' is the net length of the spillway crest in feet, N is the number of piers, K_p is the pier contraction coefficient, and K_a is the abutment contraction coefficient. For the present case, K_p and K_a are equal to 0.01, N is equal to 1, and L' is 50 ft.; consequently L is 49.4 ft.

Since the head on the crest of the Auxiliary Dam Spillway ranges from 1 to 6 ft., the effect of the end contractions is insignificant; therefore the effective length of the Auxiliary Dam Spillway crest is taken as equal to the actual length, 170 ft. The rating curves for both spillways are shown on Figures 2.4.3-3 and 2.4.3-4.

For flood routing through the reservoirs, the Modified Pulse Method was used. This method is contained in the HEC-1 computer program. The capacity curves for both reservoirs are shown on Figures 2.4.3-5 and 2.4.3-6 and the corresponding rating curves for the spillways provide data for input to the computer program for the routings.

The crest of the Main Dam Spillway is at El. 220 ft. MSL and that of the Auxiliary Dam Spillway is at El. 252 ft. MSL.

2.4.3.4 Probable Maximum Flood Flow (PMF)

Application of the PMP shown in Table 2.4.3-2 to the unit hydrographs derived from the HEC-1 computer program resulted in the PMF for the entire Buckhorn Creek basin as well as for its sub-basins. With the input shown in Table 2.4.3-3, the resulting PMF includes the initial loss and infiltration loss during the PMP. No base flow was considered in the estimate of the PMF since the mean flow in Buckhorn Creek is 87.2 cfs as shown in Table 2.4.1-1. This amount is equivalent to 1.1 cfs per sq. mi. of drainage area, which is insignificant when compared to the PMF flow.

The nearest Seismic Category I structure to the Cape Fear River is the Main Dam, which has its top at Elevation 260 ft. MSL, and the elevation of the Cape Fear River bank near Buckhorn Dam is in the vicinity of 160 ft. MSL. This large difference in elevation precludes any over-topping of the Main Dam due to backwater effects of the PMF on the Cape Fear River. Therefore, the PMF and the flood induced by the failure of dams upstream of the Buckhorn Dam in the Cape Fear River are not considered.

2.4.3.4.1 Buckhorn Creek Drainage Basin Under Its Natural Condition

The PMF hydrograph for Buckhorn Creek in its natural condition prior to the construction of the reservoirs is shown on Figure 2.4.3-7. The peak flow is 52,000 cfs, which would occur about 29 hours after the beginning of the PMP storm.

2.4.3.4.2 Drainage Basin Above the Auxiliary Dam

By considering the drainage area above the Auxiliary Dam as a sub-basin of the drainage area above the Main Dam, the intensity of the PMP of the drainage area of 71.0 sq. mi. is used to estimate the PMF. The PMF flow includes the overall runoff from the drainage area of 1.73 sq. mi., the direct rainfall on the reservoir surface area of 0.63 sq. mi., and the direct runoff from rainfall excess on the residual land area of 0.07 sq. mi. The flow was then routed through the Auxiliary Reservoir, and the peak flow is reduced from the combined overland runoff and direct rainfall of 5950 cfs to 3670 cfs, as shown on Figure 2.4.3-8.

In addition, a study with a separate and more severe local PMF was also performed. The PMP with its intensity related to a drainage area of 2.43 sq. mi. was used, as shown in Table 2.4.3-2. Since the drainage area involved is rather small, the maximum possible antecedent moisture condition was assumed, and infiltrated and retention were neglected to derive the PMF. Consequently, the PMP excess equals the PMP values shown in Table 2.4.3-2. This is the most conservative assumption that can be made with respect to rainfall. The storm period considered for this analysis is 36 hours since time to peak inflow is well within this time period, and the incremental hourly rainfall beyond this period is negligible. A unit rainfall period of 1-hour increments was adopted, and the 1 hour unit hydrograph characteristics shown in Table 2.4.3-3 were used for determining the overland flow from the drainage area of 1.73 sq. mi. In addition to the overland flow, the direct rainfall on the sum of the lake area and residual land area totaling 0.70 sq. mi. was accounted for separately, with an assumed time of concentration of zero. After obtaining the PMF hydrograph with a peak flow of 8270 cfs, the PMF was routed through the Auxiliary Reservoir, reducing the peak flow to 5030 cfs.

Figures 2.4.3-8 and 2.4.3-9 show the inflow flood hydrographs from overland flow and direct rainfall on the lake and residual land area, and the outflow hydrograph for both cases studied above.

2.4.3.4.3 Drainage Basin Above the Main Dam

The drainage area controlled by the Main Dam is 71.0 sq. mi. This area is composed of 2.43 sq. mi. above the Auxiliary Dam mentioned in Section 2.4.3.4.2, 46.37 sq. mi. of seven sub-basin areas shown on Table 2.4.3-3, and 8.68 sq. mi. and 13.52 sq. mi. of the reservoir water surface area and residual land surface area, respectively, for direct rainfall and runoff. The PMF hydrographs for each of the sub-basins were determined separately by the HEC-1 computer program, then they were summed to determine the overland PMF inflow to the Main Reservoir. Due to the proximity of all sub-basins, no adjustment was made to reflect lag time due to their relative positions in producing the peak instantaneous overland inflow.

The average hourly inflow from direct rainfall on the lake surface and the rainfall excess on the residual land areas was also determined separately for the drainage area controlled by the Main Reservoir. In addition to this inflow, the discharge from the Auxiliary Reservoir was added to obtain the total inflow to the Main Reservoir during the flood.

As shown on Figure 2.4.3-10, the overland inflow hydrograph, including the release from the Auxiliary Reservoir, has a peak of 58,790 cfs 19 hours after the start of the storm. Also shown on Figure 2.4.3-10 is the runoff from direct rainfall on the reservoir surface and residual land areas. The instantaneous combined peak inflow at the Main Dam site from all sources indicated by Figure 2.4.3-10 is about 161,710 cfs, which includes release from the Auxiliary Reservoir. This combined peak occurs about 11 hours after the start of the storm. The combined inflow hydrograph was routed through the reservoir, and an outflow hydrograph was developed with a peak flow of 11,030 cfs.

As indicated by a comparison of Figures 2.4.3-7 and 2.4.3-10, the project will afford some flood protection to the area downstream of the Main Dam during a major storm. For the probable maximum flood, the peak outflow is reduced from 52,000 cfs for the natural condition to 11,030 cfs after construction of the project.

An analysis was also made of the PMF approaching the Main Reservoir by assuming conservatively that the PMF begins five days after the start of a less severe storm such as the standard project flood resulting from 1/2 PMP. For this assumed antecedent condition, the reservoir level at the end of the fifth day resulting from the standard project flood would be about Elevation 225.2 ft. MSL, and the corresponding discharge would be about 2000 cfs. Starting with this flood surcharge at the beginning of the probable maximum flood, the peak outflow is 14,190 cfs, peaking about 33 hours after the start of the PMP storm. Figure 2.4.3-11 shows the inflow and outflow hydrographs for the probable maximum flood following the standard project flood.

2.4.3.5 Water Level Determination

The water levels in the Main Reservoir and the Auxiliary Reservoir are related to the safety of the safety related structures such as the Main Dam, the Auxiliary Dam, and those structures situated at the plant site. With the known PMF flow over the spillways of both reservoirs, the PMF stillwater levels in the reservoirs were determined by the corresponding spillway rating curves shown on Figures 2.4.3-3 and 2.4.3-4.

The PMF flow over the Auxiliary Reservoir Spillway is 3670 cfs, and the corresponding water level elevation in the reservoir is 255.2 ft. MSL. When the more severe local PMP is considered, the corresponding PMF flow is 5030 cfs, and the water level elevation is 256.0 ft. MSL. The reservoir water level elevation hydrographs for both cases are shown on Figures 2.4.3-8 and 2.4.3-9.

The PMF flow over the Main Reservoir Spillway is 11,030 cfs without the antecedent standard project flood, and the resulting water level elevation is 236.2 ft. MSL. When the antecedent standard project flood is taken into consideration, the PMF flow becomes 14,190 cfs, and the water level elevation in the reservoir is 238.9 ft. MSL. Both reservoir water level elevation hydrographs are shown on Figures 2.4.3-10 and 2.4.3-11.

2.4.3.6 Coincident Wind Wave Activity

The coincident wind wave activities were determined in accordance with the procedures and methods presented in the U.S. Army Corps of Engineers' ETL 1110-2-221 (Reference 2.4.3-10) and in the Shore Protection Manual (Reference 2.4.3-8). For this study, the first reference was used to determine the wave characteristics, while the second reference was employed in

computing the wave runup. Since no long term wind records are available for the plant site, the maximum wind velocity charts in Reference 2.4.3-10 were utilized to determine the design wind velocity. The PMH wind speed was taken from Section 2.4.5.3.1.

2.4.3.6.1 Wind Setup, Wave Height, and Wave Period

The wind setup, wave height, and wave period are a function of the effective fetch length, wind speed, wind duration, and water depth. These values are shown in Tables 2.4.5-1 and 2.4.5-2 for various critical locations where the maximum wind runup occurs. Figure 2.4.5-1 provides the locations of various fetchs used in computing the wind setup, wave height, and wave period.

2.4.3.6.2 Wave Runup

Wave runup is a function of wave height, wave period, slope of the approach, bottom structure slope, and the water depth at the toe of the structure. The upstream faces of the Main Dam and the Auxiliary Dam are protected by riprap; the former has a slope of 1 (vertical) to 2 (horizontal) and the latter has a slope of 1 (vertical) to 2 1/2 (horizontal). By considering the slope and characteristics of the protected surface and the wave steepness (ratio of wave height to wave length), the relative wave runup (ratio of wave runup to wave height) was obtained (Reference 2.4.3-8). The wave runup for wave steepness greater than 0.08 was estimated by extrapolating. The wave runup thus obtained was primarily based on the results of small scale hydraulic model tests. The results are presented in Tables 2.4.5-1 and 2.4.5-2.

The maximum wave runup at the Main Dam is 4.1 ft. This value in combination with the wind setup, (0.1 ft.) and the PMF stillwater elevation, (238.9 ft.MSL as determined in Section 2.4.3.5) produces a probable maximum water level at the Main Dam of approximately 243.1 ft. MSL due to the PMF in the Main Reservoir coincident with wave activity. This maximum water level is 16.9 ft. below the top of the Main Dam, 260 ft. MSL.

The maximum wave runup on the upstream face of the Auxiliary Dam is 1.9 ft. This value in conjunction of the wind setup, (0.1 ft.), and the PMF stillwater level (256.0 ft. MSL in the Auxiliary Reservoir determined in Section 2.4.3.5) results in a probable maximum water level at the Auxiliary Dam of approximately 258.0 ft. MSL. This maximum water level is 2.0 ft. below the top of the Auxiliary Dam.

On the plant island, the southerly fill portion of the Emergency Service Water Intake Channel and the embankment faces of the plant island which face the Main Reservoir are protected by sacrificial spoil fill. The fill at the southeast corner has a slope of 1 (vertical) to 5 (horizontal) from plant grade 260 ft. MSL to Elevation 245 ft. MSL (a variable-width berm at that elevation), and a slope of 1 (V) to 10 (H) from the berm to the existing ground. The locations of this fill are shown on Figure 2.4.1-2. Wave runup is estimated for a 1 (V) to 10 (H) smooth slope for the sacrificial fill area, and a 1 (V) to 5 (H) slope for the natural ground surface adjoining the Auxiliary Reservoir. The results are shown in Tables 2.4.5-1 and 2.4.5-2.

The plant is generally protected from wind-generated waves by high ground from all quadrants. The most critical wind fetch at the site is for a wind coming from the west over the Auxiliary Reservoir. The maximum wave runup and wind setup level on the side of the plant island is 1.7 ft. For a maximum PMF stillwater level of 256.0 ft MSL in the Auxiliary Reservoir, the maximum water level is estimated to be 257.7 ft. MSL, 2.3 ft. below the plant grade.

For design of safety related structures against static and dynamic effects of water waves, see Sections 2.5.6.4.5 and 3.8.4.3.1.

2.4.4 POTENTIAL DAM FAILURES, SEISMICALLY INDUCED

The following discussions in this section are based on the guidance presented in Regulatory Guide 1.59, Revision 2, Appendix A.

2.4.4.1 Dam Failure Permutations

A map of the entire Buckhorn Creek drainage basin area at the plant site, and the area up to its confluence with the Cape Fear River, is shown on Figure 2.4.3-2. There are no existing water control structures in this drainage basin other than the Main Dam, the Auxiliary Separating Dike, and the Auxiliary Dam.

Figure 2.4.1-1 shows the location of the dikes and the dams with respect to the plant site. The Auxiliary Separating Dike and Auxiliary Dam are located on Tom Jack Creek to the west of the plant site, whereas the Main Dam is located on Buckhorn Creek downstream and south of the plant site.

The Auxiliary Reservoir was formed by the construction of the Seismic Category I Auxiliary Dam and Spillway. The Auxiliary Separating Dike and Auxiliary Reservoir Channel form a cooling loop in the reservoir. The maximum water level in the Auxiliary Reservoir, as indicated in Tables 2.4.5-1 and 2.4.5-2, is expected to be Elevation 258.0 ft. MSL for a probable maximum flood (PMF) with 52.9 mph wind. The top of the Auxiliary Dam and the plant island grade are at Elevation 260 ft. MSL. The PMF is assumed to occur with the Auxiliary Reservoir at a normal water level of 252.0 ft. MSL (Auxiliary Dam Spillway crest level).

The Auxiliary Reservoir Channel, a Seismic Category I structure, is designed to remain stable when subjected to the Safe Shutdown Earthquake or the most severe cases of other postulated natural phenomena. The Auxiliary Reservoir Channel is sized such that, if any earth slippage did occur, the flow due to the PMF would still pass through the channel at a velocity not in excess of 2 ft/sec, and no substantial differential head (less than one foot) would be created across the Auxiliary Separating Dike. A typical cross section of the Auxiliary Reservoir Channel is shown on Figure 2.5.6-6.

The top of the Auxiliary Separating Dike is at Elevation 255 ft. MSL. The Auxiliary Separating Dike is a Seismic Category I structure. Due to the low differential head across the Auxiliary Separating Dike, its failure would not cause loss of water or storage capacity or result in failure of any other structure. Due to short-circuiting of the cooling pathway, the ability of the Auxiliary Reservoir to perform its emergency cooling water function would be adversely affected by the loss of the Auxiliary Separating Dike. Under this postulated condition, the Main Reservoir would function as a backup source of cooling water with discharge of water into the Auxiliary Reservoir and then into the Main Reservoir over the Auxiliary Dam Spillway. The pathway thus established exceeds the heat dissipation requirements normally provided by the Auxiliary Reservoir alone, and it is more than adequate for plant requirements.

The Auxiliary Dam is a Seismic Category I structure with the top of the dam at Elevation 260 ft. This will prevent overtopping of the dam due to maximum wave runup associated with a 52.9 mph wind coincident with the PMF (see Table 2.4.5-1). The preferred source of emergency

service water for the plant is the Auxiliary Reservoir. In the unlikely event of a failure of the Auxiliary Dam, emergency service water will be supplied from the Main Reservoir.

Failure of the Auxiliary Dam will not induce failure of the Main Dam since the two dams are widely separated (over four miles). As discussed in Section 2.4.4.2, the rise in water level in the Main Reservoir would be approximately 1.5 ft. to Elevation 221.5 ft. and there would be no wave action to impose unusual forces on the Main Dam.

The Main Reservoir was formed by the construction of a Seismic Category I dam and spillway on Buckhorn Creek. The maximum water level, including wave runup and wind setup in the Main Reservoir as indicated in Table 2.4.5-1, is expected to be at Elevation 243.1 ft. MSL. The flood is assumed to begin five days after the start of a less severe storm such as the standard project flood. For this assumed antecedent condition, the Main Reservoir level at the end of the fifth day as a result of the standard project flood would be Elevation 225.2 ft. MSL. Starting with this flood surcharge at the beginning of the PMF, the maximum still water flood level would be Elevation 238.9 ft. MSL and wave runup for a 50.4 mph wind would be to Elevation 243.1 ft. MSL. The top of Main Dam is at Elevation 260 ft. MSL.

Failure of the Main Dam would not induce failure of the Auxiliary Dam. As the lowest ground level at the Auxiliary Dam is only 10 to 15 ft. below the normal water level in the Main Reservoir, rapid drawdown of the Main Reservoir would have a negligible effect on the downstream face of the Auxiliary Dam. Furthermore, the dam is conservatively designed using a loading condition of no water downstream. If the Main Dam failed, emergency service water for the plant would be available from the Auxiliary Reservoir.

U.S. Highway No. 1 and the relocated Norfolk Southern Railroad, Durham Line, cross portions of the Auxiliary Reservoir, as shown on Figure 2.1.1-1. The highway and the railroad embankment have reservoir water on both sides which are connected by culverts. Embankment failure would not affect the safety related function of the Auxiliary Reservoir.

The Plant access road on the east side of the plant site crosses the Main Reservoir at Thomas Creek with the top of the road at approximate Elevation 243 ft. MSL. The embankment has Main Reservoir water on both sides and its failure would not affect the safety related function of the Main Reservoir, Main Dam, or Auxiliary Dam. An additional temporary construction road crosses Thomas Creek upstream of the plant access road. The failure of the construction road cannot adversely affect the safety related functions of the Main Reservoir, Main Dam, or Auxiliary Dam.

Should any of these embankments fail in such a way that the culverts are blocked, the volume of water that would be impounded is insignificant to the cooling function of the reservoirs. There are additional roads which cross fingers of the Main Reservoir. The failure of these roads would not reduce the amount of water available to the plant, nor would they cause failure of any safety related structures.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failure

An analysis was made of the effect of failure of the Auxiliary Dam on the Main Reservoir and Main Dam. The following conservative assumptions were made:

a) The Auxiliary Dam fails instantaneously (disappears) with the water level in the Auxiliary Reservoir conservatively chosen to be at Elevation 253 ft. MSL and at Elevation 220 ft. MSL in the Main Reservoir.

b) The wave height and velocity developed at the Auxiliary Dam site travels downstream without any attenuation or dampening through Tom Jack Creek just downstream of the Auxiliary Reservoir.

Based on the assumption of an instantaneous dam failure and the method of analysis given in "Water Waves" by J.J. Stoker (Reference 2.4.4-1), the height and velocity of the hydraulic bore were calculated to be 10 ft. and 44 ft. per second, respectively. The corresponding instantaneous discharge rate at the time of failure is 1,744,000 cfs. The velocity of the negative wave moving upstream was determined on the basis of the average depth (d) at each section using the formula $v = \sqrt{gd}$, where g is 32.2 ft/sec^2. The time required for this wave to move upstream and reflect back to the Auxiliary Dam site is the total time required to empty the Auxiliary Reservoir down to Elevation 220 ft. MSL.

On the basis of the instantaneous velocities and cross sectional areas at the stations upstream of the reservoir, the time required to completely empty the Auxiliary Reservoir was determined to be approximately 400 seconds. However, the Auxiliary Reservoir would not drain completely, since the normal water level in the Main Reservoir (Elevation 220 ft. MSL) is higher than the bottom of the Auxiliary Reservoir.

The time required for the positive wave to move through Tom Jack Creek into the Main Reservoir is approximately 200 seconds, during which time about 144×10^6 cu ft of water would be emptied from the Auxiliary Reservoir. Since the volume of storage between Elevation 220 ft. MSL and 230 ft. MSL (10 ft. higher) in Tom Jack Creek is 154×10^6 ft.^{3} which is greater than the volume emptied from the Auxiliary Reservoir, the bore cannot be maintained due to the lack of continuing supply of energy, and it becomes a non-linear wave with a highly dissipative impulse. For this condition, an exponential type of decay can be assumed; therefore the bore will quickly dissipate and enter the Main Reservoir with little disturbance. The average rise in the water level of the Main Reservoir would be approximately 1.5 ft., which results from a uniform spread of the total amount of water stored behind the Auxiliary Dam over the surface of both reservoirs to Elevation 221.5 ft. MSL.

Since there is no wave action in the Main Reservoir, the failure of the Auxiliary Dam would impose no unusual forces on the Main Dam.

2.4.4.3 Water Level at Plant Site

The plant site, as indicated on Figure 2.4.1-1, is bounded by the Main Reservoir on the east, south, and south-west sides, and by the Auxiliary Reservoir on the west and north-west sides. The maximum water level in the Auxiliary Reservoir would be at Elevation 258.0 ft. MSL; and the maximum water level in the Main Reservoir would be at Elevation 243.1 ft. MSL (see Tables 2.4.5-1 and 2.4.5-2). The Auxiliary Reservoir maximum water level of Elevation 258.0 ft. MSL includes wave runup and wind setup superimposed on the PMF. Failure of the Auxiliary Dam, Auxiliary Reservoir Separating Dike, or Main Dam would not result in any rise of water level above Elevation 258.0 ft. MSL and therefore the plant site with its grade at Elevation 260 ft. MSL will not be flooded by dam failure.

2.4.5 PROBABLE MAXIMUM SURGE FLOODING

The probable maximum hurricane (PMH) could cause a water level change of the Main Reservoir and the Auxiliary Reservoir. The resulting high water levels, if not considered in the project design, could affect the safety of the Main Dam, Auxiliary Dam, or other safety related structures located at the plant island. The design considerations afforded these facilities due to flooding are discussed in the following sections. Preceding the PMH, the normal water level elevations in the Main Reservoir and in the Auxiliary Reservoir are 220 ft. MSL and 252 ft. MSL, respectively.

2.4.5.1 Probable Maximum Wind and Associated Meteorological Parameters

The meteorological characteristics used to calculate the probable maximum hurricane (PMH) are those reported by the U. S. National Oceanic and Atmospheric Administration (NOAA) in their unpublished report HUR 7-97 (Reference 2.4.5 1). HUR 7-97 describes the PMH as "a hypothetical hurricane having that combination of characteristics which will make it the most severe that can probably occur in the particular region involved". The hurricane should approach the point under study along the critical path at a critical speed. The hurricane characteristics used in establishing the PMH include:

a) Central pressure index (CPI) - the minimum surface pressure in the eye of the hurricane.

b) Radius of maximum wind (R) - the distance from the eye of the hurricane to the locus of maximum wind.

c) Forward speed (T) - the rate of forward movement of the hurricane.

d) Maximum gradient wind (V_{gx}) - the absolute highest wind speed in the belt of maximum wind.

e) Peripheral pressure (P_n) - the surface pressure at the outer limits of the hurricane where hurricane circulation ends.

HUR 7-97 presents values for each of those characteristics for each degree of north latitude along the east coast. Single values are presented for CPI and P_n, and three values are given for both R and T. Since V_{gx} is dependent upon P_n, CPI, and R, three values are also given for this parameter.

At the latitude (35.6N) where the SHNPP is located, the following PMH characteristics are recommended in HUR 7-97:

a) CPI: 26.91 in. Hg.

b) R for: small radius storms (RS) - 7 nautical miles (NM); medium radius storms (RM) - 17NM; large radius storms (RL) - 34 NM.

c) T for: slow forward speed (ST) - 5 knots; medium forward speed (MT) - 18 knots; high forward speed (HT) - 40 knots.

d) V_{gx} for RS: 148.9 mph

RM: 147.2 mph

RL: 144.1 mph

e) P_n: 30.90 in. Hg.

The maximum gradient wind (V_{gx}) for various radii (R) of maximum wind shown above is a 30-foot, 10 minute average over-water wind speed. When a hurricane moves inland, its intensity is reduced. The overwater wind speed is then reduced by an adjustment ratio which is a function of its overland travel time. SHNPP is located approximately 140 miles inland from the coast line. With a forward speed of 40 knots, the overland travel time is approximately 3-1/2 hours, and the adjustment ratio is 0.84, as recommended by HUR 7-97. If 147 mph is adopted as the maximum gradient over-water wind speed, the maximum gradient overland wind speed at the site would be 123 mph.

2.4.5.2 Surge and Seiche Water Levels

For the SHNPP site the only dynamic mechanism considered to be credible for the production of high water levels is the probable maximum wind discussed in Section 2.4.5.1. Section 2.4.5.3 discusses the effects of the probable maximum wind on the plant reservoirs and the resulting wave activity.

2.4.5.3 Wave Action

2.4.5.3.1 Wave Generating Wind Activity

Since the 123 mph wind speed at the site, as discussed in Section 2.4.5.1, is a 30-ft., 10 minute average over-land wind speed, and if it is assumed that a wind duration time of 30 minutes is required to generate the maximum wave, then a further correction to account for the variation of wind speed with duration should be made. The ratio of the 30-minute wind speed (V_{30}) to the 10-minute wind speed (V_{10}) is given by the equation (Reference 2.4.5-2):

$V_{30}/V_{10} = 1.45 - (0.07)\ (\ln 1800) = 0.925,$

and

$V_{30} = 0.925\ V_{10} = 0.925 \times 123 = 114$ mph.

To account for the increase in wind speed at the site as the air trajectory moves over the smoother and more uniform water surface of the reservoir, the ratio of over-water to overland wind speed for a conservatively long fetch of 0.5 miles was applied. According to Reference 2.4.5-3, this ratio is 1.08 which would result in a 123-mph wind speed. This wind speed, in conjunction with the normal water levels in the Main Reservoir and the Auxiliary Reservoir, was used to estimate maximum wave runup. All wind wave activities were determined in accordance with the procedures and methods described in the Shore Protection Manual (Reference 2.4.5-4).

2.4.5.3.2 Wind Setup, Wave Height, and Wave Period

In view of the 40 ft. difference in elevation between the top of the Main Dam and the normal water level in the Main Reservoir, wave action on the Main Dam during the PMH was not studied. The wind setup, wave height, and wave period for the critical locations at the Auxiliary Dam and around the plant island were calculated based on known values of fetch, water depth, and wind speed. These values are listed in Tables 2.4.5-1 and 2.4.5-2. Each critical fetch for which wave runup has been calculated is shown on Figure 2.4.5-1.

2.4.5.3.3 Wave Runup

Following the same procedure as described in Section 2.4.3, wave runup at the Auxiliary Dam and plant island has been calculated.

Assuming a PMH windspeed of 123 mph, the maximum runup is 3.8 ft. at the Auxiliary Dam. This value, combined with a wind setup of 0.4 ft. and the normal water level in the Auxiliary Reservoir, results in a maximum water level elevation of 256.2 ft. MSL, which is 3.8 ft. below the top of the Auxiliary Dam. Similarly, the maximum runup at the plant island is 2.7 ft. This value, combined with the wind setup of 0.2 ft. and the normal operation water level in the Auxiliary Reservoir, results in a maximum water elevation of 254.9 ft. MSL, which is 5.1 ft. below the grade elevation of the plant island.

U. S. Highway 1, which crosses a finger of the Auxiliary Reservoir, is elevated above the maximum reservoir water levels; the pre-existing alignment of the road and the pre-existing drainage structures under the road have not been changed except for lengthening of the culverts to allow for possible future widening of the road.

The plant access roadway to the site crosses the Thomas Creek area of the Main Reservoir and is protected from flooding since the lowest elevation of the roadway at the plant site is 243 ft. MSL. The maximum wave runup and wind setup level is 240.2 ft. MSL as shown in Table 2.4.5-2.

The relocated Norfolk Southern Railroad, Durham Line, crosses portions of the Auxiliary Reservoir, as shown on Figure 2.1.1-1. The railroad embankment has reservoir water on both sides. The top of the rails is set at a minimum elevation of 262 ft. MSL, and the top of the embankment is at approximate Elevation 260 ft. MSL; these elevations are approximately eight feet above normal water level in the Auxiliary Reservoir and are above all wave runup heights.

2.4.5.4 Resonance

Wave amplification due to "harbor resonance" will not occur on either reservoir at the plant site because the wind fetch is approximately 100 times longer than the significant wave length. The resonance due to such a high mode, if it does occur, would not have an appreciable effect. Normally, only the first few modes of resonance are of concern, that is, the wave length would have to be at least 500 ft.

2.4.5.5 Protection of Structures

The only Seismic Category I safety-related facilities that require design consideration due to wave action are those associated with the reservoirs; i.e., the Main Dam, Auxiliary Dam,

Auxiliary Separating Dike, Auxiliary Reservoir Channel, Emergency Service Water Intake and Discharge Channels, the Emergency Service Water Screening Structure, Emergency Service Water Intake and Discharge Structures, the Cooling Tower Makeup Water Intake Channel, and the Emergency Service Water and Cooling Tower Makeup Water Intake Structure.

The upstream face of the Main Dam, both upstream and downstream faces of the Auxiliary Dam, and both sides of the Auxiliary Separating Dike are protected by riprap designed for the worst postulated wave action. Additional description of the riprap design is in Section 2.5.6.

The downstream face of the Main Dam is protected by a layer of oversized rock, as indicated on Figure 2.5.6-2. As discussed in Section 2.4.2.2, the backwater effects of the Cape Fear River on the downstream face of the Main Dam are not expected to be significant. However, protection of the downstream face, as described in Section 2.5.6.4, serves as an additional safety precaution.

The Emergency Service Water and Cooling Tower Makeup Water Intake Structure, the Emergency Service Water Screening Structure, and the Emergency Service Water Discharge Structure are designed to withstand forces that could result from the worst postulated flood and wind conditions.

The plant intake and discharge channels and the Auxiliary Reservoir Channel are designed to withstand the worst postulated natural phenomena, as described in Section 2.4.8.

The embankment of the plant island along the Main Reservoir is protected by sacrificial spoil fill, as shown on Figure 2.4.1-2. The berm of the sacrificial spoil fill at Elevation 245 ft. MSL is above the maximum Main Reservoir water level of 240.2 ft. MSL (see Table 2.4.5-2) and it has a width of 300 feet on the south and southeast exposures. The nearest Seismic Category I structure (Diesel Generator Building) on the plant island is about 2000 ft. from the edge of the Main Reservoir at its normal water level of 220 ft. MSL and approximately 900 ft. from the edge of the plant island grade at Elevation 260 ft. MSL. The southerly edge of the Emergency Service Water Intake Channel is also 1000 ft. from the edge of the Main Reservoir at its normal water level of 220 ft. MSL. The extent of erosion due to the two worst fetches (S and SSE, see Table 2.4.5-2) is estimated to be 150 ft. resulting from a PMH having a duration of 48 hours. This estimation is based on methods described in References 2.4.5-5 and 2.4.5-6. The 300 foot wide sacrificial spoil fill, therefore, provides a very conservative design. Additional description of the sacrificial spoil fill is in Sections 2.4.3.6.2 and 2.5.6.

All safety related structures on the plant island are protected from high water levels up to Elevation 261 ft. which is higher than any anticipated flood levels due to wave runup in the reservoirs or direct rainfall on the plant island. For further discussion see Sections 2.4.1.1 and 3.4.1.

2.4.6 PROBABLE MAXIMUM TSUNAMI FLOODING

The areas of the U. S. that are susceptible to tsunamis are those bordering on the Pacific Ocean or the Gulf of Mexico. As it is located approximately 140 miles inland on the Atlantic coast, the Shearon Harris Nuclear Power Plant is not subjected to tsunamis.

2.4.7 ICE EFFECTS

Ice formation in this locality is not expected to be severe enough under any circumstances to jeopardize the operation of the Cooling Tower Makeup Water System or the Emergency Service Water System. Minimum average temperatures, as recorded at Raleigh Airport, for the months of December, January and February are 30.5, 30.0 and 31.1°F, respectively.

The Emergency Service Water Screening Structure draws cooling water by gravity from the Auxiliary Reservoir, the preferred source of emergency service water, through the Emergency Service Water Intake Channel. Pump suction for all pumps are located more than 10 ft. below the low water level. Therefore, considerable icing would be required in order to affect the operation of these pumps. This amount of icing is unlikely, based on data available from the National Oceanic and Atmospheric Administration Office at the Raleigh-Durham airport which show ice formation on large bodies of water in Central North Carolina is limited to minor freezing along the shoreline.

However, in the unlikely event that ice is present in the Auxiliary Reservoir the Emergency Service Water Screening Structure is protected from ice entering the intake bays by means of a concrete baffle wall which extends from the deck (Elevation 262 ft MSL) to Elevation 247.5 ft MSL. This is below the normal Auxiliary Reservoir level. In addition each bay has a course screen consisting of 3-in. by 3/8 in. bars with a clear space of 3-in. between bars. Any ice fragments smaller than 3-in. that pass through the course screens will be picked up and disposed of by the traveling screens. Ice buildup on the traveling screens will be prevented by the use of heated hoods and by continuously running the screens if potential icing conditions warrant. The inlet of the gravity discharge pipe is located more than eight ft. below low water.

The Emergency Service Water and Cooling Tower Makeup Intake Structure on the Main Reservoir serves as the backup source of emergency service water and the sole source of Cooling Tower makeup water. This structure draws water from the Main Reservoir through the Cooling Tower Makeup Water Intake Channel. The structure is protected from ice entering the pump bays by means of a concrete baffle wall which extends from the deck to one foot below the normal water level. In addition each pump bay has a coarse screen and traveling screen. The coarse screens and traveling screens are of the same description and have the same function as those for the Emergency Service Water Screening Structure described above. In addition, the traveling screens in the bays housing the emergency service water pumps are equipped with heated hoods. The pump suction inlet of all pumps in the Emergency Service Water and Cooling Tower Makeup Intake Structure is located more than 10 ft. below low water level. Therefore considerable icing would be required in order to affect the operation of these pumps.

Ice formation in the Emergency Service Water Discharge Channel cannot jeopardize emergency service water system performance since the elevation of the Emergency Service Water System discharge point is above the high water Level resulting from the PMF.

The plant will be shutdown and cooled down if ice formation in either intake channel would jeopardize the emergency service water supply.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

The safety related cooling water channels (canals), reservoirs, and water control structures within the reservoir system of the Shearon Harris Nuclear Power Plant consist of the Main Reservoir, the Auxiliary Reservoir, the Auxiliary Reservoir Separating Dike, the Auxiliary Reservoir Channel, the Emergency Service Water Intake and Discharge Channels, and the Emergency Service Water and Cooling Tower Makeup Intake Channel.

The design bases and operating modes of the reservoir system are described in relation to the safety-related Emergency Service Water System, Ultimate Heat Sink, and the Cooling Tower Makeup Water System; these discussions appear in Sections 2.4.11, 9.2.1, 9.2.5, and 10.4.5.

Shearon Harris Nuclear Power Plant complies with NRC Regulatory Guide 1.127 (refer to Section 1.8) and Ebasco Specification CAR-SH-CH-24, "Reservoir, Dams and Dike Instrumentation Program (Non-Nuclear Safety)." In addition, the North Carolina Utilities Commission requires a dam inspection program involving private consultants. As a minimum, the inspection program will include the water-control structures discussed in Section C.2 of Regulatory Guide 1.127. Periodic monitoring of embankment instrumentation will be performed. The Emergency Service Water Channels and Auxiliary Reservoir are monitored for sediment buildup.

The Shearon Harris Nuclear Power Plant reservoir system constitutes the only water bodies that are of concern regarding protection of plant facilities from flood and wave runup; discussion of the protection of channels and reservoirs is contained in Sections 2.4.2, 2.4.3, 2.4.4, and 2.4.5.

The only locations where potential blockage is of concern to safe plant operation are the Emergency Service Water Intake and Discharge Channels, and the Auxiliary Reservoir Channel. These channels are Category I structures and are designed to remain stable when subjected to the Safe Shutdown Earthquake or the most severe cases of other postulated natural phenomena (see Section 2.5.6). In the unlikely event of a slide of the earth slopes, the size of the channels is sufficient to pass the minimum required service water flow at a maximum velocity of 2 ft. per second under the conditions of maximum drawdown of the Main Reservoir and the Auxiliary Reservoir, as indicated in Section 2.4.11. Channel plans and sections are shown on Figures 2.5.6-6, 2.5.6-7, 2.5.6-8, and 2.5.6-28.

The use of screens for the Emergency Service Water Screening Structure and the Emergency Service Water and Cooling Tower Makeup Intake Structure, the location of the intake structures, and the maximum velocity of 2 ft. per second in the channels provide assurance that no blockage of the intake structures, damage to the intake structures or damage to the emergency service water pumps can occur.

The effects of failure of the Auxiliary Separating Dike are discussed in Section 2.4.4.

The design bases for reservoir operation during periods of low water level are discussed in Section 2.4.11.

Uncontrolled spillways at both dams are designed to provide release of flood waters so that the reservoir water levels do not exceed the design bases of the dams (see Section 2.4.3). There is no operational need for emergency storage evacuation of reservoir inventories, however, a low level release system is provided for the Main Reservoir to adjust reservoir discharge water

quality to assure that downstream release criteria are met. Incorporated into the Main Dam spillway, it consists of three (3) Howell Bunger valves located in the central pier and side abutments of the spillway. The valves have intakes in the reservoir at different elevations and locations. The arrangement is shown in Figures 2.5.6-1, 3.8.4-34, and 3.8.4-36 and the discharge capacity curves are shown in Figure 2.4.8-1.

The Howell Bunger valve in the central pier is a 24-inch valve with center line at El. 206.7 ft MSL. A 36-inch diameter steel pipe with intake at El. 195.0 ft MSL in the reservoir conveys water to the valve. The valves in the two abutments of the spillway are 36-inch valves with center lines at El. 213.0 ft. MSL. The intake for the West abutment valve is in the abutment at El. 213.0 ft MSL, whereas the East abutment has its intake inside the reservoir at El. 180.0 ft MSL, connected to the valve by a 48-inch diameter steel pipe.

2.4.9 CHANNEL DIVERSION

In view of the lack of historical evidence of any realignment of Buckhorn Creek, future realignment is considered to be extremely remote. Moreover, realignment in such a way that the runoff of the drainage basin would be diverted away from the reservoir system is impossible due to the contours of the basin.

Likewise, due to the topography of its valley, realignment of the Cape Fear River is also considered extremely remote. As explained in Section 2.4.1, the comprehensive development plan for the Cape Fear River Basin proposed by the Corps of Engineers will function to control flood flow on the river and rather than diverting flow, will ensure the availability of a 600 cfs minimum flow at Lillington.

Regardless of the continued availability of runoff to the reservoirs, the safety of the plant cannot be jeopardized by diverted flow. Operational commitments require shut down of the plant when reservoir water levels reach designated low points. At these low points there will still be sufficient water in the reservoirs to achieve safe shut down of the plant.

2.4.10 FLOODING PROTECTION REQUIREMENTS

The safety related facilities will not be affected by flooding resulting from reasonably possible combinations of the probable maximum flood (PMF), probable maximum wind (PMW), and the probable maximum precipitation (PMP).

Sections 2.4.3, 2.4.4, and 2.4.5 discuss the maximum water levels in the Main and Auxiliary Reservoirs and around the plant island, where most of the safety related facilities are located. Section 2.4.2.3 discusses the water level on the plant island due to the PMP.

The facilities located on the plant island will not be subjected to any flooding, as the plant grade, at Elevation 260 ft. MSL, is 2.3 ft higher than the maximum water levels around the plant island. The protections of structures on the plant island against the PMP are discussed in detail in Section 3.4.1.1. For the area between west face of Fuel Handling Building and the retaining wall see Section 3.4.1. Flood protection for safety related systems is discussed in Sections 3.4.1.

The Emergency Service Water Screening Structure, the Emergency Service Water Discharge Structure, and the Emergency Service Water and Cooling Tower Makeup Intake Structure are

designed so that their decks are above all calculated water levels. Since they extend down below normal water levels, they are also designed to withstand forces that could result from the worst postulated flood, wave, and wind conditions.

Safety related facilities other than those located on the plant island are the Main Dam, the Auxiliary Dam, the Auxiliary Reservoir Separating Dike, and the Auxiliary Reservoir Channel. The dams, dike, and channel are discussed in Section 2.4.4, 2.4.8, and 2.5.6. The top of the Main Dam is at Elevation 260 ft MSL, which is 16.9 ft above the maximum water level at the Main Dam. Therefore, the dam will not be overtopped. The top of the Auxiliary Dam is at Elevation 260 ft MSL, which is 2.0 ft higher than the maximum water level in the Auxiliary Reservoir at the Auxiliary Dam.

The Auxiliary Separating Dike, with its crest at Elevation 255 ft. MSL, will be subjected to overtopping due to waves generated by the probable maximum hurricane (PMH) wind (123 mph) acting on the normal reservoir level, Elevation 252 ft MSL or winds up to 54.4 mph combined with the PMF level, Elevation 256 ft MSL. The upstream and downstream slopes of the dike are protected by riprap, as discussed in Section 2.5.6. The failure of the Auxiliary Reservoir Separating Dike, as discussed in Section 2.4.4, will not result in loss of water or storage capacity and the Main Reservoir will function as a back-up source of cooling water to provide an adequate cooling circuit through the reservoirs.

As discussed in Section 2.4.4, the Main and Auxiliary Dams will not be subjected to any dynamic forces due to flooding, other than local wave action which is dissipated by the use of riprap. The dams and associated spillways have been designed for hydrostatic forces corresponding to the PMF levels in the two reservoirs.

2.4.11 LOW WATER CONSIDERATIONS

2.4.11.1 Low Reservoir Level

The synthesized flows of Buckhorn Creek for 1924 to 1981 were analyzed to determine the most critical low flow period. This period was determined to be a 19 month historical drought of May 1980 through November 1981. The actual analysis was carried out for May 1980 through May 1982 to demonstrate reservoir recovery.

Although the earliest synthesized Buckhorn Creek flow data dates back only to 1924, there are good precipitation records at the Raleigh National Weather Station which go back to 1867. A review of these records indicates that the lowest annual precipitation occurred in 1933, with a total of 29.93 in. Near record lows were also experienced in 1930, 1941, 1951, 1968, and 1977. It is conceivable, therefore, that the minimum flow experienced during the period of synthesized runoff may represent the lowest values of runoff dating back to 1867, the beginning of precipitation data. However, drought frequency analyses were made based on the period when flow could be synthesized from regional streamflow data, and therefore the results should be conservative.

2.4.11.1.1 Worst Critical Period

From a seven-year reservoir operation study for the period 1973 through 1980, the mean water level in the Main Reservoir is 219.4 ft. MSL as discussed in the SHNPP Environmental Report, Section 2.4.2.3.2.2.3. This elevation varies depending on actual inflow conditions, consumptive

use, and downstream releases. Calculations were made to determine the reservoir level fluctuation during the worst drought period of record for Buckhorn Creek under the worst monthly evaporation conditions. Table 2.4.11-18 provides results of analysis for the period May 1980 through May 1982 with the assumption that the reservoir was at the minimum predicted normal operation level at 216.3 ft. MSL.

The total annual evaporation of 63.67 inches for the worst meteorological conditions was used in Table 2.4.11-18. This estimate of natural evaporation was obtained from a method of maximization using the meteorological data shown in Table 2.4.11-3. The basis for selecting these monthly meteorological data was the maximum monthly average temperature of record at the first-order National Weather Station at Raleigh, North Carolina during the period 1931-1970. Plate 2 of the Environmental Science Service Administration (ESSA) Climatic Atlas (Reference 2.4.11-2) shows the mean annual lake evaporation in the Raleigh area to be approximately 41.0 in. Plate 3 of the Atlas shows a mean annual Class A pan coefficient of 75.5 percent for the site area, and Plate 5 indicates a standard deviation of annual Class A pan evaporation of about 3 in. The estimates of evaporation under normal meteorological conditions of 53.35 in. and under worst meteorological conditions of 63.67 in. are considerably more conservative than the 41 in. shown on Plate 2 of the Atlas. Another direct indication of the degree of conservatism in these estimates is provided by a calculation of the number of standard deviations from the average evaporation value of 41 in. The estimated normal evaporation of 53.35 in. under normal meteorological conditions is 4 standard deviations above the Atlas average. The evaporation under worst conditions (63.67 in.) represents 7.5 standard deviations above the Atlas average.

The period had a 4-month average flow of 10.5 cfs, a 7-month average of 11.5 cfs and a 12-month average of 25.1 cfs and an average flow over the drought period of 26.6 cfs (DA = 71 sq. mi.).

The maximum storage use was 34833 ac. ft. and the minimum water level was 209.4 ft. MSL.

2.4.11.1.2 100-Year drought

The average 4-month, 7-month, and 12-month flow of Buckhorn Creek corresponding to the 100-year drought discussed in Section 2.4.11.2 was utilized in the analysis. Evaporation rates are for worst monthly evaporation conditions. As shown in Table 2.4.11-19, the maximum Main Reservoir storage utilized for this extreme drought condition would be 30342 ac. ft. drawing the reservoir down to Elevation 211.0 ft. MSL.

As discussed in Section 2.4.11.7, the Tech Spec minimum Main Reservoir Level is 206 ft. to ensure flow requirements to safety related heat exchangers, cooled by Emergency Service Water, are met. The unit will be shutdown if the water level in the Main Reservoir falls below 206 ft.

A study was performed to address the statistical probability of Main Reservoir level falling below a level of 215.0 ft. MSL. A Main Reservoir impoundment analysis was performed using historical synthesized monthly average flows of Buckhorn Creek taken from Table 2.4.1-1 and Table 2.4.11-18. From this study, average monthly reservoir levels were calculated for the period from 1923 to 1982 (713 months). These monthly levels generally trended down each summer and fall and then recharged to normal pool levels during the winter and spring. This coincides with the normal climatological precipitation patterns for this region. However many months experienced low precipitation and occasionally this occurred for many consecutive

months. During a few of these abnormally lengthy dry periods the Main Reservoir level actually dropped below elevation 215.0 ft. and is summarized below:

1. October 1951 through January 1952 (4 months)
2. July 1981 through January 1982 (7 months)

Both of these dry periods were preceded by an unusually dry winter and spring whereby the reservoir was not recharged to normal pool level. Then during the climatologically drier summer and fall, the reservoir levels dropped quickly to less than 215.0 ft. The lowest level calculated was in November 1981 when the reservoir fell to 212.9 ft.

The monthly average reservoir levels for each of the 713 months in the study were evaluated. The calculations showed there is less than a 1.0% chance for any given month that the Main Reservoir level will fall below a level of 215.0 ft. MSL.

2.4.11.2 Low Flow in Streams

Short duration minimum flow in Buckhorn Creek has little effect on the project due to the large storage in the Main Reservoir. Therefore, isolated periods of less than four months were not considered. Drought periods of four months and seven months were determined for the period of synthesized record of Buckhorn Creek from 1924 to 1981. The two worst drought periods were the periods February 1951 through January 1952 and August 1980 through July 1981. The 1950-1951 period had a 4-month average of 5.8 cfs, a 7-month average of 8.5 cfs, and a 12-month average of 26.5 cfs at the Main Dam. The 1980-1981 period had a 4-month average low flow of 10.5 cfs, a 7-month average low flow of 11.5 cfs and a 12-month average low flow of 25.1 cfs.

Based on the available data and the principle of the Markov chain process, synthesized flow of Buckhorn Creek was generated according to a statistical method described in Reference 2.4.11-2. The frequency analysis of the synthesized flow was then utilized to estimate a severe drought having a return period of 100 years. The following gives the average minimum flow for the various 100-year return period drought durations:

	Buckhorn Creek Flow at Main Dam (DA=71.0 sq.mi.) (cfs)
Average minimum 4-month flow	3.7
Average minimum 7-month flow	6.9
Average minimum 12-month flow	23.2

2.4.11.3 Low Water Resulting From Surges, Seiches, and Tsunamis

The water level in the Auxiliary Reservoir is maintained at a minimum elevation of 250 ft. MSL by pumping water from the main reservoir. Since the bottom of the Emergency Service Water Intake Channel is at Elevation 238 ft. MSL, a minimum water depth of 12 ft. will be maintained in the channel. During the probable maximum hurricane (PMH), the maximum wave height would be 5.4 ft. and the maximum wind set-up would be 0.4 ft. (see Table 2.4.5-1). Therefore the lowering of the water level resulting from a PMH induced surge in comparison with the 12-foot

depth of intake water would not significantly affect the intake capacity of the Emergency Service Water System.

The normal operation water level in the Main Reservoir is at Elevation 220 ft. MSL. Since the bottom of the Cooling Tower Make-Up Water Intake Channel is at Elevation 194 ft. MSL, this 26-foot difference provides sufficient margin to withstand the lowering of the water level in the Main Reservoir by surges due to the PMH because the resulting wave height would be 6.7 ft. and the wind setup would be 1.0 ft. (see Table 2.4.5-2).

Due to its inland location the site is not subject to tsunamis (see Section 2.4.6).

As the site is located in a region of minor seismic activity and less severe meteorological events, the occurrence of a seiche is a remote possibility. In the unlikely event of a seiche, the wave amplitude would be such that surges due to the PMH described above still present the worst case.

The absence of expected icing concerns and the design consideration given to unlikely ice formation in the reservoir system is discussed in Section 2.4.7.

2.4.11.4 Historical Low Water

Historical low flow data for Buckhorn Creek were derived from the recorded data of Middle Creek because only limited low flow data during isolated periods is available for Buckhorn Creek prior to June 1972, as described in Section 2.4.1.2.1.1. Based on the drainage areas ratio, the calculated minimum flow data for Buckhorn Creek from 1940 to 1978 are presented in Table 2.4.11-12. Calculated data, based upon actual measured data as adjusted by drainage area relationships, are also shown in this table for periods starting with 1973, the first whole water year following the establishment of a USGS gage station. As can be seen from this table, Buckhorn Creek has experienced some periods with no flow. The frequency analyses for 1, 7, 30, and 60 consecutive days low flow are shown on Figures 2.4.11-2, 2.4.11-3, 2.4.11-4, and 2.4.11-5, respectively. The water level in the Main Reservoir associated with the drought periods on record has been studied in detail; the minimum water level elevation (conservatively based on four unit operation) following a subsequent 30-day emergency condition, is 204.4 ft. MSL as discussed in Table 2.4.11-14.

2.4.11.5 Future Controls

No future uses of Buckhorn Creek which could significantly affect the natural creek flow are foreseen at this time. The minimum flow conditions for the creek are presented in Section 2.4.11.2.

Since the ability of safety related facilities to function adequately under drought conditions has been confirmed in Section 2.4.11.1, provisions for stream flow augmentation for plant use are not required.

2.4.11.6 Plant Requirements

Safety related water requirements for the plant are supplied by the Emergency Service Water System, described in Section 9.2.1. The Emergency Service Water System is a once through, open cycle design with respect to the reservoir system of SHNPP.

Service water during normal plant operation is supplied from and returned to the Cooling Tower. The make-up requirement to the Cooling Tower from the Main Reservoir constitutes a major plant use during normal operation; Cooling Tower make-up is a maximum of 16,000 gpm operating at peak evaporative rates (12,000 gpm evaporation, 4,000 gpm blowdown, 10 gpm drift).

Additional quantities of reservoir water pumped by the Cooling Tower make-up pumps are diverted for use as make-up to the Water Treatment System (200 gpm), as Auxiliary Reservoir makeup (2,000 gpm), by the screen wash pumps (540 gpm), and by the self-cleaning strainers of the trash collection system (360 gpm). The total withdrawal from the Main Reservoir is therefore a maximum of 19,100 gpm.

However, Cooling Tower blowdown (4,000 gpm), screen wash water, and strainer backwash are returned to the Main Reservoir. The net consumptive use of reservoir water is therefore 14,200 gpm for the plant under maximum evaporative conditions, assuming all secondary services of the Cooling Tower make-up pumps are required simultaneously.

During emergency service water operation, the Auxiliary Reservoir is the preferred source of water and the discharge is returned to the Auxiliary Reservoir. When the Auxiliary Reservoir is not available for supplying emergency service water, the Main Reservoir will serve as the backup source with discharge to the Auxiliary Reservoir.

The safety related cooling water flow requirement for various modes are provided in Table 9.2.1-1. The availability of water during the 30-day period following an accident is discussed in Section 2.4.11.7 and Table 2.4.11-14. Figures 3.8.4-25 through 3.8.4-31 show the intake structures including sump invert elevation and configuration, minimum design operating water level, and pump submergence elevation.

The thermal effluent released during emergency shutdown will have a temperature not exceeding 120°F, while the radioactive release is maintained as low as reasonably achievable. The effluent will be discharged into the Auxiliary Reservoir through the Emergency Service Water Discharge Channel. The design bases for effluent mixing and dispersion from the Service Water System under emergency conditions are the same as for the Circulating Water System, described in Section 2.4.12.

The 100-year drought water level elevation in the Main Reservoir has been incorporated in the study of the previously mentioned 30-day period following an accident (see Table 2.4.11-14).

2.4.11.7 Heat Sink Dependability Requirements

The ultimate heat sink is a complex of water sources, including associated retaining structures and any canals or conduits connecting the sources with the cooling water intake structures of the plant. The sources of water for the normal and emergency cooling modes are the Cooling Tower and the Main or Auxiliary Reservoirs, respectively. The retaining and conveyance systems consist of the Main Dam, the Auxiliary Dam, the Auxiliary Reservoir Separating Dike, the Auxiliary Reservoir Channel, the Emergency Service Water Intake and Discharge Channels, and the Cooling Tower Makeup Water Intake Channel. These facilities are described in Sections 2.4.4 and 2.4.8.

The normal source of plant service water is the circulating water Cooling Tower (see Section 9.2.1). The preferred source of emergency cooling water, if the non-safety related Cooling Tower or its associated components are not available, is the Auxiliary Reservoir. The backup source of emergency service water is the Main Reservoir if the Auxiliary Reservoir is not available. Emergency Service Water from either source is discharged to the Auxiliary Reservoir.

The Auxiliary Reservoir will perform its function as the ultimate heat sink in the event of a loss of service water from the Cooling Tower. Figure 2.4.3-6 shows the area-capacity curve for the Auxiliary Reservoir. A minimum level of 250 ft. Elevation in the Auxiliary Reservoir is maintained at all times during normal operation by creek inflow above the Auxiliary Dam and by pumping water from the Main Reservoir. The Main Reservoir is normally maintained at a level of 220 ft. MSL, but may decrease to 206 ft. MSL during normal operation. The UHS analysis uses the Main Reservoir level of 205.7 ft. as the starting point to determine final UHS temperature and level and if an adequate volume of water exists to remove the heat generated by the plant.

Level was determined to decrease to 203.6 ft. MSL during emergency conditions. While at their minimum normal operation levels, both the Auxiliary and Main Reservoirs, taken separately, are more than adequate to permit emergency shutdown and cooldown of the plant.

The Main Reservoir will function as a cooling reservoir in the case where the Auxiliary Reservoir is not available. The Auxiliary or Main Reservoir will function as cooling reservoirs during an emergency.

Emergency and normal shutdown heat loads are discussed in Section 9.2.1. The service water system heat exchanger design basis temperature is 95°F, and the estimated maximum service water system inlet temperature for the postulated conditions of emergency shutdown and cooldown is 94.2°F. 94.2°F is acceptable as a maximum pre-accident temperature since the Ultimate Heat Sink analysis does not account for thermal stratification which would result in a maximum, post-accident (30-day) pump suction temperature below 95°F (Ref. SW 0085). The meteorological conditions postulated in this case are considered to be the most critical conditions for maximum inlet temperature. The basis for these conditions is a statistical analysis of the meteorological parameters directly related to water temperature; that is, solar radiation, ambient air temperature, dew point temperature, and wind speed. The meteorological conditions that maximize the service water temperature are high solar heating, high ambient air temperature, high relative humidity, and low wind speed. The worst meteorology for one day occurred on June 27, 1952, and the worst month occurred between July 18 and August 15, 1949. The average values of these meteorological parameters for 1-day to 30-day periods were calculated based on these days to estimate the maximum service water system inlet temperature. The limiting analysis calculated a maximum pre-accident reservoir temperature of 94.2°F using a composite 10-day period including the worst 9-day consecutive meteorology (7/22/49 - 7/30/49) plus the worst 1-day (6/27/52). The auxiliary Reservoir initial water temperature of 82.2°F is assumed in the analysis based on the July reservoir equilibrium water temperature for the normal meteorological conditions. The analysis also found that a pre-accident reservoir temperature of 94°F would result in a final, 30-day, post-LOCA temperature of 95.17°F, which is just slightly above the ESW design-basis temperature of 95°F. This is acceptable because the reservoir analysis does not account for thermal stratification which would result in a pre-accident temperature below 94°F and a post-accident temperature below 95°F. The analysis accounts for auxiliary reservoir inventory loss via valve seat leakage into a completely empty main reservoir. (Reference Calc SW-0085.)

The normal meteorological conditions of each month for the site area are presented in Table 2.4.11-2 and the corresponding reservoir equilibrium water temperatures (in °F) are:

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
39.6	43.2	52.6	62.9	73.1	79.5	82.2	80.7	74.3	62.2	49.5	39.1

The consumptive use of water during the emergency period is a function of plant heat rejection and evaporation. Using the most severe monthly evaporation rates based on the worst single month meteorological data (July, 1932, listed in Table 2.4.11-3), the total drawdown of the Auxiliary Reservoir for the 30-day emergency period was computed for accident conditions for one unit. In estimating the evaporation rate, the Auxiliary Reservoir initial water temperature was conservatively assumed to be 95°F.

For the purpose of the calculation, rainfall and inflow are assumed negligible during the emergency period. The Auxiliary Reservoir water level decreases only 1.5 ft. Therefore, the reservoir provides an adequate supply of water during the emergency period.

The analysis is performed to demonstrate the adequacy of the Main Reservoir to provide at least a 30-day supply of water for emergency shutdown and cooldown of one unit in the event that the level in the Main Reservoir drops to Elevation 205.7 ft. MSL. This analysis is based upon the assumptions that:

1. Rainfall during the 30-day emergency period is zero,

2. Buckhorn Creek inflow is zero,

3. The auxiliary reservoir water level is maintained at a minimum elevation of 250 ft. MSL during normal operation, and the spillover elevation to provide the spill rate of 200 cfs into the Main Reservoir is 252.5 ft. MSL.

4. Worst single month meteorological data (July 1932) is used for the maximum evaporation.

5. The Main Reservoir and the Auxiliary Reservoir initial water temperatures are 95°F.

The Main Reservoir water level at the end of the 30-day emergency period would be Elevation 203.6 ft. MSL which is above the minimum operating level of the Service Water and Cooling Tower Make-up Water Pumps.

The UHS analysis uses the Main Reservoir level of 205.7 ft. as the starting point to determine final UHS temperature and level, and if an adequate volume of water exists, to remove the heat generated by the plant. However, to ensure the flow requirements for safety related heat exchangers cooled by Emergency Service Water (see Section 9.2.1) are met, the UHS minimum Main Reservoir level is 206 ft.

Ample warning of low water level in the Main Reservoir will be available since the low water conditions produced by the 100-year drought would be of such unusual severity that the conditions will be obvious weeks prior to reaching MSL critical level. Plant operators, especially, will be cognizant of the condition in the Main Reservoir due to the unusual make-up requirements of the Auxiliary Reservoir during these weeks. In addition, a level transmitter is

located at the Emergency Service Water and Cooling Tower Make up Water Intake Structure which alarms in the Control Room when the water level in the Main Reservoir decreases to Elevation 207 ft. MSL.

Section 2.4.11.1 discusses the reservoir capacity for heat dissipation and the assumed concurrent water losses under normal operation and emergency conditions, respectively. For the description of compliance with Regulatory Guide 1.27, see Section 1.8.

Other than service water, the Auxiliary Reservoir is the source of firefighting water and certain non-safety related HVAC cooling water makeup. It is not postulated that a fire and an accident that requires plant shutdown would occur simultaneously. The firefighting water system has a design capacity of 2500 gpm, and it is activated only if a fire occurs. HVAC cooling water makeup occurs on an intermittent basis and requires a maximum of 100 gpm. These requirements are considered insignificant.

Section 2.4.4.1 discusses the capability of each (Main & Auxiliary) dam to withstand the failure of the other dam.

Volumes of potential sediment deposit from Buckhorn Creek were determined to be on the order of 460 and 20 ac-ft., respectively, in the Main and the Auxiliary Reservoirs for the length of the plant life of 40 years. These amount only to 0.7 percent and 0.4 percent of the respective reservoir capacities at their normal water levels.

The USGS sediment sampling data for Buckhorn Creek at Corinth, North Carolina were analyzed to obtain a sediment rating curve, as shown on Figure 2.4.11-9. This rating curve was combined with 40 years of synthetic daily streamflow to estimate the quantity of direct sediment inflow from Buckhorn Creek. To generate the synthetic daily streamflow data, the streamflow records of Buckhorn Creek were supplemented with the records of Middle Creek at Clayton, North Carolina and two computer programs (References 2.4.11-4 and 2.4.11-5) were used.

In view of the small quantities predicted, the effects of sediment deposit on reservoir operations and cooling capacities will be negligible. The critical levels of reservoir drawdown during drought or emergency periods discussed above will be essentially unaffected because the sediment aggregation found for the reservoirs is not significant enough to cause any appreciable changes to the reservoir area-capacity curves. The same argument applies to the effects of sedimentation on reservoir cooling capacities.

During the License Renewal review, the evaluation of sedimentation in the Main and Auxiliary Reservoirs was considered. Sedimentation effects were determined to be negligible and would be managed by monitoring activities during the period of extended operation. Refer to Chapter 18 for additional information.

2.4.12 DISPERSION, DILUTION AND TRAVEL TIME OF ACCIDENTAL RELEASES OF LIQUID EFFLUENTS IN SURFACE WATERS

Accidental discharges of radionuclides to the surface waters could conceivably occur due to failure of storage tanks which contain potentially radioactive liquids or due to inadvertent discharge of liquid waste to cooling tower blowdown. Inadvertent discharges are precluded by automatic termination of liquid waste discharge on high radiation signal (see Section 11.2).

2.4.12.1 Storage Tank Failure.

The contents in the tanks located in either the Waste Processing Building (Figures 1.2.2-47 through 1.2.2-54) or the Tank Building (Figure 1.2.2-84) are potentially radioactive.

The Waste Processing Building is a Seismic Category I concrete structure which contains such tanks as the floor drain tank, laundry and hot shower tank, and waste hold-up tanks. The building is designed to retain the liquid contents of all tanks containing potentially radioactive liquids. The tanks are located below plant grade, and therefore their failure would not result in runoff to the surface waters.

The Seismic Category I Tank Building contains large capacity tanks, such as the refueling water storage tank (RWST), the condensate storage tank, and the reactor makeup water storage tank (RMWST). Each of these tanks is housed in an individual compartment capable of retaining the tank contents.

As discussed in Section 11.2, an inadvertent release of radionuclides via the normal liquid effluent release pathways will be automatically terminated. For the purpose of investigating the effects of an accidental release of liquid effluents in surface waters, the unlikely failure of a storage tank and its concrete enclosures is postulated to occur.

The RWST has both the highest specific and total activity of the three tanks located above grade in the Tank Building. Table 2.4.12-1 lists the concentrations of nuclides contained in the refueling water storage tank.

Any accidental releases into the Main Reservoir are afforded dilution by the Main Reservoir, Buckhorn Creek, and the Cape Fear River before reaching Lillington, the location of the closest downstream surface water user (see Tables 2.4.1-5 and 2.4.1-6). Any spillage from the plant island would enter the reservoir in its northern reaches and would discharge into the Cape Fear River of the Main Dam Spillway at the reservoir's extreme southern end. Any spillage would essentially achieve complete mixing before being discharged since the flow through the reservoir is relatively slow.

Even though it is not considered likely, if the entire contents of the RWST (470,000 gals, see Table 6.2.2-9) were released to the lake instantaneously, the dilution provided by the volume of water in the Main Reservoir, conservatively chosen to be 62,000 acre feet (volume of the Main Reservoir when two ft. below normal operating level), would be on the order of 10^5.

Also, note that the following assumptions add to the conservatism of the analysis. The radionuclide content in the RWST was maximized by assuming the reactor coolant activity to be based on 1 percent failed fuel. The design basis activity in Table 11.1.7-1 is also homogeneously mixed in the RWST. Further, it was assumed that the Spent Fuel Pool Filtration System was not operating during refueling. The instantaneous mixing of the tank contents in the reservoir is also a conservative assumption. The RWST is located approximately 1500 ft. from the reservoir inside the Tank Building. Even spilled liquid would have to travel 1500 ft. overland where some absorption and retention of radionuclides in the soil would take place. Instantaneous mixing removes from consideration the travel time (and consequently the radioactive decay) for the water to migrate to the reservoir and neglects the possibility that portions of the waste would be prevented from reaching the reservoir by retention in the soil. If

the real case were analyzed, absorption and retention would lessen the amount of activity present in the reservoir at any given moment.

Specific concentrations at Lillington, provided in Table 2.4.12-1, show the results of plant operation (reservoir volume - $2.92x10^9 ft^3$ and reservoir discharge = 43 cfs). The reservoir flow was diluted by the minimum annual average flow of 1350 cfs (occurred in 1981, see Table 2.4.1-3) in the Cape Fear River. Any natural runoff into Buckhorn Creek between the Main Dam and its confluence with the Cape Fear River was conservatively assumed to be negligible. It can be seen from Table 2.4.12-1 that the C/MPC at Lillington is well below the allowable concentrations of 10 CFR 20.

2.4.13 GROUNDWATER

The project is located essentially within the Buckhorn Creek watershed of the Piedmont providence near the Fall line, the physiographic limit between the Coastal Plain and the Piedmont Plateau, in east-central North Carolina. Buckhorn Creek is a tributary of the Cape Fear River. The towns of Bonsal, New Hill, and Corinth are located within the watershed. Based on Raleigh Durham Airport (RDU) data, the average annual temperature in the area is 59.1 F and the mean annual precipitation is 42.5 in. This section presents groundwater conditions, sources, and usage of the aquifer in the area and at the site (Figure 2.4.1-1).

2.4.13.1 Description and Onsite Use

2.4.13.1.1 Regional Groundwater Conditions

The entire drainage area of Buckhorn Creek northwest of the Jonesboro Fault is underlain by Triassic rocks of the Newark Group. The drainage area of Buckhorn Creek that is located southeast of the Jonesboro Fault is relatively small and is underlain by Paleozoic crystalline rocks and igneous intrusives, as well as metamorphic rocks of the Carolina Slate Belt. Both the Triassic and Pre-Triassic rocks are overlain by clayey soils and saprolite.

a) Overburden - The plant area is covered with residual soils derived from the underlying rocks. Numerous soil borings drilled at the plant island, as well as in the Auxiliary Reservoir area, confirm the existence of up to about 15 ft. of clayey soil and saprolite overlying the Triassic rocks.

Excavation and mapping of trenches in the plant site area, as well as excavation and borings for the Site Fault Investigation (Reference 2.4.13-1), also indicate the preponderance of clayey and silty loam soils.

b) Triassic Rocks - The source of groundwater in the area is the rock units of the Sanford Formation of the Newark Group (Triassic). They consist of claystone, shale, siltstone, sandstone, conglomerate, and fanglomerate. An exception to this lithology is the intrusion of thin diabase dikes in the rock (Reference 2.4.13-1 and 2.4.13-2). The dikes were mapped in connection with the site fault investigation in the plant and Auxiliary Dam areas (Reference 2.4.13-1). The diabase rock is weathered near the surface and is unweathered below depths of about 20 ft.

The primary permeability of the Triassic rocks is very low and the rocks appear to be essentially dry. Some lenses of relatively higher permeability exist within the Triassic rocks.

However, they are not extensive and are surrounded by materials with relatively lower permeability. The Triassic rocks have fractures resulting from stress releases; the fractures provide secondary permeability in the rocks and are filled with water below the water table. The fractures are common to depths of about 100 ft., but become less prevalent and are tight below that depth. Below about 400 ft., the fractures are closed and sealed to water flow, as shown by tests and by experience gained through well drilling in the area. Recharge in the area occurs by percolation of precipitation through the overburden; however, most of the precipitation is either returned to the atmosphere through evapo-transpiration or becomes surface runoff. The predominance of surface and near-surface deposits with extremely low permeability results in rapid runoff of precipitation. Therefore, natural recharge to the aquifer occurs at a very low rate.

The precipitation which percolates downward is confined laterally by the diabase dikes and vertically by the absence of open fractures or joints at depth in the Triassic rocks. Numerous attempts to develop groundwater supplies from deep Triassic rocks have not been successful since these rocks are tight and relatively dry. However, groundwater is developed in the Triassic basin from hornfels zones adjacent to diabase dikes. The relationship of dikes and fractures to groundwater flow is illustrated diagrammatically on Figure 2.4.13-3.

The use of groundwater in the site region is limited because of the low yield of the aquifer; most of the wells are for domestic use. A few small towns in the area (see Table 2.4.13-1 and Figure 2.4.13-4) use the Traissic rocks as a source of water. However, total groundwater usage is still small, as discussed in Section 2.4.13.2.

2.4.13.1.2 Site Groundwater Conditions

Investigations conducted at the site reveal that geologic and hydrologic conditions in the site area are essentially the same as the regional conditions described in Section 2.4.13.1.1.

The plant site is located on a ridge bounded by Thomas Creek to the east, Tom Jack Creek to the west, and White Oak Creek to the southeast. These creeks are tributaries of Buckhorn Creek, which enters the Cape Fear River about seven miles south-southwest of the site. The plant site has been graded to Elevation 260 ft. msl. The pre-graded site elevations ranged from about 210 ft. to 280 ft. msl; the land surface generally sloped towards the east and southeast.

a) Top Layer - The Soil Conservation Service soil survey of Wake County, 1970, classified the site soils as the Creedmoor-White Store Association (Reference 2.4.13-3). Some typical engineering properties of the Creedmoor White Store soil series, as mapped in the site area and taken from the Soil Conservation Service soil survey of the Wake County, 1970, are listed below. They indicate that the Creedmoor-White Store soil conditions are relatively impervious. The surficial clay and saprolite zones prevent ready recharge to the rocks below them, as indicated by the dry state of the rocks (Reference 2.4.13-1).

CREEDMOOR-WHITE STORE ASSOCIATION

CREEDMOOR SOIL (Typical Profile)

Depth (in.)	Texture	Percentage Passing Sieve No. 200 (0.074 mm)	Permeability (in./hr.)	Shrink-Swell Potential

0-12	sandy loam	30-45	2.0 - 6.3	Low
12-29	clay loam	35-85	0.63 - 2.0	Moderate
29-58	clay	70-95	0.2	High
58-96	clay	35-90	0.2	Moderate

b) Triassic Rocks - The plant site and peripheral lands are underlain by Newark Group rocks (Triassic). The tightness of the rock formations, as a result of compaction and cementation, is evident from most of the cores extracted from the borings. Surface percolation of precipitation is controlled by the location of joints and fractures in the rocks. The low permeability of the Triassic rocks suggests that any movement of groundwater within the rocks also will be controlled by the interconnecting patterns of joints and fractures. Groundwater at and around the site occurs principally within jointed rock, generally at depths of 30 to 90 ft. below the original ground surface. Within the Newark Group, larger reserves of groundwater occur in the proximity of diabase dikes. Several small dikes were found in the plant area; groundwater supplies for use during plant construction have been developed in the proximity of the dikes.

Seven wells with a total capacity of about 200 gal./min. were completed during 1973 and are being used during the construction phase. Additionally, eight new wells, which increased the total capacity of all wells to about 450 gal./min., were developed in the proximity of diabase dikes during 1977-1979; three more in 1980 and two more in 1981 (Table 2.4.13-2).

2.4.13.1.3 Onsite Use of Groundwater

Site wells are listed in Table 2.4.13-2 and are shown on Figure 2.4.13-1. Groundwater is being used at the site during the construction phase for (1) concrete batch plant and concrete placement, (2) office and plant use, and (3) grouting. Groundwater is not expected to be used for plant operation after the plant potable water system becomes operational. Estimated monthly groundwater consumption at the site for March, 1978, through February, 1980, is shown in Table 2.4.13-3. The estimated plant water requirements through the year 1982 are shown in Table 2.4.13-4. Carolina Power & Light Company is the principal user of groundwater within two miles of the plant; there are only two domestic users within two miles of the plant, and both are up gradient near the 7,000 ft. radius boundary.

2.4.13.2 Source

2.4.13.2.1 Regional Use of Groundwater

Public wells within ten miles of the site are listed in Table 2.4.13-1 along with their maximum pumpage rates, water levels, and use; Figure 2.4.13-4 shows the locations of the wells.

The nearest communities that are using groundwater for public water supply are Holly Springs and Fuquay-Varina; both are in Wake County. Holly Springs, about seven miles east of the plant site, has two wells which supply a total of about 40,000 gallons per day. Fuquay-Varina, about ten miles southeast of the plant site, has eight wells which supply a total of about 400,000 gallons per day. The wells produce water from crystalline rocks of the Carolina Slate Belt; in the plant area, the same crystalline rocks are buried a few thousand feet beneath the Triassic sediments. The Holly Springs and Fuquay-Varina wells are not located in the Triassic Basin.

The closest community downstream of the plant site is Corinth, approximately five miles to the southwest, where a group of houses have individual wells of minimal production from the Triassic, Newark group aquifer. The well depths range from 62 ft. to 140 ft., while their production varies from 0.5 to 13 gpm. The relative yields of the wells at Corinth are all less than 0.10 gpm per foot of uncased hole.

2.4.13.2.2 Groundwater Levels and Movement

A piezometric-level map (Figure 2.4.13-1), based on water-level measurements taken before commencement of full-scale plant construction, shows that the general groundwater movement in the plant area at that time was to the southeast toward White Oak Creek. Most of the original site-area piezometers have been lost due to construction activities; therefore, sixteen new piezometers were constructed in December, 1979. A piezometric-level map, based on water-level readings in production wells, in the sixteen new piezometers, and in two old piezometers, taken during the winter of 1979-1980, is shown on Figure 2.4.13-2. The map is based on the highest water levels observed during this period (Tables 2.4.13-5 and 2.4.13-6) and they do not necessarily represent static water levels. Figure 2.4.13-2 shows that the general direction of groundwater movement at the site is still to the southeast toward White Oak Creek. However, the water levels have been significantly altered due to the ongoing pumpage from the site wells. Figure 2.4.13-2 shows that cones of depression have developed on the northeastern and southwestern sides of the plant. Three of the piezometers have since been abandoned and one piezometer was destroyed as indicated on Table 2.4.13-2.

2.4.13.2.3 Aquifer Characteristics

The Triassic rocks underlying the plant site constitute the principal aquifer in the area of the plant and reservoirs. The thin layer of overburden overlying the Triassic rock consists of clayey soils and saprolite which yield little or no usable groundwater.

The Triassic rocks of the aquifer are quite thick and widespread in extent. However, because of compaction and cementation of individual rock layers, it can be regarded only as a minor aquifer. Yields from known wells in the area generally range up to 20 gpm, but average only about 5 gpm or about 0.03 gpm/ft. of well (References 2.4.13-2 and 2.4.13-4). Generally, the principal areas of groundwater storage in the Triassic Basin are found near diabase dikes which have intruded the Triassic sediments. Twelve wells which were developed in the proximity of the dikes in the site area are providing water for use during construction of the plant.

Even though Triassic rocks constitute the major groundwater source within the site environs, they exhibit very low permeability for groundwater storage and movement. Of the 57 wells with an average depth of 158 ft. that have been constructed in the Triassic formation in western Wake County, 16 percent yield less than 1 gpm, while the average production rate is 5 gpm. Such relatively low permeability also explains why the Triassic formation is the lowest producing groundwater source in the region (Reference 2.4.13-2). Numerous borings carried out for soils and geologic information in the plant site and reservoir areas confirm the very low permeability of the Triassic formation. The permeabilities of materials in the Auxiliary Reservoir and SHNPP site areas, based on pump-in packer tests, are summarized in Table 2.4.13-7.

Six site wells located in the proximity of diabase dikes yielded specific capacity values from 24-hour driller's tests that range from 0.16 gpm/ft. to 0.59 gpm/ft. The specific capacity values correspond to transmissivity values of about 40 $ft.^{2/}$day to 130 $ft.^2$/day (Reference 2.4.13-5).

The beds below the surface clay and saprolite zones appear to have two distinct components of permeability. There is a very low permeability in the materials themselves. The second component of permeability comes from fractures that have resulted from stress release. This is the principal component which is measured as permeability during pump-in tests at the site.

The fractures in the Triassic rocks are filled with water below the water table. The fractures are common to depths of 100 ft. but become less prevalent and tight below that point. Below 400 ft., the fractures are closed and sealed to water flow, as shown both by tests and by experience gained through private well drilling in the area. This fracture relationship is illustrated on Figure 2.4.13-3.

Small amounts of groundwater were encountered in some trenches where fractured rock was evident. After their excavation, the trenches continued to hold surface runoff due to the low permeability of the fine-grained soil and rock materials.

Down-hole pressure testing of the soils and of the Sanford formation was carried out in borings located in the plant site area and in the main dam area (see Sections 2.5.1 and 2.5.6). In the plant site area, 10 ft. intervals were tested under pressures up to 110 psi in borings BP62, BP68, and BP70 (Figure 2.5.1-14). Intervals tested ranged from depths of 10 ft. to 145 ft. Several isolated zones registered small water losses under high pressure.

The results of the pressure tests, coupled with the soil conditions observed and mapped in the trenches, confirm that the soils and foundation materials and the permeabilities listed in Table 2.4.13-7 are representative of those found at the site. At the plant site area, the few zones that exhibited small water losses during pressure testing were isolated intervals that are located between dense, impervious rock layers which registered no water losses during pressure testing. The impermeable zones ranged in thickness from 10 to 50 ft. above and below each interval that had a water loss.

Hydrogeologic information from borings and published data indicate that the small water losses in the above mentioned borings were due primarily to fracture confluence instead of formation texture or permeability changes in the Sanford formation of the Newark Group.

2.4.13.2.4 Effects of Groundwater Usage

The population in the vicinity of the plant is small and groundwater usage is minimal due to low yields of wells. Most of the land within a two-mile radius, and some beyond this distance, has been acquired by Carolina Power & Light Company. Therefore, the population in the plant vicinity is not likely to increase much and groundwater usage will remain essentially the same.

The yield of the Triassic aquifer is low and only a limited supply of groundwater is obtainable from the proximity of diabase dikes. Therefore, any increase in groundwater usage will be limited because of the poor permeability and storage characteristics of the aquifer.

Groundwater is being utilized at the site during construction. Table 2.4.13-4 shows the total site groundwater use for the years 1980 through 1982.

Site groundwater usage is expected to gradually decrease due to the decline in construction activities.

Figures 2.4.13-1 and 2.4.13-2 compare the pre-construction piezometric level to that existing during the winter of 1979-1980. The groundwater levels, which have been affected considerably, are declining due to pumpage from the site wells. Cones of depression have developed on the northeastern and the southwestern sides of the plant around wells which are being pumped. Directions of groundwater movement have been reversed in the proximity of some wells, as depicted by the cones of depression. The levels are expected to return to near normal as the construction use of water declines.

The reservoirs at the Shearon Harris Nuclear Power Plant site comprise a total of approximately 4,417 acres in surface area and contain approximately 77,500 acre-feet of water at the normal pool elevations. The main reservoir operating level is Elevation 220 ft. MSL, and the elevation of the Auxiliary Reservoir is Elevation 252 ft. MSL. Three main cones of depression have groundwater levels lower than Elevation 190 ft. MSL. When the reservoirs are at operating levels, the subsurface flow of water is toward the cones of depression from the two reservoirs and the water levels in the cones of depression are expected to gradually achieve partial recovery. After construction is completed and the groundwater level has recovered, groundwater will move toward the Main Reservoir.

Water is supplied to the reservoirs by stream flow, direct precipitation and runoff, and an insignificant quantity of groundwater influent from springs of intercepted permeable zones associated with intrusive rocks where they are in hydraulic contact with the reservoirs.

Because of the impervious nature of the soils and country rock, there is only insignificant interchange of water between the reservoirs and the aquifer. This condition is verified as shown in Figure 2.4.13-5. Note that the water levels in piezometers 8A and LP13 are at elevations 102.5 ft. (affected by pumping) and 189.3 ft., respectively, while the water level in the emergency intake canal, approximately 50 feet from both wells, is at elevation 245 ft.

In Table 2.4.13-7, the results of permeability determinations from downhole pressure tests show that permeability values for the country rock range from 0.0096 to 0.265 gallons per day per square foot (gpd/ft^2) within the plant site. According to the USDA Soil Conservation Service soil survey of Wake County, 1970, the permeability values of the upper 96 inches of soil range from 29.9 gpd/ft.^{2} to 94.2 gpd/ft.^{2} in the uppermost 12 inches of sandy loam, from 9.4 to 29.9 gpd/ft.^{2} in the next 17 inches of clay loam, and 3 gpd/ft.^{2} in the next 79 inches of clay. The saprolite zones below the surficial clay have much lower permeability values, as mentioned above, and prevent ready movement of water from the surface to the deeper soils.

The lack of data points outside the immediate vicinity of the plant island makes it impossible to prepare an accurate map of the piezometric surface in the offsite areas. However, in Figures 2.4.13-6 and 2.4.13-7 pre-construction, current, and post-construction water-level conditions in the plant island area are illustrated. The post-construction water levels are anticipated to closely duplicate the preconstruction conditions except where altered by the plant structure and, to some extent, in the immediate proximity of the reservoir and canals.

The permeability values of the soils and saprolite that underlie the reservoir are so low as to require near vertical gradients to drive even a small amount of water from the reservoir bottom to the water table. In areas where there may be a flow of water from the reservoir to the water table, the steep hydraulic gradient will confine the flow path to within approximately 100 feet of the shoreline. Where fracture systems of intrusive dikes may be in hydraulic contact with the reservoir and the head relationships are such as to allow flow from the reservoir into the aquifer,

the gradients will be less than in the country rock, but the flow path will be narrow and confined very closely to the fractured zones in the dikes. According to the observed behavior of water in the fracture system during the pumping test on wells 13 and 15, it is possible that measurable changes in the water level may occur a few hundreds of feet from the reservoir in such fracture systems. The reservoirs will produce no observable effects on the groundwater levels outside the Shearon Harris Nuclear Power Plant site.

2.4.13.3 Accident Effects

The contents in the tanks located inside the Waste Processing Building or in the Tank Building, including the reactor make-up water storage tank, the refueling water storage tank, and the condensate storage tank are potentially radioactive.

2.4.13.3.1 Groundwater Pathway

The only possible groundwater path to water users following a radioactive spill would be seepage through the soil. The plant and peripheral lands are underlain by the Triassic, Newark Group aquifer. An accidental release of radionuclides at the site can be assumed conservatively to percolate downward to the aquifer instantaneously. The general direction of groundwater movement in the aquifer at the site is toward the southeast. However, ongoing pumpage at the site for construction water has altered the flow direction locally toward the pumping wells (Figure 2.4.13-2).

The value for porosity in the groundwater movement analysis was based on a measured value for permeability for the fracture system of the intrusive-rock dike between wells 13 and 15 (Figure 2.4.13-8). Inasmuch as hard-rock fracture systems are heterogeneous and anisotropic, hydraulic characteristics for these systems can be grouped only in a broad category. In the system between wells 13 and 15, the measured permeability value of 2841 gpd/ft.^{2} compares with the lower part of the scale of values for gravel as given in Walton, pp. 33-36 (Reference 2.4.13-6). Values were estimated for porosity and "effective porosity" (specific yield) by using the same relative position as "permeability" on scales of these values given in that publication.

The range of values for permeability of gravel is given as 1,000 to 15,000 gpd/ft.^{2}. Proportionally, the value of total porosity is estimated at 31 percent and the value of effective porosity (same as specific yield in Walton, 1970) is estimated at 17 percent.

Assuming the maximum parameters, it is established that the minimum time required for the groundwater to reach the closest community downstream from the plant would be about 144 years. This time estimate is based upon the following parameters: Corinth is the nearest town, approximately five miles to the southwest, where residents have wells of minimal production from the Triassic, Newark Group (Figure 2.3.2-18). The maximum measured site coefficient of permeability is 520 ft./yr. (Table 2.4.13-7). The maximum measured site hydrologic gradient is 0.06 ft./ft. towards the SE from the Waste Processing Building (Figure 2.4.13-2). The effective porosity is 0.17.

The effective travel time of radionuclides which may contaminate the aquifer following a tank rupture would be considerably greater due to absorption and ion exchange on the underlying rock. The distribution coefficients (Kd) for cesium and strontium, the critical radionuclides, are assumed to be 20 and 2, respectively. These values were taken from Table VII 3-7 of Appendix VII of WASH 1400 and are conservative when compared to values reported in the literature

(Reference 2.4.13-7). The calculated retention factors using these values for Kd, an effective porosity of 0.17 and a bulk dry weight density of 2.6 (Table 2.5.4-1; 162.8 lbs/ft.^{3}) are 307 for cesium and 32 for strontium. Using these retention factors, the travel time for Cs-137 and Sr-90 for transport to the nearest community would be:

Cs-137 = (144 yrs) (307) = 4.4 x 10^4 yrs

Sr-90 = (144 yrs) (321) = 4.6 x 10^3 yrs

Assuming tritium to be in the form of water, the effective travel time for tritium would be 144 years. Based upon these effective travel times, radioactive decay would reduce the amount of tritium, Cs-137, and Sr-90 which could potentially reach Corinth to negligible levels.

Although the hydraulic gradients at the site vary considerably, the maximum gradient is about 0.06 ft./ft. toward the southeast from the Waste Processing Building (Figure 2.4.13-2). The distance between the Waste Processing Building and nearest site well that has created a cone of depression is about 2000 ft. Any spills could travel toward the well.

The permeability of the aquifer material adjacent to the site dikes is significantly higher than values for the country rock; a value of 500 ft./yr. could represent the dike zone or some extensive fracture zone in the country rock.

Assuming that the aquifer is homogeneous, with a conservative value for the coefficient of permeability of 500 ft./yr. and a porosity of 30 percent, the water would travel at a rate of about 100 ft./yr. and would cover a distance of 2000 ft. in about 20 years.

Table 2.4.13-7 shows that all but two permeability values for the materials at the plant site are less than 10 ft./yr. These two values could represent fractures which may not be extensive. The aquifer material, which has a permeability of 10 ft./yr., a porosity of 30 percent, and a hydraulic gradient of 0.06 ft./ft., would have a water movement rate of 2 ft./yr. and it would take about 1000 years for water to move through 2000 ft.

The closest community downstream from the plant site is Corinth (Figure 2.3.2-18), approximately five miles to the south-southwest, where residents have wells of minimal production from the Triassic, Newark Group. Taking the above two groundwater movement rates of 100 ft./yr. and 2 ft./yr. it would take about 54 years and 13,200 yrs., respective]y, for the water to reach Corinth.

2.4.13.4 Monitoring of Safeguard Requirements

Fourteen piezometers that were installed in 1979, as well as two pre-construction piezometers and one new well, are available at the plant site. The piezometers and site wells provide data on water levels, hydraulic gradient, and direction of flow. Water levels in piezometers and site wells are measured periodically and analyzed to assess the effect of construction on the site groundwater regime. Water samples from three wells were analyzed to determine baseline water quality parameters (Table 2.4.13-8).

Table 2.4.12-1 shows the C/MPC at Corinth are approximately one-half of the allowable 10 CFR 20 concentrations. The closest privately-owned well to the reservoir is on N. C. State Road 1128 at approximately 600 feet from the shoreline. The ground surface elevation at this

well is greater than 30 feet above the normal pool level. The direction of groundwater flow would be from the well to the reservoir, in this case. Inspection of the topographic maps of the area indicates the expected direction of groundwater flow all around the reservoir to be towards the reservoir. Possible exceptions may be in the stream valley immediately and around the dam, and within a few feet of the general shoreline as the gradients adjust to the water levels in the reservoir.

The chemical and biological requirements for the plant make-up water are quite stringent and dictate that the high quality of the reservoir water must be maintained. Should any reservoir water seep into the surrounding streams, it would be filtered within the aquifer and would be of better quality than the water in the receiving streams.

2.4.13.5 Design Bases for Subsurface Hydrostatic Loading

The subsurface portions of Seismic Category I structures on the plant island are designed for hydrostatic loadings with groundwater at Elevation 251 ft. MSL. A permanent dewatering system is not utilized for the Shearon Harris Nuclear Power Plant. Groundwater occurring in widely separated joints in the rock did not significantly affect construction. Any rain or surface water that accumulated during construction was pumped out by sump pumps.

The pre-construction piezometric-level map, shown on Figure 2.4.13-1, indicates that the piezometric levels were higher than Elevations 251 ft. MSL under some sections of the plant. However, the lack of significant inflow of groundwater in the completed plant block excavation indicates that groundwater in the rock occurs only in widely separated joints and bedding planes. Except for the west side of the fuel handling building and the adjacent portion of the waste processing building the perimeter of the plant structure up to the top of the foundation mats is in direct contact with rock that is essentially impermeable, and the portion between the plant structures and rock above the top of the mats has been backfilled with residual soil which is of very low permeability (estimated to be less than 10 ft./yr). Additionally, the winter 1979-1980 piezometric-level map (Figure 2.4.13-2) shows that water levels beneath the plant area are well below Elevation 251 ft. MSL.

The source of surface water higher than the design basis groundwater level is the Emergency Service Water Intake Channel of the Auxiliary Reservoir, which has an operating level at Elevation 252 ft. MSL, the closest point of which comes to within about 300 ft. of the plant island. The Auxiliary Reservoir will not raise the groundwater elevation beneath the plant island above Elevation 251 ft. MSL for the following reasons:

a) The residual soil underlying the reservoir is of very low permeability, as indicated by testing.

b) After the Auxiliary and Main Reservoirs are filled, groundwater will move from the reservoirs to the cones of depression created by the pumpage from wells. However, the major portion of the groundwater flow will be from the Auxiliary Reservoir to the Main Reservoir.

c) Groundwater from the Auxiliary Reservoir will start moving toward the plant island at an Elevation of about 252 ft. MSL. However, the water level will be at a much lower level than Elevations 251 ft. MSL by the time it reaches the plant island due to the hydraulic

head loss as it flows through the low permeability materials for a distance of about 300 ft.

d) Drainage for the retaining wall and seepage through open joints in the retaining wall further reduce the hydraulic head beneath the plant area.

2.4.14 TECHNICAL SPECIFICATION AND EMERGENCY OPERATION REQUIREMENTS

As described previously in this section, the Main and Auxiliary Reservoirs have been designed and constructed to withstand without operational restrictions the most adverse hydrological conditions, including flooding, that can occur. Actions that will be taken to assure that reservoir conditions are within the analyzed parameters are as follows:

a) The plant will be shut down when the water level in the Main Reservoir falls to the minimum level specified in Technical Specifications. This action will assure adequate pump submergence and cooling capability in the Main Reservoir.

b) The plant will be shut down when the water level in the Auxiliary Reservoir falls to the minimum level specified in Technical Specifications. This action will assure adequate pump submergence and cooling capability in the Auxiliary Reservoir.

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2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

2.5.0 SUMMARY

2.5.0.1 Basic Geologic and Seismic Information

2.5.0.1.1 Regional Geology

The SHNPP site is located in the Deep River Triassic Basin, a trough-like topographic lowland located mostly within the Piedmont Plateau physiographic province. The upland surface of the Piedmont Plateau is an ancient, deeply weathered erosion surface. It is characterized by gently rolling topography with shallow valleys and rounded divides. Upland elevations range from 300 to 600 ft. above sea level along the eastern border of the plateau to about 1500 ft. above sea level at the foot of the Blue Ridge scarp. Immediately west of the Deep River basin, the upland surface rises to the west and north from between 350 and 375 ft. in northern Lee County to about 600 ft. above sea level in the northwest corner of Chatham County, with the slope of the surface averaging about 10 ft. per mile. Elevations in the Deep River Triassic lowland are generally 50 to 200 ft. lower than those of the Piedmont upland which borders its western and northeastern margins; as a result these borders are marked by abrupt erosional escarpments in many places. Elevations range from less than 160 ft. above sea level along the Cape Fear River to more than 500 ft. in some places in the northern part of the basin. Local relief is less than 100 ft. except near the main streams. The lowland is generally more intricately dissected than the Piedmont upland to the west, and streams are better adjusted to the structure of the bedrock. Most streams have steep valley sides and narrow bottoms, but some streams such as the Deep, Haw, and Cape Fear Rivers, are bordered by terraces which form extensive flat areas.

The igneous and metamorphic rocks which underlie the Piedmont Plateau can be divided into several broad northeast-southwest trending belts on the basis of differences in metamorphic grade. Metamorphic grade is highest in the westernmost belt, the Inner Piedmont, which contains upper amphibolite grade gneisses and schists. Rocks of lower amphibolite grade are present in the Charlotte Belt, which borders the Inner Piedmont on the east, and the Raleigh belt, which lies near the eastern margin of the Piedmont. Metavolcanics and metasediments of mostly greenschist grade characterize the Carolina Slate Belt, which lies between the Charlotte Belt and the Raleigh Belt, and the Eastern Slate Belt, which flanks the Raleigh Belt on the east and forms the easternmost portion of the Piedmont. The Deep River Triassic Basin is a sediment-filled trough located between the Carolina Slate Belt on the west and the Raleigh Belt on the east.

The Carolina Slate Belt rocks form a section that is believed to be at least 30,000 ft. thick in North Carolina. This section consists largely of metavolcanic rocks and subordinate metasediaments, of Late Precambrian and Cambrian age, intruded in places by granitic plutons. Volcaniclastic rocks, including siltstone, claystone, and graywackes are interbedded with rhyolitic, dacitic, andesitic, and basaltic volcanic rocks, including tuff, lapillistone, pyroclastic breccia, and lava flows. These rocks are deformed into a series of northeast-trending folds, the major one being the Troy anticlinorium near the center of the southern portion of the belt. Minor folds to the east and northeast of the anticlinorium are either asymmetrical or overturned to the southeast and have axial plane dips that range from vertical to less than 60° northwest. Folds to the west and southwest of the anticlinorium are broad and open and only slightly asymmetrical with axial planes dipping steeply northwest. Some rocks have closely spaced slaty cleavage, hence the name "Slate Belt"; however, many others are rather massive with little or no cleavage. Slaty cleavage is particularly common in the Gold Hill - Silver Hill fault zone, a major fault zone along the southwest border of the Carolina Slate Belt. Radiometric age dating indicates that the volcaniclastic sequence is mainly 650-500 million years old, although some members may be older, and that many of the plutons, ranging from gabbro to granite in composition, are contemporaneous with or slightly younger than the volcanic rocks. Greenschist grade metamorphic granitic plutons on the eastern margin of the belt have been dated at 326 million years (Lilesville pluton) and 285 million years. (Wilton pluton).

The Raleigh Belt is essentially a southward-plunging anticlinorium which disappears beneath a cover of Coastal Plain sediments south of Smithfield. The axial surface of the anticlinorium strikes N20° E and dips 70° E. The axial portion of the anticlinorium is occupied by a granitic core, generally referred to as the Rolesville granite, which is about 50 miles long and 10 to 15 miles wide. The granite is flanked by amphibolite grade metamorphic rocks consisting mostly of biotite-muscovite gneiss and schist with some interlayered hornblende gneiss. Near the Rolesville granite the gneisses and schists are injected in places by pegmatites up to 50 ft. or more in width. Small bodies of altered ultramafic rock are common in the northwest part of the belt. Scattered small plutons of granitic rock are present in the northern and western part of the belt; those that have been dated radiometrically have ages in the 325-265 million years range. The metamorphic rocks on both flanks have steep dips in most places and in some places appear to have been isoclinally folded. A northeast trending mylonite zone thought to represent a major fault, the Nutbush Creek Fault, is reported to cut the west flank of the anticlinorium. Apparent offset on the fault is right-lateral.

The Deep River Triassic Basin is a structurally complex, northeast trending trough containing a wedge-shaped block of mostly clastic sediments belonging to the Upper Triassic Newark group. Maximum thickness of the sedimentary wedge is believed to be 10,000 ft. or more. Strata within

the basin generally dip as well as thicken toward the southeast. The sediments are mostly of fluvial or lacustrine origin and are characterized by abrupt lateral changes in texture and composition. The oldest rock unit, the Pekin formation consists largely of red or brown, fine-grained clastic rocks with a few beds of conglomerate in the lower part. The middle unit, the Cummock formation, contains two coal beds in its thickest part in the southeastern part of the basin; one bed is generally less than two ft. thick and the other is generally less than four ft. thick. The remainder of the formation consists of gray or black shale, claystone, siltstone, and sandstone. The youngest unit, the Sandford formation, consists mostly of red or brown fine-grained clastics except along the southeast edge of the basin, where it consists of conglomerate and fanglomerate. The rocks of the basin are broken by two systems of normal faults, northeast-trending longitudinal faults and northwest-trending minor cross faults. The faults divide the basin into triangular or diamond-shaped blocks with dimensions as small as one mile by two miles. The Jonesboro Fault zone, which forms the eastern border of the Basin is a diagonal dip slip fault with a total vertical displacement of 5,000 to 10,000 ft. and unknown right lateral displacement. Major longitudinal faults within the basin have vertical displacements of several hundred ft. and minor cross faults have displacements from a few feet to a few hundred feet In many places the Triassic sediments are intruded by diabase dikes of late Triassic or early Jurassic age. The dikes range up to 300 ft. wide and up to seven miles long. Most dikes trend N15° - 40°W, but more northerly trends are common in the northern part of the basin.

The geologic history of the central and eastern Piedmont region is poorly known because fossil-bearing strata are extremely rare and geochronology is based largely on radiometric dating of igneous events. The geologic record suggests that island arc volcanism was the dominant activity from Late Precambrian through Cambrian time. A period of major deformation of early volcanogenic deposits around 600 million years ago formed the major folds of the Carolina Slate Belt. This deformation was accompanied or closely followed by emplacement of granitic plutons. The deformational event was followed by renewed volcanic activity possibly indicating development of a new island arc system during Cambrian time. This volcanism continued through late Cambrian time and was followed in early Ordovician time by a metamorphic event which produced greenschist metamorphism in Carolina Slate Belt rocks. Another major deformational event occurred during Devonian time which involved major movement in the Gold Hill fault zone, greenschist metamorphism, and emplacement of granitic plutons in the Charlotte belt. The last clearly indicated major Palezoic event was emplacement of granitic plutons in the Charlotte and Carolina Slate Belts during Pennsylvanian time. The earliest clearly recorded Mesozoic event is the deposition of late Triassic sediments in subsiding northeast-trending troughs in the eastern (Deep River - Wadesboro Basin) and western (Dan River Basin) parts of the Piedmont. In the Deep River Basin normal fault movement along segments of the Jonesboro fault system and the resulting differential subsidence caused eastward tilting of sedimentary strata. Accumulation of the sedimentary wedge was followed by continued movements in the Jonesboro fault zone and development of cross-basin faults. Emplacement of diabase sills and dikes followed formation of the cross faults and continued into Jurassic time. Final movement of the Jonesboro fault during late Triassic-early Jurassic time was followed by widespread zeolite mineralization related either to lowgrade burial metamorphism or to high heat flow and hydrothermal activity. Little is known of late Mesozoic and Tertiary history. The region apparently has been relatively stable tectonically since late Mesozoic time. Crustal movement has largely been limited to vertical isostatic adjustments possibly related to periodic uplift of the Appalachians to the west and subsidence of the Coastal Plain to the east.

2.5.0.1.2 Site Geology

The SHNPP site is located near the eastern edge of the Cape Fear River drainage basin. The plant site is on an upland area of gently sloping hills and ridges located between Tom Jack Creek on the west and Thomas Creek on the east. Elevations of hill tops and ridge crests are mostly between 250 and 275 ft. and local relief is generally less than 60 ft. The area is mostly in woodland with scattered small farms. Drainage from the site is southeast through Tom Jack and Thomas Creek to Whiteoak Creek, which flows southwestward into Buckhorn Creek, which in turn flows southward and empties into the Cape Fear River about a quarter mile below Buckhorn Dam.

The soils around the site are mostly residual soils derived from sedimentary rocks and diabase dikes underlying the area. Soil depth ranges from 0 to 15 ft., but is commonly between five and 10 ft. The soil is generally thinnest over sandstone and thickest over diabase dikes. Most residual soil is silty clay in texture, but silty sand may be found along streams and in limited areas overlying sandstone. Residual soils observed in trench excavations were medium stiff to hard. Permeability values of most soils are extremely low, resulting in rapid precipitation runoff.

The site is underlain by gently dipping rocks of the Upper Triassic Sanford formation. The bedrock is mostly siltstone and fine-grained sandstone interbedded with subordinate shale, claystone, and conglomerate. These rocks consist mostly of alluvial fan, stream channel and floodplain deposits and are characterized by abrupt changes in composition and texture, both horizontally and vertically. Beds range in thickness from less than an inch to a maximum of 20 ft. They interfinger and overlap into compact masses with no structural weakness. Several north to northwest trending diabase dikes of Triassic-Jurassic age have intruded the Triassic bedrock in the site vicinity. These dikes are near vertical and one to 15 ft. thick. Bedrock adjacent to the dikes is commonly baked to a dark gray or black color. Most dikes are deeply weathered to a mixture of clay and rounded cobbles of residual diabase. Layering in the Triassic sedimentary rocks strikes N5°-15°E and dips 9° to 17° to the southeast. Three joint sets are present; the two dominant sets are vertical, one striking N40°-50°E and the other N20°-30°W. A third set trends north-northwest and dips 55° to 70° to the southwest. A minor fault uncovered in the plant excavation trends nearly east-west across the site. The fault is a normal fault with downthrow on the south. The fault surface is somewhat undulatory with dips ranging from vertical to 55° southward. The vertical component of movement is between 80 and 100 ft; the maximum horizontal offset could not be determined, although dikes intruded during the period of movement are offset by an amount ranging from a minimum of one-half foot to a maximum of 13 ft. Drag folding of Triassic beds is present on the hanging wall of the fault. A detailed investigation of this fault, described in Section 2.5.3, determined that the fault is a minor tensional normal fault whose last movement was prior to 150 million year ago.

Several small, non-capable faults were found in the foundations of Main Dam structures. These are described in Section 2.5.0.3. No other significant structural features were found.

The Triassic strata underlying the site have very low primary permeability and yield little or no groundwater except where secondary permeability is provided by fractures, which are common to depths of 100 ft. Below 100 ft. fractures are less common and tighter and below about 400 ft. are closed and sealed to water flow. Highest groundwater yield is found adjacent to the diabase dikes, which act as impermeable barriers to groundwater flow. Piezometric data from boreholes in the plant area indicate that groundwater movement in the plant area is, in general, to the southeast toward WhiteOak Creek.

Historical records of earthquake activity indicate that the site is virtually aseismic. There is little history of felt earthquakes in the site area and no historical accounts of the behavior of the site during the few earthquakes which have been felt. A seismic monitoring network covering the plant site began operation in 1977; it has recorded no local earthquakes through four years of continuous monitoring. During historical time no earthquake has occurred within 60 miles of the site. No geological evidence of Holocene earthquake activity in the vicinity around the site was found.

The geologic history of the site through Paleozoic time is poorly known, since the only Paleozoic rocks exposed in the plant area are Raleigh Belt gneisses and schists exposed in the main dam foundation south of the plant. It is uncertain whether these rocks represent Precambrian continental crust of possible Grenville age upon which Slate Belt volcanogenic deposits were formed or whether they represent Slate Belt rocks which have undergone a higher degree of metamorphism than other Slate Belt rocks. The sedimentary rocks underlying the site were deposited in Late Triassic time contemporaneously with normal faulting along the Jonesboro fault zone. Filling of the Deep River Basin, mainly from eastern sources, continued until rocks now exposed at the site were buried under several thousand feet or more of sediments. A change in stress orientations resulted in the formation of the Shearon Harris site fault and other cross-basin faults. Three separate periods of dike emplacement occurred contemporaneously with the movement of the site fault. A likely sequence of events was (1) sequential intrusion of the two oldest dikes and continuation of fault movement (2) crystallization of zeolites (low-barium laumontite and saponite) in the fault-dike intersections (3) intrusion of the third dike as fault movement continued (4) cessation of movement on the fault, and (5) further crystallization of zeolites harmstome, heulandite, barite, and high-barium laumontite in dike fault intersections. These events probably culminated during Jurassic time. The site was probably buried under Coastal Plain deposits similar to those that overlap the basin to the southwest during Cretaceous time and may have remained buried under Coastal Plain cover for much of Tertiary time. Exhumation of the present day surface may have occurred as late as Pliocene time. The site has been relatively stable tectonically since late Jurassic time, with tectonic activity limited largely to minor vertical movements.

2.5.0.2 Vibratory Ground Motion

The region in the immediate vicinity of the site is characterized by low-level seismicity. Moderate levels of earthquake activity occur in the surrounding region at distances greater than 130 miles from the site. During the period 1754-1977, eight earthquakes of epicentral intensity VII or greater occurred within about 200 miles of the site. Six were of intensity VII, the closest of which occurred at a distance of about 133 miles from the site. The Charleston, South Carolina earthquake of August 31, 1886, which occurred about 200 miles south of the site, has an intensity of X and was probably felt with an intensity of VI in the site area. The earthquake in Giles County, Virginia on May 31, 1897, which occurred about 160 miles northwest of the site, had an intensity of VIII and was probably felt in the site area with an intensity of about V.

The relationship between earthquake activity and known geologic structures or tectonic provinces in the southeastern region of the United States is poorly known because earthquakes are not associated with visible surface faulting and because seismograph station coverage of the region has been inadequate to determine focal depths or focal mechanisms of most historical earthquakes. Three seismic zones have been recognized which border the site region. These are the Central Virginia seismic zone, a relatively narrow zone of activity located in the Piedmont province and oriented obliquely to the NE-SW structural trend; the South

Carolina-Georgia seismic zone, a broad zone spanning both the Piedmont and Coastal Plain Provinces and oriented transversely to the regional structure of the crystalline basement rock; and the Southern Appalachian seismic zone, which includes the Blue Ridge and the Valley and Ridge Provinces from southwestern Virginia to central Alabama and parallels regional structural trends. However, the many faults mapped in each of these zones have no record of surface rupture during historical times.

Historical seismicity related to the filling of large reservoirs in the southeastern Piedmont is limited to a few reservoirs in the South Carolina-Georgia seismic zone. These reservoirs range in volume from 0.5 km^3 to 2.5 km^3 and water depths near seismically active areas range from 25 m to 110 m. All earthquakes reported to date have had local magnitudes less than five. Reservoir-induced seismicity has not been reported from the Piedmont of North Carolina despite the presence of several large reservoirs, some of which cross major faults.

Based on historical seismicity, the maximum potential earthquake which might affect the site would be a recurrence of the Charleston, South Carolina earthquake of 1886 which was probably felt as an intensity VI at the site. The largest earthquakes in the site region which are not attributable to any particular geologic structure or seismic zone have been of intensity V. However, it is considered possible that some intensity VII earthquakes in the eastern Piedmont and the Coastal Plain may have been related to exposed or buried Triassic Basins. Therefore, a shock of intensity VII occurring in the Deep River Basin is considered to be the maximum potential earthquake.

Seismic wave transmission characteristics of the site were determined from seismic refraction measurements and ambient vibration measurements. Compressional wave velocities range from 1250 to 2000 ft./sec. for residual soils and/or highly weathered bedrock, from 5000 to 7150 ft./sec. for weathered and/or fractured bedrock, and from 10,900 to 13,650 for sedimentary bedrock. Measured shear wave velocities were 2500 ft./sec. for weathered and fractured rock and 5600 ft./sec. for sound bedrock; a velocity of 500 ft./sec was assumed for residual soil based on previous experience under similar conditions. Computed values of Poisson's ratio were 0.44 for residual soil, 0.37 for weathered and fractured bedrock, and 0.35 for sound bedrock. Observed characteristic frequencies of the site from ambient ground motion measurements were 100, 55 1/2, and 25 Hz. The maximum observed level of ground motion, which occurs in the vertical component, was 0.48×10^{-3} in./sec. at about 25 Hz.

The safe shutdown earthquake is designated as an intensity VII earthquake occurring close to the site. The resulting maximum horizontal ground acceleration at foundation level within the competent bedrock at the site is estimated to be less than 12 percent of gravity. In order to provide an additional margin of conservatism, a value of 15 percent of gravity is assigned as the maximum horizontal ground acceleration. All safety related structures and systems are designed to assure safe plant shutdown for two horizontal excitations and one vertical excitation simultaneously. Seismic Category I systems and components are designed for a minimum of 10 loading cycles under safe shutdown earthquake conditions. The horizontal and vertical response spectra for the SSE, prepared in accordance with NRC Regulatory Guide 1.60, are presented on Figures 2.5.2-12 and 2.5.2-13.

The operating basis earthquake is designated as one with half the accelerations of the safe shutdown earthquake and equivalent to an Intensity VI earthquake near the site. The corresponding horizontal acceleration at foundation level in the bedrock would be less than 7.5 percent of gravity. The horizontal and vertical response spectra for the OBE, prepared in

accordance with Regulatory Guide 1.60 and scaled to .075g horizontal ground acceleration, are presented on Figures 2.5.2-14 and 2.5.2-15.

2.5.0.3 Surface Faulting

At the time of preparation of the PSAR for the SHNPP, the only fault known to exist within five miles of the plant site was the Jonesboro Fault, whose trace approximates the course of Buckhorn Creek three miles southeast of the site. Site investigations in the plant and Auxiliary Reservoir area, which included 12,125 ft. of trenching at depths from two to twelve ft. , numerous geologic borings 100 to 250 ft. deep, and approximately 5000 linear ft. of seismic refraction survey lines, failed to uncover any evidence of surface faulting. However, during excavation activities a minor normal fault, herein referred to as the Site Fault, was exposed in the foundations of the plant Waste Processing Building and approximately 20 small faults were exposed in the foundations of Main Dam structures about 4.5 miles south of the plant site.

2.5.0.3.1 Jonesboro Fault

The Jonesboro Fault is a northeast-trending diagonal slip fault whose total length exceeds 100 miles. It forms the eastern edge of the Deep River Basin and marks the contact between Triassic sedimentary rocks to the west and Paleozoic volcaniclastic and crystalline rocks to the east. It is nearly vertical with 8,000 to 10,000 ft. of vertical displacement and an unknown amount of right-lateral displacement. In places south of the site it offsets diabase dikes and is overlain by undisturbed, flat-lying Cretaceous sedimentary deposits. The age of last movement on the Jonesboro is bracketed between intrusion of Late Triassic-Jurassic diabase dikes and the deposition of Cretaceous sediments overlying it. Therefore, the Jonesboro Fault is not considered to be a capable fault.

2.5.0.3.2 Site Fault

A comprehensive investigation of the Site Fault was performed for Carolina Power & Light Company by Ebasco Services Incorporated in order to determine (1) age of last movement on the fault, (2) fault length, (3) vertical and horizontal components of movement on the fault, and (4) alignment and attitude of the fault from the plant excavation to and through the Auxiliary Reservoir area.

A program of trenching perpendicular to the trend of the fault, supplemented by further exploratory borings, was carried out to trace the fault beyond the limits of the excavation. Magnetometer surveys were used to locate and trace diabase dikes which served as markers to aid in determining fault displacement. This program traced the fault for a total of 8000 ft. across the plant and Auxiliary Reservoir area and located five diabase dikes trending north northwest across the fault. It was found that the fault, when exposed in sedimentary beds, exhibited a southerly dip between 55 and vertical. Drag folding was present on the hanging wall of the fault in all exposures, but bedding planes of strata on the northern or foot wall were rarely disturbed.

Nine core borings were completed in sedimentary rocks on both sides of the fault to determine the vertical component of offset along the fault. The vertical component of movement was determined to be greater than 80 ft. and less than 100 ft. as measured at three locations. The horizontal component of movement on the fault was determined from offsets of diabase dikes, which are essentially vertical. The horizontal offsets range from 0.5 ft., up to a maximum of 13 ft. A large horizontal component of movement is precluded by the undulatory nature of the fault

surface, which changes strike about every 300 ft. Movement on the fault was primarily tensional with a minor left-lateral component.

Remote sensing imagery, including ERTS, SLAR, and Skylab as well as conventional aerial photos, were analyzed in an attempt to determine the total length of the fault. The attempt was unsuccessful because the site fault was not detected by any imagery technique. Several hundred regional and local linear features were identified, but none were identified as capable faults on the basis of imagery evaluation. Nineteen lineaments were considered significant to the site and required field investigation; none that were field checked were identified as capable faults.

Field observations of undisturbed soil, saprolite, and, in one place, a post-Triassic sedimentary deposit overlying the fault indicated only that the age of last movement was probably greater than one million years. Therefore, detailed laboratory studies of samples from diabase dikes, associated sedimentary rocks, and undeformed crystals of secondary zeolites found in the fault gouge were undertaken to determine the geochronology of dike emplacement and fault movement. These studies included paleomagnetic age dating of diabase dike samples, potassium-argon age dating of diabase dike and zeolite samples, and strontium isotope studies of diabase, zeolites, and associated sedimentary rock.

Potassium-argon age dates on the diabase dikes range from a minimum of 168 million years to 260 million years, while dike ages based on remanent magnetization determinations ranged from an absolute minimum of 150 million years to a maximum of about 225 million years. The intrusion of the dikes at the site was largely contemporaneous in time with movement on the fault, but the youngest of the dikes showed slight displacement by the fault, indicating that some movement took place after dike emplacement. Secondary zeolite minerals were present in fault gouge at dike-fault intersections which showed no evidence of deformation by fault movement. Minimum potassium-argon age dates on the zeolites indicated ages up to 35 million years. These minimum ages were believed to be spuriously low because of argon loss from the zeolites since their formation. A comparison of strontium 87/86 ratios of the zeolites, dike rocks, and associated sedimentary rocks established that the diabase dikes were the source of the chemical ingredients which formed the zeolites. These minerals are formed at temperatures between 100 and 225 degrees centigrade. The two events which could be associated with such temperatures in the Triassic basin are the emplacement of the diabase dikes and a period of regional low-grade burial metamorphism, both of which occurred prior to 150 million years BP. Since the secondary minerals were emplaced prior to 150 million years ago and have not been disturbed by subsequent faulting, last movement on the fault was prior to that time.

As a result of these investigations, CP&L concluded in its Fault Investigation Report (Reference 2.5.1-29) that the Site fault was not a capable fault and would not move under reservoir loading or other proposed construction.

On January 6, 1976, Carolina Power & Light Company was formally notified by the NRC that the fault discovered at the Shearon Harris Nuclear Power Plant is not a capable fault as defined in Appendix A to 10 CFR Part 100 (see Reference 2.5.3-5 and Supplement 3 to SER dated July 1977). This fault is further discussed in Section 2.5.1.2.3, where it is referred to as a minor high-angle fault. The NRC also requested that seismic monitoring be performed at the site to confirm the NRC staff conclusion that the proposed reservoirs at the site will not cause fault movement during and after filling.

In response to the NRC's request, CP&L submitted a proposal on February 13, 1976, to establish a seismic monitoring network which would encompass the SHNPP plant site area. Although this proposal called for monitoring to begin in January of 1979, the network was installed and became operational on September 30, 1977, in order to obtain more definitive baseline data prior to the filling of the reservoirs.

2.5.0.3.3 Main Dam Faults

The minor faults exposed in the foundations of the main dam structures are in Paleozoic crystalline rocks. The faults are all minor normal faults with lengths measured in 10's of feet and displacements measured in inches.

Because the small amount of movement along these faults took place prior to deformation-mineralization which occurred more than 225 million years ago, the faults are not considered to be capable faults as defined in Appendix A to 10 CFR Part 100. Reports on these small faults are catalogued in the Foundation Report, Appendix 2.5E. The NRC concurred with this conclusion based on detailed reports which were submitted and on field inspections by their geological staff. The NRC's concurrence was primarily verbal. However, copies of internal NRC memoranda from Mr. Sydney Miner to Mr. Olan D. Parr state concurrence with CP&L's findings that certain faults are considered non-capable. Also, in IE Inspection Report Nos. 50-400/79-07, 50-401/79-07, 50-402/79-06, and 50-403/79-06 concerning inspections conducted by Mr. John R. Harris of the NRC, Region II, certain faults are identified as being considered as non-capable.

2.5.0.4 Stability of Subsurface Materials

The plant is founded on gently dipping, well-consolidated Triassic siltstones and sandstones. Rock beds range in thickness from a few inches to a maximum of around 20 ft., and are commonly lenticular. Depth of weathering is commonly between five and 10 ft. but tends to be shallower over sandstones and thicker over diabase dikes. Joints are irregularly spaced at intervals of a few feet and are mostly vertical. The dominant joint set is oriented N40° - 50° E, a secondary set is oriented N20° - 30° W, and a tertiary trends north-northwest and dips 55° to 70° to the southwest. An east-west trending, non-capable normal fault with drag folds on its downthrown south side crosses the plant and Auxiliary Reservoir areas. No other folds, faults, shears, or zones of structural weakness were noted in the plant foundations.

Programs of subsurface exploration based primarily on trenching and borehole sampling and drilling were conducted for both the preliminary site investigation and the site fault investigation. In addition, the floors and walls of excavations for plant foundations were mapped geologically. No areas of actual or potential surface or subsurface subsidence, uplift, or collapse were found.

The static and dynamic engineering properties of subsurface materials were determined by laboratory testing of rock samples obtained from core drilling and by field geophysical measurements. The index properties determined were dry density and rock quality designation (RQD) for selected rock samples and grain size distribution for selected residual soil samples.

The static modulus of deformation was computed from unconfined compression tests. Compressional wave and shear wave velocities were determined from seismic refraction measurements in the field. These velocities were used in the computation of Poisson's ratio and the dynamic modulus of deformation.

The plant excavation includes the foundations for the Waste Processing Building, four Turbine Buildings, a Fuel Handling Building, four Reactor Auxiliary Buildings, four Containment Buildings, and four Tank Buildings. It encompasses a total area of approximately 837,500 sq. ft. and has a total volume of approximately 1,200,00 $yd.^3$. Excavation began with the leveling of the ground surface in the plant area to 260 ft. elevation. After leveling, unclassified soil materials were excavated to ripper refusal and excavation was completed to final grade by controlled blasting. Slopes were excavated at 1:1 in overburden and at 1:4 in bedrock. After inspection and approval of final excavation surfaces by a geologist, the foundation surface was cleaned of loose materials and mapped geologically. Dewatering in the excavation was accomplished by intermittent use of sump pumps. Treatment methods used for foundation protection after excavation included slush grouting of joints to control groundwater seepage, placement of drain pipes at locations of seeps to prevent buildup of excessive pressures, shotcreting of some slopes and placement of a seal coat on foundation surfaces. Selected backfill material was compacted between structures and rock surfaces to meet requirements of 95 percent Standard Proctor Density with moisture control at ± 4 percent of Standard Proctor optimum moisture content and a maximum permeability value of 10 ft./yr.

A plot of piezometric levels across the site area show that the piezometric surface, in general, slopes southeastward, indicating that groundwater movement is toward the Whiteoak Creek arm of the Main Reservoir. Groundwater is confined mostly to joints and fractures in bedrock and to the bedrock zones adjacent to diabase dikes, which tend to act as barriers to groundwater movement; residual soils and unfractured bedrock have extremely low permeability and yield little or no groundwater. Downhole water pressure tests at borehole depths ranging from 10 ft. to 145 ft. indicate that permeabilities range from approximately 5 ft./yr. to less than 300 ft./yr. in sedimentary rock units underlying the site area. Twenty-four hour pump tests in several wells drilled near diabase dikes during 1977-78 indicated specific capacities ranging from 0.16 gpm/ft. to 0.59 gpm/ft. Minor seepage of groundwater from joints and fractures occurred in the plant excavation, particularly after rains. This water, as well as surface-water runoff, drained into sumps and was removed by occasional pumping.

All Seismic Category I structures within the plant area, except for Seismic Category I Electrical Manholes, Seismic Category I Underground Electrical Conduits, and some Seismic Category I pipes, are founded on sound rock which will not amplify ground motion from earthquakes. The ground acceleration at the foundation levels of structures supported on sound rock is equal to the baserock acceleration. The maximum horizontal accelerations of the bedrock were chosen to be 15 percent of gravity for the safe shutdown earthquake and 7.5 percent of gravity for the operating basis earthquake. The foundation of the plant has no potential for liquefaction because it consists of hard sound rock. The ground acceleration at the level of individual manholes was determined by an amplification analysis of ground motion through a vertical soil column between the bedrock and the manholes. The acceleration values obtained from the analysis were further increased by 50 percent for the equivalent static analysis of each manhole structure. Horizontal acceleration values at the level of individual manholes, as obtained from the analyses, are 0.25 g for the SSE and 0.14 g for the OBE; vertical acceleration values are 0.19 for the SSE and 0.10 for the OBE.

Static stability analysis of the plant island structures included settlement, bearing capacity, and lateral earth pressures. The average settlements under static loading computed for the various structures are very small, ranging from 0.008 ft. to 0.035 ft. The differential settlements should be even smaller and, therefore, structurally tolerable. Since the settlements consist of the pseudo-elastic compression of the bedrock, they will occur essentially upon load application.

The computed ultimate bearing capacity is 714 tons per sq. ft., but the design bearing capacity chosen is 25 tons per sq. ft., which provides a factor of safety of 28 compared with the ultimate bearing capacity.

2.5.0.5 Stability of Slopes

The plant site has no natural slopes whose failure could adversely affect the safety of the nuclear power plant.

2.5.0.6 Embankments and Dams

Two dams were constructed in the Buckhorn Creek watershed to impound cooling water for the Shearon Harris Nuclear Power Plant. The Main Dam impounds a reservoir used primarily for cooling tower makeup water which has a normal water level elevation of 220 ft. and a water surface area of approximately 4,000 acres. The Main Reservoir also serves as a backup source of emergency service water. The Auxiliary Dam impounds a reservoir for emergency service water which has a minimum pond elevation of 250 ft. and a surface area of 317 acres. An Auxiliary Separating Dike and Auxiliary Reservoir Channel control the flow of discharged emergency service water through the east and west arms of the Auxiliary Reservoir. The dike, constructed across the east arm of the reservoir, prevents discharged emergency service water from flowing directly back to the emergency service water intake area. The Auxiliary Reservoir Channel connects the east and west arms of the Auxiliary Reservoir, allowing emergency service water discharge to enter the west arm of the reservoir for maximum cooling before circulating back to the intake area.

The Main Dam, Auxiliary Dam, Auxiliary Separating Dike, Auxiliary Reservoir Channel, Emergency Service Water Intake and Discharge Channels, and Emergency Service Water and Cooling Tower Makeup Intake Channel are designed and constructed to Seismic Category I criteria and to withstand the effects of natural phenomena. The slope of the dams, dike, and channels are designed to a factor of safety of 1.5 under static conditions, 1.2 for simultaneous OBE and 100-year return period flood level, and 1.1 for simultaneous SSE and 25-year return period flood level. The simultaneous OBE and 100-year return period flood level was not analyzed for the ESW and Cooling Tower Makeup Intake Channel since the simultaneous SSE and 25-year return period flood level analysis was more conservative.

2.5.0.6.1 Main dam

The Main Dam is located on Buckhorn Creek about 4.5 miles south of the plant site and about 2.5 miles north of the Cape Fear River. It is a rockfill dam with a maximum height of 108 ft. and a length of approximately 1550 ft. at the berm elevation of 260 ft. Its outside slopes are 2.0 horizontal to one vertical. The main dam's spillway, with a crest elevation of 220 ft., is uncontrolled. The spillway crest has a net length of 50 ft. with a pier at its midlength.

The Main Dam and Spillway are located approximately 3000 ft. southeast of the Jonesboro Fault and are underlain by pre-Triassic igneous and metamorphic rocks consisting of granite, hornblende-mica gneiss, quartz-feldspar gneiss, and mica schist. The predominant foliation strikes approximately N55°E and dips 30° to 60° northwest. Most joints are spaced two to three ft. apart. The two dominant joint sets are steeply dipping, one striking northeast and the other northwest. In most places the bedrock is covered by residual soils or by alluvium in the valley bottoms. Exploration in the Main Dam area consisted of surface geological reconnaissance and

several subsurface exploration programs for the purpose of evaluating foundation conditions for the Main Dam and Spillway and for exploration and sampling of possible sources of borrow materials for use as earth and rockfill for the dam. Subsurface investigations included Seismic Refraction Surveys, detailed foundation mapping in excavations for main dam structures, excavation of trenches and test pits for borrow material sampling, and borehole drilling, testing, and sampling.

The Main Dam has a core of compacted silty clay and clayey silt material protected on each side by two 8-ft.-thick transitional filter zones and a rockfill shell. The core is founded on suitable rock (defined as rock material that cannot be removed on a production basis with a single-tooth ripper of a D-8 tractor or equivalent) and the rockfill shell is founded on weathered rock (defined as material which cannot be removed on a production basis with the blade of a D-8 tractor-dozer. The materials comprising the core, fine and coarse filters, and rockfill shell met specified design properties as discussed in detail in Section 2.5.6. The downstream face of the dam is protected by a layer of oversized rock and the upstream face by a four ft. thickness of riprap.

The stability of the Main Dam was evaluated by the slip circle method and the finite element method. The slip circle method was used to determine the static, pseudo-static, and pseudo-dynamic stability of the dam. The results of these analyses demonstrate that the slopes of the Main Dam will have an adequate factor of safety under all postulated design conditions. The finite element analysis was made to evaluate the seismic stability of the dam. Results indicate that the dam has ample safety margins during the SSE and the OBE.

Seepage control at the Main Dam is affected by the use of an impervious clay core founded in a cut-off trench and by installation of a grout curtain in the foundation rock. The cut-off trench was excavated to suitable rock, and the grout curtain, utilizing neat cement, was emplaced to a depth of 50 ft. below the floor of the cut-off trench.

2.5.0.6.2 Auxiliary Dam

The Auxiliary Dam is located across the Tom Jack Creek Basin arm of the Main Reservoir, adjacent to the southwest boundary of the plant site. It is an earth dam approximately 3903 ft. long with a maximum height of approximately 72 ft. and a berm elevation of 260 ft. Its outside slopes are 2.5 horizontal to one vertical. The dam's spillway is an uncontrolled concrete ogee section with a crest length of 170 ft. and crest elevation of 252 ft. The basis for its hydraulic design is the probable maximum flood (PMF).

The auxiliary dam area is in the Deep River Basin and is underlain by clastic sedimentary rocks which, like those underlying the plant site, belong to the lower part of the Triassic Sanford Formation. The bedrock consists of four major lithologic units: medium-to coarse-grained sandstone, fine-to medium-grained sandstone, siltstone, and shaly siltstone. These units grade into one another both laterally and vertically, and all intermediate combinations are present. The strata strike N5°-15°E with dips ranging from 9° to 17° to the southeast. The two dominant joint sets are vertical, one striking N40°-50°E and the other N20°-30°W. A third set trends north northwest and dips 55° to 70° to the southwest. The plant site fault, as detailed in the Shearon Harris Fault Investigation Report (Reference 2.5.1-29), crosses the Auxiliary Dam at Station 4 + 23, striking N87°E and dipping 65° to 75° southeast. This fault has been demonstrated to be non-capable, as discussed in Section 2.5.3.

The foundation exploration program for the Auxiliary Dam included test borings, test trenching, and seismic wave velocity measurements. The Shearon Harris fault investigation included borehole drilling and excavation of exploratory trenches in the reservoir area a few hundred feet north of the Auxiliary Dam axis. Further drilling along the dam axis and detailed geologic mapping of the foundation were done during construction. Exploration and sampling for the borrow area testing program included auger borings in the area located between the Auxiliary Dam and the Auxiliary Reservoir Separating Dike and auger boring and test pits in the area which was selected as the borrow area adjacent to the Auxiliary Dam Spillway.

The Auxiliary Dam has a compacted core of silty clay and clayey silt material protected by a transition filter zone and a random rockfill shell on each side. The downstream shell is provided with two horizontal drainage blankets, each three ft. thick, which are connected with the transition filter zone adjacent to the core of the dam. In addition, a 200 ft. wide drainage layer is provided under the shell in each of two areas where pre-existing creeks had been located. The foundation of the dam was excavated into weathered rock and the cutoff trench was excavated to suitable rock. The materials comprising the core, filters, and rockfill shell met specified design properties as discussed in detail in Section 2.5.6.

The random rockfill surfaces are protected by a layer of riprap in the areas of wave action. Surface areas outside the wave action zones are protected by oversized rock.

The static, pseudo-static, and pseudo-dynamic stability of the Auxiliary Dam was determined by the slip circle method. In addition, sliding wedge analyses were made to verify the sliding stability in the abutment areas. The results of these analyses demonstrate that the slopes of the Auxiliary Dam will have an adequate factor of safety under all postulated design conditions. A finite element dynamic analysis made to evaluate the seismic stability of the dam indicates that the dam has ample safety margins during the SSE and the OBE.

Seepage control was affected by the use of an impervious core founded in a cut-off trench and by istallation of a grout curtain in the foundation rock. The cut-off trench was excavated to suitable rock and the grout curtain, utilizing neat cement, was emplaced to a depth of 50 ft. below the floor of the cut-off trench.

2.5.0.6.3 Auxiliary Reservoir Separating Dike

The Auxiliary Reservoir Separating Dike is located across the east arm of the Auxiliary Reservoir about 1700 ft. north of the Auxiliary Dam. It is approximately 1200 ft. long with a maximum height of approximately 55 ft. Its outside slopes are 2.5 horizontal to one vertical.

The bedrock underlying the Auxiliary Reservoir Separating Dike consists of clastic sedimentary rocks of the Triassic Sanford Formation and is similar in most respects to the strata underlying the Auxiliary Dam.

Foundation conditions were explored by means of borehole drilling and measurement of seismic wave velocities along the centerline of the dike and by geologic mapping of the foundation excavation.

The Auxiliary Reservoir Separating Dike has a core of compacted silty clay and clayey silt material protected by a random-rockfill shell which is size graded near the core, with the finer material placed adjacent to the core. The core and rockfill shell are founded either on

weathered rock or on a thin layer of stiff residual soil overlying weathered rock. The material comprising the core and rockfill shell met specified design properties as discussed in detail in Section 2.5.6. Slope protection is provided by riprap placed on random rockfill. Since water level is the same on both sides of the dike, provisions for seepage control through the dam foundation were unnecessary.

The stability of the Auxiliary Reservoir Separating Dike was evaluated by the same methods used for the Main Dam and the results of the stability analyses indicated that the slopes of the Auxiliary Reservoir Separating Dike will have an adequate factor of safety under all postulated design conditions.

2.5.0.6.4 Channels

The Emergency Service Water Intake and Discharge Channels are conservatively designed to carry the service flow required for normal and emergency shutdown of SHNPP. The intake channel is approximately 3580 ft. long and 50 ft. wide at its invert elevation of 238 ft. The discharge channel is approximately 2170 ft. long. The width at its invert (Elevation 240 ft.) is 50 ft. to 80 ft. The walls of both channels have a slope of two horizontal to one vertical in soil and one horizontal to four vertical in rock. Portions of the slopes were shaped to grade by backfilling with an impervious material. Diabase dikes were capped with concrete where they crossed channels.

The Auxiliary Reservoir Channel is approximately 1570 ft. long and 140 ft. wide at its invert elevation of 235 ft. Its walls have a slope of two horizontal to one vertical in soil and one horizontal to four vertical in rock. The Auxiliary Reservoir Channel is sized to carry the maximum ultimate discharge of the Service Water System coincident with a PMF flow for the upstream drainage basin.

The Emergency Service Water and Cooling Tower Makeup Intake Channel is approximately 2500 ft. long and 45 ft. wide at its invert elevation of 194.0 ft. The walls of the channel have a slope of two horizontal to one vertical in soil and one horizontal to four vertical in rock on the north side of the channel and two horizontal to one vertical in rock on the south side. The channel was cut entirely through hard residual soil and rock. Therefore, there were no fill sections except local backfill around intake structure and local small structures such as manholes, etc.

All four channels are underlain by gently dipping strata of the Triassic Sanford Formation. Bedrock is lithologically and structurally similar to that at the Auxiliary Dam. Foundation conditions were explored by means of borehole drilling, sampling, and testing.

The stability of the channels was analyzed by means of the slip circle method. Results demonstrate that the slopes of the channels have an adequate factor of safety under all postulated design conditions.

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

2.5.1.1 Regional Geology

The Shearon Harris Nuclear Power Plant is located in the Deep River Triassic Basin near the eastern edge of the Piedmont Plateau in central North Carolina. Physiographically, North

Carolina is divided into three natural divisions: the Coastal Plain on the east, the Piedmont Plateau in the center, and the Appalachian Mountains on the west, as shown on the Regional Physiographic Map, Figure 2.5.1-1. The Deep River Basin is a northeast-southwest trending topographic trough that lies near the eastern edge of the Piedmont Plateau and is occupied by a complex wedge-shaped downfaulted block of Triassic rocks.

As defined in the FSAR, the Deep River Triassic Basin extends southwest from near Oxford in Granville County to west of Carthage in Moore County. It is a structural and topographic trough almost 100 miles long and five to 20 miles wide. It is bounded on the west, north, and east by pre-Triassic metamorphic and igneous rocks of the Piedmont Plateau. On the south and southeast, along the inner edge of the Coastal Plain, there is a cover of post-Triassic (Coastal Plain) deposits that overlap and cover parts of the Triassic and older rocks.

The Deep River Basin is divided into three principal parts (Figure 2.5.1-2). The Durham Basin, in which the plant is located, is the northernmost part; the Sanford Basin is the southernmost; the Colon cross structure separates them.

a) The Sanford Basin - The Sanford Basin extends southwest from Colon in Lee County to the end of the Deep River Basin in central western Moore County, a total distance of 32 miles. It has a maximum width of 14 miles and includes parts of Lee, Moore, and Chatham Counties. Carthage is near the south-western end and Sanford is near the northeastern end of the basin.

b) The Colon Cross Structure - The Colon Cross structure, eight miles long and five miles wide, is entirely in northeastern Lee County. It extends westerly from the northeast edge of the county to the village of Colon, which is four miles northeast of Sanford.

c) The Durham Basin - The Durham Basin, about 52 miles long with a maximum width of 20 miles, extends southwesterly from near Oxford in Granville County to the northeastern border of Lee County. It includes parts of Chatham, Wake, Orange, Durham, and Granville Counties. The City of Durham, for which it is named, lies in the north-central part of the basin. The plant, located in the south-central part, is 19 miles south of Durham and 16 miles southwest of Raleigh.

2.5.1.1.1 Regional Physiography

The Shearon Harris Nuclear Power Plant site is located in the Piedmont Plateau physiographic province. The upland surface of the Piedmont Plateau is an ancient, deeply weathered erosion surface developed on crystalline and volcaniclastic rocks of Precambrian and Paleozoic age. The Piedmont lowlands are developed on Triassic sedimentary rocks occupying structural troughs in the Piedmont Plateau.

The Piedmont upland surface is characterized by broadly undulating topography with narrow, shallow valleys and rounded divides. Upland elevations range from 300 to 600 ft. above sea level along the eastern border of the Piedmont to around 1500 ft. at the foot of the Blue Ridge scarp to the west. Immediately west of the Deep River Basin the upland surface rises to the west and north from between 350 and 375 ft. in northern Lee County to about 600 ft. in the northwest corner of Chatham County, with the slope of the surface averaging about 10 ft. per mile. In a few places monadnocks are present which have elevations several hundred feet higher than the plateau surface. The most prominent monadnocks in the eastern part of the

Piedmont are the Uwharrie Mountains in Randolph and Montgomery Counties. The Occoneechee Mountains are a less prominent series of knobs and ridges in Chatham and Orange Counties.

The Deep River Triassic lowland surface is generally 50 to 200 ft. lower than that of the Piedmont Plateau upland, which borders the lowland on the west, north, and east, and that of the coastal Plain upland, which borders it on the southeast and south. Consequently, the lowland is bordered by abrupt erosional escarpments in many places. Elevations range from less than 160 ft. near the Cape Fear River to more than 500 ft. in some places in the northern part of the lowland. Local relief is generally less than 100 ft. except near the main streams and along some border escarpments. The lowland surface is more intricately dissected than the Piedmont upland surface, and stream courses in the lowland are better adjusted to the structure of the bedrock. Stream valleys are wider in the lowland than in the upland and narrow flood plains are present along the rivers and their larger tributaries. Most smaller tributaries have steep valley sides and narrow valley bottoms. The most extensive flat areas in the lowland are formed by terraces which border some of the main streams such as the Haw, Deep, and Cape Fear Rivers.

The Triassic lowland is bordered on the south and southeast by the Coastal Plain physiographic province, whose inner edge forms a north-facing escarpment with a maximum height of about 250 ft. above the lowland at Carthage. Elevations of the inner margin of the Coastal Plain range from 400 ft. to a maximum of 580 ft. The upland surface on the inner part of the Coastal Plain is characterized by broad interstream divides and flat, wide valleys; surface relief is generally less than that of the Piedmont upland and Triassic lowland surfaces.

2.5.1.1.2 Regional Stratigraphy and Lithology

The Piedmont Plateau in North Carolina is underlain mostly by crystalline and volcaniclastic rocks of Precambrian and Paleozoic age (Figure 2.5.1-3). These rocks can be divided into several broad northeast-southwest trending belts on the basis of differences in metamorphic grade. Metamorphic grade is highest in the westernmost belt, the Inner Piedmont, which contains upper amphibolite grade gneisses and schists. Rocks of lower amphibolite grade are present in the Charlotte Belt, which borders the Inner Piedmont on the east, and in the Raleigh Belt, which lies near the eastern margin of the Piedmont. Metavolcanic rocks and metasediments of mostly greenschist grade characterize the Carolina Slate Belt, which lies between the Charlotte Belt and the Raleigh Belt, and the Eastern Slate Belt, which flanks the Raleigh Belt on the east and forms the easternmost portion of the Piedmont. The Deep River Triassic Basin is a sediment-filled trough located along the eastern margin of the Carolina Slate Belt; it forms the western margin of the Raleigh Belt in places.

The southeastern part of the site region is underlain by Atlantic Coastal Plain sediments of Cretaceous and younger age. Cretaceous sediments overlap the southern margins of the Eastern Slate Belt and the Raleigh Belt and the southeastern margin of the Deep River Basin. Scattered small deposits of Eocene age overlie Cretaceous sediments and crystalline rocks of the Raleigh Belt in places.

A generalized regional stratigraphic column compiled from several sources is shown in Figure 2.5.1-3a. There are no published regional isopach maps pertinent to the Shearon Harris site. The deposits underlying the plant (Triassic sedimentary rocks) are characterized by many lateral and vertical variations. Thus, data obtained from design and exploratory borings cannot

be adapted to generate an isopach map because there is no continuous, easily identifiable marker horizon in the Triassic rocks.

2.5.1.1.2.1 Precambrian and Paleozoic Rocks

Precambrian and Paleozoic rocks in the site region have been grouped into three broad categories. These are (1) metavolcanic rocks of the Carolina and Eastern Slate Belts; (2) gneisses and schists of the Raleigh Belt; and (3) intrusive rocks of mostly granitic composition which are present in the Raleigh Belt and both Slate Belts.

2.5.1.1.2.1.1 Metavolcanic Rocks

The metavolcanic rocks of the Carolina and Eastern Slate Belts include lithic and crystal tuffs, welded flow tuffs, flows, volcanic breccias, volcanic conglomerates, graywacke conglomerates, argillites, slates, phyllites, and thin limestone beds. (Reference 2.5.1-1) which have intertonguing relationships. Their aggregate thickness is believed to be at least 30,000 ft. in the Carolina Slate Belt. These rocks have been subjected to low-grade, greenschist facies metamorphism and in places have a well-developed slaty cleavage. The simple classification used on the Geologic Map of North Carolina (Reference 2.5.1-2) places the metavolcanics in three major groups: bedded argillites (volcanic slate), felsic volcanics, and mafic volcanics. The areal distribution of these groups in the site region is shown on Figure 2.5.1-3. The following descriptions of these three groups is based largely on References 2.5.1-1 and 2.5.1-2.

a) Bedded Argillites - The rocks mapped as bedded argillites include finely bedded meta-shale exhibiting graded bedding, slate, phyllite, and phyllonite as well as argillite. Graywacke and tuff are present in places. This group is represented by slates in the western part of the site region and by phyllites and phyllonites in the eastern part of southern Wake County. The rocks are deeply weathered in most places and outcrops of fresh rock are rare.

b) Felsic Volcanics - This group consists largely of volcanic fragmental and flow materials. The fragmental rocks range from rhyolitic to dacitic in composition and consist mostly of coarse and fine tuffs and subordinate breccias. The coarse tuff predominates and contains the breccia and fine tuff as interbedded bands and lenses. In places the finer material grades into bedded argillite. The flows are essentially rhyolite and occur as narrow bands or lenses interbedded with the tuff and breccia. Lenses of bedded slate and mafic volcanics too small to show on Figure 2.5.1-3 are also present within this group. Felsic volcanics weather to a light gray soil commonly underlain by yellow to light-red clay.

c) Mafic Volcanics - Mafic volcanics consist largely of flows ranging from andesite to basalt in composition and of fragmental rocks of mostly andesitic composition. The fragmental rocks are mostly tuffs, breccias, conglomerates, and graywackes. Andesite and basalt flows occur as narrow bands and lenses interbedded with the fragmental rocks. The andesite is dark green and coarsely porphyritic in places, but most commonly it is fine grained and massive. The basalt is dark gray to nearly black and commonly amygdaloidal. Most rocks in this group weather to a dark red to maroon clayey residuum.

Radiometric age dating of the metavolcanics indicates that they were formed during Late Precambrian and Cambrian time. R-Sr whole rock and U-Pb zircon ages of felsic metavolcanics from the Carolina Slate Belt range from 620 to 520 m.y. (References 2.5.1-3 through 2.5.1-7). However a few plutons which intrude the metavolcanics have ages older than 620 m.y. (References 2.5.1-6 and 2.5.1-8). Near Chapel Hill some metavolcanics are apparently older than the Chapel Hill pluton, which yields a R-Sr whole rock age around 700 m.y. (Reference 2.5.1-6).

2.5.1.1.2.1.2 Gneisses and Schists

Metamorphic rocks of amphibolite grade present in the Raleigh Belt include hornblende gneisses, mica gneisses and schists, and felsic gneisses. The hornblende gneisses are mostly medium-to-coarse-grained, massive amphibolites or well-foliated hornblende gneisses. They commonly contain plagioclase (oligoclase) and quartz in various amounts and lesser amounts of biotite and epidote.

Mica gneisses and schists are medium to coarse grained and consist predominantly of feldspar, quartz, muscovite, and biotite; biotite is more common in the gneisses and muscovite is more common in the schists. Feldspar is predominantly plagioclase but microcline and orthoclase are common.

Felsic gneisses are light-colored, medium-grained rocks characterized by high quartz and microline content and a predominance of muscovite over biotite. Interlayered with these gneisses are garnet-mica schist and two persistent belts of graphite schist.

The age of the Raleigh Belt gneisses and schists is poorly known. As discussed further on, they have been intruded in several places by granitic plutons which have Rb-Sr whole rock ages around 300 m.y. (Reference 2.5.1-9), and thus cannot be younger than Pennsylvanian. The fact that the gneisses and schists form part of a large anticlinorium flanked on both sides by Slate Belt rocks suggests that they are as old as, or older than, the metavolcanics. If they represent the "basement" on which the Carolina Slate Belt rocks were deposited, their age may be one billion years old or older (Reference 2.5.1-10).

2.5.1.1.2.1.3 Intrusive Rocks

The gneisses, schists, and metavolcanics in the site region have been intruded by a wide variety of igneous rocks, of which granitic rocks are by far the most common. These intrusive bodies range in size from small, lens-like bodies to large single plutons and plutonic complexes.

Granitic intrusives in the site region include granite, adamellite, granodiorite, quartz diorite, and tonalite. Most of these can be classified into two broad groups, one pre-dating major regional metamorphism and the other post-dating it. Pre-metamorphic granites characterize the western part of the site region and post-metamorphic granites the eastern part. Pre-metamorphic granites which have been dated radiometrically have ages which range from around 705 m.y. to 519 m.y. Black and Fullagar (Reference 2.5.1-6) report Rb-Sr whole rock ages of 705 m.y. for the Chapel Hill pluton and 638 m.y. for the Meadow Flats granodiorite which is a few miles north of Chapel Hill, and Fullagar (Reference 2.5.1-4) obtained a Rb-Sr age of 519 m.y. for the Farrington complex south of Chapel Hill. U-Pb zircon ages of 575 m.y. were obtained for the Roxboro metagranite (References 2.5.1-5 and 2.5.1-11) and 650 m.y. for the Flat River intrusive complex northeast of Durham (Reference 2.5.1-8). The pre-metamorphic nature of intrusives in

this group is indicated by foliation that is parallel to regional trends and by microtextures that show cataclasis and recrystallization (Reference 2.5.1-12).

Post-metamorphic granitic plutons contain massive, even grained rocks that show little evidence of metamorphism. Most rocks in this group are medium to coarse grained and light to medium gray or light to medium pink. Only a few of these plutons in the site region have been dated radiometrically. These include the Lilesville granite in the southwest corner of the site region, the Wilton pluton on the west border of the Raleigh Belt in Granville County, and the Castalia pluton which straddles the Raleigh Belt-Eastern Slate Belt boundary in the northeastern part of the site region. Ages obtained by the Rb-Sr whole rock method were 313 m.y. for the Castalia pluton, 285 m.y. for the Wilton pluton, and 326 m.y. for the Lilesville pluton (Reference 2.5.1-9).

The Rolesville batholith, which covers an area of 1710 sq km east of Raleigh, was classified as possible syn-metamorphic by Butler and Ragland (Reference 2.5.1-12). It consists of mostly medium-to-coarse-grained, foliated granitic rocks in its main portion. According to Parker (Reference 2.5.1-13) it was formed in two magmatic stages during the mid-Paleozic; however, its exact age is unknown. Structural mapping by Becker and Farrar (Reference 2.5.1-14) indicates that the batholith post-dates the formation of isoclinal folds in the surrounding country rock and pre-dates a second deformational event which refolded the isoclinal folds and produced muscovite-biotite foliation in the granite. The main part of the batholith is apparently younger than the Precambrian and Cambrian plutons found in the western part of the site region and older than the 313 m.y. old Castalia pluton, which is poorly foliated.

Rocks mapped as diorite-gabbro on Figure 2.5.1-3 as a whole are intermediate in composition between true diorite and gabbro. However, bodies of almost normal diorite are present in southeastern Guilford, southern Caswell, and southern Person Counties, and bodies of almost normal gabbro are found in northern Person and northeastern Granville counties (Reference 2.5.1-2). Included in the diorite-gabbro group are some granite-diorite bodies consisting predominantly of diorite. No radiometric ages have been reported for diorite-gabbro intrusions in the site region. Fullagar (Reference 2.5.1-4) suggests that some diorite-gabbro bodies located elsewhere in the southeastern Piedmont are 385 m.y. old or older.

The rocks shown as dunite on Figure 2.5.1-3 are actually lenticular bodies of serpentinite, soapstone, and other altered ultramafic rock. Individual lenses range in outcrop length from a few hundred feet to about three miles (Reference 2.5.1-15). All are poorly exposed and are mapped chiefly by the distribution of float of ultramafic composition on the ground surface.

2.5.1.1.2.2 Mesozoic Rocks

Mesozoic rocks in the site region comprise Triassic sediments deposited in the Deep River Basin, Coastal Plain sediments of Cretaceous age, and diabase dikes and sills of Triassic-Jurassic age.

2.5.1.1.2.2.1 Triassic Sedimentary Rocks

The Triassic sedimentary rocks of the Deep River Basin have been discussed by Campbell and Kimball (Reference 2.5.1.16) and Reinemund (Reference 2.5.1-17), who divided them into three formations: (1) a lower unit, the Pekin Formation, consisting predominantly of fine-grained clastic sediments, (2) a middle unit, the Cumnock Formation, which is coal-bearing in the Sanford Basin, and (3) an upper unit, the Sanford Formation, which constitutes the bedrock in

the vicinity of the plant site. The following descriptions of these formations are from Reinemund (Reference 2.5.1-17).

The Pekin Formation lies unconformably on the metavolcanics and intrusive rocks of the Carolina Slate Belt and crops out in a belt along the northwest side of the Deep River basin. The total thickness of exposed rocks in this formation is 1800 to 4000 ft. in the Sanford Basin, about 3500 ft. in the Colon cross structure, and 3000 ft. at the south end of the Durham Basin. Red, brown, or purple claystone, siltstone, shale, and fine-grained sandstone are the predominant rock types in the Durham and Sanford Basins, and a few beds of conglomerate are present in the basal 50 to 500 ft. of the formation. Conglomerate is abundant in all parts of the formation in the Colon cross structure.

The Cumnock Formation, which lies conformably on the Pekin, is present mainly in the Sanford Basin, where it crops out in a narrow belt. This formation is absent in the Colon cross structure; in the Durham Basin it is exposed for a distance of only 3 1/2 miles along strike in the southernmost part of the basin. The maximum thickness of the Cumnock Formation is 750 to 800 ft. in the eastern part of the Sanford Basin, where two beds of coal are present in the lower part of the formation. The lower bed, known as the Gulf coal bed, is generally less than two ft. thick, and the upper bed, the Cumnock coal bed, is generally less than four ft. thick. Below the coal beds, the Cumnock Formation contains about 250 ft. of gray shale, claystone, siltstone and sandstone and overlying the coal beds is about 500 ft. of gray and black shale and claystone. Coal is absent in the western part of the Sanford Basin, where the Cumnock Formation is about 500 ft. thick and consists mostly of siltstone and sandstone. At the southern end of the Durham Basin the Cumnock consists mostly of silty shale, siltstone, and sandstone, and contains only a few inches of coal.

The Sanford Formation crops out along the southeastern side of the Deep River Basin. It is more than 3000 ft. thick in the Sanford Basin, 2000 to 3000 ft. thick at the southern end of the Durham Basin, and from 500 to 600 ft. thick at the southwestern end of the Colon cross structure. Throughout most of the Deep River Basin, outside of the Colon cross structure, the formation consists of fine-grained red, brown, or purple clastic sediments. The rocks of this formation coarsen toward the southeast and form a nearly continuous belt of conglomerate and fanglomerate along the southeastern edge of the basin. The formation consists largely of conglomerate and fanglomerate in the Colon cross structure.

2.5.1.1.2.2.2 Triassic - Jurassic Diabase

Diabase intrusions, generally regarded as Late Triassic or Jurassic in age, are common in the Raleigh Belt, Carolina Slate Belt, and the Deep River Basin, although only the larger dikes are shown on Figure 2.5.1-3. These intrusions most commonly have the form of dikes which range from a few inches to several hundred feet in width and from a few feet to several miles in length. Sills and sill-like masses of diabase are present in the Deep River Basin and range up to 400 ft. in thickness. According to Reinemund (Reference 2.5.1.17) diabase intrusives occupy about four percent of the Deep River Basin.

The diabase dikes include both olivine-normative and quartz-normative types (Reference 2.5.1-18). Typical diabase is dark gray to greenish black, fine to medium grained, and consists of about 50 percent plagioclase, 25 to 30 percent augite, 10 to 20 percent olivine, and accessory magnetite, ilmenite, apatite, pyrite, titanite, biotite, and amphibole. Many of the dikes in the Deep River Basin and the Carolina Slate Belt show the effects of zeolitization.

Reconnaissance mapping of the dikes in the eastern Piedmont by Burt and others (Reference 2.5.1-19) indicates that two major dike trends are present, one N10°-30°W and the other north-south. A third, less prominent group of dikes, trends east-northeastward.

2.5.1.1.2.2.3 Cretaceous Sediments

Coastal Plain deposits, generally regarded as Cretaceous in age, overlap the entire southern portion of the site region (Figure 2.5.1-3). The basal unit of the Coastal Plain deposits is shown as the Tuscaloosa Formation on the Geologic Map of North Carolina (Reference 2.5.1-2). The Tuscaloosa Formation can be divided into lower and upper members. The lower member rests unconformably on Triassic rocks of the Deep River Basin and crystalline rocks of the Piedmont. According to Conley (Reference 2.5.1-20) the basal part of this member consists of gray carbonaceous clays containing lignitized plant remains and amber, with interbedded thin, gray and olive sand beds. Above the base the clays are less carbonaceous and lighter gray in color. The upper part consists of light olive clayey sand beds containing thin clay beds. The lower member of the Tuscaloosa was probably deposited in a marine environment (References 2.5.1-20 and 2.5.1-21).

The upper member of the Tuscaloosa Formation unconformably overlaps the lower member as well as Piedmont crystalline rocks and Deep River Basin sediments. It consists of a heterogeneous sequence of lenticular clays, muddy sands, clean sands, and pebbly sands (Reference 2.5.1-21). Its base is unconsolidated gravel composed of rounded quartz, ranging from one to six in. in diameter, in a matrix of kaolinitic clay and clayey sand (Reference 2.5.1-20). Most authorities consider this member to be of fluvial origin.

The Tuscaloosa Formation is overlain by the Black Creek Formation which, according to Swift and Heron (Reference 2.5.1-21), has an interfingering contact with the Tuscaloosa's upper member. The Black Creek Formation consists of laminated, medium dark gray to dark gray clay interbedded with medium gray to yellow orange sands. Primary sedimentary structures within the formation indicate that it was deposited in a near-shore marine environment.

2.5.1.1.2.3 Cenozoic Rocks

Cenozoic rocks in the site region include Tertiary marine deposits, high level surficial deposits of possible Tertiary age, and Quaternary terrace gravels and alluvium. The Tertiary marine deposits (Castle Hayne Limestone and the Yorktown Formation) are shown on Figure 2.5.1-3; the other Cenozoic deposits are mostly too small to be shown at the scale of Figure 2.5.1-3.

The main outcrop belt of the Eocene Castle Hayne Limestone lies to the southeast of the site region. It lies unconformably on older rocks and at one time evidently covered much of the Coastal Plain and even extended onto the Piedmont Plateau (Reference 2.5.1-2). It is present in the site region as scattered deposits lying on crystalline rocks of the Raleigh Belt in Wake and Johnston Counties and overlying Cretaceous rocks in the southern part of the site region. The formation consists in part of light gray fossiliferous limestone and in part of light-gray marl (Reference 2.5.1-2).

The Yorktown Formation of Miocene age overlaps Eastern Slate Belt rocks in places on the eastern margin of the site region. It consists largely of clay, sand, and shell marl in surface exposures (Reference 2.5.1-2).

High-level, unconsolidated gravels, sands, and clays of unknown age occupy a belt 10 to 40 miles wide along the western border of the Coastal Plain in North Carolina. These deposits lie unconformably in part on older Coastal Plain sediments and in part on crystalline rocks of the easternmost Piedmont (Reference 2.5.1-22). They form discontinuous sheets and patches at elevations of from 270 to about 500 ft. above sea level. These high-level deposits have not been mapped on a regional scale, but Reinemund (Reference 2.5.1-17) mapped "high-level surficial deposits" in the Deep River Basin. However, Reinemund considered all of the Coastal Plain deposits high-level gravel, not recognizing the upper member of the Tuscaloosa Formation which directly underlies the gravel deposits throughout Moore County (Reference 2.5.1-20). Conley (Reference 2.5.1-20) proposed the name Pinehurst Formation for the nonfossiliferous gravel and sand deposits which unconformably overlie the upper member of the Tuscaloosa Formation, and cap all the higher Coastal Plain Hills in central and western Moore County. The formation is normally brown or grayish brown, commonly iron stained, and in places it is cemented with hematite or, less commonly, limonite. Hematite concretions and kaolinitic clay concretions are commonly interspersed throughout the formation (Reference 2.5.1-20). Reinemund (References 2.5.1-17) thought it possible that the high-level surficial deposits he mapped included materials as old as Cretaceous and as young as Pleistocene, while Conley (Reference 2.5.1-20) thought it conceivable that the surficial gravels could be Late Miocene, Pliocene, or early Pleistocene age.

Terrace deposits are present along the Deep, Haw, and Cape Fear Rivers and along some of their larger tributaries (Reference 2.5.1-17). These deposits consist predominantly of friable silty or sandy clay enclosing irregular patches or thin lenses of sand and gravel. They are probably of Pleistocene age (Reference 2.5.1-17).

Alluvium is present along most streams flowing across the Deep River Basin and the Coastal Plain. It consists predominantly of chocolate-brown and grayish-brown silt with some gray organic clays.

2.5.1.1.3 Regional Structure and Tectonic Features

The site region is characterized structurally by northeast-trending belts of isoclinally folded and metamorphosed Precambrian and Paleozoic rocks which are partly overlain by two generally wedge-shaped, northeast-trending masses of southeast-dipping, mostly Mesozoic sediments. Triassic rocks of the Deep River Basin constitute the smaller sediment mass and the Cretaceous rocks of the Inner Coastal Plain the larger.

2.5.1.1.3.1 Precambrian and Paleozoic Features

As discussed in Section 2.5.1.1.2, the Precambrian and Paleozoic rocks of the site region can be divided into three northeast-trending belts on the basis of metamorphic grade. Metamorphic grade is highest in the Raleigh Belt (amphibolite facies) and is lowest in the Slate Belts which flank it (greenschist facies).

2.5.1.1.3.1.1 Carolina Slate Belt

The metavolcanic rocks of the Carolina Slate Belt have been deformed into a series of northeast-trending folds that range from broad and open to tightly compressed (Reference 2.5.1-23). Two major fold structures have been recognized in Carolina Slate Belt rocks of the

site region. These are the Troy Anticlinorium in the southern portion and the Virgilina Synclinorium in the northern portion.

The Troy Anticlinorium is the dominant feature of the Carolina Slate Belt in Randolph, Moore, and Montgomery Counties. It trends northeast and plunges toward the southwest (Reference 2.5.1-20). It is over 30 miles wide between the Pee Dee River on the west and the northwestern corner of Moore County on the east. A series of normally doubly plunging folds ranging in wavelengths from one to three miles are developed on the southeastern flank of the anticlinorium (Reference 2.5.1-20). These minor folds are either asymmetrical or overturned toward the southeast and have axial plane dips that range from vertical to less than 60 degrees northwest (Reference 2.5.1-1). Minor folds on the west flank of the anticlinorium are broad and open and only slightly asymmetrical with axial planes dipping steeply northwest.

The Virgilina Synclinorium, divided by the North Carolina - Virginia border, is the major Carolina Slate Belt structure, and is located in eastern Person County and northwestern Granville County. Its width in North Carolina is about 32 km (Reference 2.5.1-5). The synclinorium trends northeast but its axial trace cannot be located exactly because there is some faulting in the axial region of the structure and the axial plane cleavage is younger than the fold buckling which formed the synclinal structure. Numerous small folds of about one mile wavelength are present on the flanks of the synclinorium; these folds plunge gently northeast or southwest (Reference 2.5.1-5). According to Tobisch and Glover (Reference 2.5.1-24), folding began before the onset of metamorphism and the buckle folds produced by the initial deformation were later modified by metamorphism and a cleavage-forming deformational event which tightened the early folds where it was coaxial with them or refolded them where it was not. Glover and Sinha (Reference 2.5.1-5) have presented evidence that the major syncline was produced by a major compressional event about 575-620 m.y. ago during the Late Precambrian or early Cambrian, and that the Slate Belt regional metamorphism probably occurred between 520 and 300 m.y. ago.

No faulting of regional extent has been mapped in Carolina Slate Belt rocks of the site region (excluding the Jonesboro Fault discussed below). Conley (Reference 2.5.1-20) mapped two groups of faults of local extent in Moore County. One group trends northeast parallel to the axes of folds, and the other trends northwest across fold axes. In Person County wrench faulting is responsible for at least a mile of left lateral offset of the fold axis of the Virgilina Syncline (Reference 2.5.1-5).

Faulting of regional extent is present outside the site region along the western edge of the Carolina Slate Belt, where the Gold Hill-Silver Hill fault zone marks a sharp break between low-grade Slate Belt rocks on the east and plutonic rocks of the Charlotte Belt on the west (Reference 2.5.1-23). Butler and Fullagar (Reference 2.5.1-25) have established on the basis of radiometric age dating that major movement along the Gold Hill Fault probably occurred during the Devonian period.

2.5.1.1.3.1.2 Raleigh Belt

The Wake-Warren Anticlinorium is the dominant structural feature of the Raleigh Belt. As defined by Parker (References 2.5.1-15 and 2.5.1-26) it is at least 30 miles wide and about 90 miles long in North Carolina; it continues beyond the state boundary into Virginia. The axial plane of the anticlinorium strikes N20°E and dips 70 degrees southeast. The axis plunges southward at a very steep angle of perhaps 70 degrees (Reference 2.5.1-15). The axial portion

of the anticlinorium is occupied by a granitic batholith, generally referred to as the Rolesville granite, which is about 50 miles long and 10 to 15 miles wide. The granite core is flanked by metamorphosed rocks which dip steeply in most places and appear to have been closely and, in places, isoclinally folded. A Barrovian-type metamorphic facies series, ranging from lower greenschist to middle almandine-amphibolite facies, is concentric with the batholith (Reference 2.5.1-13).

2.5.1.1.3.1.3 Mesozoic Features

The major Mesozoic structural feature in the site region is the Deep River Basin, a complex, wedge-shaped block of rocks occupying a northeast-trending, trough-like depression in the older rocks in the Piedmont Plateau. The northwestern edge of this basin is formed in part by the unconformable and geographically irregular contact of Triassic sediments with underlying rocks and in part by northeast-trending longitudinal faults. The southeastern edge is formed by a fault zone, the easternmost fault of which is the Jonesboro Fault (Reference 2.5.1-27), a northeast-trending diagonal slip fault with a total vertical displacement of 5,000 to 10,000 ft. and unknown right-lateral displacement. Within this block, the Triassic sedimentary rocks generally dip about 15° southeast. Local reversals of dip eastward and northward also occur as a result of faulting and intrusion of diabase dikes (Reference 2.5.1-27). Two systems of normal faults, northeast-trending major longitudinal faults and northwest-trending minor cross faults, have broken the rocks of the southern half of the Deep River Triassic Basin into irregular blocks as small as 1 km x 3 km (Reference 2.5.1-27).

The Jonesboro Fault and related faults just to the west are among the most significant in North Carolina. They are part of a northwest-dipping fault zone with a vertical displacement of 5,000 to 10,000 ft. For more than 100 miles, the Jonesboro Fault forms the contact between Triassic and older rocks along the southeastern side of the Deep River Basin. The other related faults are known only from geophysical studies. Five other major longitudinal faults, all paralleling the Jonesboro Fault, but not part of the fault zone, are the Deep River, Gulf, Indian Creek, Governors Creek, and Crawley Creek faults. There are also several faults that branch from the Crawley Creek and Governors Creek faults, and a number of minor longitudinal faults. These five, their branches, and other minor longitudinal faults all occur in the northwestern part of the Sanford Basin west of the Cape Fear River. No faults of this type were mapped by Reinemund in the Durham Basin and none were recognized during the siting and design investigations of the SHNPP area. Harrington (Reference 2.5.1-28) mapped minor longitudinal faults along the northwestern edge of the Durham Basin, but these are farther from the SHNPP area than those in the northwestern part of the Sanford Basin.

2.5.1.1.4 Regional Seismicity

The region in the immediate vicinity of the site is characterized by low-level seismicity. During historical time no earthquake epicenters have been recorded within 40 miles of the plant site. Moderate levels of earthquake activity occur in the surrounding region at distances greater than 130 miles from the site. During the period 1754-1977 eight earthquakes of epicentral intensity VII or greater occurred within about 200 miles of the site. Six were of intensity VII, the closest of which occurred at a distance of about 133 miles from the site. The Charleston, South Carolina earthquake of August 31, 1886, which occurred about 200 miles south of the site, had an intensity of X and was probably felt with an intensity of VI in the site area. The earthquake in Giles County, Virginia on May 31, 1897, which occurred about 160 miles northwest of the site had an MM intensity of VIII and was probably felt in the site area with an intensity of about V.

Further details on regional seismicity are given in Section 2.5.2.

2.5.1.1.5 Regional Geologic History

The geologic history of the central and eastern Piedmont region is poorly known because fossil-bearing strata are extremely rare and geochronology is based largely on radiometric dating of igneous events. The geologic record suggests that island arc volcanism was the dominant activity from Late Precambrian through Cambrian time. A period of major deformation of early volcanogenic deposits around 600 m.y. ago formed the major folds of the Carolina Slate Belt. This deformation was accompanied or closely followed by emplacement of granitic plutons. The deformational event was followed by renewed volcanic activity possibly indicating development of a new island arc system during Cambrian time. This volcanism continued through late Cambrian time and was followed in early Ordovician time by a metamorphic event which produced greenschist metamorphism in Carolina Slate Belt rocks. Another major deformational event occurred during Devonian time which involved major movement in the Gold Hill fault zone, greenschist metamorphism, and emplacement of granitic plutons in the Charlotte belt. The last clearly indicated major Paleozoic event was emplacement of granitic plutons in the Charlotte and Carolina Slate Belts during Pennsylvanian time. The earliest clearly recorded Mesozoic event is the deposition of late Triassic sediments in subsiding northeast-trending troughs in the eastern (Deep River - Wadesboro Basin) and western (Dan River Basin) parts of the Piedmont. In the Deep River Basin normal fault movement along segments of the Jonesboro fault system and the resulting differential subsidence caused eastward tilting of sedimentary strata. Accumulation of the sedimentary wedge was followed by continued movements in the Jonesboro fault zone and development of cross-basin faults. Emplacement of diabase sills and dikes followed formation of the cross faults and continued into Jurassic time. Final movement of the Jonesboro Fault during Jurassic time was followed by widespread zeolite mineralization related either to low-grade burial metamorphism or to high heat flow and hydrothermal activity. Little is known of late Mesozoic and Tertiary history. The region apparently has been relatively stable tectonically since late Mesozoic time.

Crustal movement has largely been limited to vertical isostatic adjustments possibly related to periodic uplift of the Appalachians to the west and subsidence of the Coastal Plain to the east.

2.5.1.2 Site Geology

The Shearon Harris project site includes the Shearon Harris Nuclear Power Plant and two cooling water reservoirs. The power plant, Auxiliary Reservoir, and most of the Main Reservoir are located in the Deep River Basin and, as shown in the site area geologic map in Figure 2.5.1-4, are underlain by Triassic sedimentary rocks. The Main Reservoir Dam is located approximately 3000 ft. southeast of the Jonesboro Fault and is underlain by pre-Triassic crystalline rocks.

2.5.1.2.1 Site Physiography

The Triassic sediments of the Deep River Basin are more easily eroded than the igneous and metamorphic rocks of the Piedmont Plateau or the porous sand and gravel deposits of the Coastal Plain. As a result, the Triassic basin is a trough-like topographic lowland for most of its length. It is bounded on the northwest, north, and east by the upland surface of the Piedmont Plateau and on the southeast and south by the upland surface of the Coastal Plain.

The Triassic lowland is an intricately dissected surface with a local relief generally less than 100 ft. Interstream divides are sharp and narrow near the main streams, but away from the main drainage lines they become higher, broader, and flatter. In the southern part of the Durham Basin (the area of interest), the general slope is toward the Cape Fear River, which crosses the Triassic lowland at the south end of the Durham Basin. Most of the streams in the Traissic lowland have a dendritic pattern and are actively deepening their valleys. In general, valley sides are comparatively steep and valley bottoms are narrow, but narrow flood plains are present along some of the larger streams.

Elevations in the Triassic lowlands range from less than 160 ft. above sea level along the Cape Fear River to approximately 500 ft. above sea level in the northern part of the Durham Basin. In the general Power Plant and Auxiliary Reservoir area, the lowland is some 200 to 300 ft. above sea level and 50 to 200 ft. below the adjacent surfaces of the Piedmont Plateau and the Coastal Plain.

These gently rolling hills and low-gradient streams are a result of a long period of erosion that has stripped away large volumes of relatively soft Triassic rocks, leaving the Deep River Triassic Basin as a muted topographic low. Because of the muted topographic features and the long period of topographic inactivity that produced them, there are no landslides, areas of uplift or subsidence, or other natural features which could be potentially hazardous to the plant. There are also no activities or man-made features in the area which have the potential for affecting site safety.

2.5.1.2.2 Site Stratigraphy and Lithology

The rocks of the site area are predominantly well-consolidated Triassic sedimentary rocks. A few diabase intrusive rocks are also found in the vicinity of the site (Figure 2.5.1-4). Remnants of the flat-lying poorly consolidated sediments of the Coastal Plain, terrace gravels, and Holocene alluvial deposits occur in the vicinity of the site. The pre-Triassic crystalline rocks which underlie the Main Dam are described in Section 2.5.6.

2.5.1.2.2.1 Triassic Sedimentary Rock

The Triassic sedimentary rocks of the Deep River Basin are clastic fluvial deposits: claystone, shale, siltstone, sandstone, conglomerate, and fanglomerate. They are characterized by abrupt changes in composition. In some places coarse sediments predominate; in others fine-grained materials are present. Lithologic units are seldom more than a few feet thick. In general, three-fourths of the rocks in the Deep River Basin are red, brown, or purple; the rest are gray or black.

The terms "arkosic" or "feldspathic," as used in this report and applied to the Triassic rocks of the southern part of the Durham Basin, refer to any rock that has an appreciable quantity of feldspar (at least enough to be readily recognized in a hand specimen). Many of the sandstones are arkosic but few contain sufficient feldspar (25 percent) to be true arkoses.

The sediments of the Deep River Basin are composed largely of debris from nearby pre-Triassic metamorphic and igneous rocks; in places they contain much debris from nearby granite intrusive bodies. These sediments were deposited as alluvial fans, stream-channel and flood plain deposits, and lake and swamp deposits.

The Triassic bedrock underlying the plant site and adjacent areas is covered by overburden that is partly residual, partly transported. On the upland areas, the overburden consists of residual yellow, sandy clay and sandy loamy soil derived from the weathering of underlying rock. This soil is usually only two to six ft. thick, but in areas of erosion, it has been mostly removed by running water. In stream valleys and bottoms, yellow, sandy, clayey alluvium has often accumulated to thicknesses of from two to four ft. to as much as 10 to 12 ft. Beneath this overburden, the Triassic sedimentary rocks are usually dense, compact, and only slightly weathered, with a variably developed, thin saprolite zone.

In some areas, notably that near the west abutment and spillway of the Auxiliary Dam, as shown on Figure 2.5.1-4, the Triassic rocks are overlain by younger sedimentary rocks, which were studied intensively only at the place where they overlie and are undisturbed by the site fault (Section 2.5.3). There they were correlated with the undifferentiated high-level deposits described by Reinemund (Reference 2.5.1-17).

The sedimentary rocks of the Deep River Triassic Basin have been divided into three units which from oldest to youngest are the Pekin, Cumnock, and Sanford Formations. The Pekin and Sanford Formations are mostly red, brown, or purple siltstone, claystone, shale, sandstone, conglomerate, and fanglomerate. The Cumnock Formation consists of gray and black claystone, shale, siltstone, fine-grained sandstone, and two beds of coal.

a) Pekin Formation - The Pekin Formation, oldest of the three Triassic formations in the Deep River Basin, lies unconformably on pre-Triassic metamorphic and igneous rocks and crops out in a narrow belt one to three miles wide along the northwestern side of the basin. In general, the Pekin Formation is not particularly well exposed in the Deep River Basin and knowledge of its lithologic character is based on the examination of many small, scattered outcrops. The best exposures of this formation occur in the Wadesboro Triassic Basin near the town of Pekin, Montgomery County, for which it is named.

    The total thickness of exposed rocks in the Pekin Formation is from 1,800 to 4,000 ft. At the southern end of the Durham Basin, the thickness is approximately 3,000 ft. Practically all the rocks in this formation are red, brown, or purple. In general they are medium to fine-grained clastic rocks consisting of claystone, shale, siltstone, and sandstone, with a few beds of conglomerate and fanglomerate near the base of the formation.

    The eastern limits of the Pekin Formation are well to the northwest of the plant site; therefore, rocks of this formation are of no importance at the site.

b) Cumnock Formation - The Cumnock Formation lies conformably on the Pekin Formation and crops out in a narrow belt in the southern end of the Durham Basin. It consists of gray and black claystone, shale, siltstone, fine-grained sandstone, and two beds of coal. The formation was named for the Cumnock coal mine where 460 ft. of strata were exposed in the shaft. In the southern part of the Durham Basin the Cumnock Formation is only a few hundred feet in thickness and width of outcrop. The northernmost outcrop mapped by Reinemund (Reference 2.5.1-17) is about one mile southwest of Merry Oaks in Chatham County. The formation lies well to the southwest of the plant site.

c) Sanford Formation - The Sanford Formation was named for the town of Sanford, which lies in a belt where this formation is more than 10 miles wide. It lies conformably on the

Cumnock Formation in the Durham and Sanford Basins, but apparently lies unconformably on the Pekin Formation in the Colon cross structure, where the Cumnock Formation is missing. The Sanford Formation is more than 3,000 ft. thick in the Sanford Basin and 2,000 to 3,000 ft. thick in the southern edge of the Durham Basin. It borders the southeastern edge of the Deep River Basin throughout most of its length and reaches a width of several miles in the Sanford Basin and the southern part of the Durham Basin. Rocks of the Sanford Formation underlie almost two-thirds of the southern half of the Deep River Triassic Basin, including the plant site.

The Sanford Formation is composed of clastic sediments consisting of claystone, shale, siltstone, sandstone, conglomerate, and fanglomerate, more than three-fourths of which are red, brown, or purple. The remaining are various shades of gray. In the Durham Basin, rocks of the upper part of the Sanford Formation, consisting of conglomerate and fanglomerate, are exposed in a zone one to two miles wide along the eastern and southeastern side of the basin adjacent to the Jonesboro Fault. Between the zone of conglomerates and fanglomerates and the zone of rocks of Pekin age along the western and northwestern side of the basin, fine grained sediments of the lower part of the Sanford Formation, consisting of shale, siltstone, and sandstone, are present. The Shearon Harris Nuclear Power Plant Site is located in this area of fine-grained sediments (Figure 2.5.1-4).

The sediments of the Sanford Formation underlying the plant site and much of the southeastern part of the Durham Basin were deposited as alluvial fans and stream channel and flood plain deposits. These materials are characterized by abrupt changes in composition and texture, both horizontally and vertically. They contain few distinctive beds and subdivisions that are consistently mapable. The beds vary in thickness from less than an inch to a maximum of 15 to 20 ft. As a result, exposures only a few feet apart may vary considerably in texture and composition. Notwithstanding these variations in composition and texture, the beds and lenses interfinger and overlap into compact masses that show no structural weakness. Because the rocks dip gently to the southeast, the plant is not founded on any single layer. As shown in the foundation report (Appendix 2.5E), layers of fine to medium sandstone are the most common foundation rock.

2.5.1.2.2.2 Triassic-Jurassic Intrusive Rocks

Triassic sedimentary rocks in the Deep River Basin have been intruded by Triassic-Jurassic dikes, sills, and sill-like masses. The dikes are from a fraction of an inch to more than 300 ft. wide and from a few ft. to more than seven miles long. Sills and sill-like intrusives vary from a few inches to more than 200 ft. thick. These intrusives, commonly classed as diabase, occupy about four percent of the total area of the Deep River Trissic Basin.

Sills and sill-like intrusives are almost completely confined to the Cumnock Formation in the Deep River Coal Field. They are most abundant between the towns of Gulf and Haw Branch, where one-third to one-half of the Cumnock Formation is occupied by sills and sill-like intrusives as much as 400 ft. thick. The only exposed sill of any appreciable size known to occur in the Sanford Formation crops out near Euphonia Church in western Lee County. No sills or sill-like masses are known to occur in the southern part of the Durham Basin.

Diabase dikes are abundant in the Sanford Basin and the Colon cross structure. They generally follow northwest-trending joints and cross faults along which displacements are less than 50 ft. Dikes in the Sanford Basin trend N25 to 40W, whereas those in the Colon cross structure and the southern end of the Durham Basin trend N15 to 20W or N-S.

2.5.1.2.2.3 Post-Triassic-Jurassic Sedimentary Deposits

South of the plant site, the Triassic sedimentary rocks, the younger intrusive igneous rocks, and the trans-basin faults are overlain by undisturbed, flat-lying weakly consolidated Cretaceous marine sediments (References 2.5.1-20 and 2.5.1-21). These could, however, be remains of the Jurassic burial event described in Section 2.5.3 and therefore could be pre-Cretaceous. Probable up-dip, non-marine remnants of these deposits are found nearer the plant site, as are probable later Tertiary deposits (which record sea-level fluctuations) and Quaternary terrace gravels. None of these deposits were formed by tectonic movements.

2.5.1.2.3 Site Structural Geology

The only major structural feature present in the site vicinity is the Jonesboro Fault, whose trace crosses the lower end of the Main Reservoir less than a mile north of the Main Dam, as shown on Figure 2.5.1-4. This fault is covered in places by unbroken Cretaceous sediments southwest of the Main Reservoir Dam and is considered to be inactive. A minor high-angle fault was discovered in the foundation of the plant during excavation. This fault was subjected to an intensive investigation (Reference 2.5.1-29) which led to the conclusion, with which the NRC concurred, that the fault is not capable (see Reference 2.5.3-5). The fault and the investigation of its capability are described in Section 2.5.3, where it is referred to as the site fault. Other minor faults, all judged to be non-capable, were mapped in the foundation of the Main Dam in the pre-Triassic crystalline rocks. Most of these faults are only a few tens of feet long with only several inches of displacement. Written reports on these features as presented to the NRC are included in Appendix 2.5.E.

Folding in the site area is most common in gneisses and schists exposed in the foundation of the Main Reservoir Dam. These rocks appear to have undergone several periods of folding with isoclinal folding predominating. The magnitude of this folding could not be determined because of the limited area of exposure of the crystalline rocks. The only folding observed in Triassic sedimentary rocks were drag folds resulting from movement along the site fault.

Joints are common in both the pre-Triassic crystalline rocks and the Triassic sedimentary rocks. The dominant joint set in the crystalline rocks at the Main Dam strikes approximately N60°-70°E and dips 50° to 70° to the southeast. Another set strikes N20°-35°W and dips 70° to 90° southwest. Joints in the Triassic rocks of the power plant and Auxiliary Dam areas are most prominent in the sandstones and siltstones. Three joint sets are present. The two dominant sets are approximately vertical, one striking N40°-50°E and the other N20°-30°W. A third set strikes north-northwest and dips 55° to 70° southwest. Most of the joints are tight and do not extend vertically for more than a few feet.

2.5.1.2.3.1 Geophysical Studies of Structural Features

a) Gravity Studies - To define geological structures in the basin, Mann and Zablocki (Reference 2.5.1-30) made gravity measurements over the Deep River Wadesboro Basin to establish (1) location of concealed structures within the eastern Triassic Basin, (2) location

of the northwestern and southeastern borders of the basin where they are covered by a post-Triassic overlap, and (3) determination of the southeastern extent of the Sanford Basin. They took measurements with a Worden Gravimeter at 1,200 stations spaced about one mile apart. These measurements were relative to the gravity base station at Chapel Hill established by Woollard and Mann in 1956.

Assuming a specific gravity of 2.67, Mann and Zablocki (Reference 2.5.1-30) computed Bouguer anomalies, shown on Figure 2.5.1-5, with an isomilligal interval of 5 mgals. They also made a residual anomaly map with a contour interval of 5 mgals in an attempt to separate the deep-seated features from surface and near-surface geologic effects. The Triassic area is shown in dashed lines on Figure 2.5.1-5. The surface and near-surface geologic effects and the probable basement and intrabasement variations in lithology mask to a degree the limits of the Triassic basin.

Two gravity highs border the basin, one on the northwestern side of the Durham Basin, the other on the southeastern side of the Wadesboro Basin. Relative gravity lows, indicated by the 5 and 0 mgal isogal lines, exist in the Wadesboro, Sanford, and Durham Basins. The only negative gravity areas are located in the southeastern part of the Durham Basin and east of the Sanford Basin; these measure approximately -5 mgals. The gravity low and negative anomaly areas were chiefly attributed by Mann and Zablocki (Reference 2.5.1-30) to especially great thicknesses of Triassic rocks. The isogals cross the Jonesboro Fault at right angles in the southern part of the Durham Basin, but north and southwest of this area they cross obliquely. The isogals on the residual map also cross the Jonesboro Fault obliquely for most of its trace. Mann and Zablocki (Reference 2.5.1-30) believed that the primary reasons this great fault is masked are variations in the basement and near-surface lithology surrounding the basin.

The Colon cross structure, the constriction between the Durham and Sanford Basins, is a northwest-trending anticlinal warp. As a result of this feature, the isogals surrounding this area have a saddle-like appearance. Mann and Zablocki (Reference 2.5.1-30) correlated the longitudinal faults, cross faults, and diabase dikes mapped by Reinemund (Reference 2.5.1-17) with finger-like deviations of the isogals in the Sanford Basin. On that basis, they postulated that similar projections in the Wadesboro and Durham Basins stem from the same type of structural features.

The negative-anomaly area along the eastern border of the Durham Basin south of Raleigh is probably associated with the large granite body in that area, the gravitational effects of which extend into that part of the basin indicated by the -5 mgal isogal. With the exception of this area, the deeper parts of the eastern Triassic basin are enclosed by 5, 10, and 15-mgal isogals. Mann and Zablocki (Reference 2.5.1-30) correlated the negative anomaly in the center of the Sanford Basin with the network of longitudinal faults and cross faults indicated on the geologic map (Figure 2.5.1-3). These faults may have caused the basement in this area to subside more than in other parts of the Deep River Wadesboro Basin.

Figure 2.5.1-6 is a map of the Deep River-Wadesboro Basin showing the location of traverse lines across the basin. The profile from Pittsboro to Raleigh through Cary, and the profile from Siler City to Sanford were selected for further examination, because the power plant site lies in the area bounded by these profiles.

The interpretation of gravity anomalies is complicated by the unknown configuration and physical properties of underlying geological structures. A useful approach to interpretation can be made by application of "direct methods" described by Tanner (Reference 2.5.1-31), which obtain the configuration of a body directly from the observed anomaly, provided that the general shape and physical properties of the body are specified . Direct methods have been successfully used to interpret gravity anomalies caused by sedimentary basins and granite bodies (References 2.5.1-31 and 2.5.1-32).

Anthony Watts, of Lamont-Doherty Geological Observatory, Columbia University, used a computer program he developed to compute the basement configuration, assuming a certain density contrast. Figure 2.5.1-7 shows the residual anomaly values for the traverse from Pittsboro through Cary to Raleigh. It also shows the computed values for the basement configuration illustrated in that figure, for a density contrast of -.015 gm/cm^3. The computed values agree well with observed data. The basement rocks in the western half of the basin appear to lie closer to the surface than the rocks in the (graben-like) eastern half, which are probably 5.2 km thick. The maximum measured anomaly difference, about 12 milligals, is between the basin sediments and the adjacent bedrock. The great displacement of the Jonesboro Fault and related faults is dramatically emphasized in this profile.

The residual anomaly values along the profile through the northern part of the Sanford Basin from Siler City through Sanford are shown on Figure 2.5.1-8. This basin appears wider near the crystalline basement than does the Durham Basin. The basement floor is closest to the surface near the northwestern side. The maximum value of the residual gravity anomaly is about 8 mgals and model computations indicate the maximum thickness to be about 2.2 km for a density contrast of -0.15 gm/cm^3. The geologic model that can explain the residual anomaly is indicated at the bottom of Figure 2.5.1-8.

Conclusions

The isogals of the Bouguer and residual anomaly gravity maps cannot be correlated satisfactorily with either the outline of the basin or the entire trace of the Jonesboro Fault. The fault discovered at the power plant is a minor feature, not reflected in these records. It is clear from an examination of the Bouguer map that the deepest parts of the eastern Triassic basin are located in the northwestern and southeastern part of the Sanford Basin and the southern part of the Wadesboro Basin.

Although the geologic features are not exactly defined on the gravity maps, the gravity profiles clearly show the basement configuration of the Deep River Wadesboro Basin. Model calculations suggest a deep accumulation of Triassic sediments and also show the Jonesboro Fault as a steeply dipping feature on the southeastern side of the basin. To locate dikes and/or faults in the area, it is necessary to have station spacing of about one-tenth of a mile or less; therefore, the gravity measurements of Mann and Zablocki cannot be used to locate these features. Recontouring and computer modeling of available gravity data have not revealed any information unfavorable to the plant site. Figures 2.5.1-7 and 2.5.1-8 show schematic cross-section models of a Triassic basin which fit the gravity data.

b) Aeromagnetic Studies - To further define geologic structures in the area, previously compiled aeromagnetic studies were examined. The examination revealed northwest-trending lineaments near the site (Figure 2.5.1-9), representing diabase dike swarms in this otherwise magnetically quiet area. The apparent low-order lineament aligned east-west

near the site is probably not a true magnetic lineament, but a normal error involving the east-west flight alignment of the aeromagnetic survey. The en echelon magnetic signature of the diabase dike swarm passing immediately through the site is discussed further in Appendix D of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29).

2.5.1.2.3.2 Remote Sensing Imagery

Positive imagery acquired for the fault investigation by Side-Looking Airborne Radar has been examined exhaustively. Conventional high- and low-altitude aerial photography, Skylab imagery and ERTS composites have been examined in great detail. Location, length and alignment of several hundred linear features were identified. None were identified as capable faults on the basis of imagery evaluation, nor were any of these linear features that were field checked identified as faults.

Ground-truth assessments in the field indicated no evidence of recent earth movements in the site area. The site fault was not detected by any imagery techniques. Appendix C of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29) contains a full description of the remote sensing imagery evaluation.

2.5.1.2.3.3 Stresses

Prouty (References 2.5.1-33) noted that the absence of folding of sediments in the Durham Basin seemed to confirm the absence of strong compressional forces since the time the sediments were laid down, an observation consistent with data and interpretations of the Deep River Basin over the intervening years.

The relationship between residual stress and contemporary tectonic stress must be considered. The levels of stresses locked into some rocks by past, no longer active stress patterns are probably greater than stress levels under present tectonic conditions. For example, in some underground works it is commonly found that the crystalline rocks are under sufficiently high stress to induce explosive fragmentation of rocks, known commonly as pop-outs and rock bursts, around underground openings.

In contrast, throughout the Deep River Coal Field, students of mining activities, including Dr. J. L. Stuckey, report lack of apparent stress in underground openings, some as deep as 800 ft. below the surface. Dr. Stuckey recalls that during visits to the mines in the early 1920s, Howard N. Butler, the manager of Carolina Mine, felt that supporting sets in workings were often unnecessary, although installed by custom. The histories of these mines do not indicate incidence of wall or crown failure. The present basic stress condition in the rocks can be described as an "at rest" condition.

Sbar and Sykes (Reference 2.5.1-34) presented evidence which suggests that the locations of earthquakes in eastern North America are controlled by unhealed faults or fault zones in the presence of high stress. The orientation of the faults with respect to the stress field may be one factor that determines on which faults strain release occurs. If this hypothesis is assumed valid, then by mapping both stress and faults unhealed by metamorphism within plates, it may be possible to assess earthquake risk at specific locations.

Metamorphism, which includes crystallization in fault zones, is associated with deep burial and high temperature and is often accompanied by hydrothermal activity. If a region like the site

area has been quiet and stable and not undergone great burial or great uplift since the Mesozoic, the possibility of seismicity is extremely low. Consequently, an unhealed fault in such a region of a subdued, stable surface indicates the region has been quiet and free of great earth movements. In an area of unhealed faults, it is necessary to determine individually the time of last movement of any particular fault, since the lack of healing by metamorphism may point directly to the crustal stability of that region. Other signs of deformation, such as oversteepened slopes, landslides, high and sharp terrain, and offset stream courses are more indicative of cause for concern in areas of unhealed faulting. Harrington (Reference 2.5.1-35), in discussing his analysis of the west border of the Durham Triassic Basin, is quoted as follows:

> "Field evidence indicates that there have been no ascending mineralizing waters in post-sedimentation time. This means that in the granites and slates, except for the manganese oxide precipitates and colloidal clays which have been carried in by meteoric waters, the fault cracks are still open. The slickensides developed during faulting are often preserved as casts of manganese or clay."

The fault investigated at the plant site is an unhealed fault along which movement has not occurred since the Late Jurassic, more than 150 million years B.P. The only metamorphic event that has affected the rocks at the site was a low-grade burial event that occurred prior to that time, as indicated by chemical remanent magnetization superimposed on the original magnetization of the diabase dikes.

A sensitive indicator of the relative stress state, though not the orientation, may be the condition of the ground water in the rock mass. The Triassic basin sediments are themselves highly impermeable rocks, known, however, to contain relatively small quantities of water trapped between diabase dikes that act as vertical aquicludes and compartmentalize the basins. In the Dunbarton Basin of South Carolina-Georgia, however, the occurrence of overpressurized ground water has been reported (Reference 2.5.1-36). This is in a region of some seismic activity.

No overpressurized conditions have been found in the Durham Basin. Bain (Reference 2.5.1-37) did not note overpressured groundwater conditions in the Deep River Basin. Prouty (Reference 2.5.1-33) described a deep well (1,640 ft.) drilled in Triassic sediments in Durham and failed to note any pressure condition. In some instances, artesian groundwater has been an indicator of excessive stresses in a rock mass. Bain and Thomas listed 349 wells in Triassic rocks in 1966 (Reference 2.5.1-38) and reported that a few flowing wells are present in Durham and Chatham counties and that these are normal artesian wells. Shallow wells, down to 300 ft., in the Triassic rocks average only 0.08 gpm per foot of uncased hole and show a marked reduction in yield below 100 ft. May and Thomas (Reference 2.5.1-39) reported on 84 wells in Triassic rocks in the Raleigh area, the average depth of which is 153 ft., which produce an average of 0.04 gpm per foot of well. Reinemund (Reference 2.5.1-17) described groundwater in the Deep River Basin and found no artesian wells. He described the logs of 26 deep borings of which he personally logged 5 (deepest 2,354 ft.), but did not report water conditions. Artesian flow from these borings did not occur (Reinemund, verbal communication, 1975). The absence of overpressure groundwater conditions in the Durham Basin is further evidence of a lack of contemporary high levels of stress.

A search for geologic evidences of Holocene earthquake activity in the region surrounding the plant site was made but none were found. Geologic evidences suggesting the occurrence of earthquakes as found in areas of known seismic activity include over-steepened slopes; large

landslides; areas of trapped surface water, with poorly developed drainage; stream offsets; and other topographic irregularities. None of these features have been found in the site area. A number of investigators have suggested that clastic dikes found in the Coastal Plain might be associated with earthquake activity. Heron, Judd, and Johnson (Reference 2.5.1-40) have concluded that clastic dikes in the Carolina Coastal Plain were formed by filling of fractures that probably developed in weathered rock as a result of slump or hillside creep. They found no evidence to indicate that any of the clastic-filled fractures in the Carolina Coastal Plain were formed by tectonic activity.

Some investigators feel that the Cape Fear Arch has been alternately a positive and negative feature since the Cretaceous, long after the sediments of the Triassic basin were deposited. Ferenczi (Reference 2.4.1-41) reviewed the literature up to that time on the Cape Fear Arch and other features of the Coastal Plain. This review indicates a general agreement that the Cape Fear Arch has been alternately a structural high and structural low through geologic time since the earliest Jurassic deposits in the Coastal Plain. Swift and Heron (Reference 2.5.1-21) also report some changes with geologic time.

Certainly the Cretaceous and post-Cretaceous rocks of the Atlantic Coastal Plain are incapable of storing any significant level of stress beyond those normal body stresses associated with lithostatic pressure without soft ground rupture, similar to that which occurs in the Gulf of Mexico Coastal Plain, where the ruptures are not associated with seismicity.

Brown, Miller and Swain (Reference 2.5.1-42) concluded that the region of the plant site is in a phase of crustal deformation in which the east-west-trending fault at the site would be under secondary compressive horizontal force. This alignment of present stresses is favorable to the stability of the site fault.

An assessment of the geologic and structural history of the development of the Triassic Basin and subsequent events is outlined in the following sequence:

a) The Jonesboro Fault and other major faults of the Deep River Triassic Basin are probably reactivated older structural trends in the basement rocks. Reactivation or initiation of tensional, normal-type movement, together with a possible lateral component on the Jonesboro Fault, was followed by the deposition of Triassic sediments, which continued to be deposited during progressive movement on the Jonesboro and other longitudinal and cross faults. The episode of Triassic deposition accumulated sediments which were probably several thousand feet thicker than those remaining today and which spread over a much broader area. This load overconsolidated the deeper sediments and resulted in the lithification of the remaining rocks, with resulting high specific gravity, above normal seismic velocities, and low permeability.

b) During this time span, shallow rooted movement on the site fault took place as a complement to deeper rooted continuing movement on the Jonesboro and probably other trans-basin faults. As the fault movements continued in Triassic-Jurrassic time, diabase dikes were intruded. Some of the dikes contain abundant amygdules at present surface levels, indicating that the dikes were intruded when the ancestral surface was less than about 1,000 ft. above the present surface. This indicates that a major period of erosion followed deposition of the original thicknesses of Triassic sediments but occurred before intrusion of the diabase dikes. At this time, movement on the Jonesboro Fault remained tensional, with a right-lateral slip component. Movement on the site fault

was primarily tensional, with a minor left-lateral component. These movements produced a clockwise sense of rotation between the two faults.

This episode of movement on the site fault and other faults, the folding of the Triassic rocks to form the Colon cross structure and other more minor folds, and intrusion took place during late Triassic-Jurassic time. The zeolite laumontite was crystallized in the fault gouge from the diabase.

Emplacement of a pluton of granitic rock a few miles east of the site may have occurred late in the history of movement on the Jonesboro Fault. The pluton is expressed on the gravity map of the Triassic basin as a negative anomaly. On the aeromagnetic map, it is expressed as an irregular positive anomaly and appears to encroach into the basin. In side looking radar, the pluton is reflected as a gray tonal change and appears circular in east-west imagery. Field reconnaissance confirms the presence of the large granite body, but its relationship to the Jonesboro Fault is not defined.

c) After intrusion of the dikes, major movement continued on the Jonesboro Fault, south of the site. This movement may have ended more quickly north of the site, since a dike, probably of this period, appears to cross the Jonesboro Fault with very little off-set. Alternately, the dike was emplaced during last stages of movement on the fault. Negligible to small last movement took place on the site fault after intrusion of the youngest dikes, which are Jurassic in age.

d) During later Jurassic time, the surface of the Triassic sediments, which was only slightly higher than the present surface, was buried under a load of sediments which may have been as thick as 9,000 ft. or as thin as 2,000 ft. This burial is suggested by the regional low-grade metamorphic event determined from chemical remanent magnetization of the dikes prior to 150 m.y. BP, and the crystallization of higher temperature secondary minerals, including zeolites, in the gouge of the site fault at that time. The differences in possible depth of burial noted above reflect lack of precise knowledge of the heat flow regime in the rocks at that time. If a normal continental heat versus depth relationship is postulated to provide the temperatures needed for crystallization of the secondary zeolite minerals, then the depth of burial was great. However, if the continental crust was thin or absent at the time of burial, higher temperatures would have been achieved under much shallower burial.

The depth of Jurassic burial was relatively great or spread over much of the Piedmont, since subsequent erosion of this material and quantities of underlying Piedmont and rejuvenated Appalachian rocks have furnished about 600 million cubic miles of land-derived rock material to the Coastal Plain, more than half of it Cretaceous in age. Whatever the depth of burial at the site during Jurassic time, the site fault was not moved subsequently, since the secondary minerals that crystallized in the fault gouge are not cataclastically strained or broken.

e) Following the Jurassic metamorphic event, major transportation of sediments across the Triassic basin took place and resulted in deposition of great quantities of late Jurassic-Cretaceous marine sediments in the Coastal Plain, extending inshore to points west of the basin. Since the Jonesboro and other faults do not off-set these materials, and to

the north appear to offset only slightly a dike of Jurassic age, last movement on the Jonesboro Fault and the site fault took place no later than the late Jurassic.

f) Since the late Jurassic period, the site area has been remarkably stable. Less than 1,000 ft. of Triassic rocks have been eroded, and they have not been further folded or deformed. No faults younger than Miocene are found in the site region. It is evident that groundwater levels at the dikes have never been lower than at present because secondary minerals in the diabase dikes and fault gouge could only have been preserved below groundwater level. Streams are currently downcutting in Triassic rocks, indicating that they have not been relatively lower than at present. Levels of aggradation of sediments associated with higher stands of sea level such as during the Cenozoic are preserved in the site region. Examples include the exposure of the site fault underlying sediments at Trench FET-19W (see Section 2.5.3) (though this deposit could be a late Jurassic remnant) and Quaternary terrace deposits found along the Cape Fear River.

g) Diabase dike intrusion at the site was contemporaneous with movement on the site fault, which continued over a relatively long span of time before and during dike intrusion. The diabase dikes range in age from an absolute minimum of 150 million years to a maximum of about 225 million years on the basis of remanent magnetization studies and from a minimum of 168 million years to a maximum of 260 million years based on potassium-argon dating.

2.5.1.2.5 Site Engineering Geology

A comprehensive series of subsurface investigations was conducted to evaluate the engineering, geologic, and seismologic characteristics of the plant site and the sites of various reservoir-related structures and to sample soils for possible use as borrow materials. Figure 2.5.1-10 is an index map for FSAR figures which show the locations of boreholes, exploration trenches, test pits, and geophysical surveys in the power plant, Auxiliary Dam, and Main Dam areas. Subsurface investigations in the power plant vicinity (Figures 2.5.1-11, 2.5.1-12, 2.5.1-13, and 2.5.1-14) are discussed below. Those in the Auxiliary Dam and Main Dam areas are discussed in Section 2.5.6. Numerous boreholes drilled for these subsurface investigations are located outside the map areas outlined in Figure 2.5.1-10. These boreholes were not critical to Seismic Category I structures and are not discussed in the FSAR; however, their locations are included in the tabulations of borehole locations presented in Appendices 2.5.A and 2.5.C.

Figure 2.5.1-11 shows the locations of boreholes (Series D and Series P boreholes) and exploration trenches excavated for a preliminary subsurface investigation of bedrock composition, orientation, and quality across the sites of the power plant and the Auxiliary Dam. Soils encountered in the boreholes were described in accordance with the Unified Soils Classification System, and RQD (Rock Quality Designation) values were obtained by calculating the ratio of core four in. or more in length to the length of the full core run. Twelve thousand ft. of trenches were excavated during these investigations to supplement the information obtained from borings. Layout of two of the trenches was planned in relation to regional geology. Trench No. 1 was oriented perpendicular to the regional strike of lithologic units and Trench No. 2 was oriented perpendicular to the trend of regional cross faults. Logs of these bore holes and trenches are included in Appendix 2.5A.

The locations of subsurface investigations conducted for design of the power plant and adjacent structures' foundations are shown in Figures 2.5.1-12, 2.5.1-13, and 2.5.1-14. Boreholes in the

BP series (Figures 2.5.1-12 and 2.5.1-14) are design borings for the power plant; logs of these borings and results which are of downhole water pressure tests that were conducted on selected borings in this series are included in Appendix 2.5A. Laboratory test results on representative samples from the boreholes are included in Section 2.5.4 and Appendix 2.5B. The BC series of boreholes (Figures 2.5.1-12 and 2.5.1-13) was drilled to investigate foundations for the emergency service water channels and the cooling tower make-up water intake channel; borehole logs are included in Appendix 2.5A and laboratory test results are in Section 2.5.4 and Appendix 2.5B. Boreholes in the BD series (Figures 2.5.1-12 and 2.5.1-13) and the BX series (Figure 2.5.1-12) were drilled to investigate foundations for proposed dikes (BD boreholes) and a proposed auxiliary dam (BX boreholes) which were included in preliminary project plans. Logs of these borings are on file with CP&L but are not included in the FSAR because the structures for which they were drilled were eliminated from the final project plans. The BCT series boreholes (Figure 2.5.1-12) were drilled in the foundations of the plant cooling tower; the logs of these boreholes are not included in the FSAR because the cooling tower is not classified as Seismic Category I structures. The borehole logs are, however, on file with CP&L. The BB series of boreholes (Figure 2.5.1-12) consists of auger borings drilled to sample soils in a proposed borrow area. The logs of these auger borings and the logs of the TPY series of borrow area test pits are included in Appendix 2.5C. The seismic refraction and shear wave velocity surveys whose locations are shown in Figure 2.5.1-12 are discussed in Section 2.5.2.

Additional subsurface exploration consisting of 21 borings and 4000 ft. of trenches was completed in the power plant and Auxiliary Reservoir Dam area in 1974 during the Shearon Harris fault investigation, which is described in Section 2.5.3. Locations of these boreholes and trenches are shown in Figures 2.5.1-15 and 2.5.1-16. Borehole logs and trench wall sections are presented in Section 2.5.3.

In order to document geologic conditions during construction the excavations for the foundations of the power plant, the Main Dam, and the Auxiliary Dam were mapped at a scale of 1 in. equals 10 ft. The Auxiliary Separating Dike was mapped at a scale of 1 in. equals 50 ft. These maps are included in Appendix 2.5E.

None of the subsurface investigations revealed any zones of alteration, irregular weathering profiles, structural weakness, unrelieved residual stresses in bedrock (Section 2.5.1.2.3.3), or activities of man in the area which might adversely affect the site. The siltstones and some of the fine sandstones at the plant and Auxiliary Dam slake over a period of days on repeated wetting and drying. To prevent this, water was not used in final excavation clean-up. Rocks were blown clean using air lances, and kept covered during periods of inclement weather until concrete could be placed. Concrete was placed only on clean, fresh, unaltered rock.

As described in Section 2.5.2, there is little history of felt earthquakes in the site area. There are no historical accounts of the behavior of the site during the few earthquakes which have been felt. There is, however, no evidence of adverse behavior and no reason to expect any.

2.5.1.2.6 Site Groundwater Conditions

Site groundwater conditions are described in Section 2.4.13.

2.5.2 VIBRATORY GROUND MOTION

A review of seismic history of the region and an examination of evidences for Quaternary faulting or other forms of ground deformation provide information necessary for establishing the seismic design basis for vibratory ground motion. Earthquake activity is correlated with geologic structures or tectonic provinces (when earthquakes may not be reasonably correlated with specific structures), and earthquakes significant for determining SSE (safe shutdown earthquake) are identified. The design acceleration at the site is estimated by using an attenuation relationship appropriate to the site region. Response spectra for SSE and OBE (operating basis earthquake), for horizontal and vertical ground motion, are developed by using Regulatory Guide 1.60.

2.5.2.1 Seismicity

Table 2.5.2-1 is a catalog of earthquakes that occurred within about 200 miles of the site through 1981. Figure 2.5.2-1 (Reference 2.5.2-23) is an epicenter map of the southern Appalachian region for the period 1754-1971. Figure 2.5.2-1a shows an epicenter map of earthquakes, from the earliest time through 1981, occurring within 50 miles from the site. A large scale map showing earthquakes within 200 miles from the site, for the period 1698-1981, along with seismic zones, after Bollinger (Reference 2.5.2-23), is presented in Figure 2.5.2-17.

The region in the immediate vicinity of the site is characterized by a low level of seismicity. However, a moderate level of earthquake activity has occurred in the surrounding region at distances greater than about 130 miles from the site.

Beginning in 1977, bulletins of the Southeastern U.S. Seismic Network describing the "Seismicity of the Southeastern United States" were issued. In these bulletins the hypocenters of a large number of small magnitude earthquakes were published, thus indicating a significant improvement in detection and location capability in the region. However, the level of activity remained rather low. During the period January 1978 through December 1981, only 9 earthquakes of magnitude 3.0 or above occurred within a radius of 200 miles from the site. Only one of these (October 8, 1979 magnitude 3.6) was of magnitude larger than 3.5. All of these earthquakes occurred at distances greater than 140 miles from the site. Also during this period only three earthquakes were reported to have occurred within a radius of 50 miles from the site. The earthquakes of February 25, 1978 (magnitude 2.2), March 4, 1981 (magnitude 2.8), and October 3, 1981 (magnitude 1.1) occurred at distances of 39, 42, and 31 miles, respectively, from the site.

During the period reported (1698-1981), eight earthquakes of epicentral intensity VII and above occurred within about 200 miles of the site. Six were of intensity VII, the closest occurring about 133 miles from the site. The 1897 Giles County, Virginia earthquake, MM intensity VIII, occurred approximately 160 miles northwest of the site; the 1886 Charleston earthquake, MM intensity X, about 200 miles south of the site.

An earthquake series of considerable significance occurred near New Madrid, Missouri on December 16, 1811, January 23, 1812 and February 7, 1812, each with MM intensity XI - XII, at an approximate distance of 570 miles from the site.

Effects of some of these major earthquakes are discussed below:

a) Charleston, South Carolina Earthquakes of August 31, 1886

These were the strongest shocks in the southeastern United States in historical times. There were two main shocks, at 9:51 p.m. and 9:59 p.m. EST on August 31, 1886. Two epicentral tracks were identified, one near Summerville, 16 miles northwest of Charleston. The approximate epicentral coordinates are 32.9° North, 80.0° West, about 200 miles south of the site. The shocks were preceded by explosion-like sounds near Summerville on August 27th and 28th. The main shocks were followed by aftershocks, some of rather high intensity, into the next day. The duration of the first shock was about 35 to 40 seconds.

The shocks were felt in an area of about 2,000,000 square miles. The area within a distance of about 100 miles of Charleston was strongly shaken. The most serious reports came from the major population center of Charleston, where people were terrified and damage was extensive. About 60 people were killed. The shocks were accompanied by roaring sounds.

The degree of damage to structures could generally be correlated with the type of design and construction as well as with local geologic conditions. Much of Charleston is constructed on "made land", including filled-in creeks. Structures in these areas were severely damaged. Heavy masonry and brick buildings, especially those constructed using poor-quality mortar, were often severely damaged, while well-built wooden houses (which would tend to respond elastically to earthquake motion) were generally much less damaged.

Ground waves were reported. Near the epicentral points, cracks, craterlets and sand boils were noted and railroad rails were bent. People were thrown to the ground; many chimneys fell.

The shock was felt as far away as Boston, Milwaukee, New York, Cuba, and Bermuda. At Savannah, about 90 miles from the epicenter, about 300 chimneys were damaged. At Augusta, about 100 miles away, about 100 chimneys fell and a dam fissured and broke. At Raleigh, about 215 miles from the epicenter, the shock was reported strongly felt, with instances of cracked walls and fallen chimneys. Raleigh lies within the Rossi-Forel intensity VI Zone. It is probable that the shocks were felt in the site area with intensity of about VI.

Occasional earthquake activity in the Charleston area has continued to the present time.

b) Union County, South Carolina, Earthquake of January 1, 1913

The shock occurred at 1:28 p.m. EST on January 1, 1913. The epicenter was about 34.7° North, and 81.7° West, about 175 miles southwest of the site. The maximum intensity of the shock was VII-VIII (VIII on Rossi-Forel scale, Reference 2.5.2-2). It was felt in an elliptical area, 45 miles by 25 miles, trending north-northeast south-southwest. Wave-like motions of the ground were reported in several places. The shock was accompanied by noises like thunder. It was probably felt in the site area with an intensity on the order of I-III. The total area affected was about 43,000 square miles, with the shock felt in North Carolina, eastern Tennessee, and southeastern Georgia.

On June 26, 1945, an intensity VI shock occurred in this same area near Murray Lake, South Carolina. Chimneys were reported cracked near the epicenter. Rumbling noises also preceded this shock.

c) Giles County, Virginia, Earthquake of May 31, 1897

The shock of maximum intensity VIII occurred at 1:58 p.m. EST on May 31, 1897. The epicenter was about 37.3° North, 80.7° West, about 160 miles northwest of the site; the total affected area was about 280,000 square miles. The shock was felt from Georgia on the south to Pennsylvania on the north and from the Atlantic Coast to Indiana and Kentucky, but most strongly at Pearisburg, Giles County, Virginia.

Old brick houses and chimneys were cracked, and bricks were shaken from chimney tops. Fissures appeared in the ground, and small landslides were noted.

At Narrows, Virginia, on the New River, near the West Virginia border, it was claimed that a motion like the ground swell of the ocean was observed. Large rocks rolled down the mountains. The shock was accompanied by loud sounds. At Raleigh, North Carolina, two shocks were felt, and a few chimneys were damaged. The shock was preceded by four shocks between May 3rd and May 31st, and was followed by aftershocks until June 6. This shock was probably felt in the site area with an intensity of about V.

There have been numerous additional shocks in the Giles County area, the most recent in 1968 (intensity IV, near Narrows, Virginia).

d) Shocks of December 22, 1875

The main shock of maximum intensity VII occurred at 11:45 p.m. EST on December 22, 1875. The epicenter was probably about 37.6° North, 78.5° West (near Richmond, Virginia), about 133 miles northeast of the site. The total affected area was about 50,000 square miles. This shock, preceded by a minor shock on March 10, was felt over a relatively large elliptical area extending from Baltimore, Maryland, southwest to Greensboro, North Carolina, and from the Atlantic Coast westward to Greenbrier County, West Virginia. Near the epicenter, five shocks occurred in quick succession. Bricks were shaken from chimneys in Goochland and Powhatan Counties, and shingles were shaken from a roof at Manakin, Virginia. A chimney collapsed in Wilmington, North Carolina. At Richmond, Virginia, the shock lasted 20 to 30 seconds, and deep rumbling was noted. There were no reports of the shock having been felt in the site vicinity. Numerous other small shocks have occurred in the Richmond-Charlottesville-Arvonia region; the largest possibly had intensities approaching that of the 1875 shock. Such shocks occurred in 1774, 1833, 1852, 1885, and 1907. Minor activity has occurred as recently as 1966 (intensity V near Richmond).

2.5.2.1.1 Initiation of Seismicity Associated with Reservoir Impoundments

Through 1978, 64 instances of reservoir induced seismicity (RIS) have been reported worldwide. Although most reservoirs located in aseismic terrain fail to induce seismicity, some earthquakes have occurred near reservoirs in regions that were previously considered aseismic. Because a fault at the SHNPP underlies one of the reservoir sites, the causes and potential for reservoir-related seismicity have been reviewed.

Earthquakes were first related to reservoirs when local earthquakes were felt shortly after Lake Mead began to fill in the mid-1930's. They culminated in a magnitude five shock about a year after the reservoir had filled to 80 percent of its capacity. For a number of years thereafter, small local earthquakes showed close correlation in numbers and energy release with seasonal

peak loads. After some time, correlation was as close with unloading as with loading. However, sometimes there was no direct correlation (Reference 2.5.2-3).

A. G. Galanopolos (Reference 2.5.2-4) reported a correlation of small to moderate earthquakes felt in Attica, Greece, over the period 1931 to 1965 with peak loadings of Marathon Lake. The lake is relatively small, with a capacity of 4 x $107m^3$. Earthquake epicenters were as far as 20 km from the lake, yet most occurred when the lake was overflowing. Galanopolos (Reference 2.5.2-4) has also shown definite correlation of numerical peaks of microearthquakes with seasonal load peaks in the case of Vaiont Dam and Reservoir in Italy.

In Australia, a moderate amount of seismic activity has been reported in association with Sydney Water Board Projects. Most of the earthquakes occurred at some distance from the reservoir sites; however, the filling of Lake Eucumbene was associated with moderate seismicity.

The Kremasta Dam and Reservoir in Greece are in a seismically active area. Filling of the lake began on July 21, 1965, and the first tremors were felt in December of that year. An exceptionally long sequence of small local shocks started after damming of the Acheloos River and impoundment of the artificial lake. The number of foreshocks could be correlated with the increase of reservoir loading. The main shock occurred soon after water level had reached the maximum height of 120m. Subsequently, many aftershocks have been recorded and felt (References 2.5.2-5 and 2.5.2-6).

The most noteworthy examples of reservoir associated seismicity in aseismic areas are the earthquakes associated with Kariba Dam Project in Africa and the Koyna Dam Project in India. Koyna Reservoir is on the Deccan Lava near the crest of a regional monoclinal structure. Hot springs and other evidences of activity are known along the axis of the structure.

Many have considered the area aseismic and placed it in zone zero of the seismic zoning map of India (Reference 2.5.2-7). The filling of the reservoir was accompanied by numerous microearthquakes and by mid-September, 1967, some one hundred epicenters had been located within the reservoir area.

On September 13, 1967, two magnitude 5.0 to 5.5 earthquakes caused minor damage; on December 11, 1967, a magnitude 6.5 earthquake caused extensive damage. The epicenter of the December 11 shock was within a few kilometers of the dam. Within the following 14 days, six aftershocks, magnitudes 5.5 to 6.2, occurred in the general area. Focal depths varied downward to 20km (References 2.5.2-8 and 2.5.2-9).

On the Zambezi River, the Kariba Dam, a concrete arch 120m high, which created the largest reservoir in the world, developed a classical case of reservoir induced seismicity. The storage reaches 16,000 x $107m^3$. The dam is founded on closely folded and faulted Precambrian rocks. The reservoir is over 250km long, covering mostly Paleozoic sediments and lavas of a tectonic rift valley. Numerous faults border the Paleozoic formations, but the region was seismically quiescent before the dam was constructed and was considered aseismic.

Impounding of Kariba Reservoir started in 1955, and the dam was completed in 1959. Significant shocks were first felt in 1961, and seismic activity steadfastly progressed with the rise of water level which crested in 1963 (Reference 2.5.2-10). The peak seismic activity followed almost immediately, with nine felt shocks, magnitudes 5.1 to 6.1, from August through

November, 1963. Epicenters were localized near the deepest parts of the storage. Correlation between increased seismic activity and rising water level in the reservoir can be graphically demonstrated.

The sequence of seismic events near the Kariba Reservoir is undoubtedly such that significant activity began following impounding of the reservoir, and then slowly decayed over a number of years. Such behavior and a foreshocks-peak-aftershocks pattern is reported to be characteristic of reservoir induced seismicity (Reference 2.5.2-9).

Another documented case of reservoir-related earthquakes has recently been reported by Shen et. al. (Reference 2.5.2-11). The Hsin Feng Kiang Dam in Kwang Tung province, China was begun in July, 1958. The reservoir covers 390 sq. km. and impounds 1,150 x $107m^3$ of water. The dam is 440 m long and has a maximum height of 105 m. Before construction began, the immediate vicinity of the site exhibited the typical scattered seismicity of China.

Within a month after first impounding water in October, 1959, seismic activity started; as water level rose, so did frequency of shocks. The first earthquakes were in the region of the dam itself, but progressively the activity spread to other regions. There were relatively few events under the reservoir; the great majority were within a kilometer of the water's edge. Each new rise in water level seemed to stimulate fresh activity; the depths of earthquakes were typically 4 or 5 km. Finally, in March, 1962, a magnitude 6 event occurred within 1 km. of the dam, at a depth of 5 km. The epicentral intensity was VIII - a violent shock.

During the last 20 days before the main shock, there was marked reduction in seismic activity everywhere in the reservoir region; and the steadily occurring small events moved toward the epicenter of the main shock. Since March, 1962, aftershocks have continued, with characteristics conventionally expected of an aftershock sequence. Up to 1972, 258,247 shocks with magnitude equal to or greater than 0.2 were recorded.

Another occurrence of reservoir-related seismicity is that at Keban Dam and Reservoir in eastern Turkey. The reservoir filled after closure of the diversion tunnels in the fall of 1973. Maximum reservoir volume is 3,000 x $107m^3$, at a height of 212 m. The dam and reservoir are in a tectonically active zone between the Anatolian Fault to the north and the eastward-trending northern extension of the African Rift Zone to the south. In the 10 years preceding impoundment, eleven events (the largest of magnitude of 5.6) with epicenters in the area, were recorded at more than five international stations. A microseismic array has been in operation around the reservoir since September, 1973. The background level of microseismic events, prior to impounding, was 10-15 events per month. As the reservoir approached 100 m depth in the spring of 1974 and leveled off, the incidence of microseismic events rose to 80-90 per month, with a peak number of events lagging behind maximum pool level by about two months. The number of events declined under a stable 100 m pool depth in 1974 to the background level of 10-15 events per month. This is perceived as a normal foreshock sequence, with more events expected during flood rises in pool level. To date, microseismic epicenters generally coincide with known active or recently active faults in the reservoir area.

Talwani (Reference 2.5.2-12) has documented four cases of temporal and spatial seismicity modification at reservoirs in the Piedmont of South Carolina. Three of these cases (Lakes Jocassee, Keowee, and Monticello) are instances of RIS, while the fourth case (Clark Hill) is questionable.

The relationship of reservoir construction to local earthquake situations is observational and statistical. The existence of a seismic effect from reservoir impounding is best documented by cases such as Koyna, Kremasta, Kariba, Leak Mead, Camarillas, Talbingo, Nure, Hsin Feng Kiang, Keban, and a number of others. Earthquake swarms in the areas of these reservoirs were previously unknown. Since reservoir impoundment, observed seismic activity has been centered in, and only in, the immediate vicinity of the reservoirs. It seems evident that water impoundment triggered local earthquakes in these cases.

On the other hand, many reservoirs, some approximating Lake Kariba in volume, have been impounded with little or no association with local earthquakes.

Packer et. al. (Reference 2.5.2-13) constructed a multivariate probabilistic model for the conditional probability of RIS at a reservoir characterized by its depth, volume, stress regime, and geologic setting. Of these four variables, depth and volume showed the strongest correlation with RIS.

Various mechanisms to explain RIS have been put forward. They include increased stress due to water load (Reference 2.5.2-10), decreased effective stress due to increased pore pressure (Reference 2.5.2-14), stress corrosion in silicate rocks (Reference 2.5.2-15), and argillization of the materials of the weak structural plane and the reduction of its shearing strength (Reference 2.5.2-11). The loading and pore pressure mechanisms seem to be the most reasonable. In any case, the amount of shear stress generated by even the largest reservoirs is two orders of magnitude too small to cause the fracturing of intact rock and an order of magnitude too small to cause movement along pre-existing fracture planes. Therefore, the stress change brought about by reservoir impoundment can only act as a triggering mechanism for release of pre-existing stress.

The impoundment of every reservoir modifies the stress regime in the region immediately surrounding the reservoir. Whether or not these stress changes cause RIS depends upon many factors. The primary factors are the pre-existing state of stress surrounding the reservoir, the magnitude of the induced stress, and the geologic and hydrologic conditions of the site.

2.5.2.1.2 Seismicity and the Plant Reservoirs

The reservoirs at the SHNPP site are small compared to reservoirs associated with seismic activity. Table 2.5.2-2 lists a few reservoirs associated with seismic activity, compared with the depth and volume of the Main and Auxiliary

Reservoirs for the SHNPP. While there is no definite critical water depth for causing RIS, Packer's work (Reference 2.5.2-13) suggests that the likelihood of RIS being associated with a water depth of 19 m is extremely remote.

Talwani (Reference 2.5.2-12) has observed cases of RIS in the Piedmont of South Carolina. Because of this observation and due to the fact that every reservoir produces some modifications in the local stress field, the pre-existing state of stress, the magnitude of the induced stress, and the geologic and hydrologic setting of the SHNPP have been examined to seek out any possible correlations that might exist with the RIS sites in the South Carolina Piedmont.

The SHNPP site does not fall within any of Bollinger's zones of southeastern seismicity and no over pressured groundwaters have been noted in the Durham Basin (Section 2.5.1.2.3.3). This is not the case for the four RIS sites in South Carolina. All four lie within Bollinger's South Carolina-Georgia Seismic Zone. This zone has exhibited historical seismic activity and over pressured groundwaters have been reported (Section 2.5.1.2.3.3). Based upon these data it appears that the SHNPP site is located in a region of lower ambient stress than the RIS sites in South Carolina.

The maximum magnitude of the induced stress caused by reservoir impoundment is related to its depth and volume. Table 2.5.2-2 lists the reservoirs with RIS in South Carolina and the SHNPP site reservoirs. The maximum depth of the SHNPP site reservoirs is 19 meters. This is less than one half of the depth of the Monticello Reservoir, which is the shallowest South Carolina site of RIS. In addition the total volume of the SHNPP Reservoirs is less than 20 percent of the volume of the smallest South Carolina RIS site. The maximum stress change due to loading at the SHNPP site is 1.85 bars. This is also the maximum possible increase in pore pressure (assuming that the depth to the water table is negligible). This value is less than half the calculated stress change induced by the smallest South Carolina RIS site.

All four South Carolina RIS sites are located upon gneissic rocks of the Piedmont physiographic province. The SHNPP site reservoirs are located predominately upon clastic sediments of the Durham Basin. The site geology is discussed in detail in Section 2.5.1.2. The permeability values obtained at the site are generally low (Section 2.4.13). This suggests that the migration of pore pressures to depth will be inhibited.

A comparison of the SHNPP site reservoirs to four reservoirs with RIS in South Carolina shows no correlation in pre-existing stress, water depth, water volume, geologic or hydrologic conditions. Therefore, there is no reason to expect the SHNPP site reservoirs to induce any significant seismic activity.

2.5.2.1.3 Plant Seismic Monitoring Network

On January 6, 1976, the NRC formally notified CP&L that the NRC had concurred with CP&L's conclusion that the fault discovered at the SHNPP is not a capable fault as defined in Appendix A to 10 CFR Part 100. The NRC requested seismic monitoring at the site to confirm their conclusion that the proposed reservoirs at the site will not cause fault movement during and after filling.

In response, CP&L submitted a proposal dated February 13, 1976, to establish a seismic monitoring network encompassing the SHNPP site area. Although this proposal called for monitoring to begin in January 1979, the network was installed and became operational on September 30, 1977, in order to obtain more definitive baseline data prior to filling the reservoirs.

The seismic monitoring network consists of an array of stations, Numbers 1 through 4, covering the SHNPP site area. Each contains a Teledyne Geotech Model 18300 vertical-component, short-period seismometer. One station, No. 4, also has two horizontal-component Model 18300 seismometers aligned N-S and E-W. Data from Stations 1 through 3 are transmitted via UHF radio to the SHNPP meteorological tower. Station 4 data are transmitted to the meteorological tower via commercial telephone lines. At the meteorological tower, data from all stations are multiplexed and transmitted by telephone data lines to the central recording facility in Raleigh

and are recorded on 16 mm microfilm by a Teledyne Geotech RF 400 Develocorder. Carolina Power & Light Company committed to operate the SHNPP seismic monitoring network continuously until two years after the filling of the Auxiliary Reservoir, which reached full-pond in March 1983.

During this period, seismic events detected in the site area were logged and discussed in quarterly reports to the NRC.

The station locations are:

Surface Rock	Station	Coordinates (Lat., Long.)	Elevation (m)	Magnification (1 Hz)
Triassic Sedimentary	1	35°37'55" N 78°58'49" W	86.0	134,000
Piedmont Crystalline	2	35°33'17" N 78°59'23" W	74.7	213,000
Piedmont Crystalline	3	35°35'36" N 78°54'16" W	94.5	67,000
Triassic Sedimentary	4	35°39'57" N 78°54'16" W	93.0	169,000

2.5.2.2 Geologic Structures and Tectonic Activity

A detailed description of the geologic structures of the region and tectonic activity is presented in Section 2.5.1.

2.5.2.3 Correlation of Earthquake Activity With Geologic Structures or Tectonic Provinces

In recent years a number of authors have discussed the seismicity of various parts of eastern North America and its relation to various tectonic features. A major difficulty in such analysis is the near absence of instrumental data.

Woollard (Reference 2.5.2-16) inferred a well-defined seismic belt associated with the Appalachian Mountain Belt. He also mentioned a seismic trend that connects Charleston to the Appalachian Belt. McClain and Meyers (Reference 2.5.2-17) observed a zone of relatively high seismicity in the southern Appalachian Mountain areas of Virginia, eastern Tennessee, western Carolinas, northern Georgia, and Alabama. This zone coincides with the southern portion of the Appalachian Mountain Geologic province, its seismicity probably representing minor adjustments of the highly disturbed rocks. Nuttli (Reference 2.5.2-18) observed that the distribution of epicenters throughout the eastern United States is diffuse and that epicenters do not appear to be associated with long, active faults. Also, eastern earthquakes are not associated with visible surface faulting. Ferguson and Stewart (Reference 2.5.2-19) observed that certain parts of North Carolina appear to be more seismic than others although no faults have been defined in these areas.

A marked alignment of epicenters across western North Carolina and on into Virginia was first observed by MacCarthy (Reference 2.5.2-20), who postulated that such clear-cut linear alignment probably represents a real tectonic alignment, a belt of structural weakness in the uplifted Blue Ridge. He hypothesized that there is a zone of weakness, with numerous small movements that from time to time generate rather localized tremors and disturbances. Oliver

and Isacks (Reference 2.5.2-21) observed that in the Appalachian region, earthquake epicenters follow areas of highest topographic elevations.

Fox (Reference 2.5.2-22) observed that the Blue Ridge and Piedmont provinces are associated with moderate seismicity, i.e., shocks of intensity less than VII. Fox also observed that seismic activity is minor in the extensive area between Richmond, Virginia and Charlotte, North Carolina.

From analysis of earthquake distribution in the southeastern United States, Bollinger (Reference 2.5.2-23) concluded that epicenters occur in four zones: two that are parallel and two that are perpendicular or oblique to the dominant northeast trend of Appalachian structure (Figure 2.5.2-2). He identified the following seismic zones:

a) Southern Appalachian Seismic Zone - extends from western Virginia to central Alabama in the Valley and Ridge, and Blue Ridge provinces.

b) Northern Virginia - Maryland Seismic Zone - A diffuse northward extension of the above.

c) Central Virginia Seismic Zone - a relatively narrow, isolated zone of activity in the Piedmont province, offset from the above two zones, and oblique to the NE-SW structural trend.

d) South Carolina - Georgia Seismic Zone - a broad zone spanning the Piedmont and the Coastal Plain provinces and transverse to regional structure.

The area around the SHNPP site is located between the Central Virginia, Southern Appalachian, and South Carolina-Georgia seismic zones defined by Bollinger (Reference 2.5.2-23) (see Figure 2.5.2-2). The SHNPP site area has exhibited no seismicity. The Piedmont province, in which the site is located, is active only in central Virginia, in South Carolina, and in northwest Georgia. Appreciable earthquake activity in the Coastal Plain province occurs only in South Carolina.

In general, seismic activity occurs primarily in the Valley and Ridge and Blue Ridge provinces, although, even considering epicentral errors, earthquakes occur in all four of the geologic provinces in the southeastern United States (Figure 2.5.2-3). Furthermore, activity is oriented both parallel and transverse to the dominant northeast-trending Appalachian structures. Many thrust faults and normal faults that have been mapped in each province are very ancient and have no records of surface rupture during recent geologic time. Bollinger (Reference 2.5.2-24) identified three factors which make it difficult to correlate earthquakes with near-surface geological structures:

a) Inadequate seismograph-station density to determine focal depths or focal mechanisms.

b) Minor versus major seismic-zone characteristics, especially with regard to surface expressions of earthquake faulting.

c) Great difference in time scales between seismicity data and regional geologic data.

Focal depths and mechanisms are the critical seismic parameters that may be directly correlated with geology. Focal depths, for example, can indicate whether the casual fault was in the sedimentary section, basement rocks or deep within the crust. Focal mechanism data can yield strike and dip of the fault plane and the direction of slip on it.

The second factor, relating the levels of seismicity and tectonism, is intrinsic to the region. The lack of surface faulting during earthquakes prevents immediate correlation with many major and minor faults mapped in the area by geologic techniques.

The third relates to the relevant time frames. When earthquake data (spanning more than 200 years) are compared with the geologic data (spanning more than one billion years), the time interval difference is seven orders of magnitude. Thus, the task is deciding which, if any, of the ancestral geologic features are pertinent to observed seismicity.

2.5.2.4 Maximum Earthquake Potential

Figure 2.5.2-3 shows that earthquakes occur in all four geological provinces in the area. Seismic activity is oriented parallel and transverse to the dominant northeast-trending Appalachian structures. The many thrust and normal faults mapped in each province do not have a record of surface rupture during historical times; therefore, detailed correlation of seismicity and geology in this region is impossible without detailed geological and geophysical investigations. Moreover, an appropriate tectonic (or seismotectonic) map for the eastern United States is unavailable at present. Under the circumstance, the seismic zones defined by Bollinger (Reference 2.5.2-23) on the basis of spatial distribution of earthquakes and geodetic and tide gage data appear to provide a reasonable basis for the investigation of maximum earthquake potential and evaluation of seismic risk.

The largest historical earthquakes associated with each seismic zone in the region are identified in the following:

a) Southern Appalachian Seismic Zone

   The Giles County, Virginia, May 31, 1897, MM intensity VIII, and Gadsden, Alabama, January 27 and 28, 1905, MM intensity VII-VIII events, were the two largest earthquakes in this zone. Minimum distance between zone and site is about 165 km. An earthquake with epicentral intensity VIII would be barely perceptible at this distance and would have no damage potential at the SHNPP site.

b) Northern Virginia - Maryland Seismic Zone

   The Luray, Virginia, April 9, 1918, MM intensity VI and northern Virginia, September 5, 1919, MM intensity VI events, were the two largest earthquakes in this zone. The minimum distance between this zone and the SHNPP site is about 315 km. An intensity VI earthquake would not be felt at this distance.

c) Central Virginia Seismic Zone

   The Petersburg, February 21, 1774, MM intensity VII and Richmond, December 22, 1875, MM intensity VII events, were the two largest earthquakes in this zone. The

minimum distance between this zone and the SHNPP site is about 155 km. An intensity VII earthquake would be barely perceptible at this distance.

d) South Carolina - Georgia Seismic Zone

The largest earthquake in this zone occurred near Charleston, South Carolina on August 31, 1886, and was felt with MM intensity X in the epicentral region. Figure 2.5.2-4 is an isoseismal map of this earthquake, prepared by Bollinger (Reference 2.5.2-25). It shows that the earthquake was probably felt with a MM intensity V (but not more than VI) in the SHNPP site area.

From studies related to the Charleston, South Carolina earthquake of 1886, Rankin (Reference 2.5.2-26) concluded the following.

"The extent and boundaries of the Charleston block are not well known, but it does appear to underlie a sizeable area of the emerged and submerged Coastal Plain. The Orangeburg scarp appears to coincide with the northwestern boundary of the Charleston block and may be structurally controlled."

"Why did the 1886 earthquake occur in the Charleston-Summerville area rather than elsewhere in the Charleston block? Is it reasonable, in fact, to restrict the probability of a recurrence of an 1886 earthquake to the Charleston block at all? Clearly, we need to know more about the Charleston block and about the nature and location of the boundaries of this block."

Without prejudice to the geographical boundaries of the Charleston block and without considering whether or not a recurrence of an "1886 earthquake" may be restricted to the Charleston block, a conservative determination of the vibratory ground motion may be made by considering the recurrence of such an earthquake in the South Carolina-Georgia seismic zone at its minimum distance, about 130 km, from the site. Using the attenuation relation,

$I = I_o + 2.87 - 0.000\,52R - 2.88 \log R$,

(where R is the specified distance in km.)

developed by Bollinger (Reference 2.5.2-25) for a 50 percent fractile of the data set, a site MM intensity of 6.71 is obtained.

e) New Madrid Seismic Zone

Three large earthquakes, each with epicentral MM intensities XI-XII, occurred on December 16, 1811; January 23, 1812; and February 7, 1812, near New Madrid, Missouri. Figure 2.5.2-5 is an isoseismal map for the earthquake of December 16, 1811.

A microearthquake network study, currently under progress at St. Louis University, reveals that the activity in this region, which may be described as the New Madrid seismic zone, is strongly localized (Reference 2.5.2-27). The minimum distance of the New Madrid seismic zone from the site is about 900 km. From the attenuation relation,

$$I = I_o + 2.35 - 0.00316 R - 1.79 \log R,$$

presented by Gupta and Nuttli (Reference 2.5.2-28), it follows that a recurrence of the December 16, 1811 earthquake in the New Madrid seismic zone at the point nearest the site would be felt with a MM intensity of 5.2 in the vicinity of the site.

f) Background Seismicity in the Region

In the site region the largest historical earthquakes have been of MM intensity V. These cannot easily be attributed to any particular geologic structure or seismic zone. Included are two North Carolina coast earthquakes on January 18, 1884, and March 5, 1958, and one western North Carolina earthquake on August 26, 1916.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

To determine seismic wave transmission characteristics of the site, the following site geophysical surveys were made:

a) Seismic refraction lines to define bedrock topography.

b) A seismic refraction line to detect both compression wave and shear wave arrivals for evaluation of dynamic soil and rock properties.

c) A downhole velocity survey to further define dynamic rock properties.

d) Ambient noise studies to determine predominant periods of ground motion due to background noise levels.

Figure 2.5.2-6 shows the location of these studies. The following sections describe each phase of the geophysical exploration.

2.5.2.5.1 Seismic Refraction Surveys

Refraction surveys were made along six seismic lines totaling approximately 5,000 linear ft. Seismic lines 1 through 4 were completed at selected locations adjacent to their respective exploration trench numbers. Seismic lines 5 and 6 extend southeast from Trench 2 to the vicinity of Borings P6 and P7, respectively. These lines are approximately perpendicular to Trench 2.

The recording equipment used for this refraction investigation were a portable Electro-Tech ER-75-12 refraction seismograph and Electro-Tech EV-5-4 geophones with a natural frequency of 14 cycles per second. The geophones were fitted with spike attachments for coupling with the underlying soil. Geophone spacings of 25 to 50 ft. were used. Explosives (normally 5 lb. charges of Nitramon-S placed in 10 ft. deep drill holes) were detonated in the center, ends and beyond the ends of each line.

The geophysical crew consisted of two geophysicists who supervised the field investigation, operated the recording instruments, and made preliminary interpretations of geophysical data in the field. A licensed powderman handled and placed the explosive charges.

The results of the geophysical refraction survey, showing profiles of various strata and compressional-wave velocities in soils and underlying bedrock, are given in Figures 2.5.2-7 through 2.5.2-10. The velocities of compressional wave propagation in the upper soils and underlying rock were computed from the plotted data. In addition to the geophysical profiles, the plots of the time-distance data resulting from the survey are shown immediately above the corresponding profile. The accuracy of the calculated depth to bedrock is considered to be within 10 percent for the major portion of the survey; however, in areas where sound bedrock is indicated at shallow depths, the precision is probably less.

Velocities of the different strata, as evaluated from the refraction surveys, are as follows:

a) Residual soils and/or highly weathered rock; the velocity range is 1,250 to 2,000 ft./sec.

b) Weathered and/or fractured bedrock; the velocity range is 5,000 to 7,150 ft./sec.

c) Sedimentary bedrock (probably unaltered); the velocity range is 10,900 to 13,650 ft./sec.

2.5.2.5.2 Shear-Wave Velocity Survey

A shear-wave velocity survey was conducted along a 3,400 ft. section of Trench 1. Shear-wave velocities were computed from the recordings of two Sprengnether Engineering Seismographs placed at 350 ft. intervals along portions of the trench. Shot holes were located at varying distances, up to 3,400 ft. from the farthest geophone. Traces from the Sprengnether seismographs, and eight 1-component geophones placed at 100 ft. intervals were recorded on an Electro-Technical SDW-100 oscillograph.

Survey results, with compressional wave velocities and computations of Poisson's Ratio, are summarized in Table 2.5.2-3. These data refer to subsurface conditions in the vicinity of Boring P5, which are considered typical.

2.5.2.5.3 Downhole Velocity Survey

A downhole velocity survey in Boring P6 provided a check on the compressional wave velocities measured during the seismic refraction surveys. The boring was cased with steel casing to 20 ft. below the ground surface. Small explosive charges were buried approximately 5 ft. deep at distances of 10 to 25 ft. from the boring. In the boring, seismic response to the detonations was detected with a 12-trace geophone cable and was recorded with an Electro Technical Labs M-4-E amplifier and a SDW-100 oscillograph.

Figure 2.5.2-11 shows results of the downhole velocity survey. The compressional velocity of the bedrock measured in this survey is greater than the corresponding compressional velocity measured by seismic refraction, because seismic refraction surveys record average dynamic bedrock properties over a lateral distance, while downhole velocity surveys record the properties at an isolated point. Thus, compressional velocities measured during the seismic refraction survey are more representative of the actual dynamic bedrock properties. A zone of slower velocity material (9,250 ft./sec.) is apparent between depths of 93 and 130 ft.

2.5.2.5.4 Ambient Vibration Measurements

The last phase of geophysical field studies was measuring the level of ambient ground motion (microtremors) caused by various natural and man-made sources. On September 18, 1970, measurements were taken at the five locations shown on Figure 2.5.2-6; all were taken when no equipment or drills were in operation at the site.

A direct-writing Model VS-1100, Sprengnether Engineering Seismograph recorded ambient ground motion. A VS-1100D amplifier was used with the seismograph, resulting in a maximum gain level of 2,000. The three components of ground motion measured were radial, vertical and transverse.

Table 2.5.2-4 shows the results of ambient ground-motion measurements. The observed characteristic frequencies of the site are 100, 55 1/2, and 25 Hz. The maximum observed level of groundmotion, which is in the vertical component, is 0.48×10^{-3} inches/second at about 25 Hz. Location No. 5 at the northwestern end of Trench 1, appears to be the quietest location, whereas locations 3 and 4, near Borings P6 and P7, respectively, appear to be least quiet. See Section 2.4.13 for groundwater levels, as they may affect seismic velocities in unconsolidated/loosely consolidated sediments.

2.5.2.6 Safe Shutdown Earthquake

To establish criteria for the SSE, the degree of remotely possible ground motion has been considered in light of the seismic history and the geologic structure of the region and the site. This included the effects upon the site of the recurrence of the earthquakes discussed in Section 2.5.2.1, when located so that their epicenters, or regions of maximum intensity, were at minimum distance from the site. The maximum effect at the site would be caused by a recurrence of the 1886 Charleston earthquake. The result would be a MM intensity 6.7 earthquake if it were assumed to occur in the South Carolina-Georgia seismic zone at its minimum distance from the site, or VI, if it were considered localized to a structure near Charleston.

The general pattern of seismicity within the Piedmont was also considered. Shocks in the Piedmont, such as the 1875 intensity VII shock near Richmond, have been known to occur in the vicinity of Triassic basins. An intensity VII shock near Wilmington, Delaware in 1871 has not been related to known geologic structure. Because of limited documentation, its exact epicenter cannot be fixed.

Coastal Plain sediments mask the bedrock in the area south and east of Wilmington, and much of the bedrock geology is still unknown; consequently, it is possible this shock is related to some unidentified Triassic structure. Hence, the remote possibility of a shock of intensity VII occurring in the Deep River Basin, close to the site, has been considered.

Based on the foregoing, the SSE is designated an intensity VII earthquake with its epicenter near the site. At foundation level within the competent bedrock at the site, the maximum horizontal ground acceleration due to such a shock is estimated to be less than 12 percent of gravity.

In order to provide an additional margin of conservatism, a value of 15 percent of gravity is assigned as the maximum horizontal ground acceleration. All safety related structures and

systems have been designed to assure safe plant shutdown for two horizontal excitations and one vertical excitation simultaneously.

The SSE is postulated as a shock of low magnitude close to the site, of less than 10 seconds expected duration, and producing no more than 5 to 10 cycles of motion at the maximum predicted level. This pattern of motion should be similar to the Golden Gate (San Francisco) earthquake of 1957 or the Helena, Montana, earthquake of 1935. Records from these earthquakes indicate only 2 to 3 cycles of strong motion; therefore, the selection of 10 cycles of strong motion for the SSE is conservative.

Seismic Category I systems and components, therefore, have been designed for a minimum of 10 loading cycles under SSE conditions. Figures 2.5.2-12 and 2.5.2-13 show the horizontal and vertical response spectra for the SSE, prepared in accordance with Regulatory Guide 1.60.

Recent seismic network monitoring supports the observation that the SHNPP site is located in an area of low historic seismicity that is lower than that of neighboring areas of the Piedmont Province to the north and south in the eastern United States (EUS). Inasmuch as MMI = VII is the largest intensity earthquake known to have occurred in the Piedmont Province in the southeastern United States and only one event of that intensity has been associated with Triassic basin areas in the Piedmont (the Richmond, VA earthquake of December 23, 1875), it was concluded above that a random MMI = VII event near the site provided an adequately conservative basis for the SSE design response spectrum for the SHNPP.

In consideration of the above discussion, the maximum random earthquake near the Harris site is estimated to be associated with MMI = VII. There do not exist sufficient instrumental data to define an intensity - magnitude relationship for the Piedmont Region, nor are there numerous near-field strong motion records from the region to estimate a site-specific spectrum. As a result, we adopt general intensity - magnitude relationships and data obtained in other regions. Studies of earthquake intensities and magnitudes indicate that MMI = VII would be associated with an earthquake of local magnitude ML approximately equal to 5.3. For purposes of generating a site-specific spectrum, it is appropriate to consider strong motion records within one-half magnitude unit of this value, i.e., $4.8 \leq M_L \leq 5.8$.

As an alternative to developing new site-specific spectra to compare to the Harris SSE, one option is to use available compilations of spectra for the appropriate magnitude range. One compilation is presented in Appendix A of NUREG/CR-1582, Volume 4, prepared by Lawrence Livermore National Laboratory (LLNL). Figure 4-7 of that document presents mean and mean-plus-one-standard deviation spectra for records in the magnitude range $4.8 \leq M_L \leq 5.8$ obtained at distances less than 25 km. (The authors of that document consider these spectra representative of MMI = VII motion). These spectra, which are for five percent damping, are reproduced in Figure 2.5.2-18; also shown in that figure is the SSE spectrum for the Harris site, for five percent damping. The LLNL spectra were originally published in 1979 and do not include more recent data, nor do they include data from prior earthquakes which indicate low amplitude motions. Generation of new site-specific spectra which include these additional data in the appropriate magnitude and distance range may ultimately be appropriate, but all these additional data are from NON-EUS sites. For the present, it is sufficient to compare the Harris SSE spectrum with the LLNL spectra, to obtain an indication of the SSE's validity.

The LLNL spectra shown in Figure 2.5.2-18 were generated for two classes of site conditions (rock and soil). The rock spectra are most appropriate for comparison to the SSE spectrum

because the Shearon Harris facility is founded on competent lithologic units (Triassic) as discussed in the FSAR. For completeness, both the rock and soil spectra are compared to the Shearon Harris SSE spectra in Figure 2.5.2-18.

These comparisons show that the Harris SSE design spectrum (five percent damping) envelopes or matches the rock mean +1σ (one standard deviation) site specific spectrum over the entire frequency range. From this, we conclude that the Harris SSE spectrum is appropriately conservative for representing the maximum random earthquake in the Piedmont province, for which the facility is designed.

As an aside, we note that the soil mean +1σ spectrum is somewhat higher than the SSE spectrum above 12 hz. The high frequency end of the soil spectrum is of questionable reliability for several reasons:

a) It has essentially the same values at high frequencies as the M_L = 5.8 soil curve based on 5.3 ≤ M_L ≤ 6.3 events (NUREG/CR-1582, Volume 4, Figure 4-8 compared to Figure 4-7) whereas the rock site-specific spectra differ systematically at all frequencies as expected for larger vs. smaller magnitude events.

b) The soil spectra for $M_L \simeq 5.3$ were derived by including in the analysis three Japanese records with peak accelerations as high as 0.6 g. Reanalysis of these accelerograms to account for instrument correction generally reduced the values at high frequencies: the 0.6 g. peak acceleration was reduced to 0.5 g., for example (Matushka, Personal Communication, 1982). Moreover, there may have been soil resonance effects affecting these records which would not be relevant for a rock site.

   Inasmuch as the rock site-specific spectrum is the most appropriate for the Shearon Harris site conditions, we conclude from the comparisons presented that the SSE design is adequately conservative for the maximum random earthquake that would occur near the site.

2.5.2.7 Operating Basis Earthquake

The study discussed in Section 2.5.2.6 examined the degree of ground motion which is considered possible during the economic life of the facility based on the seismic history of the region and the site area. It is concluded that it is unlikely that the site would be subjected to ground motion above intensity VI during the life of the facility; therefore, the accelerations of the operating basis earthquake were taken as half those of the SSE. The corresponding horizontal acceleration at foundation level in the bedrock would be less than 7.5 percent of gravity. Accordingly, the OBE is designated an intensity VI earthquake, with its epicenter near the site.

Figures 2.5.2-14 and 2.5.2-15 show the horizontal and vertical design response spectra for the OBE, prepared in accordance with NRC Regulatory Guide 1.60 and scaled to .075g horizontal ground acceleration.

Figure 2.5.2-16, Earthquake Occurrence Probability, plots the seismic history within 250 miles of the site. Assuming the seismicity of the area to be relatively uniform within the 500-mile diameter circle, a parallel relationship was drawn representing the earthquake occurrence probability within a 10-mile radius. This is a very conservative approximation, since earthquake occurrence in the region is not random and uniform but is generally confined to well-defined

local areas far distant from the site. On this statistical basis, it is conservatively concluded that the site will undergo intensity VI ground motion (the OBE) on the order of once every 3,500 years.

2.5.3 SURFACE FAULTING

Studies of regional and site geology, detailed in Section 2.5.1, provided information on overall geologic conditions in the area. In addition, to provide assurance against the presence of capable faults in the area as well as to furnish basic engineering data, a number of trenches and boreholes were completed at locations shown on Figures 2.5.1-10 through 2.5.1-14.

The trenches varied in depth from 2 to 12 ft., depending on the nature of the overburden and the depth to rock too hard to cut with a backhoe. In the upland areas the overburden, consisting of residual yellow sandy clay and sandy loam, is only 2 to 4 ft. thick. It is quite uniform in texture and composition and stands well in the sides of the trenches. Where this overburden is only 2 to 4 ft. thick, rocks too hard to cut with a backhoe were usually reached at depths of 5 to 6 ft. Along stream courses and other low areas, yellow sandy, clayey alluvium is commonly 3 ft. to 10 ft. thick; it generally caved badly, and satisfactory information could not be obtained on the bedrock in portions of trenches. In these areas trenching was supplemented by drill holes 40 to 100 ft. deep, in order to determine the character and soundness of the bedrock beneath the alluvium. A number of drill holes, 100 to 250 ft. deep, were put down to determine the soundness and loadbearing capacity of the rocks underlying the plant site.

The plant site area was explored by four major trenches having a total length of slightly over 12,000 ft. They were supplemented by five minor trenches, 50 to 100 ft. long, and numerous drill holes, 50 to 152 ft. deep. Figure 2.5.1-11 shows trench locations; detailed logs of the four major trenches are presented in Appendix 2.5A.

Minor cross faults are characteristic of the Triassic bedrock surrounding the site. However, the exploratory trenches at the site revealed no evidence of cross faults. Several small diabase dikes were exposed in Trenches 1 and 2, west and south of the proposed reactor site.

During excavation of the Waste Processing Building, a small cross fault, hereafter referred to as the "site fault," was discovered. Thus, two faults, the Jonesboro fault and the site fault are relatively close to the site. Further studies were made to determine their possible relationship to each other, and the capability of the site fault (Reference 2.5.1-29).

As a result of these studies, the Jonesboro Fault and the site fault are considered to be of the same general age; the site fault, however, having a history of movement later than that of the Jonesboro fault, along which movement occurred for a very long time that included the beginning of deposition of Triassic sediments and that ended after the intrusion of Jurassic diabase dikes. Movement along the site fault occurred after deposition and lithification of several thousand feet of Triassic Basin sediments and ended shortly after intrusion of the latest of the Jurassic dikes.

Evaluation of the small vertical and horizontal components of movement of the site fault suggests it is short, with shallow-roots, compared to the Jonesboro Fault, along which movement began much earlier. Both faults are considered to be rooted in the crust. These shallower upper crustal fractures are in contrast to the deeper seated north-northwest oriented lines of tension revealed by the contemporaneous intrusion of diabase dikes, probably from the

upper mantle. Such upper mantle material, intruded as diabase dikes into the Triassic Basin and the Piedmont, did not follow either of the faults.

The site fault and the Jonesboro Fault belong to a conjugate set of normal faults. Stresses that produced them caused rotational deformation, emphasized by major motion along the Jonesboro Fault, the primary fault of the set. The site fault is an antithetic normal fault of this conjugate set. Normally the primary and antithetic fault planes tend to have parallel strikes, but in regions of differential vertical motion, the strike of the antithetic set may not be parallel; however, the geometry of the set precludes intersection of the faults. East of the site, stresses were most probably released by faults parallel to the site fault in an "En Echelon" arrangement.

Since the Late Jurrasic, the site area has been remarkably stable. The Triassic rocks have not been further faulted, and no faults offsetting strata younger than Miocene have been found in the site region.

The site fault is overlain by sedimentary rock which has not been offset or otherwise disturbed by movement on the fault. Evidence from comparative lithology, depth of oxidation, and soil-profile thickness indicates the sedimentary rock is older than one million years and probably much older, perhaps as old as Jurassic.

Intrusion of diabase dikes at the site occurred before and during fault movement, which continued for a relatively long time. The dikes range in age from an absolute minimum of 150 million years to a maximum of about 225 million years, based on remanent magnetization studies, and from a minimum of 168 million years to a maximum of 260 million years, based on Potassium/Argon (K/Ar) dating.

The Jonesboro Fault and the site fault are relict structural features in a tectonically very stable region. Neither they nor other faults in the site area are capable faults. Further details of these investigations are presented below and in the Shearon Harris Fault Investigation Report (Reference 2.5.1-29).

Several small non-capable faults were found in the foundation of the Main Dam as described in Section 2.5.6.2.

2.5.3.1 Geologic Conditions of the Site

Details of the geologic conditions of the site are presented in Section 2.5.1.

2.5.3.2 Evidence of Fault Offset

2.5.3.2.1 The Jonesboro Fault

The southeastern side of the Deep River Basin is formed by the Jonesboro Fault, a northeast-southwest trending diagonal slip fault, dipping northwest, with vertical displacement of 8,000 to 10,000 ft. and unknown right-lateral displacement. For more than 100 miles along the southeast side of the Deep River Basin, the fault is the contact between Triassic and Paleozoic rocks. There are five other major northeast trending longitudinal faults within the northern half of the Deep River Basin. Of the six, the nearest to the site is located four miles to the southeast. It is the only one identified in the site area investigation (see Figure 2.5.1-4). The Jonesboro Fault

and the other major faults of the Deep River Basin are probably reactivated older structural trends in the basement rocks.

In addition, a system of northwest-trending minor cross faults fractured the rock of the southern half of the Deep River Basin. During the site investigation, no evidence was discovered to suggest the Jonesboro Fault is offset by cross faults.

Campbell and Kimball (Reference 2.5.1-16), who believe that final movement on the Jonesboro Fault was the last episode in the structural development of this region, reported that the fault cuts all other structures.

Reactivation or initiation of tensional, normal-type movement along some possible lateral components on the Jonesboro Fault was followed by deposition of Triassic sediments. Prouty (Reference 2.5.1-33) concluded that movement on the Jonesboro Fault continued sporadically from the beginning of sedimentation until after it ended.

Magnetic and reconnaissance surveys were conducted on diabase dikes and "cross faults" occurring along the Jonesboro Fault in an effort to date the site fault. Five of these locations, shown on Figure 2.5.3-1, were studied and mapped. Magnetic data was collected at each location using a portable proton magnetometer along traverses which were surveyed by Brunton compass. This magnetometer data was used along with information obtained from standard field investigation methods to correlate (based on geometric relationships) similarities and differences between dikes encountered on either side of the Jonesboro Fault. In addition, seismic and plane table surveying methods were used at location 5. The information gathered at locations 1-5 indicates that none of the dikes mapped at these locations are continuous across the Jonesboro Fault. Evidence, although inconclusive, suggests that at least one pair of dikes at each of the first four locations were at one time continuous and have subsequently been offset by the Jonesboro Fault. The amount of offset between dikes at southern locations 1-4 varies from 1300 ft. at location 4 to 420 ft. at location 3. However, possible offset at the northernmost area, location 5, is only 60 ft. An alternative interpretation at location 5 is that diabase was intruded into the fault zone concurrent with last movement of the fault. Consequently, dating of the last movement of the Jonesboro Fault by the absolute dating of diabase does not appear possible at locations 1-4 and is questionable at location 5.

The age of last movement on the Jonesboro Fault therefore bracketed between the intrusion of Late Triassic-Jurassic dikes and the deposition of the Cretaceous marine sediments overlying it. Appendix M of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29) discusses the Jonesboro Fault diabase dike relationship fully.

2.5.3.2.2 The Site Fault

The site fault was discovered in the excavation of the plant Waste Processing Building. Figures 2.5.1-15 and 2.5.3-2 show the fault in relation to the plant excavation; Figures 2.5.1-4 and 2.5.3-3 show the fault in relation to the geology of the site.

2.5.3.2.2.1 Investigation Procedures and Results

2.5.3.2.2.1.1 Locating the Fault

After discovery of the site fault, it was traced some 8,000 ft. east and west by digging short trenches normal to the fault. When exposed in sedimentary beds, the fault exhibits a southerly dip between 60 and 90 degrees, always exhibits drag folding on the hanging wall, and seldom exhibits any disturbance of bedding planes on the northern or foot wall. The fault tends to become oversteepened in coarser grained sandstones and is nearly vertical adjacent to diabase dikes offset by it.

In all cases, identification of the site fault was positive. Furthermore, 4,000 linear ft. of trenching revealed no additional faults. Details of procedures and observations along the fault are described in Appendix P of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29). The alignment of the fault and detailed trench maps are shown on Figures 2.5.1-15, 2.5.1-16, 2.5.3-4, and 2.5.3-5.

2.5.3.2.2.1.2 Identification of Linear Features by Remote Sensing

Investigation of the fault discovered in the excavation of the plant included extensive use of remote sensing techniques to seek other linear features in the site region and area. These techniques included conventional high and low altitude aerial photography Side Looking Airborne Radar, including false color enhancement, and Skylab and Landsat imagery.

Location, length, and alignment of several hundred linear features were identified. Throughout this study, interpretations of lineaments were conservative. Only those features identified with some certainty were recorded; other marginal alignments were not included.

In the field, direct visual observation was made (geologist and photogeologist) of at least one checkpoint along each significant lineament, with emphasis on possible manifestations of ground movement. Lineaments close to the site, of unusual length, or parallel to known regional fault trends were given special attention.

No linear features were identified as capable faults on the basis of imagery evaluation, nor were any that were field checked identified as faults. The site fault was undetected by any imagery technique.

No evidence of surface or near-surface faulting was found near the site or regionally during remote sensing investigations and fuel checks, nor was any evidence found of other features potentially inimical to the safety of the plant. Remote sensing investigations are discussed in detail in Appendix C of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29).

2.5.3.2.2.1.3 Locating Diabase Dikes

A magnetic survey, using a proton magnetometer, was made across the SHNPP plant area in conjunction with the fault investigation. The purpose was to locate and trace all diabase dikes in the plant area. Readings were taken at 10 to 50 ft. intervals along several east-west traverses approximately parallel to the fault zone. Each dike was traced within the site by taking numerous readings along 25 to 200 ft. long traverses perpendicular to the estimated strike of

each dike. Only the highest reading, usually representing the down-dip edge of the dike, along each traverse was recorded.

Readings recorded along the trace of each dike were plotted on profile sheets. Complete results of site magnetic surveys are given in Appendix A of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29).

2.5.3.2.2.1.4 Vertical Component of Fault Offset

Nine core borings were completed in sedimentary rocks on either side of the site fault to determine the vertical component of offset. The cores were carefully studied for marker beds that could be correlated across the fault and from which offset could be determined. Correlation of marker beds in the site area proved to be extremely difficult because sediment lithology changes radically over short lateral distances.

On the west wall of the plant excavation, near the fault exposure on the north, or upthrown, block, there is a distinctive, thick lens of medium to coarse-grained, white to gray, arkosic sandstone. As shown on Figure 2.5.3-6, boring FB-1-74 in the excavation on the south or down-thrown side of the fault, penetrated this same sandstone lens, thus determining the vertical offset. The vertical component of offset at this location was determined to be 83 ft. Petrographic analyses were made of samples taken from the sandstone outcrop and from the sandstone in the boring. These analyses indicate that the petrography of the two sandstone samples is sufficiently similar to have come from different thin sections cut from the same hand specimen. However, this is not conclusive, since many arkosic lenses near the site are petrographically similar.

Borings in sedimentary rocks were made outside the plant excavation in two areas that straddled the fault trace to determine vertical offset in those areas. One area was about 800 ft. east of the plant excavation; a second was about 1,200 ft. west of it (Figure 2.5.1-15). In each area, four borings were made, consisting of a pair on each side of the fault to help identify marker beds on one side, which could be correlated across the fault to the second pair. All borings were drilled at least 30 ft. from the fault to minimize effects of drag folding along the fault.

East of the excavation, two sandstone marker beds were correlated across the fault, as shown on Figure 2.5.3-7. The vertical component of fault offset in the east area was thus determined to be 98 ft.

West of the excavation, the four borings passes through a sandstone marker bed which had been offset about 92 ft., as shown on Figure 2.5.3-8. The results of this program indicate that vertical offset on the fault is greater than 80 ft. but less than 100 ft.

2.5.3.2.2.1.5 Horizontal Component of Fault Offset

Marker beds in the sedimentary rocks could not be used for determining horizontal offset at the fault because of the gentle dip of beds, lack of distinctive physical characteristics, and lateral depositional lithology changes over short distances. The near-vertical diabase dikes proved to be the best references for determining horizontal offset. However, offset of dikes only shows postintrusive movement, not necessarily the total offset of the sediments.

The best exposure of dike offset is illustrated on Figure 2.5.3-9, which shows the offset of composite East Dike 2 at several elevations. The apparent horizontal offset of the dike segments is quite small for a dike of irregular thickness to have been offset vertically above 90 ft., as were the sedimentary rocks. This suggests that the dike segments have not been offset vertically as much as the sediments. Figures 2.5.3-9 and 2.5.3-10 show that the easternmost segment of the dike displays more apparent horizontal offset than the smaller, westernmost segment at each elevation. This could be explained by separate emplacement of the dike segments during the period of fault movement. The easternmost segment should show more displacement because it is older and has been subjected to a longer period of activity on the fault. Paleomagnetic data (see Appendix G, Shearon Harris Fault Investigation Report, Reference 2.5.1-29) show that the smaller westernmost dike segment is younger than the other segments. This suggests that the sequence of events was (1) movement on the fault, (2) intrusion of the easternmost dike segment, (3) continued movement along the fault, (4) intrusion of the central dike segment, (5) continued movement along the fault, (6) intrusion of the westernmost dike segment, (7) minor continuing movement along the fault, (8) crystallization of laumontite, (9) final movement on the fault, and (10) low-grade burial metamorphism, with crystallization of zeolites harmotome and heulandite.

The minimum horizontal offset is 0.5 feet, ranging up to a maximum of 13 feet. A large horizontal component of movement is precluded in that the fault changes strike about every 300 feet.

2.5.3.2.2.1.6 Width of Fault Gouge Zone

The fault-gouge zone varies from a few inches to about 3 ft. in width. Figure 2.5.3-10 illustrates the varying width of the zone; Figure 2.5.3-11 shows the zone in relation to offset at one of the dikes.

2.5.3.2.2.1.7 Age of Movement on the Site Fault

The last movement on the site fault was geologically ancient. Independent lines of evidence strongly indicate there has been little or no movement on it in the past 150 million years or more.

a) Evidence from Secondary Mineralization

A number of secondary minerals are found in the fault gouge at the intersection of the fault by diabase dikes. These include zeolites harmotome, heulandite, and laumontite occurring in the fault gouge near both West Dike 3S and East Dike 2. Minimum age of the zeolites minerals has been determined from their K/Ar content. That these are minimum ages is supported by other special studies. Other secondary minerals identified include calcite, f-saponite, pyrite, and barite. Coal is also present in minor amounts. All secondary materials and coal are found below the present groundwater table. Coal and pyrite occur with carbonaceous shale in the fault plane at the diabase dike-fault intersection at East Dike 2. F-saponite occurs as a secondary mineral in the fault gouge at East Dike 2. While these minerals (other than the zeolites) and coal probably cannot be dated, they formed in the fault plane subsequent to latest movement.

K/Ar data firmly establish that heulandite sample 4 KA 74-210 is greater than 10 million years old, as shown with other zeolite ages on Figure 2.5.3-12, but, as the Sr^{87}/Sr^{86} ratios

shown on Figure 2.5.3-13 indicate, heulandite is probably cogenetic with emplacement of the diabase dikes and is therefore more realistically 150 million years old.

That the age from K/Ar data is much less than that of the dike is easily accounted for by the recognized tendency for zeolites to lose argon. The intact condition of some of the very brittle, delicate zeolites indicates that they were formed after faulting, so that the last movement on the fault was certainly more than 10 million years ago, most probably before the final cooling of the dike about 200 million years ago or at least during a burial metamorphic event before 150 million years ago.

The remaining K/Ar minimum ages for zeolite samples on Figure 2.5.3-12 range up to 35.3 ± 14 million years before present (B.P.) for sample 2-ETB. The K/Ar studies further demonstrate that significant amounts of argon are readily lost during preparation of samples for mass spectrometric analyses. It is believed that even at room temperature, argon will be lost from zeolites under vacuum, because of diffusion due to reduced partial pressure outside the crystal structure. Because of this, the calculated dates given for this last group of analyses must be considered absolute minimum ages. From the nature of zeolites, it is apparent that most of the argon is lost naturally, prior to mass spectrometric analysis. Indeed, since the potassium in zeolites resides in large cation exchange locations, there is little tendency for the argon to accumulate.

Studies of the Sr^{87}/Sr^{86} ratios of zeolite samples indicated the likely source of the zeolites which occurred in the fault zone at the intersections with diabase dikes. The Sr^{87}/Sr^{86} ratios of the zeolites in diabase and fault gouge are consistent with zeolite formation from late-phase hydrothermal solutions derived from the cooling diabase dike and contaminated by no more than 20 percent of the strontium dissolved from the surrounding fault-gouge material. Since the volume of the enclosing sedimentary rocks is much greater than the volume of the diabase, it is extremely unlikely that the zeolites could have precipitated from groundwater solutions with a strontium content derived from the sedimentary rocks reduced in Sr^{87}/Sr^{86} ratio by later contact with the diabase dike.

Zeolite sample 2-ETB has a Sr^{87}/Sr^{86} ratio so close to that of the adjacent dike that it appears to have undergone very little strontium-cation exchange. This sample, and perhaps all the zeolites, formed with a Sr^{87}/Sr^{86} ratio of less than 0.7060. The zeolites could not have formed with a Sr^{7}/Sr^{86} ratio of much less than 0.704 to 0.705, unless they formed from strontium in magma or rocks derived directly from the upper mantle. The initial Sr^{87}/Sr^{86} ratio of the diabase was 0.704. Therefore, it appears that the only suitable source of the strontium in the zeolites is the same as that of the diabase. Therefore, the diabase and zeolites are genetically related.

Secondary mineral samples obtained from the fault zone at East Dike No. 2 and West Dike 3S were studied. Zeolite minerals were not observed in the fault zone at West Dike 3; in the fault zone, away from the dikes; nor in representative rock samples from surrounding Triassic sediments.

The zeolite mineral assemblages are hydrothermal and/or "burial" metamorphic and are always associated with the diabase dikes. Evidence for this association can be found in their occurrence only in the dike-fault intersections or as small veins or amygdule fillings in the dikes, and in their absence from the fault zone away from the dikes.

Although it is difficult to determine the exact temperature of formation of these minerals, a conservative assessment would be about 150 ± 50 C for heulandite and harmotome, and 200 ± 50 C for laumontite and f-saponite. Ambient temperatures in the sedimentary rocks at these shallow depths of intrusion (as suggested by undeformed amygdules and crystal-lined open cavities in the dikes) were certainly lower than 100 C (around 40 C at 0.3 km with a 50C/km geothermal gradient). There is no evidence of activity in this area since the middle Mesozoic. Hence, it is reasonable to assume that the heat necessary for these hydrothermal solutions could only have come from the dikes. Magmas of this composition begin crystallizing at about 1250 C and are completely solid at about 1140 C. Thus, crystallization of the secondary minerals occurred after the dikes had completely solidified and mostly cooled. The wall rocks immediately adjacent to the dike would have reached 200 C in a geologically short time after intrusion of the dike, given the following reasonable conditions:

1) liquidus temperature of 1250 C

2) depth of intrusion of 0.3 km

3) geothermal gradient of 50 degrees C/Km

4) width of dike of 1 m

Obviously, the larger the dike, the higher the geothermal gradient, and the deeper the depth of intrusion, the longer the time that is required for the dike and adjacent rocks to cool.

If some of these secondary minerals owe their existence to the later burial metamorphic event, rather than deuteric hydrothermal activity, then continued downfaulting of the graben and sediment accumulation must have buried the dikes much deeper than the level of crystallization. Assuming a high geothermal gradient (50C/km), a temperature of 150 C requires a depth of burial of almost three km; it would have been necessary for this much overburden to have been removed since middle Mesozoic time. The Jurassic burial-metamorphic event, more than 150 million years B.P. (before present) is the only such event recorded as paleomagnetic chemical remanent magnetization (CRM) in the dike rocks.

All the secondary minerals can be found as free-growing crystals in small druses, vugs, and cavities, or as mosaics in veins in the fault zone, so there is no doubt that these minerals are younger than at least the initial movement on the fault. It is critically important to know whether there are any zeolites younger than the last movement on the fault, a question that can be answered by examining each zeolite mineral megascopically and microscopically, looking for evidence of cataclastic deformation or fault-induced strain in the crystals. Thus, zeolites can be readily studied for evidence of age and movement; for this reason, they were the only minerals considered in these studies.

Some of the blade-like laumontite crystals examined exhibit undulatory extinction; others did not. Such extinction does not necessarily indicate that crystals were deformed by movement on the fault. Zeolite crystals are very fragile; even grinding the thin-section may cause some strain. Dehydration of laumontite to leonhardite by exposure to air perhaps can also cause strain. Optical properties of this "laumontite" indicate that it is now leonhardite, although it was true laumontite when the x-ray diffraction analysis was made.

However, there is some stronger evidence to suggest that at least some of the laumontite in the fault zone was cataclastically deformed. A photomicrograph of a laumontite vein in sample ED2-5 (see Appendix H of Shearon Harris Fault Investigation Report, Reference 2.5.1-29) shows clearly that this material has been cataclastically deformed, as evidenced by shearing, and also shows mechanical disaggregation and rotation of laumontite grains. All this deformation could not have occurred in preparing the thin-section. Hence, there was some movement on the fault after crystallization of at least some laumontite.

Photomicrographs of harmotome and heulandite show anhedral to subhedral polycrystalline mosaics in veins cutting the fault zones. Undulatory extinction is slight to nonexistent, and the cataclastic deformation seen in some laumontite is not observed. The perthite-appearing structure in the harmotome is complex twinning, a characteristic of almost all harmotome crystals, which can appear superficially as undulatory extinction, but the sharp interface between twin domains and the simultaneous extinction of many domains in a simple grain rule out simple undulatory extinction.

Moreover, the small, delicate, pink heulandite crystals of sample D3S-SHNPP (see Appendix H of Shearon Harris Fault Investigation Report, Reference 2.5.1-29) that occur in apparently undisturbed veins at inclined angles to the fault plane could not have withstood movement on the fault without being deformed. Both megascopically and in the microdetails of thin sections, the zeolite minerals do not show effects of mechanical deformation. Therefore, the final movement on the fault can be bracketed between the crystallization of laumontite and harmotome at East Dike 2 and before crystallization of heulandite at West Dike 3S.

If the dikes and surrounding rocks cooled rapidly, it would seem coincidental that the crystallization of laumontite and harmotome or (laumontite and heulandite) would bracket the time of last movement on the fault, assuming they were derived from the same hydrothermal solutions. The time interval between laumontite and harmotome crystallization would have been relatively short. It has been reported that Mesozoic diabase dikes of eastern North America underwent burial metamorphism several million years after the dikes were intruded. Paleomagnetic dating showed that the diabases at the Shearon Harris site underwent burial metamorphism about 20 million years after dike intrusion. Accordingly, it is reasonable to conclude that laumontite was associated with the original deuteric hydrothermal alteration of the dike shortly after its crystallization, whereas harmotome and heulandite were formed during the burial metamorphism some 20 million years later. It is concluded that the undisturbed secondary zeolites in the fault gouge have minimum radiometric ages ranging up to about 35 million years but are more realistically older than 150 million years B.P.(before-present).

The material presented above has largely been extracted from reports of individual consultants. Their full reports and findings are presented in Appendices B, E, F, and H of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29).

b) Evidence from Soil, Saprolite, and Sediment Exposures

1) Soil and Saprolite Overlying Triassic Sedimentary Rocks

The fault has not moved during formation of existing soil and saprolite on Triassic sedimentary rocks, as seen in trenches and cuts. Clay mineralogy studies revealed this to be an inplace weathering profile.

The soil is described as White Store, a brownish yellow, fine sandy clay to clay. A complete description is included in Appendix I of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29). As seen in outcrop, hand specimen, or microscopically, the residual soil horizon has not been disturbed by the fault. Below this soil horizon, weathering decreases with depth; from 2 to 15 ft. below it, the material is classified as saprolite. Saprolite is thin over the sedimentary rocks because of the impermeable soil and because the sedimentary rocks approach chemical equilibrium under nearsurface conditions much more closely than do the diabase intrusives. Thin secondary clay infillings, primarily montmorillonites and clay-sized micas, are undisturbed along the fault plane and in adjacent joints and fractures. Plaster and Sherwood (Reference 2.5.3-1) state that residual soils developed in a temperate non-glaciated area in the mid-Atlantic region of the United States are quite old, and typically show classical A, B, C, and D profile development. They indicate that a strong profile development would be expected, because residual soils in this area have been variously estimated to be as old as Miocene. More rapid erosion has robbed the plant-site soils of the opportunity to develop this strong profile (see Appendix I of the Shearon Harris Fault Investigation Report, Reference 2.5.1-29). Thorp, quoted in Buol and others (Reference 2.5.3-2), states that a red-yellow podsolic soil, which belongs to a group that includes the soil developed over the Triassic sedimentary rocks at the plant site, is preserved in Georgia under early Pleistocene deposits; it may be Pliocene, and therefore older than one million years.

2) Soil and Saprolite Overlying Diabase Igneous Intrusive Rocks

No movement occurred on the fault during weathering and formation of the soil-saprolite profile, 35 ft. thick, overlying the fault trace at East Dike 2. This weathering profile is undisturbed in the saprolite above the fault plane, and there is no differential depth of weathering across the fault. If movement along the fault had occurred during the formation of the thick saprolite, the soft clay materials would have been easily deformed. Any deformation would have been preserved as shearing of the clays, resulting in rearrangement of the normal weathering profile, both in the saprolite and as differential depth of weathering across the fault. This is not the case; the profile is uniform.

The only available evidence tracing the fault across East Dike 2 in the saprolite zone is the apparent offset of individual apophyses of the diabase and at some elevations the deposition in the fault trace by groundwater of what appears to be secondary manganese. This finding is consistent with that of Harrington (Reference 2.5.1-28).

Evaluation of the minimum time required to form a saprolite cover over 35 ft. thick at East Dike 2 is based on the following considerations:

a) There is no evidence of shearing in the saprolite or residual soil overlying hard rock in East Dike 2 at the fault.

b) Groundwater levels at the site have been higher than at present, or stable since the Late Jurassic, because the ages of secondary minerals in the fault gouge are of that order of age. These secondary minerals are found only below the mixing zone of oxidation that underlies the groundwater table; they were leached out of the diabase at elevations above this zone. Therefore, it can be interpreted that the secondary minerals would have been leached out of the gouge had the groundwater table been lower during any long period since their formation. These minerals do not form below 100 C, and there is no evidence of a high temperature regime in these rocks since the period of regional low-grade metamorphism following intrusion of the diabase dikes. The minerals occur only in association with the dikes.

A dynamic equilibrium exists between the development of the weathering profile of saprolite and surface erosion. Such equilibrium is necessary, but the degree to which it can be refined is questionable. At some places near the site, diabase is weathered to depths as great as 90 ft.; therefore, the equilibrium postulated is largely controlled by such factors as degree of slope, and others relating to the degree of protection of a location from surface erosion over long periods; i.e., mineralogy, climate, and degree of fracturing.

The diabase dikes tend to act as barriers to lateral groundwater movement. All springs known in association with them flow from hornfels adjacent to the dikes. The degree of oxidation and saprolitization of the diabase is greatest near the ground surface and above the groundwater table. From the groundwater table down to approximately 30 ft., changes in the degree and intensity of oxidation are very gradual, decreasing with depth. Below 30 ft., however, there is a relatively rapid change with increasing depth to unweathered hard rock. For example, at the East Dike 2 fault, this transition occurs and is complete at a depth of 42 ft.

Carson (Reference 2.5.3-3) studied the maximum depth of oxidation in glacial deposits of the Olympic Peninsula in Washington. The greatest depth occurs in the oldest of these deposits, the relatively permeable Wedekind Creek Formation, which is oxidized to depths below 30 ft., largely above groundwater level. The age of this deposit is 530,000 to 700,000 years B.P. Near the site fault the depth of oxidation below the groundwater table in the diabase of East Dike 2, a profoundly less permeable though coarser grained material, is approximately 40 ft. On the basis of comparative depth of oxidation, the diabase has not been disturbed by movement on the fault for more than 500,000 years.

The development of weathering rinds on basalt clasts in the glacial deposits studied by Carson (Reference 2.5.3-3) is analagous to the development of spheroidal weathering rinds commonly seen in diabase at the plant site. This weathering process, carried to its extreme phase, results in the complete decomposition of the diabase, which is chemically unstable under oxidizing near-surface conditions. In the material Carson studied, the greatest mean thickness of such rinds is 6 mm, on clasts from

the Wedekind Creek Formation. According to Carson, the youngest age for this formation is 530,000 years B.P.; therefore, extrapolation of a weathering-rind formation rate of 6 mm/530,000 years would indicate that the complete weathering to clay of a 6-ft. diameter boulder of the basalt would take some millions of years. Inasmuch as the diabase is commonly weathered to clay to depths greater than 30 ft. at East Dike 2 and 90 ft. elsewhere, it may be concluded that the time of last movement of the fault, as indicated by lack of shearing in the saprolite, is at least a few millions of years B.P.

Certain variables, however, introduce uncertainty into such a comparison. One of these is the variation Carson found in the basalt clasts he studied and the possibility they underwent some degree of weathering prior to their deposition. Another variable is the relationship of the rate of weathering and oxidation to the location of individual basalt fragments, with respect to groundwater level and the rate of groundwater movement. Still another is the chemical differences between the basalts studied by Carson and the diabase found at the site. Taken together, however, the evidence of a state of equilibrium between erosion and weathering, the impermeability of the diabase, the depth of oxidation, and the rate of speroidal weathering clearly suggest extremely great age for the undisturbed saprolite profile in diabase at the fault.

Undisturbed secondary clay infilling crosses the fault in weathered diabase at East Dike 2. Undisturbed spheroidal weathering in diabase was observed in places.

3) Undisturbed Sedimentary Rocks Overlying the Fault

In trench FET-19W, the fault trace underlies an uncemented sedimentary deposit, a relationship illustrated on Figure 2.5.3-14. In this exposure the trace of the fault does not offset the alluvial rock unit overlying the fault. A strongly developed soil profile was observed in the alluvium. Fault movement has not occurred since deposition of the alluvial sediments. It has not been possible to date the sediments by radiometric methods because radioactive isotopes are absent. The alluvium is oxidized throughout its entire depth; therefore any pollen or microfossils would have been leached out. Because of the lithologic character, distance from the Cape Fear River, and the compactness of these deposits resting unconformably on Triassic bedrock traversed by the fault, it is concluded that these sediments should be included in the group mapped and described by Reinemund (Reference 2.5.1-17) as "high level surficial deposits." The material is a dense, compact clayey sand containing small lenses of clean, medium to coarse angular quartz sand. At the base, a thin layer of rounded quartz gravel incorporates material from the underlying Triassic sedimentary rocks. Reinemund (Reference 2.5.1-17) summarized the age relations of these deposits as follows:

"Within the mapped area, the evidence indicates only that the deposits are post-Triassic. Observations made a few miles east of the Deep River field, by Stephenson, indicate that some of the materials are post-Cretaceous, and the deposits as a whole have generally been regarded as Tertiary, possibly Pliocene. It

is possible, however, that these deposits included materials as old as Cretaceous and as young as Pleistocene."

At minimum distance, these deposits are over five miles from Cape Fear River; Quaternary deposits are markedly closer. Reinemund describes Quarternary terrace deposits as increasing in age with elevation and distance from the river, which is reasonable, and remarks that the higher of these terraces may be prePleistocene.

The material overlying the fault at trench FET-19W is uniformly graded sand, markedly different from Reinemund's Quarternary terrace deposits which were found in field investigations to be grossly gap graded, coarse gravels, cobbles and small boulders in a silt-clay matrix. The Quaternary terrace deposits appear to have been formed in low-energy environments in which letdown of coarser clasts from older, higher terraces tool place into the silt-clay regime of deposition.

The soil profile developed in the undisturbed sedimentary rock unit in trench FET-19W is approximately 6 ft. thick, a thickness consistent with that of the soil profile of similar quartz sands, known to be Pliocene, in middle Coastal Plain sediments.

This correlation is further evidence that the deposit at trench FET-19W is older than one million years.

Since the deposit seen in trench FET-19W is at approximate Elevation 240 ft., and there are other outcrops of these deposits at elevations above 600 ft., it may be presumed that the deposits, as described by Reinemund, were some 400 ft. thick in this area, and have subsequently been eroded away, an interpretation confirmed by John Reinemund during this examination of the outcrop in December, 1974. Reinemund interprets these materials as probably Tertiary but agrees they could be as old as Jurassic. These deposits may represent updip outcrops of strata identified as Cretaceous by Conley (Reference 2.5.1-20) and Swift and Heron (Reference 2.5.1-21). They could represent a remnant of the Jurassic burial episode described in Section 2.5.1.1.5.

Since the deposit is perched some 25 ft. above the Holocene stream channel to which its surface drains, and is distinctly different than the gray sand silts of the present stream channel, it cannot be of Holocene age.

c) Evidence from General Geology

The fault is in a geologic setting of about 150 to 250 million years B.P. Faults displacing strata younger than Paleocene were neither found during this study nor previously in the Carolina Piedmont-Coastal Plain. Therefore, by association, the fault is considered a very ancient feature in a historically and geologically aseismic area. Ancient, currently stable faults in the site region do not offset present topographic features or drainage patterns. The topographic relief is subdued.

Conley (Reference 2.5.1-20) states that cross faults can be traced into this Triassic basin and that these faults have displaced Paleozoic Slate Belt rocks as much as a mile along strike but have displaced the Triassic sedimentary rocks only a few hundred feet.

This indicates that major movement on the faults occurred before the Triassic, with only minor subsequent movement.

There has been no movement on the Jonesboro Fault, one of the great faults crossing North Carolina, since deposition of Cretaceous marine sediments over it. Major faults of this type, as well as minor faults, sometimes undergo recurrent movement, with a different sense of movement than before, when stresses in the earth's crust trigger macroseismic activity. Recurrent movement has not occurred since the pre-Cretaceous on the Jonesboro Fault or on the cross faults described by Conley. After minor deformation following deposition, about 150 million to 250 million years B.P., the Triassic basin must have been quite stable, because sedimentary beds are not closely folded.

From the geologic setting and additional points developed in other sections of this report, it is concluded that the site fault moved late in the history of the other areal/regional faults, is millions of years old, and is a relict feature, inactive and not capable.

d) Evidence from Geometry and Field Observations

The general spatial relationship of the dikes, site fault and plant as discussed below is shown on Figures 2.5.1-15 and 2.5.1-16.

1) East Dike 2

From evidence throughout the hard rock exposure developed by excavation on composite East Dike 2, faulting occurred after crystallization of the youngest of the three dike segments. At the dike-fault intersection, however, there are marked differences in the apparent offset among the three dike bodies constituting what is called East Dike 2, as shown on Figures 2.5.3-9 and 2.5.3-10.

The East Dike 2 exposure supports the following important interpretations:

a) Sedimentary beds are not offset across the three individual dike bodies, as shown on Figure 2.5.3-9. Therefore, the dike does not follow a discernible fault. The three bodies were separately intruded and crystallized separately.

b) The vertical component of movement along the fault after intrusion of the youngest dike segment must have been small, at most a few tens of feet.

c) The horizontal component of movement after intrusion of the youngest dike segment also must have been small, possibly a foot or less.

d) The differences in horizontal offset of the individual dike bodies reflect their sequential intrusion during the period of movement on the fault, as seen in the table below, from Figure 2.5.3 9:

HORIZONTAL OFFSETS IN FEET			
DIKE SEGMENT			
Elevation	Youngest/Western	Central	Oldest/Eastern
247	1 - 3	1 - 3	6
244.8	2 - 3.5	4 - 6	5 - 6
243.1	3	3 - 4	6

238.5	?	5 - 6	8
234.6	2 - 6	6 - 7	4 - 8
218.5	6 - 7	6 - 7	10
210.3	5	8 - 9	10 - 13

Therefore, in spite of the overall vertically sinuous nature of the dike segments, the paleomagnetically oldest eastern dike segment is clearly displaced horizontally, and probably vertically through a depth range of 37 ft., more than the younger western segment. The horizontal relationship is demonstrated on Figure 2.5.3-10, where the two easternmost dike segments are shown offset and deformed, with left lateral sense, by a splay in the fault, while the western segment is not affected, strongly suggesting that the splintering movement took place prior to intrusion of the western segment.

Sequential intrusion of dike segments during faulting proves the contemporaneous nature of movement on the fault with intrusion of the dikes, which are known radiometrically and paleomagnetically to be more than 150 million years old. These conditions also offer supporting evidence that the site fault is a minor, late contemporary feature to the Jonesboro Fault. Last movement on the Jonesboro Fault was before deposition of Cretaceous sediments that are present south of the plant site. The diabase dike identified as Location 5 in Appendix M of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29) may have been intruded during the last phase of movement on the Jonesboro Fault.

2) West Dike 3S

West Dike 3S intersects West Dike 3 at a point several hundred feet north of their intersection with the fault (See Figure 2.5.3-15). West Dike 3S is younger than West Dike 3. Deeper excavation revealed a second intersection of the dikes, about 8 ft. lower than the first. At the lower exposure, the smaller West Dike 3S intrudes but does not cross the larger West Dike 3, proving that the smaller dike is younger. The outcrop pattern of the dike intersection is unequivocal. Textural differences in the two dikes are distinct. Paleomagnetic sample PM 7 from West Dike 3, at the intersection between the two dikes, shows a reset of remanent magnetization in the larger dike, also providing further confirmation that West Dike 3S is the younger (Figure 2.5.3-11).

Sedimentary beds on either side of West Dike 3S are not offset. At Elevation 249 ft., the outcrop of the intersection of West Dike 3S and the fault is such that West Dike 3S, if any apparent offset can be detected, is only a few in. out of line. This is seen on Figure 2.5.3-11. Subsequent excavation to Elevation 233 ft. revealed relative change in location of all key elements of the intersection. Analysis of these changes indicates that they could not have been produced by fault offset if the dike were as planar at the fault as elsewhere.

The exposure of the intersection of West Dike 3S and the fault at Elevation 239 ft. offers strong evidence that West Dike 3S was intruded after movement on the fault (Figure 2.5.3-15). As observed and described, the dike is straight everywhere but at the fault, where, however, it would be expected to have been straight originally if intruded before faulting. The irregular aspect of the dike at this exposure is apparently related to intrusion, not faulting, because the adjacent sedimentary bedding appears to be less disturbed than would be expected if faulting had caused the existing dike configuration

(Figure 2.5.3-15). Furthermore, as can be seen on Figures 2.5.3-11 and 2.5.3-16, diabase of the West Dike 3S overlaps the sheared clay of the fault zone; thus, two apophyses at the end of the northern limb of this exposure appear to offer compelling evidence that the diabase was intruded after faulting. There is a smaller, football-shaped apophysis immediately west of the stub end of the southern limb of West Dike 3S. Zeolite samples were obtained from this apophysis and adjacent fault gouge; a detailed portion of this area is shown diagramatically on Figure 2.5.3-17.

Where apophyses enclose country rock against the main body of the dike, the intensity of alteration in the surrounded rock is greater than in other immediately adjacent areas. This is consistent with evidence that alteration at this point was proceeding from two sides, from both the diabase dike and the apophyses.

Clay mineralogy studies of gouge material at the stub ends of the dike at Elevation 239 ft. did not reveal characteristic changes expected from thermal alteration. The ends of the dike so overlap the clay gouge that a zone of sheared clay only one fourth to one inch thick is exposed.

At lower excavation elevations, burned and baked typical fish-scale clay gouge is found in place at the stub ends of the dike segments and extends as a band between the segments. During the final stages of crystallization and initial cooling of the dike, some very slight movement must have occurred on the fault, sweeping thermally altered material off the most protrudent incursions of the dike into the gouge. In contrast, if the main movement along the fault had occurred after dike emplacement, the thermally altered material would have been wiped from the ends of the dike at all points and chaotically rearranged within the fault gouge. In East Dike 2,, a similar intrusion, not associated with faulting, is shown on Photo V of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29). This illustrates the precise structural control of the dike emplacement.

Figure 2.5.3-18 represents possible solutions to fault movements, focusing on the West Dike 3S-site fault intersection. West Dike 3S is shown at true dip (about 80°) to illustrate what happens during various fault movements. From general aspect, either Panel 3 or 4 could be interpreted to represent the movement of the fault on the basis of the change in dip of the diabase, as shown in Panel 3, or on the basis of the alignment of the two ends, as shown in Panel 4. However, these solutions are ruled out, because the dip of the diabase remains the same both north and south of the fault intersection.

3) West Dike 3

West Dike 3, which was not excavated to depths sufficient to expose hard rock at the fault intersection, exhibited about 10 ft. of apparent left-lateral offset, as shown on Figure 2.5.3-15.

4) West Dike 4

This dike was not excavated, because the fault crossing underlies a trunk rail line.

5) West Dike 5

West Dike 5 is unique in having pronounced, probably superficial, westerly dip north of the fault and right-lateral apparent offset. It was not excavated to depths sufficient to expose hard rock at the fault intersection. The anomalous, near-surface westerly dip of this dike may be the result of the intrusion into broken ground adjacent to the fault.

e) Evidence on Dike Intrusion Sequence from Petrography and Chemistry

Petrographic and chemical work on the dikes provided a critical test for field observations concerning relative ages of dikes and site fault. Field evidence established that composite East Dike 2 was intruded during fault movement; West Dike 3S was intruded after most movement had occurred, but before a final, minor element of movement; and West Dike 3 (Figure 2.5.3-13) was probably intruded early during movement and offset about 10 ft. left laterally.

At first inspection, the chemical variation diagrams (Figures 1 and 2 of Appendix H of the Shearon Harris Fault Investigation Report, Reference 2.5.1-29) contradict field observations concerning the relative ages of West Dikes 3 and 3S. West Dike 3S comprises the more mafic rocks and would be expected to be older. However, the generalization that the more mafic rocks are intruded first only holds in the case of non-porphyritic rocks, i.e., when 100-percent magma rather than a "crystal mush" is intruded and the chemical trend on the variation diagram is a true "liquid line of descent." West Dike 3 was intruded as a nearly 100-percent magma, while West Dike 3S was intruded as a "crystal mush." Both were derived from the same crystallizing magma chamber (comagmatic), which was tapped early, before many crystals had formed; West Dike 3 was the result. A subsequent tap of the chamber, after many crystals (phenocrysts now) had formed, produced West Dike 3S.

A test of this hypothesis would be whether the groundmass of West Dike 3S is less mafic than that of West Dike 3. Unfortunately, a chemical analysis of the West Dike 3S groundmass is not available and would be difficult to determine accurately. Modal analysis would not suffice, as the critical factor is the composition of individual minerals in the groundmass. The only mineral composition readily determined without a microprobe is that of plagioclase; fortunately, plagioclase alone can provide an adequate test of the hypothesis.

Comparative results for plagioclases in West Dike 3 with those in West Dike 3S are given in Table 2 of Appendix H of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29). West Dike 3 plagioclases are anorthite-rich, thus agreeing with field observations that West Dike 3 was intruded first.

Other reasoning can argue that West Dike 3 was intruded before West Dike 3S, which is more altered deuterically than West Dike 3, contains more volatiles, and contains hydrous silicate-filled amygdules. Isotope data on Sr^{87}/Sr^{86} for the dikes and associated zeolites indicate that the water necessary to form the zeolites came from the diabasic magmas, rather than from groundwater or sediment pore water. This water presumably also altered the dikes deuterically. The magma must have been supersaturated with respect to the fluid phase when the dike crystallized, as indicated by presence of amygdules. During crystal fractionation, volatiles are enriched in the magma so that late-stage differentiates are more volatile-rich than early differentiates. This agrees with

a younger age than West Dike 3 for the more volatile-rich and more highly altered West Dike 3S.

In conclusion, chemical and petrographic data cannot conclusively determine the relative age of East Dike 2 with respect to the other two dikes. However, they substantiate the field observations that West Dike 3S is younger than West Dike 3. This subject is more fully discussed in Appendix H of the Shearon Harris Fault Investigation Report (Reference 2.5.1-29).

2.5.3.2.2.2 The Fault as an Oriented Plane of Weakness

Brown, Miller, and Swain (Reference 2.5.1-42), after their exhaustive work on the structural architecture of the region, concluded that the area including the plant site is under east-west horizontal compressive force. They do not associate the secondary level and orientation of such stress with movement along faults aligned east-west in the contemporary stress field. Their conclusions confirm present stability of the site fault.

Observations in mines of the Deep River Coal Field, where mine openings in both sedimentary rocks and diabase dikes extend to depths of 800 ft. below the surface (and 600 ft. below sea level) have not revealed over-stressed rock. Timbered supports in one of these mines have been considered unnecessary for long-range stability, by the mine manager. Observation of representative room and pillar openings after supports were removed at the conclusion of mining indicated that large spans were self-supporting (Reference 2.5.3-4).

These reported conditions, in contrast to other areas where pop-outs and rock bursts are common occurrences at depths as little as 60 ft. below the surface because of excessive stored stresses in the rocks, indicate that the Triassic rocks of this area, including that of the site fault, are in an at-rest, stable condition.

Kiersch (Reference 2.5.1-36) reports overpressurized deep groundwater in Triassic sedimentary rocks of the Dunbarton Basin, South Carolina-Georgia. This should be a very sensitive indicator of stress levels in these impermeable rock types, compartmentalized as they are by impervious diabase dikes. Occurrence of such overpressurized groundwater in the Durham basin, where the plant site is located, has not been reported in data covering some 450 wells in the Durham Basin. This also leads to the conclusion that these rocks are in an at-rest, stable condition.

Talwani (Reference 2.5.2-12) has observed cases of reservoir induced seismicity (RIS) in the piedmont of South Carolina. Because of this observation, the pre-existing state of stress, the magnitude of the induced stress, and the geologic and hydrologic setting of the Shearon Harris Nuclear Power Plant have been examined to seek out any possible correlations that might exist with the RIS sites in the South Carolina Piedmont. This study is discussed in detail in Section 2.5.2.1.2. The comparison of the Shearon Harris site reservoirs to the four cases of reservoir induced seismicity in South Carolina shows no correlation in pre-existing stress, water depth, water volume, geologic or hydrologic conditions. Therefore, there is no reason to expect the SHNPP site reservoirs to induce any significant seismic activity.

2.5.3.3 Earthquake Associated with Capable Faults

A letter to Carolina Power & Light Company from the NRC (Reference 2.5.3-5), concurs with the Shearon Harris Fault Investigation Report (Reference 2.5.1-29) in concluding that the site fault is not a capable fault as defined in Appendix A to 10 CFR Part 100.

In summary, there are no capable faults that could affect the site and therefore this section is not applicable to SHNPP.

2.5.3.4 Investigation of Capable Faults

For the reasons given in Section 2.5.3.3, this section is not applicable to SHNPP.

2.5.3.5 Correlations of Epicenters with Capable Faults

For the reasons given in Section 2.5.3.3, this section is not applicable to SHNPP.

2.5.3.6 Descriptions of Capable Faults

For the reasons given in Section 2.5.3.3, this section is not applicable to SHNPP.

2.5.3.7 Zones Requiring Detailed Faulting Investigation

For the reasons given in Section 2.5.3.3, this section is not applicable to SHNPP.

2.5.3.8 Results of Faulting Investigations

For the reasons given in Section 2.5.3.3, this section is not applicable to SHNPP.

2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

2.5.4.1 Geologic Features

The plant is founded on well-consolidated Triassic sandstone, siltstone, shale, and claystone. The individual beds are generally lenticular; hence, rock types change abruptly horizontally and vertically. No areas of active or potential surface or subsurface subsidence, uplift, or collapse have been found; none are thought likely to be present.

Geologic evidence suggests that a considerable thickness of Triassic rocks has been eroded from the site; it is probable that some unconsolidated sediments of Cretaceous and younger age have also been deposited and eroded from the site. Most erosion probably occurred during the Mesozoic Era. Evidence suggests that erosion over the last few million years has been relatively slight.

Generally, groundwater in the site area is sparse, as the rocks have very low primary permeability. Water does occur in joints, fractures and bedding planes, however, and reaches useable quantities near some diabase dikes.

The depth of weathering in the bedrock varies essentially from 5 to 10 ft., depending upon the type of underlying rock. Local exceptions to this were found along the eastern portions of

Trenches 2 and 3 (for locations see Figure 2.5.1-11), where residual soils are from 0 to 4 ft. thick, and over diabase dike materials, where weathering was observed to a depth of 15 ft. or more. Other zones of irregular weathering and alteration have not been found.

Joints are somewhat irregularly spaced in the site area, most of them at intervals of a few feet and steeply dipping. The predominant trend direction is approximately northeast. A few joints trend approximately north-northwest; north-south trending joints are uncommon. Some joints parallel bedding. High angle joints in other directions and joints with moderate dips occur sporadically. Joints observed adjacent to diabase dikes are generally perpendicular to the strike of the dike, and extend 2 to 10 ft. laterally from the dike. When rock is weathered, light gray clay forms in the joints and fractures. Joints and other geologic features at the plant, the Main Dam and the Auxiliary Dam were mapped at a scale of 1 inch = 10 ft. (1 inch = 50 ft. outside of the core trench of the Main Dam). The maps are included in the Foundation Report which is Appendix 2.5E.

Low-angle (0 degrees to 45 degrees to the horizontal) and high-angle (46 degrees to 90 degrees to the horizontal) fractures were found at various elevations in the P and D series test borings shown in Appendix 2.5A. Fractures were most common in test borings P16-B and P17. Several borings penetrated thin layers of gray or green clay that occupied fractures in claystone strata.

Many fracture surfaces in the weathered rock were slickensided, but slickensided fracture surfaces were generally absent in the joints and fractures in the fresh rock forming the plant foundation; they are not thought to be related to tectonic activity. During excavation, a fault was discovered in the foundation of the Waste Processing Building; studies of this noncapable feature are reported i Section 2.5.3 and in the Shearon Harris Fault Investigation Report (Reference 2.5.1-29). No other folds, faults, shears, or other zones of structural weakness were noted at the plant excavation.

As noted in Section 2.5.1.2.3.3, there is no evidence of residual stresses in the bedrock.

The siltstones and a few sandstones in the plant area are subject to spalling if repeatedly wet and dried over a period of weeks. To minimize this problem, once the last phase of excavation cleanup was begun, the rocks were protected against inclement weather. Final cleanup used only air lances; water was not applied. After rock was cleared, it was moistened and the 4 in. thick concrete protective mat was applied, followed by the design concrete seal mat. In some areas, depending on the construction schedule, the design seal mat was placed after final cleanup without the 4 in. protective mat. No other potentially hazardous rock or soil conditions were noted.

Geologic conditions of the site are discussed fully in Section 2.5.1.

2.5.4.2 Properties of Subsurface Materials

2.5.4.2.1 General

The site is underlain by clastic sediment, principally shale, claystone, siltstone, and sandstone. These materials are characterized by changes in composition and texture, both horizontally and vertically. The beds vary in thickness from a few inches to a maximum of 15 to 20 ft.

Notwithstanding these variations in composition and texture, the beds and lenses interfinger and overlap into compact masses that show no structural weakness.

All safety related plant structures are founded on this compacted mass of clastic sediments. The plant is built on sound rock and plant grade is established at Elevation 260 ft. Plant structure foundations utilize reinforced concrete mats and spread footings to spread loads to an acceptable bearing value determined by laboratory tests. A discussion of dams, dikes, spillways, and channels, and the related properties of the subsurface materials at these locations, is presented in Section 2.5.6.

Test results for subsurface materials, in addition to those presented in this section, are included in Appendix 2.5B.

2.5.4.2.2 Static Properties

Static properties that were determined included index properties, compressive strength, and deformation properties.

2.5.4.2.2.1 Index Properties

2.5.4.2.2.1.1 Dry Density

Dry density determinations were performed on selected rock samples and are summarized in Table 2.5.4-1. The average dry density for the rock material is 162.8 pounds per cubic foot.

2.5.4.2.2.1.2 Grain Size Analysis

Grain size distribution (GSD) curves for the residual soil samples are presented on Figures 2.5.4-1 through 2.5.4-53. Tests were performed in accordance with ASTM D-422.

2.5.4.2.2.1.3 Core Recovery

Core recovery is defined as the length of core recovered divided by the length of the core run, expressed as a percentage. Core recovery values were computed for all rock cores, and results are presented in logs of borings in Appendix 2.5A.

2.5.4.2.2.1.4 Rock Quality Designation

Rock quality designation (RQD) is defined as the total length of sound rock core pieces, 4 in. long and over, divided by the length of the core run and expressed as a percentage (Reference 2.5.4-1). In cases of cores broken by mechanical fractures during drilling or handling (i. e. the fracture surfaces were fresh, irregular breaks rather than natural joint surfaces), the fresh broken pieces were fitted together and were considered as one piece, provided they formed the requisite length of 4 in. RQD values were computed for all cores and results are presented in the logs of borings in Appendix 2.5A. The average RQD value below Elevation 235 ft. for each BP boring in the plant foundation area is shown in Table 2.5.4-2. The average RQD value below Elevation 235 ft. in the foundation area is 92.7 percent.

2.5.4.2.2.2 Strength Determinations

2.5.4.2.2.2.1 Unconfined Compression Test

The compressive strength of selected rock core samples was determined by unconfined compression tests. Tests were performed in general accordance with ASTM D-3148. The average unconfined compressive strength for the rock material is 8148 psi. Individual test results are presented in Table 2.5.4-3. Stress-strain curves, developed by utilizing the test results, are presented on Figures 2.5.4-54 through 2.5.4-65.

2.5.4.2.2.2.2 Triaxial Compression Test

Triaxial compression tests were performed on selected rock core samples. The test results are summarized in Table 2.5.4-4. Individual stress-strain curves based on the triaxial compression tests are presented on Figure 2.5.4-66. A Mohr's circle plot, developed by utilizing both the unconfined and triaxial compression test results, is presented on Figure 2.5.4-67. The rock samples utilized in the triaxial compression tests exhibited an average friction angle (φ) of 45 degrees and cohesion (C) of 2400 psi based on the Mohr envelope shown on Figure 2.5.4-67.

2.5.4.2.2.3 Deformation Properties

2.5.4.2.2.3.1 Poisson's Ratio

Poisson's ratio (μ) was computed, as discussed in Section 2.5.4.7, using shear wave (Vs) and compressional wave (V_p) velocities determined from field geophysical data. The shear wave and compressional wave velocities of the subsurface materials were seismically determined, as discussed in Sections 2.5.2.5 and 2.5.4.4. Values computed for Poisson's ratio based on wave velocity measurements results are summarized in Table 2.5.2-3.

In addition, values of Poisson's ratio were determined from laboratory unconfined compression testing in which both axial and radial strains were measured. The results of the axial and radial strain measurements, and the computed Poisson's ratio values from laboratory testing, are presented on Figures 2.5.4-68 through 2.5.4-91 and summarized in Table 2.5.4-5.

The results computed for Poisson's ratio, especially those based on field geophysical data, are in good agreement with typical values for sandstones and siltstones available in the literature (Reference 2.5.4-2).

2.5.4.2.2.3.2 Static Modulus

The static modulus of deformation was computed from the unconfined compression tests. Values determined for static modulus are summarized in Table 2.5.4-3. These values are then reduced by a factor dependent on rock quality designation, RQD (Reference 2.5.4-1), as discussed in Section 2.5.4.7.

2.5.4.2.3 Dynamic Properties

The dynamic properties of the rock material were investigated through field and laboratory measurements of compressional and shear wave velocities. The dynamic modulus was then computed from the wave velocities, as discussed in Section 2.5.4.7.

2.5.4.2.3.1 Field Geophysical Measurements

2.5.4.2.3.1.1 Compressional Wave Velocity (V_p)

The compressional wave velocity (V_p) was determined through geophysical measurements made during seismic refraction surveys. A downhole velocity survey was performed to provide a check on the compressional wave velocity measurements obtained during the seismic refraction surveys. Discussion of the seismic survey program for geophysical exploration are presented in detail in sections 2.5.2.5 and 2.5.4.4. The test results are summarized in Table 2.5.2-3.

2.5.4.2.3.1.2 Shear-Wave Velocity (V_s)

Shear wave velocity (V_s) measurements were determined by using Sprengnether Engineering Seismographs. Details of the tests are discussed in Section 2.5.4.4. The results of these tests for each stratum are summarized in Table 2.5.2-3.

2.5.4.2.3.2 Laboratory Geophysical Measurements

The shockscope, an instrument developed by Dames & Moore (Reference 2.5.4-3) to measure the velocity of cmpressional wave propagation in laboratory samples, was utilized in measuring the compressional wave velocity of selected rock core samples. In the shockscope tests, samples are subjected to a physical shock 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. The compressional wave velocity (V_p) measurements based on the shockscope test are presented in Table 2.5.4-6. The velocity measured in the shockscope is used for correlation with field velocity measurements obtained in geophysical refraction and uphole surveys. Additional testing to determine the compressional wave velocities (V_p) of rock samples was conducted by Law Engineering in general accordance with ASTM D-2845. The results are also presented in Table 2.5.4-1.

2.5.4.3 Exploration

2.5.4.3.1 General

Field investigations, at locations shown on Figures 2.5.1-11 through 2.5.1-16, were performed to evaluate the engineering geologic and seismologic characteristics of the site. Field exploration consisted of: 1) an engineering geologic survey of the site and surrounding areas, 2) a preliminary test boring program, borings for design of plant structures and additional borings for the site fault investigation, 3) a trench excavation program, also expanded for the fault investigation, and 4) geologic mapping of plant foundations. The exploration also included installation and monitoring of wells and piezometers. Discussions of the installation and monitoring of wells and piezometers are presented in Sections 2.5.4.6 and 2.5.4.13.

The initial exploration program was performed by or conducted under the technical direction of Dames & Moore geologists and engineers. Moore, Gardner and Associates, Incorporated of Asheboro, North Carolina made surveys to establish horizontal and vertical controls at the site. Law Engineering Testing Company completed borings for design of structures and did geotechnical testing of materials as required. Ebasco Services, Incorporated geologists carried out the site fault investigation and mapped the plant foundation for Carolina Power & Light

Company (CP&L) during construction, and CP&L personnel performed the magnetometer traverses.

2.5.4.3.2 Engineering Geologic Survey

A comprehensive survey was conducted to identify the engineering geologic characteristics of the site and surrounding area. It included detailed inspections of 1) rock cores from test borings, 2) surface features, 3) exposed road cuts, 4) excavated trenches, 5) bedrock outcrops, and 6) a Brunton Compass survey. Geologic maps, literature, gravity-survey data, aerial photographs, and topographic maps were examined. Representatives of local and state agencies, universities, and private organizations were interviewed to obtain engineering geologic data.

2.5.4.3.3 Geologic Borings

Borings were drilled to investigate the surface soils, underlying sedimentary bedrock and the several diabase dikes in the investigation area. Preliminary borings of the P and D series (Figures 2.5.1-11) explored the plant and Auxiliary Dam vicinity, which were followed by closely spaced borings for foundation design of specific plant structures. Boring locations are shown on Figures 2.5.1-12 through 2.5.1-14. During construction additional borings were completed for the site fault investigation at locations shown on Figures 2.5.1-15 and 2.5.1-16.

Project borings, including those for dams, bridges, and other structures totaled about 1250 and are tabulated by purpose with coordinates and depths in Appendix 2.5A.

As indicated in the tabulation, most logs of the borings in the plant vicinity are also included in the Appendix. Logs for some proposed but abandoned structures such as the East Auxiliary Dam and for structures which are not Seismic Category I, such as the Cooling Tower, are not included in the FSAR, but are maintained in CP&L files. Logs for the fault investigation are shown on Section 2.5.3 figures.

Truck-mounted and skid-mounted rotary wash, wire-line drilling rigs were employed in the field programs. Drillers used a 3 7/8 in. diameter rotary core barrel to penetrate and sample the soils at each test boring location. When rock was encountered, a 2 1/8 in. double-tube core barrel and diamond drilling bit were used to advance the borings and collect continuous samples of bedrock.

A standard two in. outside diameter split-barrel sampler with an inside diameter of one and three-eighths in. was used to obtain soil samples for soil classification and for laboratory tests. In soils, the standard penetration test was made at every change of strata; and within strata, at intervals not exceeding five ft. For the fault investigation, some of the borings were sampled continuously using the standard penetration test.

Cased borings of sufficient size were made to accommodate either a three in. diameter thin-wall tube (Shelby Tube) sampler, a three in. diameter piston sampler, or a Denison-type, double-tube core barrel in order to obtain relatively undisturbed soil samples. Twenty-five pound bag samples of soil were obtained from uncased auger borings. The rock drilling program produced a rock core sample approximately two in. in diameter. Rock cores were examined in the field, logged, and stored in standard core boxes for future use and testing.

All borings were logged in detail by geologists. The soils encountered were described in accordance with the Unified Soil Classification System. RQD values (Reference 2.5.4-1), calculated for each core length and indicated on the boring logs, indicate the ratio of core, four or more inches in length, to the full core run.

2.5.4.3.4 Trench Excavation Program

More than 12,000 ft. of trenching was performed at the site during the original site studies to supplement the information obtained from the borings. Additional trenches were excavated during the fault investigation.

Locations of the four major trenches are shown on Figures 2.5.1-11, 2.5.1-15, and 2.5.1-16. Portions of Trenches 1 and 2 are adjacent to the plant site; trenches 3 and 4 are on the Auxiliary Dam alignment. Trenches were generally excavated to refusal of the backhoe equipment. Most trenches were 2 to 10 ft. deep; however, those in residual soil derived from diabase dikes were advanced to the full limit of the excavating equipment, 15 ft., without meeting refusal. Trenches for the fault investigation were excavated by backhoes and bulldozers to depths of 5-40 ft. One trench for the fault investigation required blasting.

All excavations were inspected in regular increments to evaluate lithology, quality, and continuity of the bedrock as well as overburden composition and consistency. Trench logs prepared for the original site studies are provided in Appendix 2.5A. Hand-penetrometer tests were made at intermittent intervals.

Layouts of trenches 1 and 2 were planned with regard to the regional geology. Trench 2 was normal to possible cross-faults, whereas Trench 1 was normal to the regional structural trend. Thus, trenches with these orientations would traverse any longitudinal and cross-faults that exist in the investigation area. Geological cross-sections of the trench excavations are presented in Figures 2.5.4-92 through 2.5.4-98.

Borings P9 through P18 and D11 through D18 (Figure 2.5.1-11) were drilled in the stream valleys which crossed Trenches 1 and 3 as a substitute for trenching. The loose and deeper overburden soils in these areas made excavation and data acquisition impossible by trenching operations alone.

2.5.4.3.5 Geologic Mapping

The plant excavation has been mapped at a scale of 1 in. = 10 ft. The foundation rock was found to be sound and competent. No features inimical to the safety of the plant were noted. Full discussion and geologic maps of each surface of the foundation are presented in the Foundation Report (Appendix 2.5E). Logs of all borings and trenches are presented in Appendix 2.5A.

2.5.4.4 Geophysical Surveys

The following geophysical surveys were conducted at the site:

a) Surveys to determine seismic wave transmission characteristics of the site (discussed in Section 2.5.2.5).

b) Magnetometer traverses (discussed in Section 2.5.3.2.1).

c) Microearthquake monitoring in the site area (discussed in Section 2.5.2.1.3).

2.5.4.5 Excavation and Backfill

Rough excavation for the Shearon Harris plant structures was completed in 1974 as shown on Figure 2.5.3-2. Excavation activities were resumed in 1977 and continued to completion in 1979.

A topographic map of the ground surface prior to excavation is shown on Figure 2.5.1-15. A plot plan and also a section of the excavation, as well as a plan showing the location of backfilled areas, are shown on figures presented in Appendix 2.5E. Additional geologic maps and sections of the excavation are included in the Foundation Report for the plant presented in Appendix 2.5E. The excavations for the Main Dam, Auxiliary Dam, Auxiliary Reservoir Channel, Emergency Service Water Intake Channel, and Emergency Service Water Discharge Channel are described in Section 2.5.6.

The plant excavation included the foundations for the Waste Processing Building, four Turbine Buildings, a Fuel-Handling Building, four Reactor Auxiliary Buildings, four Containment Buildings, and four Tank Buildings. It had a maximum lateral extent of approximately 925 ft. in the north-south direction and 990 ft. in the east-west direction, and encompassed a total area of approximately 837,500 sq. ft. Floor elevations of the various levels within the excavation ranged from Elevation 234 ft. for the shallowest part to Elevation 179 ft. for the deepest level. The total volume of the excavation was approximately 1,200,000 cubic yards. After cancellation of Units 3 and 4 the area west of the Fuel Handling Building has been backfilled to plant grade. The backfill is supported by a retaining wall to isolate the Fuel Handling Building from the backfill (see Section 2.5.4.5.3). After cancellation of Unit 2, the Turbine, Reactor Auxiliary Building and Containment 2 areas have been backfilled to Elevation 261 ft., Elevation 242 ft. and Elevation 236 ft., respectively. This backfill is supported by the retaining wall to isolate the Fuel Handling Building.

All excavation and backfill were performed in accordance with Ebasco Specification No. CAR-SH-CH-8, which is included in Appendix 2.5I.

2.5.4.5.1 Excavation

Plant excavation began with the leveling of the ground surface in the plant area to Elevation 260 ft. Depressions in the surface were brought up to grade with random fill consisting of local Triassic sedimentary rock material and residual soils. After leveling, unclassified soil materials were excavated to ripper refusal by common excavation using bulldozers and scrapers. The excavation was completed to final grade by controlled blasting in successive lifts and by using power shovels and backhoes to load the broken rock onto trucks for removal up an access ramp cut into the south end of the excavation.

Slopes were excavated through the overburden at 1:1. Slopes in bedrock were excavated at 1:4. Rock slopes were shaped by presplitting in accordance with specifications which did not permit heavy blasting closer than 3 ft. to the rock which forms the final foundation of concrete structures. Presplitting did not completely preserve the rock surface of the slopes because adverse jointing across bedding planes which dipped toward the excavation, as in the west wall

of the excavation, allowed small blocks to slide down bedding planes toward the excavation. This situation was somewhat aggravated during the period from late 1974 to early 1977 when excavation was halted. All loose blocks of rock were removed when excavation work was resumed.

Weathering of the sandstones exposed in the excavation during the period of project delay was negligible. Weathering of the finer-grained rocks, caused by wetting of clays contained in these rocks, did not exceed several inches below excavation levels established in 1974. The coating provided by the uppermost disintegrated rock tended to protect the underlying material from further rapid advance of the weathering process.

However, weathering along natural joints, fractures, and bedding planes as in the fine-grained rocks necessitated some additional excavation of foundation areas to assure adequate intact condition of the rock prior to placement of the concrete seal coat. The excavated rock was replaced by equivalent concrete so that the stability of structures founded on the rock was not affected.

Treatment methods used for foundation protection after excavation included slush grouting, shotcreting, and placement of drain pipes as well as placement of seal coating. Following final excavation, joints were filled with slush grout after which the invert rock was protected with Class F mortar prior to placement of the concrete seal mat over completed foundation areas. In certain instances the foundation mat structural concrete was poured directly against the rock surface after treatment with slush grout and Class F mortar. Also, in Units 2, 3 and 4 (subsequently cancelled) a vertical mat was formed and poured with concrete. Drain pipes were placed through the shotcrete at the locations of seeps to prevent the buildup of excessive pressure.

A geologist was resident at the site throughout the cleanup of the excavation to assure that foundations were prepared as designed and to document foundation conditions by geologic mapping. Following inspection and approval by the geologist, loose materials were removed from excavated surfaces by backhoe and hand tools, and the surface was blown clean with compressed air. A geologic map was then made of the rock surfaces at a scale of 1 in. = 10 ft., as documented in the Final Foundation Report for the plant site. During construction the geologist inspected all foundation surfaces and assured that any defects were treated prior to recommending approval.

2.5.4.5.2 Dewatering

A permanent dewatering system is not utilized for the plant. Groundwater seepage into the excavation was minimal because of the low permeability of the rock. Most of the inflow was due to rainfall, although minor seepage of groundwater did occur along some joints and bedding planes. Drainage was accomplished by the intermittent use of sump pumps. This drainage procedure had no adverse effect on the quality and condition of the foundation.

2.5.4.5.3 Backfill

Materials and placement requirements for backfill around Seismic Category I plant structures are discussed in Ebasco Specification CAR-SH-CH-8 "Excavation, Backfill, Filling, Grading", presented in Appendix 2.5I. The bulk of the backfill consisted of a mixture of silt and clay derived from the local Triassic sedimentary rocks. Selected backfill material was compacted

between structures and rock surfaces to meet requirements of 95 percent standard proctor density with moisture control at ± 4 percent of optimum moisture content and a maximum permeability value of 10 ft./yr.

The fill and placement requirements used to support Seismic Category I pipes and conduits are described in Section 3.7.3.12.

The sources of backfill materials were various in-progress excavations in the plant area. At completion of backfilling (later) cubic yards of fine-grained random fill, (later) cubic yards selected backfill, (later) cubic yards of crushed rock, (later) cubic yards of riprap, and (later) cubic yards of class D concrete had been placed adjacent to Category I plant structures.

These volumes do not include backfill placed during construction of the intake and discharge channels, as discussed in Section 2.5.6, or random backfill placed in large exploration trenches beyond the limits of the plant excavation.

2.5.4.5.3.1 Properties of Backfill Material

2.5.4.5.3.1.1 Grain Size

Grain size distribution (GSD) curves for the backfill material used for the plant and channel areas are presented on Figures 2.5.4-1 through 2.5.4-53.

2.5.4.5.3.1.2 Compaction

Standard proctor compaction tests were performed on the backfill material to determine the optimum moisture content and maximum density required in the placement of the material. The results are presented on Figures 2.5.4-106 through 2.5.4-110.

2.5.4.5.3.1.3 Triaxial Shear

Consolidated undrained triaxial shear tests with pore pressure measurements were performed on selected backfill material. A Mohr-Coulomb failure envelope summarizing the undrained triaxial shear test results is presented on Figure 2.5.4-111. Individual stress-strain curves as well as Mohr's circle plots are presented on Figures 2.5.4-112, 2.5.4-113, and 2.5.4-120 through 2.5.4-129.

2.5.4.6 Groundwater Conditions

2.5.4.6.1 Groundwater Conditions Relative to Stability of Safety-Related Facilities

Pre-excavation boring and piezometric records in the plant island area indicate that piezometric levels range from about 240 to 272 ft. above mean sea level and that they follow the original topography. However, the completed plant block excavation reveals that groundwater occurs in the widely separated joints and fractures in the Triassic rocks and that unfractured rock materials remain dry. The higher piezometric levels represent perched conditions, as indicated by down-hole pressure tests showing several impermeable zones at various depths.

Except for the west face of Fuel Handling Building the perimeter of the plant structure up to the top of the foundation mat is in direct contact with rock which is essentially impermeable. The

portion between the plant structure and rock has been backfilled with residual soil which is of very low permeability (estimated to be less than 10 ft./yr.). Additionally, the winter 1979-1980 piezometric-level map (Figure 2.4.13-2) shows that water levels beneath the plant area are well below 251 ft.

The source of surface water higher than the design basis groundwater level is the Emergency Service Water Intake Channel of the Auxiliary Reservoir, which has an operating pool elevation of 252 ft., the closest point of which comes to within about 300 ft. of the plant island. The Auxiliary Reservoir will not raise the groundwater elevation beneath the plant island above an elevation of 251 ft. for the following reasons.

a) The residual soil underlying the reservoir is of very low permeability, as indicated by testing.

b) After the Auxiliary Reservoir and the Main Reservoir are filled, groundwater will move from these reservoirs to the cones of depressions created by the pumpage from wells. However, the groundwater movement ultimately will be from the Auxiliary Reservoir to the Main Reservoir after construction is completed.

c) Groundwater from the Auxiliary Reservoir will start moving toward the plant island at an elevation of about 252 ft. However, the water level will be at a much lower elevation than 251 ft. by the time it reaches the plant island due to the hydraulic head loss as it flows through the low permeability materials for a distance of about 300 ft.

2.5.4.6.2 Design Criteria for the Control of Groundwater Levels or Collection and Control of Seepage

The subsurface portions of Seismic Category I structures in the plant island are designed for hydrostatic loadings with groundwater at Elevation 251 ft. A permanent dewatering system is not utilized for the Shearon Harris Nuclear Power Plant. Groundwater occurring in widely separated joints in the rock did not significantly affect construction. Any rain or surface water accumulated during construction was pumped out by sump pumps.

The design plant grade is Elevation 260 ft. for the minimum safety factor for load combinations, including the flood buoyant force. The groundwater drainage will be from the Auxiliary Reservoir (Elevation 252 ft.) and the plant site to the Main Reservoir (Elevation 220 ft.); therefore, groundwater levels will tend to remain below the design level.

2.5.4.6.3 Dewatering Requirements

After excavation at the site, seepage from some of the joints and fractures occurred a few days after rains, but the volume was small compared to the volume of surface-water runoff into the excavated areas. Where necessary, water was drained into sumps and pumped out of the excavated areas; continuous pumping was not required. Below the Containment Building mat, a porous concrete network drains into a permanent sump. For the Containment Building, a waterproof membrane was also placed under the mat. Consequently, a permanent dewatering system for lowering groundwater levels is not needed. Dewatering of Unit 2 Containment mat will be included as a portion of the outside area drainage.

2.5.4.6.4 Groundwater Conditions During Construction

Groundwater was encountered in the joints and fractures, but the rest of the rock materials were dry. Minor seepage occurred from some of the joints and fractures for a few days after rains; however, joints and fractures were slush grouted to minimize their ability to serve as water conduits. Isolated areas of continuous water seepage through weep holes in the seal mat concrete occurred in lower elevations. Otherwise, drainage into the excavated areas was limited to surface water which was collected in sumps and pumped out as required.

2.5.4.6.5 Field and Laboratory Permeability Tests

Down-hole water pressure tests were conducted in selected borings and the results are shown in Appendix 2.5A. In the plant site area, these tests were performed at 10 ft. intervals under pressures up to 110 psi at depths ranging from 10 to 145 ft. Several isolated zones registered very small water losses under high-pressure tests, which are recorded in Appendix 2.5A. Table 2.4.13-7 shows permeability values based on test results in these borings.

2.5.4.6.6 Monitoring and Fluctuations of Groundwater Levels

Tables 2.4.13-5 and 2.4.13-6 show water levels in the site wells and site piezometers, respectively, observed at weekly intervals during the winter of 1979-1980. Comparison of Figures 2.4.13-1 and 2.4.13-2 shows the difference between the preconstruction piezometer levels and the winter of 1979-1980 water levels.

2.5.4.6.7 Groundwater Movement

The general direction of groundwater movement in the plant area (Figure 2.4.13-2) is southeast, toward White Oak Creek. Groundwater flow, gradients, and velocities are discussed in Section 2.4.13.

2.5.4.6.8 Potential for Subsidence

Except for the west face of the Fuel Handling Building, the perimeter of the plant structures up to the top of the foundation mat will be in direct contact with the essentially impermeable Triassic rock. The rock surface west of the Fuel Handling Building is protected by structural concrete. The top clay/saprolite layer has been removed at the site; therefore, the potential for subsidence is negligible.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

All Seismic Category I structures within the plant area, except the Class IE electrical manholes and Class IE underground electrical conduits and some Seismic Category I pipes, are founded on sound rock. There will be no amplification in the sound rock. The acceleration at the foundation levels of those structures supported on sound rock is equal to the baserock acceleration.

The maximum horizontal accelerations which will be experienced during a safe shutdown earthquake are 15 percent of gravity. For an operating basis earthquake, the maximum accelerations will be one-half those of the SSE; i.e., 7 1/2 percent of gravity. A further

discussion of the earthquake design basis is contained in Section 2.5.4.9, and a complete discussion is presented in Section 2.5.2.

The soil-structure interaction analyses for the structures founded on bedrock were performed by using the fixed-base approach based on the lumped mass-spring method. A general discussion of the analyses is presented in Section 3.7.2.4.

The engineering properties of the bedrock at the site used for dynamic soil-structure interaction analyses are discussed below:

Poisson's ratio was estimated from the following equation derived from solid mechanics:

$$\mu = \frac{1}{2}\,\frac{\left(\frac{V_p}{V_s}\right)^2 - 2}{\left(\frac{V_p}{V_s}\right)^2 - 1}$$

where V_p and V_s are respectively the compressional and shear wave velocities determined from field geophysical data (Table 2.5.2-3). The average value of Poisson's ratio used in the analyses is 0.35, which is consistent with values reported in the literature (Reference 2.5.4-2).

As a basis with which to compare the dynamic modulus values, the static modulus of deformation was calculated from unconfined compression tests, and is presented in Table 2.5.4-3 and then reduced by a factor dependent on rock quality designation, RQD, (Reference 2.5.4-1). This reduction procedure is necessary when calculating moduli values from laboratory tests on intact cores. Intact cores tested in the laboratory do not take field discontinuities into consideration.

The dynamic modulus was determined from the following equation from solid mechanics:

$$E = V_p^2\,(\rho)\,\frac{(1+\mu)(1-2\mu)}{(1-\mu)}$$

where ρ is the mass-density of the rock.

Discussions of the determinations of the mass density and compressional and shear wave velocities are presented in Sections 2.5.4.2 and 2.5.4.4.

To be conservative, the effect of confining pressure on the modulus was neglected. Since moduli can normally be assumed to vary as a function of applied stress, this effect was examined by comparing results of laboratory velocity measurements. Therefore, the variation of moduli as a function of strain rate was observed. Since static tests for rock exhibit strains of about 10^{-2} to 10^{-3} in/in and field geophysical measurements were conducted at strains of about 10^{-6} in/in, it is possible to determine if any significant modulus variation occurs. The moduli values of the competent rock were shown to be insensitive to changes in strain rate above and below that expected of the SSE.

The formulae used in computing the spring constants for the foundations consisting of rigid rectangular footings or mats resting on elastic half space are presented in Table 2.5.4-8.

The horizontal spring constant, kx, considers the sliding of the foundation on its supporting material. To consider the effect of resistance of rock material surrounding the foundation, a spring constant was used which is analogous to the vertical spring constant, oriented to the horizontal.

The Class IE Electrical Manhole Structures consist essentially of very rigid reinforced concrete boxes fully buried in the surrounding soil. Because of their relatively small sizes, the individual structures are assumed to be single mass points excited by the same accelerations as for the surrounding soil mass. The ground acceleration at the level of individual manholes was determined by an amplification analysis of ground motion through a vertical soil column between the bedrock and the manholes by using the computer program SHAKE developed by the University of California, Berkeley.

The program SHAKE computes the responses associated with vertical propagation of shear waves through the linear viscoelastic system. The program is based on the continuous solution of the wave equation adapted for use with transient motion through the Fast Fourier Transform algorithm. The nonlinearity of the shear modulus and damping is accounted for by the use of equivalent linear soil properties using an iterative procedure to obtain values for modulus and damping compatible with the effective strain in each layer.

The input soil and rock properties were obtained from the field geophysical measurements and the dynamic laboratory testing, and are summarized in Appendix 2.5C. The model is divided into a layer system and each layer is completely defined by its characteristics of shear modulus, critical damping ratio, unit weight and thickness. Both of the horizontal and vertical safe shutdown earthquake motions were input separately at the base of the model (bedrock). Through an iterative process, subsequently the strain compatible solutions were obtained, and the new motions at the top of each layer were computed. Vertical soil column models are shown on Figure 3.7.2-8.

The ground accelerations at the level of manholes obtained through the above analysis were further increased by 50 percent for the equivalent static analysis of each structure.

The accelerations used for design of the manholes, as obtained from the above procedure, are as follows:

a) Horizontal SSE Acceleration: 0.25g

b) Vertical SSE Acceleration: 0.19g

c) Horizontal OBE Acceleration: 0.14g

d) Vertical OBE Acceleration: 0.10g

The response of buried pipelines to dynamic loading is discussed in Section 3.7.3.12, while those of the dams, dike, and channels are discussed in Section 2.5.6.

2.5.4.8 Liquefaction Potential

The foundation of the plant is hard, sound rock, and has no potential for liquefaction.

2.5.4.9 Earthquake Design Basis

The plant site lies in an aseismic area; no earthquakes have been reported within 40 miles of the site. Eight earthquakes of epicentral intensity VII and above have occurred within about 200 miles of the site. The largest of these is the Charleston earthquake of August 31, 1886, of Modified Mercalli Intensity X.

Most earthquakes in the region occur in four major seismic zones (Figure 2.5.2-2); the Southern Appalachian Seismic Zone, the Northern Virginia-Maryland Seismic Zone, the Central Virginia Seismic Zone, and the South Carolina - Georgia Seismic Zone. If the largest earthquake in any one of these zones is assumed to occur at the point in that zone nearest the plant, the largest site intensity that can be calculated, using Bollinger's (Reference 2.5.2-25) attenuation relationship for the Southeast, is 6.71. This results from the Charleston earthquake of 1886, and is larger than the motion calculated for the New Madrid earthquakes of 1811-1812.

To be conservative, we have assumed that the SSE will be an earthquake of intensity VII, and that this earthquake will occur close to the site, although no earthquakes have been recorded within 40 miles of the site. Such an earthquake would result in an acceleration of 0.12 g at the site. To be conservative, a value of 0.15 g has been adopted.

The shock is expected to be of less than 10 seconds duration, producing no more than 5-10 cycles of motion at the maximum predicted level. This level of motion should be similar to the Golden Gate (San Francisco) earthquake of 1957, or the Helena, Montana earthquake of 1935. Since these earthquakes resulted in only 2 or 3 cycles of strong motion, selection of 10 cycles is conservative. Horizontal and vertical response spectra for the SSE, prepared in accordance with Regulatory Guide 1.60 (see Section 1.8) are presented on Figures 2.5.2-12 and 2.5.2-13.

The operating basis earthquake accelerations are considered to be one-half those of the SSE (namely, 0.075g), equivalent to an Intensity VI earthquake near the site. The horizontal and vertical response spectra of the OBE are presented on Figures 2.5.2-14 and 2.5.2-15.

Complete discussion of the earthquake design basis is presented in Section 2.5.2.

2.5.4.10 Static Stability

The geologic conditions underlying the site are discussed in detail in Section 2.5.4.1. The in-situ rock properties are discussed in Sections 2.5.4.2 and 2.5.4.5. Static stability analyses for the dams, dike and channels are discussed in Section 2.5.6. This section discusses static analyses of the plant island structures, including settlement, bearing capacity, and lateral earth pressures.

Major facilities are founded on fresh sound siltstone, sandstone, shale, and claystone of the Sanford Formation. The rock provides adequate support for the units under static conditions.

2.5.4.10.1 Settlement

2.5.4.10.1.1 Methods of Analysis

The average settlement of the foundation under combined static and dynamic loading is found by using the equation derived from Boussinesq for the normal displacement of a semi-infinite

elastic solid under the action of a normal load (Reference 2.5.4-4). The equation can be expressed as:

$$S = \frac{\bar{m}\, P\left(1-\mu^2\right)}{E_E\sqrt{A}}$$

where: S = average settlement

$\bar{m}$ = displacement coefficient dependent on the shape, loaded surface, and distribution of load (Reference 2.5.4-4)

μ = Poisson's ratio

E_E = design static modulus

P = total applied load

A = total area on which load is applied

2.5.4.10.1.2 Selection of Deformation Properties

Two parameters are required to define the in-situ static deformation properties, the elastic modulus and Poisson's ratio. The frequency and nature of geologic discontinuities are significant factors in determining the properties of the rock and their effect can be incorporated in the design equations by use of the reduction factor applied to Young's modulus.

2.5.4.10.1.2.1 Design Static Modulus EE

The representative tangent modulus E_T is selected based on the statistical method by using calculated values from unconfined compression and triaxial tests on selected core specimens (see Section 2.5.4.2 and Table 2.5.4-3).

Using the reduction factor described above, the representative tangent modulus, E_T, is reduced to incorporate the effects of in-situ geologic discontinuities. The design elastic modulus is:

$$E_E = E_T \text{ (RF)}$$

where: E_E = design static modulus

E_T = average value of laboratory tangent modulus

RF = reduction factor based on RQD.

The reduction factor is based on the RQD of the rock, as discussed in Section 2.5.4.2. The average value of the laboratory modulus selected is 1.7 x 10^6 psi. The average RQD is 92.7 percent (Table 2.5.4-2). Based on a conservative RQD of 75 percent, the reduction factor RF is 40 percent (Reference 2.5.4-5). The value of the design elastic modulus obtained is 6.8 x 10^5 psi.

2.5.4.10.1.2.2 Static Poisson's Ratio μ

Values of Poisson's ratio, μ, were obtained from both laboratory compression tests on selected core specimens (discussed in Section 2.5.4.2), and from field geophysical measurements (discussed in Section 2.5.4.4). The average design value of the selected μ is 0.35 based on field geophysical measurements. This value is consistent with the values reported in the literature (Reference 2.5.4-2).

2.5.4.10.1.3 Computed Settlement

The average settlements under static loading computed for the various structures are presented in Table 2.5.4-9.

It can be seen that these settlements are very small. The differential settlements should be even smaller and, therefore, structurally tolerable. Since the settlements consist of the pseudo-elastic compression of the underlying rock, they will occur essentially upon load application.

2.5.4.10.2 Bearing Capacity

2.5.4.10.2.1 Method of Analysis

The ultimate bearing capacities for strip foundations on rock can be obtained by using the following equation from Stagg (Reference 2.5.4-4).

$$q_{ult} = 0.5\bar{\gamma}\ BN\gamma + CN_c + qN_q$$

where: $\bar{\gamma}$ = effective unit weight

B = width of the foundation mat

C = cohesion

q = surcharge pressure

and $N\gamma$, N_C, and N_q are bearing capacity values depending on the friction angle of the foundation rock.

This equation is identical with Terzaghi's bearing capacity equation (Reference 2.5.4-6).

For square foundations, the corresponding bearing capacity equation is:

$$q_{ult} = 0.4\gamma\ BN\gamma + 1.2\ CN_c + qN_q$$

2.5.4.10.2.2 Selection of Strength Properties

The appropriate properties required for the bearing capacity computation are the density and shear strength of the rock.

2.5.4.10.2.2.1 Density

The results of laboratory density tests are described in Section 2.5.4.2. The average density of the rock, as shown in Table 2.5.4-1, is 162.8 pcf. A value of 160 pcf was used for design calculations.

2.5.4.10.2.2.2 Shear Strength

The shear strength of the foundation rock was determined from laboratory unconfined and triaxial tests on selected rock cores. The details of the tests are discussed in Section 2.5.4.2. The results are summarized in Tables 2.5.4-3, 2.5.4-4, and Figure 2.5.4-67. Based on the unconfined compression test data, the average shear strength of the rock is about 4000 psi. To be conservative, this shear strength is reduced by the factor RF described in Section 2.5.4.10.1.2.1 earlier. The value of the shear strength used for bearing capacity analysis is 1600 psi.

From the Mohr's circle plot shown on Figure 2.5.4-67, the average friction angle of the rock is about 45 degrees. For design, however, the friction angle of the foundation rock is conservatively chosen to be equal to zero.

2.5.4.10.2.3 Computed Bearing Capacity

Based on a friction angle of zero, the bearing capacity equation becomes:

$$q_{ult} = 1.2CN_c + q$$

By disregarding the surcharge term q and putting $N_C = 5.14$ (Reference 2.5.4 6),

$$q_{ult} = 6.2C$$

The computed ultimate bearing capacity is 714 tsf.

The design bearing capacity chosen is 25 tsf, which provides a factor of safety of 28 compared with the ultimate bearing capacity (Reference 2.5.4-7).

2.5.4.10.3 Lateral Earth Pressures

The Plant Island Structures were designed for static and dynamic earth pressures.

The following assumptions were implemented for purposes of this analysis:

a) Water level was assumed to be at Elevation 251 ft. (Grade level Elevation 260 ft.).

b) The depth of backfill considered for at-rest pressure calculations extends from grade level (Elevation 260 ft.) to the top of the foundation mat (elevation varies with structures).

c) Pressure due to groundwater was considered only for the depth between Elevation 251 ft. and the top of the foundation mat.

d) The total lateral pressure is equal to the effective earth pressure plus the hydrostatic pressure.

2.5.4.10.3.1 Method of Analysis

2.5.4.10.3.1.1 Static Earth Pressure

The foundation walls of the structures were designed for at-rest earth pressure and hydrostatic loading. The at-rest earth pressure coefficient, K_o, was computed by using the following formula:

$$K = 1 - \sin \varphi$$

where φ is the internal friction angle of the backfill soil. The value of the friction angle is discussed in Section 2.5.4.10.3.2.

It is assumed that full at-rest pressure is developed by backfill, disregarding the possible compensating effect of the rock layer in reducing at-rest pressure against the walls.

2.5.4.10.3.1.2 Dynamic Earth Pressure

A dynamic lateral earth pressure analysis was performed for all seismic Category I structures. The procedures and parameters utilized in this analysis are presented below.

The backfill against the exterior walls of the structures is divided into two levels or regions, as shown on Figure 2.5.4-130.

a) The first level - depth d_1 - is confined by the exterior wall surface on one side and by the soil on the other.

b) The second level - depth d_2 - is confined by the exterior wall surface on one side and by the rock on the other.

For the first level it is assumed that full passive pressure may be realized.

To compute the pressure on the face of the wall (between points A and B) due to an earthquake, it is necessary to compute the movement of the wall as it varies with depth below grade. The design earth pressure is then obtained from the strain vs. pressure coefficient curve shown on Figure 2.5.4-131 using the computed strain.

The strain is computed as the wall movement at a particular depth divided by the length of the horizontal component of the Rankine failure surface at that depth.

The movement of the wall relative to the soil for the first level, d1, is the arithmetic sum of the movement obtained from the dynamic analysis and the maximum soil movements as determined by using the deflection of the 15 percent damping curve of the ground response spectra for the corresponding soil column frequency, as discussed in Section 3.7.2.4A.

For the second level - depth d_2 - it is assumed that confinement of the backfill by the very steep rock face will prevent the formation of a passive pressure wedge.

To compute the state of pressure on the face of the wall (between points B and C) due to an earthquake, the strain at various elevations is computed by dividing the total relative movement of the wall with respect to the rock at that level by the width of the backfill layer - between the concrete wall and the rock - at that level.

The movement of the wall relative to the soil over region d_2 is obtained by the same procedure as that employed for region d_1 (see above).

Once the strain is computed for various levels, it is related to the triaxial test stress-strain curves for the correct overburden pressure used as σ_v for the backfill material to obtain the corresponding pressure for each level. This pressure strain curve is shown on Figure 2.5.4-131 and is modified from data presented in the literature (Reference 2.5.4-2).

2.5.4.10.3.2 Selection of Design Properties

The properties of the backfill used in the computation of the static and dynamic earth pressures are summarized below:

a) Friction angle = 20 degrees

b) Cohension C = 400 psf

c) Dry Density = 115 pcf (silty clay)

d) Saturated Density = 130 pcf

e) Submerged Density = 67.6 pcf

f) Coefficient of at-rest pressure Ko = 1 - Sin φ = 0.7

These backfill properties were based on results of laboratory testing which are presented in Section 2.5.4.5. The values for friction angle and cohesion were based on results of laboratory triaxial tests with pore pressure measurements on samples compacted to a dry density equal to 95 percent of the maximum dry density obtained from the Standard Proctor test. The actual test results are shown on Figures 2.5.4-106 through 2.5.4-113 and 2.5.4-120 through 2.5.4-129.

2.5.4.11 Design Criteria

The design criteria and methods of analysis used in the stability studies of settlement, bearing capacity, and earth pressure are discussed in detail in Section 2.5.4.10.

The settlement analyses were based on the assumption that settlement is due essentially to the pseudo-elastic compression of the rock. The formula used in the computation of settlement is based on the theory of elasticity. In view of the very high safety factor against bearing capacity failure, the assumption of elasticity is valid. The modulus obtained from laboratory compression testing of intact core has been properly adjusted for discontinuities in the rock mass. The results indicated that the total, and hence the differential settlement, should be small.

The bearing capacity of the foundation is computed by using Terzaghi's bearing capacity equation (Reference 2.5.4-6) which is generally conceded to be conservative. For added

safety, the shear strength of the rock, obtained from the laboratory compression test of the rock core, was reduced to account for presence of discontinuities in the rock mass. The factor of safety based on a design bearing capacity of 25 tsf is given in Section 2.5.4.10.2.3; however, the required factor of safety is three. The design bearing capacity of 25 tsf is consistent with what is allowed by most building codes for sound rock. The maximum computed vertical bearing pressure on the foundation rock from any plant structure is 39 ksf which results in a computed factor of safety of 36.

The coefficient of lateral earth pressure used for design under static conditions is the "at-rest" value, which combined with the water table at Elevation 251 ft., results in a safe total lateral pressure condition.

The dynamic lateral pressure is established from structural displacements obtained from the dynamic analysis. The strain is computed by dividing the total relative movement of the wall with respect to the rock at that level by the width of the backfill (i.e., between the wall and the rock at that level). The stress is obtained by utilizing the stress-strain relationship for passive loading conditions reported in the literature (Reference 2.5.4-2). This results in a dynamic lateral earth pressure that is conservative.

2.5.4.12 Techniques to Improve Subsurface Conditions

In preparing the final excavation, all weathered and broken rock was removed, and dust and fine materials were cleaned off the rock surface by using air lances. While this final clean-up was going on, the resident geologist inspected the surface to ensure that no loose blocks were present; any found were removed with picks and bars.

After horizontal surfaces had been cleaned and mapped, joints and fractures were slush grouted until refusal. If leakage of grout occurred at lower elevations, the leaking fracture was normally caulked with rags or oakum and wedges. Where the leakage was from a feature which could not be caulked, several pours of slush grout were made, allowing each to set before continuing with another placement. This process continued until leakage ceased.

Vertical faces were covered with four in. wire mesh suspended from pieces of rebar cemented into the face. In certain instances this procedure proved unnecessary and for economic reasons, foundation mat or structural concrete was placed directly against the rock walls, with no loss of design intent. Rock bolts and rock anchors were generally not required.

2.5.4.13 Subsurface Instrumentation

During construction, close survey control was maintained on a series of monuments on horizontal and vertical faces to ensure there was no movement. Piezometers were installed to monitor groundwater levels during excavation as discussed in Sections 2.5.4.6 and 2.4.13.

2.5.4.14 Construction Notes

LATER

2.5.5 STABILITY OF SLOPES

There are no natural slopes at the project site the failure of which could adversely affect the safety of the nuclear power plant. Stability of slopes of man-made structures, such as the Main Dam, the Auxiliary Dam, the Auxiliary Reservoir Separating Dike, and emergency service water channels is discussed in Section 2.5.6.

2.5.6 EMBANKMENTS AND DAMS

2.5.6.1 General Information

2.5.6.1.1 Purpose

The primary purpose of the dams discussed in this section is to impound water for the circulating water and service water systems for the Shearon Harris Nuclear Power Plant. The Main Dam impounds a reservoir with a normal water level at Elevation 220 ft. and a water surface area of approximately 4,000 acres. During normal operation, the Main Reservoir functions as a storage reservoir and is used as the source of cooling tower makeup water. The Main Reservoir also serves as an alternative source of emergency service water supply or ultimate heat sink.

The Auxiliary Dam impounds a reservoir with a minimum normal water level at Elevation 250 ft. and a surface area of approximately 317 acres. The Auxiliary Reservoir serves the Emergency Service Water System. An Auxiliary Separating Dike across the east arm of this reservoir acts as a barrier to prevent discharged emergency service water from flowing directly back to the emergency service water intake area. The Auxiliary Reservoir Channel conveys discharged emergency service water into the west arm of the reservoir so that maximum cooling can be attained before the discharged water circulates back to the intake area.

The Auxiliary Reservoir must remain operative under the safe shutdown earthquake (SSE) condition. Consequently, the Auxiliary Dam, Auxiliary Separating Dike, Auxiliary Reservoir Channel, and Emergency Service Water Intake and Discharge Channels are Seismic Category I facilities. The Main Dam and Emergency Service Water and Cooling Tower Makeup Intake Channel, because of their importance, have also been classified as a Seismic Category I structure.

2.5.6.1.2 Location

A location map, which shows the plant area, Main Reservoir, Main Dam, Auxiliary Reservoir, Auxiliary Dam, Auxiliary Separating Dike, Auxiliary Reservoir Channel, Emergency Service Water Intake and Discharge Channels, and Cooling Tower Makeup Water Intake Channel, is presented on Figure 2.5.1-10. The location of these facilities is based on a siting study performed by Ebasco Services Incorporated in October, 1970 (Reference 2.5.6-1).

The Main Dam is located on Buckhorn Creek about 4.5 miles south of the plant site and about 2.5 miles north of the Cape Fear River.

The Auxiliary Dam is located across the Tom Jack Creek arm of the Main Reservoir, adjacent to the southwest boundary of the plant site.

The Auxiliary Separating Dike is located about 1700 ft. north of the Auxiliary Dam, between the emergency service water intake area and the emergency service water discharge area. The Auxiliary Reservoir Channel is located northwest of the Auxiliary Separating Dike. The Emergency Service Water Intake and Discharge Channels are located on the plant island southwest and northeast of the plant, respectively. The Cooling Tower Makeup Water Intake Channel is located southeast of the plant.

2.5.6.1.3 General design features

The Main Dam, Auxiliary Dam, Auxiliary Reservoir Separating Dike, Auxiliary Reservoir Channel, Emergency Service Water Intake and Discharge Channels, and Emergency Service Water and Cooling Tower Makeup Intake Channel are designed and constructed to Seismic Category I criteria and also to withstand the effects of credible combinations of natural phenomena. The slopes of the dams, dike, and channels are designed for a factor of safety of 1.5 under static conditions, 1.2 for simultaneous OBE and 100 year return period flood level, and 1.1 for simultaneous SSE and 25 year return period flood level.

The simultaneous OBE and 100-year return period flood level was not analyzed for the ESW and Cooling Tower Makeup Intake Channel since the simultaneous SSE and 25-year return period flood level analysis was more conservative.

2.5.6.1.3.1 Main dam and spillway

The Main Dam is a rockfill dam approximately 1550 ft. long at the crest, Elevation 260 ft. It is founded on rock and has a maximum height of approximately 108 ft. It has a core of compacted silty clay and clayey silt material protected on each side by two 8-ft.-thick fine and coarse filter zones and a rockfill shell. The outside slopes are two horizontal to one vertical. The Main Dam plan is shown on Figure 2.5.6-1, and a profile and section are shown on Figure 2.5.6-2.

The general plan and details of the Main Dam Spillway are shown on Figures 3.8.4-34, 3.8.4-35, and 3.8.4-36.

The basis for the hydraulic design of the spillway is to accommodate the probable maximum flood (PMF). The spillway is uncontrolled and consists of two ogee sections, each 25 ft. wide, separated by a concrete pier, with a crest at Elevation 220 ft.

The average velocity of flow in the spillway's approach channel is approximately 1 ft./sec. for a 100-year return period flood and approximately 8 ft./sec. for the PMF. The channel is cut into rock for a distance of approximately 200 ft. upstream of the ogee crest structure and in soil approximately 105 ft. further upstream. The invert and sides of the channel cut in rock are lined with concrete. The concrete lining has dowels across longitudinal and transverse contraction joints. The side lining and part of the invert lining adjacent to the ogee structure are secured to rock by rock anchors. The transverse contraction joints in the concrete lining are spaced at approximately 40 ft. centers.

The portion of the approach channel which is excavated entirely in soil is designed for an average flow velocity of 4 ft./sec. for the PMF. The invert and sides of the channel are protected by riprap placed on bedding. The riprap is sized to withstand a velocity of 8 ft./sec. to permit the use of the same size riprap for the soil side slopes above the concrete lined portion of the channel.

The discharge channel downstream of the ogee crest structure has a super-critical slope of 0.02 ft. per ft., except for a portion of the channel upstream of the stilling basin which is sloped at two horizontal to one vertical to join the discharge channel to the stilling basin. The average velocity of flow in the channel for the PMF is approximately 45 ft. per second; however, the velocity increases to approximately 72 ft./sec. at the stilling basin. The Froude number of incoming flow at the stilling basin is six. The stilling basin has concrete chute blocks and a dentated end sill.

The spillway's discharge channel and stilling basin are excavated in rock with side slopes of one horizontal to four vertical. The rock in the invert and on the sides is lined with concrete that is secured to bedrock by rock anchors. The top of the lining has a minimum freeboard of 3 ft. above the PMF water surface profile. Transverse contraction joints in the concrete lining are spaced at approximately 40 ft. centers. Transverse drains, consisting of perforated concrete pipe placed in a trench filled with crushed rock, are located at each transverse joint in the invert lining in order to minimize uplift. The transverse drains lead to a longitudinal collector drain which has its outlet in the stilling basin. The side lining has drainage holes drilled into rock in order to relieve water pressure. Drainage holes, provided in the bed lining, are drilled 20 ft. deep into the rock in order to drain the rock and reduce uplift.

The average velocity in the discharge channel downstream of the stilling basin is approximately 6 ft./sec. for the PMF. The discharge channel is cut in rock for a distance of approximately 125 ft. downstream of the concrete end sill of the stilling basin, which precludes undermining of the structure. The sides and invert of the channel, downstream of the rock cut section, are excavated in soil and protected by sacrificial rockfill laid on bedding.

2.5.6.1.3.2 Auxiliary dam and spillway

The Auxiliary Dam is a random rockfill dam approximately 3903 ft. long, a maximum structural height of approximately 72 ft., and a crest at Elevation 260 ft. Its outside slopes are 2.5 horizontal to one vertical. It has a core of compacted silty clay and clayey silt material protected on each side by a transition filter zone and a random rockfill shell. The downstream shell is provided with two horizontal drainage blankets, each 3 ft. thick, which are connected to the transition filter zone adjacent to the core of the dam. In addition, a 200 ft. wide, 3 ft. thick drainage layer is provided under the shell in each of two areas where pre-existing creeks had been located. The core of the dam is founded on weathered rock and the core (cutoff) trench is excavated to suitable rock. The filters and random rockfill shells are founded on weathered rock at the center of the dam and on firm residual soil near the abutments. The Auxiliary Dam plan is shown on Figure 2.5.6-3 and a profile and sections are shown on Figure 2.5.6-4.

The general plan and details of the Auxiliary Dam Spillway are shown on Figures 3.8.4-37 and 3.8.4-38. The spillway is an uncontrolled concrete ogee section with a crest length of 170 ft. and crest at Elevation 252 ft. The basis for its hydraulic design is the probable maximum flood (PMF).

The ogee crest of the spillway is joined to the stilling basin by a sloping apron. The stilling basin is conservatively designed for a tailwater level corresponding to a 100 year drought water level in the Main Reservoir. Since the main reservoir water level varies from a low water level of Elevation 204.4 ft. during drought conditions (coincident with emergency shutdown) to the maximum still water level of Elevation 238.9 ft. during the PMF, the sloping apron allows for proper formation of the hydraulic jump at all tailwater levels. The incoming flow at the stilling

basin has a maximum velocity of approximately 45 ft./sec. and a Froude number of 10. Control of the velocity of water in the stilling basin is provided by concrete chute blocks and baffle piers.

All Auxiliary Dam Spillway structures are founded on rock. Over-excavated areas were filled with concrete. Rock slopes in the excavation for the sloping apron and stilling basin have a slope of approximately one horizontal to four vertical. Rock in the invert and sides of the structures are lined with concrete secured to rock by rock anchors. The lining slabs are joined by dowels across longitudinal and transverse contraction joints. The transverse joints are spaced at approximately 25 ft. centers. Transverse drains, consisting of perforated concrete pipe laid in a trench filled with crushed rock, are located at each transverse joint in the invert to minimize uplift. The drains have outlets in the invert. The side lining has drainage holes drilled into rock to relieve water pressure. Drainage holes drilled into rock to drain the rock and to reduce uplift have been provided through the invert.

The side walls of the stilling basin and sloping apron, which joins the ogee crest to the stilling basin have sufficient height to retain the embankment fill and to confine the hydraulic jump within the side walls for all possible locations of the formation of the jump. The walls have a minimum freeboard of 2 ft. at the end of the jump. The additional height required for the side walls is provided by construction of retaining walls on top of the rock.

A discharge channel is located downstream of the stilling basin. The invert of the channel immediately downstream of the stilling basin is cut into rock for a distance of approximately 300 ft., which precludes undermining of the stilling basin. On the east side of the channel, up to Station 40+00 and on the west side up to Station 20+00, where the sides of the discharge channel are excavated in soil, the sides are protected by riprap placed on bedding below Elevation 225 ft. to preclude side erosion. The riprap is sized to withstand a velocity of 6 ft./sec. The top of the discharge channel excavation is more than 400 ft. from the toe of the Auxiliary Dam at its nearest location.

2.5.6.1.3.3 Auxiliary Reservoir Separating Dike

The Auxiliary Reservoir Separating Dike is approximately 1200 ft. long and has a maximum height of approximately 55 ft. Its outside slopes are 2.5 horizontal to one vertical. The dike has a core of compacted silty clay and clayey silt material protected by a random rockfill shell which is graded near the core with the finer materials adjacent to the core. The core and rockfill shell are founded either on weathered rock or on a thin layer of stiff residual soil overlying weathered rock (see Appendix 2.5E, Figure 9). The upstream and downstream slopes of the Auxiliary Reservoir Separating Dike are protected by riprap as shown on Figure 2.5.6-5. The plan, section, and profile of the Auxiliary Reservoir Separating Dike are shown on Figure 2.5.6-5.

2.5.6.1.3.4 Auxiliary Reservoir Channel

The Auxiliary Reservoir Channel is approximately 1570 ft. long and 140 ft. wide at its invert Elevation of 235 ft. Its sides have a slope of two horizontal to one vertical in soil and one horizontal to four vertical in rock. The plan, profile, and sections are shown on Figure 2.5.6-6. The Auxiliary Reservoir Channel is sized to carry the maximum ultimate discharge of the Service Water System coincident with the PMF flood flow for the upstream drainage basin.

To facilitate construction of the Auxiliary Separating Dike, a diversion channel, 25 ft. wide with an invert at Elevation 225 ft., was excavated in the Auxiliary Reservoir Channel.

2.5.6.1.3.5 Emergency service water channels

The Emergency Service Water Intake and Discharge Channels (Figure 2.5.1-10) are designed to carry the service water flow required for normal and emergency shutdown of SHNPP.

2.5.6.1.3.5.1 Emergency service water intake channel

The Emergency Service Water Intake Channel is approximately 3580 ft. long and 50 ft. wide at its invert elevation of 238 ft. The channel walls have a slope of two horizontal to one vertical in soil and one horizontal to four vertical in rock. Where cut in soil, an impervious lining 50 ft. thick was provided to form the side-slopes of the channel. Areas along the channel invert which contained sandy alluvium material were removed and replaced with an impervious material to a depth where insitu impervious material was encountered. The plan, profile, and sections of the channel are shown on Figure 2.5.6-7.

2.5.6.1.3.5.2 Emergency service water discharge channel.

The Emergency Service Water Discharge Channel is approximately 2170 ft. long. The width of the channel varies from 50 ft. to 80 ft. at the channel's invert Elevation of 240 ft. The channel walls have a slope of two horizontal to one vertical in soil and one horizontal to four vertical in rock. The plan, profile, and sections of the channel are shown on Figure 2.5.6-8.

2.5.6.1.3.5.3 Emergency Service Water and Cooling Tower Makeup Intake Channel.

The Emergency Service Water and Cooling Tower Makeup Intake Channel is approximately 2500 ft. long and 45 ft. wide at its invert elevation of 194.0 ft. The channel walls have a slope of two horizontal to one vertical in soil, one horizontal to four vertical in rock on the north side of the channel, and two horizontal to one vertical in rock on the south side. The plan, profile and sections of the channel are shown on Figure 2.5.6-28.

2.5.6.2 Exploration

A comprehensive series of exploration programs was conducted in the main dam and auxiliary reservoir areas for evaluation of foundation conditions and to locate and sample possible sources of borrow and quarry materials for construction of the dams. An index map of FSAR figures, which show the locations of various exploration activities, is presented on Figure 2.5.1-10.

2.5.6.2.1 Main dam area

The Main Dam area, as discussed in this section, includes the Main Dam, the Main Dam Spillway, and the main dam impervious borrow and quarry areas.

2.5.6.2.1.1 Local geologic features

The site of the Main Dam and Spillway (Figure 2.5.1-4) is approximately 3000 ft. southeast of the Jonesboro Fault; it is underlain by pre-Triassic igneous and metamorphic rocks which have no direct relationship to the Triassic sedimentary rocks which lie on the northwestern side of the fault and underlie the plant site. The main dam quarry area (Appendix 2.5F, Figure 12) is also located in pre-Triassic crystalline rocks. Main dam impervious borrow areas M and W (see

Appendix 2.5F, Figures 9 and 10), are located on the northwestern side of the fault in Triassic sedimentary rocks of the Sanford Formation, the same rock formation that underlies the plant and Auxiliary Dam site.

The lower part of this formation underlies the plant site, whereas the upper conglomeratic part underlies the borrow areas.

All Seismic Category I structures in the main dam area are on the southeastern side of the Jonesboro Fault. Therefore, the following discussion of lithology and structure is limited to the igneous and metamorphic rocks underlying the main dam and spillway area.

2.5.6.2.1.1.1 Lithology

Significant rock exposures in the main dam area are limited to stream beds and railroad cuts. In most places, the rock is covered by residual soil (saprolites) or by alluvium in the valley bottoms. As exposed in the excavations for the main dam core trench, diversion conduit, and spillway, the bedrock is generally hard, strong, and fresh to moderately weathered. Four main types of rock were distinguished for the purpose of foundation mapping. These are (1) granite, (2) hornblende-mica gneiss, (3) mica schist, and (4) quartz feldspar gneiss. The distribution of the rocks types is shown on the geologic maps of the Main Dam (Appendix 2.5E, Figures 3 through 6).

1. Granite - Rocks of granite composition are the most common; they underlie most of the left abutment of the Main Dam, most of the upstream part of the diversion conduit, and a major part of the spillway. These rock are typically hard to very hard, strong, and medium grained. They are composed predominately of feldspar (orthoclase and plagioclase) and quartz, with minor amounts of biotite, chlorite, muscovite, epidote, and pyrite. They are light gray when fresh, creamy white when slightly weathered, and tan or buff when moderately to highly weathered. The granites are commonly highly foliated, although foliation is weak or absent in places. In man places they are interlayered with intricately folded mica schist and/or hornblende-mica gneiss. Xenoliths of hornblende gneiss are common in the granite where foliation is weak or absent. Much of the highly foliated granite appears to contain indistinct compositional layering. The granitic rocks normally weather to light gray, fine silty to coarse sandy saprolite; the upper portion of such saprolite is commonly weathered to tan, slightly clayey sand, silt, or silty sand.

2. Hornblende-Mica Gneiss - Hornblende-mica gneiss is the second most common rock type in the foundations; it underlies much of the right abutment of the dam, the downstream part of the diversion conduit, and most of the downstream part of the spillway. This gneiss is a hard, strong, medium to fine-grained rock composed of hornblende and subordinate amounts of plagioclase and biotite. Chlorite and pyrite are common accessory minerals. The gneiss is gray to black when fresh, bluish or greenish gray when slightly weathered, and rusty brown when moderately weathered. Foliation in the gneiss is poorly to very poorly developed; in places the foliation is so weak that the rock resembles diorite. This rock type normally weathers to a red or tan saprolite composed of micaceous sandy silt; the upper portion of the saprolite is commonly weathered to a deep-red clayey sandy silt or sandy clayey silt.

3. Mica Schist - Mica schist occurs as complexly folded layers 0.01 to 10 ft. thick within the granite and as thin layers along joint surfaces within the hornblende-mica gneiss. The schist is soft to moderately hard, weak to moderately strong, moderately weathered, fine grained, highly fissile, and composed predominately of chlorite and biotite with subordinate muscovite and amphibole. Some schist layers contain abundant quartz. This rock type normally weathers to a very micaceous silty saprolite; the upper portion of the saprolite is commonly weathered to a red micaceous clayey silt.

4. Quartz-Feldspar Gneiss - Quartz-feldspar gneiss occurs in the central section of the main dam cutoff trench, in a small area at the intersection of the cutoff trench and the diversion conduit, and in part of the spillway. This rock is light gray to dark brown, fine to medium grained, moderately hard, moderately strong, and slightly to moderately weathered. It is characterized by close interlayering of gneissic rock composed mostly of quartz and feldspar with schistose rock composed of biotite and muscovite or, less commonly, hornblende and mica. Individual layers are from 0.01 to 10 ft. thick. A micaceous, quartz-rich variety of gneiss containing few schistose layers is present in the spillway area. The higher quartz content of the gneiss distinguishes it from rocks mapped as granite.

Other rock types in the foundations occur mostly as veins and small pods. Quartz veins are common in all rock types, and residual boulders of vein quartz are abundant on the ground surface. Most quartz veins cross-cut the foliation of the host rock, but some are parallel to the foliation. Veins and pods of quartz-feldspar pegmatite are also common in most areas of the foundations. Small veins and veinlets of epidote occur mostly in the hornblende-mica gneiss.

2.5.6.2.1.1.2 Structure

The orientation of the foliation and compositional layering in rocks in the main dam area is highly variable due to the highly complex nature of deformation apparent in these rocks, particularly in the layered quartz-feldspar gneiss, and in the mica schist and hornblende-mica gneiss layers in the granite. The rocks appear to have been affected by several periods of folding, with isoclinal folding predominating. However, no consistent pattern in the orientation of the folds is discernable in the foundation rocks. In most places the foliation is parallel or sub-parallel to the compositional layering. Northeast strike directions and northwest dip directions predominate. Foliation striking north-northwestern and dipping southwest is also present in places, but is much less common.

Joints are most common in the granite and the hornblende-mica gneiss. The dominant joint set strikes approximately N 60°-70° E and dips 50° to 70° to the south. Another set strikes N 20°-35° W and dips 70° to 90° southwest. Joint spacing ranges from a few inches to a few feet.

Several minor, non-capable faults with lengths measured in tens of feet were mapped in the foundation of the Main Dam. Most of the faults have strikes between N30°E and N75°E and dip steeply to the southeast. A few strike northwest and have steep northeast or nearly vertical dips. A majority of the faults exhibit right lateral strike separation, but those showing left lateral separation are also common. Apparent displacement is rarely more than two feet and is commonly less than 6 in. Detailed investigations of the faults, described in Section 2.5.6.2.1.2.2, indicate that any movement along the faults must have occurred prior to or during

deformation-mineralization processes which terminated more than 225 million years ago. Written reports on the faults as presented to the NRC are cataloged in Appendix 2.5E.

2.5.6.2.1.2 Exploration programs

An extensive exploration program was conducted in the vicinity of the Main Dam in order to evaluate foundation conditions for various reservoir-related structures and for highway and railroad relocations, and to explore and sample potential sources of borrow and quarry materials for use as impervious fill and rockfill for the Main Dam. Exploration consisted of a surface geological reconnaissance survey and subsurface investigations including detailed excavation mapping, seismic refraction surveys, borehole drilling and sampling, and excavation of test pits and trenches for soil sampling.

The locations of various preconstruction exploration activities in the vicinity of the Main Dam are shown on Figures 2.5.6-9 and 2.5.6-10. The purpose of the various borehole series shown on the figures is indicated in Appendix 2.5A. Appendix 2.5A includes a tabulation of preconstruction boreholes drilled in the plant and reservoir areas, including some boreholes located in areas not shown on Section 2.5 figures. The boreholes not shown on the figures were drilled for the make-up water system, for highway and railroad relocations, and for proposed structures which were deleted from the final project design (e.g. east auxiliary reservoir structures, skimmer wall, afterbay dam, various channels, dikes, and saddle dams). The following discussion is limited to the exploration program which was related to construction of the Main Dam and Spillway.

2.5.6.2.1.2.1. Geological reconnaissance

A geologic reconnaissance survey of the main dam and spillway area was made in 1972 by Law Engineering Testing Company. The purpose of this reconnaissance was to determine the trends of the geologic structures in the area. Special attention was given to any structural planes of weakness that could affect the stability of the cut slopes for the main dam spillway channel.

The surface reconnaissance consisted of traversing those areas likely to contain rock or saprolite exposures. In addition to the traverses, a series of inspection trenches were excavated with a backhoe near and approximately parallel to the spillway centerline. Detailed observations of the structural characteristics of the bedrock and saprolite were made in the trenches, but detailed trench logs were not prepared for the trenches because they were excavated only for observational purposes and not for sampling and testing.

At each rock or saprolite exposure, the rock type (or saprolite parent rock type) was noted, and the strike and dip of foliation and joint planes were measured with a Brunton compass. The data were recorded for later map plotting and analysis.

The results of this reconnaissance survey is contained in Appendix 2.5G.

2.5.6.2.1.2.2 Excavation Mapping

After initial excavation was completed in the diversion conduit, core trench, and spillway invert and walls, the foundations of the Seismic Category I structures (i.e., areas to be mapped at a

scale of 1 in. to 10 ft.) were cleaned by a backhoe, hand tools, and by compressed air or compressed-air-water jets.

In general, a 50 ft. reach of core trench was cleaned at a time, then a survey crew marked a grid pattern of 10 ft. centers in preparation for mapping by a geologist. Compositional layers, joints, fractures, faults, veins, pegmatites, and other mappable features in the foundation were located with respect to the grid by measuring with a tape. Attitudes of planar features, such as joints and foliation, were measured with a Brunton compass and plotted. Areas that contained faults or other anomalous features, or that required special detail were mapped at a scale of 1 in. to 5 ft.

A summary of foundation features and faults encountered at the site is presented in Table 2.5E-2.

When mapping was completed, a second geologist checked the field map and noted areas of discrepancies, if any, for correction. Final drafts were checked against the original field maps by the person who performed the mapping. The geologic maps are presented in the Geologic Report on Foundation Conditions Power Plant, Dams, and Related Structures (Appendix 2.5E, Figures 3 through 6).

Foundation areas not mapped in plan at a scale of 1 in. to 10 ft. were mapped at a scale of 1 in. to 50 ft. These areas were cleaned with a bulldozer blade in smooth areas and a Gradall in irregular areas. A grid was surveyed on 25 or 50 ft. centers.

In accordance with established CP&L procedures, unusual features and suspected faults encountered in the foundations were promptly reported to the NRC staff along with submission of a tentative evaluation of the capability of the feature. This was followed by an investigation of each feature which was documented by detailed geologic mapping, photography, and, if appropriate, other investigative methods, such as petrographic analysis. None of the features were covered by construction until they had been inspected by the NRC staff and judged by them to be non-capable based on their inspection and their evaluation of the investigation results.

During mapping at the Main Dam, 28 features were reported to the NRC; of these, 22 were determined to be faults. Most of the faults were only a few tens of feet long, with only several inches of displacement. Several schistose zones which pass through the foundation were also reported and investigated. All but one of the reported features were encountered in the right abutment and spillway area, as shown on the geologic maps (Appendix 2.5.E, Figures 3 through 6). Table 2.5E-2 lists locations of the features and dates of the reports. The written reports on the features, as presented to the NRC, are listed in Table 2.5E-3.

Regardless of the type of feature, it was documented that any fault movements occurred prior to or during deformation-mineralization processes which terminated more than 225 million years ago. Therefore, none of the features affect the safety of the Seismic Category I structures, and are not capable faults, as that term is defined in Appendix A to 10 CFR 100.

2.5.6.2.1.2.3 Seismic Refraction Survey

A seismic refraction survey that was conducted in the main dam area included one survey line (Seismic Line No. MD5) along the centerline of the Main Dam and another (Seismic Line MD6)

along the centerline of the spillway. The locations of the lines are shown on Figures 2.5.6-9 and 2.5.6-10; seismic lines MD1 through MD4, also seen on these figures, were established during siting studies and were not used in the design of the Main Dam or Spillway.

The purpose of the survey was to determine the general excavation conditions and depth of rock along the survey lines. A Dresser RS-4A seismograph unit was used in the investigation. An alignment of twelve geophones was used to register the arrival of direct and refracted waves; the wave arrival times were recorded on linagraph paper to provide a semi-permanent record. The twelve geophones used to pick up the direct and refracted waves were placed at approximately 20 ft. intervals over a controlled distance of up to 220 ft. Energy sources consisted of small "Kinepak" explosives detonated 5 ft. from the first geophone and slightly larger explosives detonated 100 ft. from the first geophone. Explosives were detonated at both ends of the alignment to record data in a forward and reverse direction. The long shots were made to detect arrivals of waves refracted from the higher velocity layers at depths that would not normally be detected by the short shots. The forward and reverse shots were required to permit an evaluation of the sloping and uneven boundaries of the subsurface layers of different velocities.

The records obtained in the field were analyzed, and graphs were developed for initial compressive wave arrival time versus the distance from the explosions. Pertinent corrections were applied to adjust the data for sloping or undulating terrain. The corrected data were used to compute the velocity of propagation of the compressive wave in the various layers and the depth to the layer boundaries beneath each geophone. The depths were calculated by a computer and the results plotted on seismic profiles. These profiles are contained in CP&L files, but are not included in the FSAR because the seismic refraction results were superseded by detailed geologic mapping of rock formations in excavations of the main dam cutoff trench and spillway.

2.5.6.2.1.2.4 Borehole Drilling and Sampling

A series of boreholes were drilled in the main dam and spillway area for exploration and sampling of foundation materials for the Seismic Category I structures. The locations of the boreholes, designated as BM1 through BM51, are shown on Figures 2.5.6-9 and 2.5.6-10. Boring and sampling methods were in accordance with ASTM D-1586. A standard 2 in. outside diameter split barrel sampler with an inside diameter of 1 3/8 in. was used to obtain soil samples for soil classification and for laboratory tests. In soils, the standard penetration test was made at every change of strata and within strata at intervals not exceeding 5 ft. Undisturbed soil samples were obtained from cased borings of sufficient size to accommodate either a 3 in. diameter thin wall Shelby tube sampler, a 3 in. diameter piston sampler, or a Denison-type double-tube core barrel. Rock core borings were made through the casings used for soil borings. Core drilling was in accordance with ASTM D-2113. NQ wireline or NX coring was used to recover rock core samples approximately 2 in. in diameter. Rock cores were examined in the field, logged, and stored in standard core boxes for future use and testing. Logs of the foundation borings are presented in Appendix 2.5A.

A series of boreholes, consisting mostly of uncased auger borings, were drilled to define a potential source of borrow materials (Borrow Area M) for use as impervious fill in the Main Dam. Twenty-five-pound bag samples of soil were collected from the borings for laboratory analysis. The locations of the boreholes (BB806 through 809, 811 through 815, 858 through 860, and 862

through 869) are shown on Figure 2.5.6-9. Borehole logs are included in Appendix 2.5A. Results of laboratory analyses are given in Appendix 2.5C.

After construction of the Main Dam began, eighteen additional auger borings were made in Borrow Area M. The locations of the borings and the results of laboratory tests on soils collected from the boreholes are given in Appendix 2.5F, Figure 9.

Six NX wireline borings and 38 air track percussion borings were made immediately north of the Main Dam to outline a quarry site for use as a source of rockfill for the Main Dam. The locations of the boreholes are shown on Appendix 2.5F, Figure 11 (Area "A"). The NX wireline borings were logged and RQD values were determined for the core. The logs of the boreholes are included in Appendix 2.5F.

During construction of the Main Dam, NX core borings were drilled to a depth of 50 ft. on 40-ft. centers located 5 or 10 ft. upstream and/or downstream of the main dam centerline. The holes, which served as primary grout holes, were logged and RQD values were determined for the core in order to verify that foundation conditions were as expected. The locations of the boreholes are shown in Appendix 2.5E, Figure 12. Details of the borings are in Appendix 2.5E.

2.5.6.2.1.2.5 Test pits and trenches

Six test pits were excavated in Main Dam Borrow Aera M during PSAR investigations and an approximately 300=pound representative soil sample was obtained from each test pit for laboratory testing. Each sample contained the proper proportion of the different types of soil observed in the pit. The locations of these pits, designated as TPM1 through TPM6, are shown on Figure 2.5.6-9. Test pit logs are contained in Appendix 2.5A. Laboratory test results are included in Appendix 2.5C.

During construction, twenty-eight test trenches were excavated by backhoe in Borrow Area M for further soil sampling. Twenty-five-pound bag samples of soil, representing a composite mix of the vertical profile of each trench, were collected by hand excavation. The locations of these trenches and the results of laboratory testing of the soil samples are shown on Appendix 2.5F, Figure 9.

Further exploration for borrow materials was required because Borrow Area M was found to be undesirable as a source of impervious fill. Additional test trenches were excavated by backhoe or bulldozer in order to locate an alternative borrow area (Borrow Area W) northeast of Borrow Area M. The locations of the trenches and the results of laboratory tests on trench samples are included in Figure 10 of Appendix 2.5F.

2.5.6.2.2 Auxiliary dam, dike, and channel areas

2.5.6.2.2.1 Local geologic features

The sites of the Auxiliary Dam, Auxiliary Separating Dike, Auxiliary Reservoir Channel, Emergency Service Water Intake and Discharge Channels, Emergency Service Water and Cooling Tower Makeup Channel, and Auxiliary Dam Borrow Area Z are located in the Deep River Triassic Basin; they are underlain by sedimentary rocks of Triassic age. The rocks, like those underlying the plant site, belong to the lower part of the Sanford Formation. However, the

strata underlying the dam and dike sites occupy a lower position in the stratigraphic succession, and consequently are older.

2.5.6.2.2.1.1. Lithology

The sedimentary rocks underlying the auxiliary dam area are characterized by four major lithologic units: medium to coarse-grained sandstone, fine-to medium-grained sandstone, siltstone, and shaley siltstone. These units grade into one another both laterally and vertically, and all intermediate combinations are present.

The medium to coarse-grained sandstone is arkosic and ranges in color from light gray to chocolate brown as the content of silt and clay in the matrix increases. It is conglomeratic in part.

The fine to medium-grained sandstone is typically chocolate brown, with silt and clay present in the matrix. In some places it is light-gray, clean, well sorted, and composed predominantly of quartz.

The siltstone ranges in color from chocolate brown to dark gray or greenish gray. It is commonly mottled and contains pebbly, sandy, shaly, and/or carbonaceous beds.

The shaley siltstone is chocolate to dark brown, green, purple, or gray. It is composed predominantly of silt and clay, although one recurring variety is pebbly.

The sandstones are the most resistant to excavation and slaking and tend to form resistant ridges in the foundation area for the random rockfill zone of the Auxiliary Dam.

The shaley siltstones are the least resistant to slaking and form a characteristic hackly weathered surface after about a week of surface exposure.

2.5.6.2.2.1.2 Structure

The Auxiliary Dam area is located near the eastern margin of the Durham Triassic Basin, which is bounded on the east by the northeast-trending Jonesboro Fault. Smaller normal faults, transverse to the Jonesboro Fault, are common in the basin. The plant site fault, as detailed in Section 2.5.3, crosses the Auxiliary Dam at Station 4+23, striking N 87° E and dipping 65° to 75° southeast. This fault has been demonstrated to be non-capable.

The sedimentary strata exposed in the auxiliary dam area strike N5°-15°E with dips ranging from 9 to 17 degrees southeast. The two dominant joint sets are vertical, one strikes N 40°-50° E and the other N 20°-30° W. A third set trends north-northwest and dips 55° to 70° to the southwest.

2.5.6.2.2.2 Exploration Programs

Most of the exploration programs conducted for the plant site also included some or all of the auxiliary reservoir area. The initial engineering geology survey of the plant site and surrounding area, conducted by Dames and Moore in 1970, included test borings, trench excavations, and a seismic refraction survey along the axis of the Auxiliary Dam. The foundation exploration program for Seismic Category I structures included test borings in the foundations of the

Auxiliary Dam, Auxiliary Dam Spillway, Auxiliary Separating Dike, Auxiliary Reservoir Channel, and Emergency Service Water Intake and Discharge Channels. Test trenches and seismic wave velocity measurements were made in the auxiliary dam foundation. Exploration and sampling for the borrow area testing program included auger borings in Borrow Area X, located between the Auxiliary Dam and the Auxiliary Separating Dike (Figure 2.5.6-12) and auger borings and test pits in Borrow Area Z, located adjacent to the Auxiliary Dam Spillway (Figure 2.5.6-11). The Shearon Harris Site Fault investigation in 1974 (Reference 2.5.1-29) included exploratory trenches and borehole drilling in the Auxiliary Reservoir area a few hundred feet north of the auxiliary dam axis. Further drilling along the auxiliary dam axis and detailed foundation mapping in the Auxiliary Dam and Auxiliary Separating Dike excavations were performed during construction.

2.5.6.2.2.2.1 Excavation mapping

The foundation of the Auxiliary Dam was geologically mapped by using the general procedure described in Section 2.5.6.2.1.2.2. The foundation was excavated to suitable rock. Those areas where suitable rock was exposed in the core trench were hand cleaned and air blown prior to mapping. Mapping in the core trench was performed on a 10 ft. grid tied to the dam centerline at a scale of 1 in. to 10 ft. Mapping in the impervious core area outside the core trench was performed on a scale of 1 in. to 50 ft. on a 25-foot grid. The maps are presented on Appendix 2.5E Figures 7 and 8.

Areas of the auxiliary dam foundation outside the core trench and impervious core area were excavated to weathered rock or firm residual soil with the blade of a D-8 tractor-dozer and sketch mapped on a scale of 1 in. to 50 ft. on a 25-ft. grid tied to the dam centerline (Appendix 2.5E, Figure 8).

The foundation of the Auxiliary Reservoir Separating Dike was excavated to firm residual soil, as required by the specifications, in the shallower parts of the excavation, and to weathered rock in the deeper parts. After excavation, the foundation surface was cleaned by a bulldozer, mapped geologically at a scale of 1 in. to 50 ft. and then photographed. No faults or other unusual features were observed and no foundation treatment was required. A geologic map of the Auxiliary Reservoir Separating Dike foundation is shown on Appendix 2.5E Figure 9.

The Auxiliary Reservoir Channel, Emergency Service Water and Cooling Tower Makeup Intake Channel, and Emergency Service Water Intake and Discharge Channels were excavated to design grade and slope by scrapers and dozers with rippers. Shallow blasting was required in portions of these channels. The channel excavations were inspected by a geologist; geologic mapping of the excavations was not required.

2.5.6.2.2.2.2 Borehole drilling

Borings were drilled to investigate the rock composition, orientation, and quality across the auxiliary dam site for the PSAR. Boring areas included the Auxiliary Dam Spillway, the auxiliary dam axis, and a line that extends several hundred feet east of the east abutment of the dam. The location of the borings, designated as D2 through D19, are shown on Figure 2.5.1-11. Truck and skid-mounted rotary wash, wire-line drilling rigs were employed in the boring program. Drillers used a 3 7/8 in. diameter rotary core barrel to penetrate and sample the thin layer of soils at each test boring location. When rock was encountered, a 1 3/4 in. double-tube core barrel and diamond drilling bit were used to advance the boring and collect continuous

samples of rock. The borings were logged in detail by an engineering geologist; they are shown in Appendix 2.5A. The soils encountered were described in accordance with the Unified Soil Classification System. RQD values, which indicate the ratio of core which was 4 in. or more in length to the full core run, were calculated for each core length and indicated on the boring logs.

Additional borings were drilled for exploration and sampling of the foundations for all Seismic Category I structures in the Auxiliary Reservoir. Drilling and sampling methods were the same as those described in Section 2.5.6.2.1.2.4. Boreholes BX1 through BX16 were drilled in the auxiliary dam foundation; BX40 through BX61 in the auxiliary dam spillway foundation, BD16 through BD20 in the Auxiliary Reservoir Separating Dike foundation, and BC45 through BC48, BC165, and BC166 in the Auxiliary Reservoir Channel. Locations of the borings are shown on Figures 2.5.6-11 and 2.5.6-12. Boreholes BC71 through BC74, BC116, BC17, BC120 through BC122, BC161, BC162, and BC179 through BC190 were drilled for the Emergency Service Water Intake Channel, and BC151 and BC170 through BC178 for the Emergency Service Water Discharge Channel. The locations of these boreholes are shown on Figures 2.5.6-7 and 2.5.6-8. Logs of all of the foundation borings are contained in Appendix 2.5A. Additional boreholes, BC-191 through BC 199, were drilled for the Emergency Service Water and Cooling Tower Makeup Intake Channel, but the cores were not recovered. These boreholes provided rock depth information only since the cores were not recovered.

Thirty-two uncased auger borings (BB155 through BB186) were drilled in auxiliary reservoir Borrow Area Z in order to obtain 25-lb. bag samples of soil for laboratory testing. Four cased borings, BB5 through BB8, were also drilled in Borrow Area Z. The locations of the boreholes are shown on Figure 2.5.6-11; logs of the borings are included in Appendix 2.5A.

Six additional boreholes were drilled during 1974 for test purposes in the Auxiliary Reservoir area as part of the Shearon Harris Site Fault Investigation. The locations of the borings, designated as TB-1-74 through TB-6-74, are shown on Figures 2.5.1-15 and 2.5.1-16. Water pressure tests were conducted in the three borings located near the Auxiliary Dam Spillway (TB-1-74, TB-2-74, and TB-3-74). The other three borings, located on the east side the reservoir between the Auxiliary Dam and the Auxiliary Reservoir Separating Dike, were used for overburden permeability tests. The borings are discussed further in Section 2.5.4.3.2; borehole logs and test results are contained in Reference 2.5.1-29.

Additional NX core borings were drilled to a depth of 50 ft. on 40 ft. centers along the axis of the Auxiliary Dam during construction. These borings, which served as the primary holes for the grout curtain, were logged and RQD values were determined for the cores to verify that foundation conditions were as expected. Locations of the boreholes are shown on Appendix 2.5E Figure 14; borehole logs are maintained in CP&L files.

2.5.6.2.2.2.3 Trenching and test pits

During the PSAR investigation in 1970, two trenches with a combined total length of 4500 ft. were excavated in the auxiliary dam and spillway area in order to evaluate the lithology, quality, and continuity of the rock, and the composition and consistency of the overburden. One trench, approximately 3500 ft. long (identified as Trench No. 3), was excavated along the axis of the Auxiliary Dam. The other trench (identified as Trench No. 4) was excavated across the spillway area. The locations of the trenches are shown on Figure 2.5.1-11. Trenches were generally excavated to refusal to the backhoe equipment that was used over most of the area. Most trenches were 2 to 10 ft. in depth. All excavations were inspected in regular increments to

evaluate overburden and rock conditions, and hand penetrometer tests were performed at intermittent intervals. Trench logs were prepared from the observations and are included in Appendix 2.5A.

Two test trenches were excavated in the foundation for the Auxiliary Dam with a Case 580B backhoe for the purpose of obtaining undisturbed representative block samples of the dam's foundation soils. The location of the trenches (identified as TPA1 and TPA2) are shown on Figure 2.5.6-11. Four (identified as TPA1 and TPA2) are shown on Figure 2.5.6-11. Four undisturbed block samples were recovered from the test trenches; three from TPA1 and one from TPA2. The block samples were cut in-situ to approximately 1-ft. cubes, sealed with wax, and placed in wooden boxes. The trench logs are included in Appendix 2.5A.

During construction of the channels, one excavation in the Emergency Service Water Intake Channel and two excavations in the Auxiliary Reservoir Channel were made to obtain undisturbed block samples of soil in order to further define in-situ residual soil properties. The locations of these excavations and laboratory test results on the samples collected from them are included in Appendix 2.5K.

Twenty-two trenches were excavated in the auxiliary reservoir area in order to trace the westward extension of the Shearon Harris Site Fault, which is located parallel to, and about 700 ft. north of, the auxiliary dam axis. The trenches were oriented approximately north-south and ranged in length from less than 100 ft. up to 700 ft. Trenches were excavated with a Link-Belt LS-4000 crawler hydraulic backhoe with 1 1/4 cubic yard bucket. In general, trench depths ranging from seven to 13 ft. were required to provide a satisfactory definition of fault features. All but three of the trenches provided good to excellent exposures of the fault trace. The fault was not observed in trenches FET-9W, 17W, and 18W, where caving conditions, resulting from a combination of deep alluvium and groundwater, precluded obtaining desired exposures. Trench locations and trench wall sections are shown on Figure 2.5.3-4.

Four test pits were excavated in auxiliary dam Borrow Area Z. An approximately 300-lb. representative soil sample was obtained from each test pit. Each sample contained the proper proportion of the different types of soil observed in the pit. The locations of the pits, designated as TPZ1 through TPZ4 are shown on Figure 2.5.6-11; the test pit logs are shown in Appendix 2.5A.

2.5.6.2.2.2.4 Seismic refraction survey

A seismic refraction survey was conducted to define the topography of the rock surface along the auxiliary dam axis and across the spillway. The location of the survey lines, identified as Seismic Lines Nos. 3A, 3B, and 4, are shown on Figures 2.5.6-11 and 2.5.6-12.

The recording equipment used for the refraction investigation was an Electro-Tech, ER-75-12 transistorized, portable, 12-channel, refraction seismograph. Electro-Tech EV-5-4 geophones having a natural frequency of 14 cycles per second were used. Geophone spacings of 25 and 50 ft. were used. Explosives were detonated in drill holes approximately 10 ft. deep at the ends, center, and beyond the ends of each line.

The results of the seismic refraction survey showing profiles of various strata, are presented on Figure 2.5.2-9. These results show compressional wave velocities in soils and underlying rock. The velocities of compressional wave propagation in the upper soils and underlying rock were

computed from the plotted data. In addition to the interpreted cross-section, the plots of the time-distance data resulting from the survey are presented immediately above the corresponding profile.

The accuracy of the calculated depth to rock is considered to be within 10 percent for the major portion of the survey; however, in the areas where rock is indicated at shallow depths, the precision is probably less.

2.5.6.2.2.2.5 Seismic Wave Velocity Measurements

Seismic wave velocity measurements were made at locations along the axes of the Auxiliary Dam and Auxiliary Reservoir Separating Dike. The locations where measurements were made are shown in Appendix 2.5C. The main purpose of the measurements was to determine compression-wave (P-wave) velocities (V_p), shear-wave (S-wave) velocities (VS), and Rayleigh-wave (R-wave) velocities (VR) of in-situ residual soil; in addition, the seismic wave velocity of the transitional material and the upper portion of the weathered rock were determined. Two methods were used to measure seismic wave velocities; (1) pulse arrival measurements of compression-wave (P-wave) velocity (V_p) and shear-wave (S-wave) velocity (VS) and (2) steady-state vibration measurements of Rayleigh-wave (R-wave) velocities (VR).

2.5.6.2.2.2.5.1 Pulse Arrival Measurements of P-Wave andS-Wave Velocities

Pulse arrival measurements were made by using a Sprengnether VS 1200 seismograph and a three-component geophone; a sledge hammer impact was used as the energy source. An Electro-Tech vertical geophone, located adjacent to the impact station, was used to provide zero time.

Pulse arrivals were recorded for both vertical and horizontal impacts; several records were made at each location to examine the repeatability of each measurement. The three-component geophone recorded the propagated seismic waves in three planes at right angles.

P-wave pulse arrivals were measured by the horizontal component of the geophone that was oriented along the line between the geophone and the impact station. S-wave pulse arrivals were measured by the two components of the geophone oriented perpendicular to the line between the geophone and impact station.

To create maximum S-wave energy, horizontal impacts were oriented perpendicular to the direction of the measurement line; in addition, this minimized the P-wave energy. By reversing the impact direction, the S-wave was reversed; by comparing the two records, which were symmetrical with respect to the time axis, accuracy of the interpretation was increased.

The impact-to-receiver distance was measured to an accuracy of ±0.1 ft; the time of the first pulse arrival was scaled from the records to an accuracy of ±1 millisecond (msec). Velocities were calculated by dividing the impact-to-receiver distance by the time.

At the auxiliary dam locations, P-wave and S-wave velocity measurements were made for impact receiver spacings of (1) 15 ft. and 20 ft. in the residual soil layer; (2) 10 ft., 12.7 ft., 20 ft., and 22 ft. in the transitional material; and (3) 12 ft. and 26 ft. in the weathered sandstone. At the auxiliary reservoir separating dike locations, P-wave and S-wave velocity measurements were made for impact-receiver spacings of 25 ft. for residual soil and 20 ft. for weathered rock.

The results of the compressional wave and shear wave velocity measurements are presented in Appendix 2.5C.

2.5.6.2.2.2.5.2 Steady-state vibration measurements of R-wave velocities

Steady-state vibration measurements were made by using a Heathkit audio generator (1 G-72), a Dyna Kit Mark III preamplifier, and a Goodman vibrator to generate R-waves. The velocities of the R-waves were measured by using two Electrotech EV-17 vertical geophones; their response was observed on a Tectronix R-5030 dual beam oscilloscope.

R-wave velocities were measured by determining the frequency required to create an in-phase response of two geophones spaced at a selected distance. The frequency was varied in increments of one hertz by the audio generator. The geophone response was displayed on the oscilloscope screen, and the in-phase response was determined to an accuracy of ±0.5 hz for frequency ranges of approximately 30 hz to 110 hz, and ±5 hz above frequencies of 110 hz. The distance between geophones was measured to an accuracy of ±0.1 ft.

Results of the Rayleigh wave velocity measurements are presented in Appendix 2.5C.

2.5.6.3 Foundation and Abutment Treatment

Foundations for the Main Dam, Auxiliary Dam, Auxiliary Reservoir Separating Dike, and the four channels (Emergency Service Water Intake and Discharge Channels, Emergency Service Water and Cooling Tower Makeup Channel, and Auxiliary Reservoir Channel) were excavated and treated in accordance with project specifications (Appendix 2.5I). For control of seepage through the foundations of the Main and Auxiliary Dams, a cutoff trench was excavated in the foundation to suitable rock (defined as rock material that cannot be moved on a production basis with a single-tooth ripper of a D-8 tractor or equivalent) and a grout curtain was emplaced for each dam.

After approval of foundation reaches by the NRC, open fractures were filled with slush grout and dental concrete was placed to modify steep slopes.

Grouting in the cutoff trenches at both dams was completed with neat cement in accordance with project specifications. Consolidation holes were drilled to 20 ft. depths on 10 ft. centers. Primary curtain holes were cored and logged to 50 ft. depths on 40 ft. centers. Secondary curtain holes without coring were then splitspaced between the primary holes. All curtain holes were water tested in five stages and then grouted in a minimum of three stages. Check holes were grouted for both consolidation and curtain grouting as required to assure closure. Grout takes were very low at both dams, as anticipated.

Details of the foundation excavation and treatment, including foundation maps, borehole plans and sections, grout takes, mixes, and pressures are given in Appendix 2.5E. Summaries for the Main Dam, Auxiliary Dam, Auxiliary Separating Dike, and the three Seismic Category I channels are given below.

2.5.6.3.1 Main Dam

The Main Dam is founded on granite and gneisses, with interlayered schists, as described in Section 2.5.6.2.1.1. The foundation of the dam was excavated to weathered rock (defined as

material which cannot be removed on a production basis with the blade of a D-8 tractor-dozer) and the cutoff trench (core trench) was excavated to suitable rock, which consists mostly of fresh to slightly weathered crystalline rock. Approximately 300,000 cubic yards of material were excavated to expose the foundation.

After initial excavation, the foundation was cleaned and mapped geologically, as described in Section 2.5.6.2.1.2.2. Foundation geology of the Main Dam is shown on Appendix 2.5E Figures 4, 5, and 6. A number of minor faults and anomalous features, with offsets measured in inches, were noted during mapping. The faults, which are discussed in more detail in Section 2.5.6.2.1.2.2, were determined to be non-capable after suitable study by Ebasco geologists and inspection by NRC staff geologists.

In general, grout takes for both consolidation and curtain holes were very low, averaging 0.02 bags per ft. of hole for consolidation holes and 0.03 bags per ft. of hole for curtain holes. A total of 24,403 linear ft. of holes was drilled, which required a total of 1,424 bags of cement for grouting and backfill. Grout hole locations for the Main Dam are on Appendix 2.5E Figures 11 and 12.

2.5.6.3.2 Auxiliary Dam

The Auxiliary Dam is founded on gently-dipping sedimentary rocks. The foundation of the dam was generally excavated into weathered rock. The impervious core is founded on weathered rock and the cutoff trench (core trench) was excavated to suitable rock. The filters and random rockfill shells are founded on weathered rock at the center of the dam and on firm residual soils near the abutments. Approximately 402,000 cubic yards of material were excavated to expose the foundation of sedimentary rock beneath the dam's shell and to provide for the cutoff trench.

The foundation was cleaned and mapped geologically, as described in Section 2.5.6.2.2.2.1. Foundation geology and foundation elevations for the Auxiliary Dam are provided on Appendix 2.5E Figures 7 and 8. The plant site fault, exposed as anticipated at Station 4+23, was the only unusual feature encountered in the excavation. This fault, which was demonstrated to be non-capable, is discussed in detail in Section 2.5.3. It was treated by a special grout pattern, as detailed in Appendix 2.5E.

Total drilling for consolidation and curtain grouting was 32,630 linear ft., which required a total of 2,652 bags of cement for grouting and backfill. Grout takes were low, averaging only 0.06 bags per ft. of hole for the curtain. Grout hole locations for the Auxiliary Dam are shown on Appendix 2.5E Figures 13 and 14.

2.5.6.3.3. Auxiliary separating dike

The Auxiliary Separating Dike is founded on gently-dipping sedimentary rocks. The geologic map of the foundation for the Auxiliary Separating Dike indicates that most of the excavation extended through firm soil into weathered rock. No faults or other unusual features were noted. Neither cutoff trench nor grouting was required because there is no differential head, as the water level is the same on both sides of the dike. The exposed foundation materials were mapped to verify that they are equal to, or better than, what was assumed for design. Prior to embankment placement, the entire foundation was proof-rolled to assure stability.

2.5.6.3.4 Channels

Each of the four Seismic Category I channels were excavated to design grade and slope. Subsequent foundation inspections by geologists indicated that neither geologic mapping nor special treatment were required. Fill sections in the Emergency Service Water Intake and Discharge Channels were constructed by using impervious fill. Moisture, density, and permeability controls were utilized to ensure the integrity of the fill sections (see Specification CAR-SH-CH-4, Appendix 2.5I).

2.5.6.4 Embankments

2.5.6.4.1 General

The locations of the two Seismic Category I dams, the Seismic Category I Auxiliary Separating Dike, and the four Seismic Category I channels (the Emergency Service Water Intake and Discharge Channels, Emergency Service Water and Cooling Tower Makeup Channel, and the Auxiliary Reservoir Channel) are shown on Figure 2.5.1-10.

2.5.6.4.1.1 Main dam

The Main Dam is approximately 1550 ft. long and has a maximum height of 108 ft. The general plan, typical cross section, and longitudinal profile of the Main Dam are shown on Figures 2.5.6-1 and 2.5.6-2. The Main Dam has a core of compacted silty clay and clayey silt material. The core is protected on each side by 8-ft. thick fine and 8-ft. thick coarse filter zones, and a rockfill shell. The core is founded on suitable rock and the rockfill shell is founded on weathered rock. Riprap slope protection placed on crushed rock bedding is provided on the upstream face, as shown on Figure 2.5.6-2. The downstream face is protected by an oversized rock zone, as shown on Figure 2.5.6-2.

The rock at the foundation of the Main Dam consists of granite, gneisses, and schists, as discussed in Section 2.5.6.2.

2.5.6.4.1.2 Auxiliary dam

The Auxiliary Dam is approximately 3903 ft. long and has a maximum structural height of approximately 72 ft. The general plan, typical cross sections, and longitudinal profile of the Auxiliary Dam are shown on Figures 2.5.6-3 and 2.5.6-4. The Auxiliary Dam has a core of compacted silty clay and clayey silt material protected on each side by a single transition filter zone and random rockfill shell; there are two downstream blanket drains in the shell and two 200 ft. wide, 3 ft. thick drainage layers in areas where preexisting creeks had been located. Riprap slope protection placed on crushed rock bedding is provided, as shown on Figure 2.5.6-4. The core of the dam is founded on weathered rock and the core (cutoff) trench is excavated to suitable rock. The filters and random rockfill shells are founded on weathered rock at the center of the dam and on firm residual soil near the abutments.

The rock at the foundation of the Auxiliary Dam consists of Triassic sandstones and siltstones.

2.5.6.4.1.3 Auxiliary reservoir separating dike

The Auxiliary Reservoir Separating Dike is approximately 1200 ft. long and has a maximum height of approximately 55 ft. A typical cross section, longitudinal profile, excavation plan, and fill plan of the Auxiliary Reservoir Separating Dike are shown on Figure 2.5.6-5. The dike has a core of compacted silty clay and clayey silt which is protected by a random rockfill shell. The rockfill shell is graded with the finer material placed adjacent to the core and the coarser particles placed to the outside. The core and rockfill shell are founded on weathered rock or locally on stiff residual soil overlying weathered rock. Riprap slope protection placed on crushed rock bedding is provided, as shown on Figure 2.5.6-5.

The engineering properties of the rock and residual soil at the foundation of the Auxiliary Reservoir Separating Dike are essentially the same as those at the foundation of the Auxiliary Dam (see Section 2.5.6.2).

2.5.6.4.1.4 Channels

The profiles and typical sections of the Auxiliary Reservoir Channel and Emergency Service Water Channels are shown on Figures 2.5.6-6, 2.5.6-7, 2.5.6-8, and 2.5.6-28.

The Seismic Category I channels were constructed mainly by excavation either into residual soil or into rock. Portions of the slopes of the Emergency Service Water Intake Channel were shaped to grade by backfilling with random fill and/or modified random fill materials. Portions of the slopes of the Emergency Service Water Discharge Channel were shaped to grade by backfilling with random fill material. The side slopes of the channels are two horizontal to one vertical in soil and one horizontal to four vertical in rock.

2.5.6.4.2 Material properties and placement

2.5.6.4.2.1 Fill materials

The fill materials used for construction of the Main Dam, Auxiliary Dam, Auxiliary Reservoir Separating Dike, and Seismic Category I channels are in accordance with established project specifications (Appendix 2.5I).

The criteria for the material used in the core of the dams are that 40 percent of the material passes the No. 200 sieve and the plasticity index of the placed material is greater than ten. In order to provide the specified materials, an extensive borrow investigation was performed, as described in Section 2.5.6-2. The logs for the auger borings in Borrow Area Z are shown in Appendix 2.5A. Grain size analysis, Proctor compaction test results, and triaxial shear test results are shown in Appendix 2.5C. The logs for the test pits and test trenches in the borrow areas are shown in Appendix 2.5A. The grain size analyses and Proctor compaction test results for representative test pits are shown in Appendix 2.5C.

The results of testing at the Main Dam core material from Borrow Area W are shown on Appendix 2.5F Figure 10.

The results of laboratory investigations of the static and dynamic properties of the core material for the Auxiliary Dam and Auxiliary Separating Dike are presented in Appendices 2.5C and 2.5D.

An additional laboratory testing program was conducted to evaluate the use of Material Z in portions of the auxiliary dam core, compacted at a moisture content as high as three percent above the optimum moisture content (see Section 2.5.6.9). This program included static drained and undrained shear strength tests, cyclic shear strength tests, and cyclic properties tests. The results are shown in Appendix 2.5C. These tests were performed on materials compacted to approximately 97 percent Standard Proctor Density at a moisture content of four percent above optimum moisture.

The test results indicated that for the particular locations in the auxiliary dam core, this material could be compacted at a moisture content as high as four percent above optimum moisture and still exhibit the required static and dynamic properties necessary for dam stability.

The test results, therefore, provided justification for increasing the required core moisture content during compaction to three percent above optimum moisture.

2.5.6.4.2.2. Foundation Materials

From samples of representative borings made at the auxiliary dam and spillway sites, grain size analyses have been performed on surface soils, and unconfined compression tests and laboratory determination of compressional wave velocities have been made on rock samples. The tests are listed in Table 2.5B-2 and the results are shown in Table 2.5B-4 and pages 2.5B-25 through 2.5B-41, and 2.5B-66 through 2.5B-80 of Appendix 2.5B.

Selection of the average to most conservative soil samples was based upon determining their properties for use as foundation materials under the dam shell or for use as fill materials. The tested rock samples were selected to obtain average to conservative rock properties of in-situ rock.

From borings made at the main dam site, grain size analyses were performed on the surface soils, and unconfined compression tests and laboratory determination of compressional wave velocities were made on rock samples. The tests are listed in Table 2.5B-1 and the results are shown in Table 2.5B-3 and pages 2.5B-5 through 2.5B-24 and 2.5B-42 through 2.5B-65 of Appendix 2.5B.

Water pressure tests were performed and the results for boring BM-8 through BM-10 and BM-13 through BM-16 are shown in Appendix 2.5A.

The results of the laboratory investigation of the static and dynamic properties of the foundation soil for the Auxiliary Dam and Auxiliary Separating Dike are shown in Appendices 2.5C and 2.5D.

2.5.6.4.3 Placement Requirements

All embankment materials, including those for the dams, dike, and channel slopes, were placed in accordance with project specifications shown in Appendix 2.5I.

Field testing and construction control procedures used to assure that the required soil properties were obtained are included in the specifications for the dams and embankments. Quality control and quality assurance compliance requirements are documented in the plant's excavation and backfill procedure.

Grain size distribution tests before and after compaction were performed to determine breakdown of the rockfill and random rockfill materials. In-place density and permeability tests were conducted after compaction. These tests verified that, in the test fills, the design strength and permeability that were assumed for the rockfill in the analysis provide a sufficient margin of safety. Strength tests were performed by the U. S. Army Corps of Engineers on test fill materials. Compaction procedures, equipment, and techniques established by the in-place test fill program were used during actual dam construction. The results of the programs were submitted to the NRC for review and approval prior to the start of actual dam construction; the results are included in Appendix 2.5H.

The maximum compacted lift thickness for the impervious core was 8 in. and for the transition filter zone materials, it was 16 in. During construction of the filters, compaction tests were taken to assure compliance with the specifications.

The specification allowed a maximum compacted lift thickness of 2 ft. for the rockfill and random rockfill shell materials; the actual maximum lift thickness was 2 ft. as determined by an in-place test fill program.

In-place test fill programs were performed to establish a method of construction that yielded effective and efficient compaction of rockfill and random rockfill materials. To accomplish the objectives, studies of roller passes and lift thicknesses of the random rockfill and rockfill material were performed. The test fills were a minimum of three lifts in height and were constructed by using vibratory rollers having a dynamic force of not less than 40,000 pounds.

2.5.6.4.4 Gradation Requirements and Compaction Criteria

Representative laboratory tests were performed to determine compaction criteria for all engineered backfill.

The material in the impervious core of the Main Dam was compacted to 97 percent Standard Proctor density at plus or minus 2 percent of optimum moisture content. The lifts of the impervious core of the Main Dam shown in Appendix 2.5F Figure 14 were compacted to 100 percent Standard Proctor at a moisture content between optimum and plus 4 percent. The impervious core of the Auxiliary Dam below Elevation 225 ft. was compacted to 97 percent Standard Proctor density at plus or minus two percent of optimum moisture content. The impervious core of the Auxiliary Dam above Elevation 225 ft. was compacted to 97 percent Standard Proctor density at plus three to minus one percent of optimum moisture content. The Auxiliary Separating Dike central core was compacted to 97 percent Standard Proctor density, below Elevation 220 ft. and 100 percent Standard Proctor density above Elevation 220 ft. at plus or minus two percent of optimum moisture content. The results of the tests for the Main Dam, Auxiliary Dam, and Auxiliary Separating Dike are discussed in Section 2.5.6.4.2.1.

Compaction criteria for filter materials were determined on the basis of static and dynamic analyses and their ability to perform satisfactorily as a filter. Static and dynamic analyses indicated that the relative density of filter materials should be equal to or greater than a certain value, as discussed in the following paragraphs, to assure adequate stability under static and dynamic loading conditions. Over-compaction of filter materials was avoided.

In dams with wide sloping cores (Main and Auxiliary Dams), the transition filters do not form a structural member to the same extent as in dams with narrow cores, since the transition filters

are held in place by the weight of the rock or random rock shells. The criteria for compacting filter layers have been reported by Newmark (Reference 2.5.6-2) and Terzaghi and Lacroix (Reference 2.5.6-3).

On the basis of the dynamic analysis presented in Appendix 2.5D, average relative density criteria were determined for the filters. The fine and coarse filters for the Main Dam were specified to be compacted to an average relative density of 75 percent, except for the upstream coarse filter above Elevation 220 ft. which was specified to be compacted to an average relative density of 80 percent. The filters for the Auxiliary Dam were specified to be compacted to an average relative density of 75 percent below Elevation 220 ft. and to an average relative density of 80 percent above Elevation 220 ft. For both dams, a five percent increase in the specified average densities was permitted during construction. The actual test results were analyzed statistically.

The criteria was determined on the basis that the filters would be constructed by using satisfactory and uniform materials and methods, that construction would be continuously inspected, and that an adequate number of relative density tests, in accordance with the provision of ASTM D2049, "Relative Density of Cohesionless Soils", would be made. For relative density determination, the minimum density was determined in accordance with ASTM D2049. Maximum density was determined either by test fills that utilized actual construction equipment and placement procedures, or in accordance with ASTM D2049, whichever gave the higher density.

In order to assure the design intent of the dynamic analysis and to establish a practical and workable construction criteria, the test results requirement was that only ten percent of the test densities were allowed to fall below the specified requirement, with no single value below 90 percent of the specified requirement (i.e., 67 percent for 75 percent specified; 72 percent for 80 percent specified) and that no filter (excluding blanket drains) had more than five percent of the test densities between 90 and 95 percent relative density. Therefore, the actual relative density values achieved in the construction of the dams exceeded the specified average relative density values. This procedure enabled the attainment of the densities necessary to ensure adequate embankment strength and filter flexibility while minimizing settlement. Thus, the filter performs its function of stability during dynamic conditions, and also as a transition filter should any healing of cracking in the core be necessary.

The filters used for the dams consist of processed, very well graded, coarse, cohesionless mixtures of hard, dense, durable rock materials. In the Main Dam, the fine filter materials are less than 1/2 in. in size, with a maximum of 13 percent passing the No. 200 sieve; the coarse filter materials are less than 6 in. in size, with a maximum of 21 percent passing the No. 4 sieve. In the Auxiliary Dam, the transition filter materials are less than 3 in. in size, with a maximum of 15 percent passing the No. 200 sieve. Gradation criteria over the full range of particle sizes are provided in the Project specifications (Appendix 2.5I) for all of the filter zones.

The gradation criteria for rockfill used in the Main Dam were that a minimum of 75 percent of the material ranged in size from 1/4 in. to 22 in. The maximum rock size did not exceed a 22 in. intermediate dimension and was not greater than 90 percent of the lift thickness. The gradation criteria for random rockfill used in the Auxiliary Dam and Auxiliary Reservoir Separating Dike were that a minimum of 75 percent of the material ranged in size from No. 10 Sieve to 22 in. The maximum rock size was 90 percent of the lift thickness that was determined by the test fill programs.

The stability analyses of the Seismic Category I dams and dike presented in Appendix 2.5D have been made by using the expected constructed values of the static and dynamic properties of the locally available materials with reasonable variations in the properties. Breakdown of the rockfill material caused by compaction, especially of the random rockfill material for the Auxiliary Dam and Auxiliary Reservoir Separating Dike, was taken into account when both the static and dynamic properties were selected. The selection of the strength parameters were based on laboratory test results and experience with rockfill and random rockfill materials, as well as on literature values of similar gradations of materials. Literature (References 2.5.6-4, 2.5.6-5, 2.5.6-6, and 2.5.6-7) on similar materials within the specified gradations show that the selected strength parameters are conservative and justifiable without deleting the minus 2 in. materials.

The random rockfill material used in the Auxiliary Dam and Auxiliary Reservoir Separating Dike consists of sedimentary rocks which were expected to break down during handling and compaction. The low strength properties utilized for these materials considered the breakdown characteristics. In addition, three horizontal drainage blankets, each 3 ft. thick, are provided in the downstream shell of the Auxiliary Dam. The blankets are connected to the transition filter zone adjacent to the core of the dam. The lowest blanket drains into the existing creek. The upper two blanket drains connect to the downstream riprap bedding material (see Figures 2.5.6-3 and 2.5.6-4). The transition filter blankets provide collection and positive drainage of internal embankment seepage.

As previously stated in Section 2.5.6.4.3, the maximum compacted lift thickness for the rockfill and random rockfill shell materials was determined on the basis of in-place test fill programs. The test fill sections were constructed by using vibratory rollers having a dynamic force of not less than 40,000 pounds. The maximum compacted lift thickness of the shell materials was 2 ft. Tests were performed on the test fill sections to demonstrate that the specified gradation for rockfill was achieved. The test results were submitted to the NRC for their review and approval prior to the start of construction of the dams. The triaxial shear strengths of the rockfill and random rockfill materials were measured by the U. S. Army Corps of Engineers using the results of tests on test fill compaction materials. Large diameter test specimens (15 in.) were used. The rockfill and random rockfill both exhibited shear strength parameters of: friction angle (φ) = 40 degrees and cohesion, (C) = 0 psf.

The rockfill and random rockfill surfaces of the dam and dike structures are protected by a layer of riprap in the areas of wave action. The riprap is sized to withstand the wave forces on each structure. The riprap provided to protect the rockfill and random rockfill is bedded on crushed rock. Bedding materials are placed beneath the riprap and are graded to prevent movement of the bedding materials. Riprap and bedding requirements are defined by project specifications (Appendix 2.5I).

2.5.6.4.5 Slope Protection

In the surface areas other than where riprap is used (the wave action zones), rocks ranging up to the dimensions of Class A riprap were placed.

The crest of the Main Dam is at Elevation 260 ft. The maximum wave runup in the Main Reservoir is discussed in Section 2.4.5. The upstream face of the dam is protected from the dynamic wave forces resulting from the maximum wave runup and wind setup level by a 4 ft. thickness of riprap. The downstream face of the Main Dam is protected by a layer of oversized rock, as indicated on Figure 2.5.6-2.

The crest of the Auxiliary Dam is at Elevation 260 ft. Both faces of the Auxiliary Dam are protected against the most severe wave action by a four ft. thickness of riprap.

The crest of the Auxiliary Separating Dike is at Elevation 255 ft.; both faces of the dike from Elevation 235 to the crest are protected by a 4 ft. thickness of riprap.

The design basis of wave protection is that recommended by Karl F. Taylor in his paper "Slope Protection on Earth and Rockfill Dams," presented in the 11th International Congress on Large Dams (Reference 2.5.6-14). The average size of riprap is based on the following formula:

$$0.388\, W_{50}^{\frac{3}{8}}\ (b \cot \alpha)^{3/5} = \frac{H}{\left(\tanh \frac{2\pi d}{L}\right)^{a}}$$

where: W_{50} = Average stone weight (lbs.)

H = Wave height (ft.) (Table 2.4.5-1)

L = Wave length (ft.)

d = Depth of water at the toe of slope (ft.)

α = Angle of slope from the horizontal

$$\left.\begin{aligned} a &= 0.20 \\ b &= 0.75 \end{aligned}\right\}\ \text{empirical factors related to } \cot \alpha$$

The gradation criteria of the riprap material limits the weight of the maximum size rock to about 4 W_{50} (W_{50} is the average stone weight) and the weight of the minimum size rock to 1/4 W_{50}. The material consists of hard and dense sandstone, conglomerate, or granitic rock fragments. The service life was evaluated by conducting sodium sulfate tests to assist in the determination of rock weathering or rock deterioration potential.

The riprap is founded on a bedding of crushed rock graded to prevent movement of the bedding material into or through the riprap. The bedding material gradation is based upon the filter criteria given in Reference 2.5.6-8.

Specifications were used to assure that the riprap was carefully placed so that the rocks form an interlocked rough surface. Inspection during construction ensured compliance with the above specifications. The length to width ratio of the riprap is controlled by the specifications. The specified maximum length to width ratio is 3.3 to 1. For each type of riprap used, the maximum size, minimum size, and average size were specified, as well as a required percentage being within a size range.

The Auxiliary Dam was designed to withstand the static and dynamic water pressure forces resulting from the maximum wave runup and wind setup on one face of the dam coincident with a wave trough on the opposite face.

The Main Dam was designed to withstand the static and dynamic water pressures resulting from the maximum wave runup and wind setup on the upstream face of the dam.

The Auxiliary Separating Dike was designed to withstand the static and dynamic water pressure forces resulting from the maximum wave runup and wind setup.

The channels were designed to withstand the static and dynamic water pressures resulting from the maximum wave runup and wind setup. Any local erosion of the side slopes of the channel due to wave action will not reduce the capability of the channel to sufficiently perform its function.

The embankment of the plant island along the Main Reservoir is protected by sacrificial spoil fill, as shown on Figure 2.4.1-2. Further discussion is presented in Section 2.4.5.

2.5.6.5 Slope Stability

2.5.6.5.1 General

The slope stability of the SHNPP earth and rockfill Main Dam, Auxiliary Dam, Auxiliary Reservoir Separating Dike, and Channels was evaluated by using the results of field exploration, laboratory testing, and analytical study.

The Main and Auxiliary Dams are zoned embankment dams constructed of three materials. The impervious core consists of compacted silty clay and clayey silt, the filters are composed of compacted granular materials, and the dam shells consist of compacted rockfill or random rockfill. Typical main dam and auxiliary dam cross sections are illustrated on Figures 2.5.6-2 and 2.5.6-4, respectively. The Auxiliary Reservoir Separating Dike consists of an impervious core with a random rockfill shell. Figure 2.5.6.5 shows a typical cross section of the Auxiliary Reservoir Separating Dike.

The Main and Auxiliary Dams, the Auxiliary Separating Dike, the Auxiliary Reservoir Channel, the Emergency Service Water Intake and Discharge Channels, and Emergency Service Water and Cooling Tower Makeup Channel are designed as Seismic Category I structures. The side slopes are designed to provide adequate factors of safety under static and dynamic loadings.

The minimum factor of safety against slope stability failure of the dams under static conditions is 1.5. The dams are also designed to a factor of safety of 1.2 for simultaneous OBE and 100 year return period flood, and to a factor of safety of 1.1 for simultaneous SSE and 25 year return period flood. In addition to the slip circle analysis, a two dimensional finite element model was used to evaluate the stability of the dams under dynamic loading conditions.

The seismic design of the dams accounts for hydrodynamic pressures due to horizontal earthquake loads by increasing the hydrostatic loads as described by Zangar and Haefeli (Reference 2.5.6-9).

Profiles and typical sections of the channels are shown on Figures 2.5.6-6, 2.5.6-7, 2.5.6-8, and 2.5.6-28. The Auxiliary Reservoir Channel, Emergency Service Water Intake and Discharge Channels, and Emergency Service Water and Cooling Tower Makeup Channel were constructed by excavating into hard residual soil or bedrock. However, portions of the slopes of the Emergency Service Water Intake Channel were shaped to grade by backfilling with random fill and modified random fill materials. The Emergency Service Water Discharge Channel was predominantly cut through rock. The earthen slopes were designed to a minimum factor of safety of 1.5 for static loads and 1.1 for dynamic SSE loading, at any water level.

2.5.6.5.2 Field exploration

Field exploration of the foundation conditions at the locations of the Main Dam, Auxiliary Dam, Auxiliary Separating Dike, and Category I channels is described in Section 2.5.6.2.

2.5.6.5.3 Laboratory Testing

The laboratory testing program included index property and compaction tests of individual and composite bulk samples of the impervious core materials for the Main Dam, Auxiliary Dam, and Auxiliary Separating Dike. Based on the results of these tests, representative composite bulk samples from the borrow area for the Main Dam and representative composite bulk samples from the borrow area for the Auxiliary Dam and Auxiliary Separating Dike were prepared.

Reconstituted specimens from each composite bulk sample were prepared at the density and moisture content equal to the field placement density and moisture content. The following tests were made on the reconstituted samples: static triaxial, cyclic torsion, and cyclic triaxial. In addition, index property and cyclic triaxial tests were performed on undisturbed samples of the foundation soils from the auxiliary dam and auxiliary separating dike areas.

The results of laboratory investigation of the static and dynamic properties of the core materials for the Main and Auxiliary Dams and Auxiliary Separating Dike and the foundation soils for the Auxiliary Dam and Auxiliary Separating Dike are presented in Appendix 2.5D. Supplementary test results for the material from Borrow Area Z, used as core material for the Auxiliary Dam, are discussed in Appendix 2.5C. Results of tests on material from Borrow Area W, used as core material for the Main Dam, are shown in Appendix 2.5F.

Static consolidated drained, consolidated undrained, and unconsolidated undrained triaxial tests provide values of the static strength characteristics. The cyclic torsion tests provide values of modulus and damping at very low levels of strain. Cyclic triaxial tests, made at low and intermediate levels of strain (by using controlled strain testing procedures), yield additional data on modulus and damping. Cyclic triaxial tests were made at various levels of strain (by using controlled stress testing procedures) to obtain cyclic strength characteristics, and modulus and damping values at the strain levels.

Based on the results of cyclic torsion and triaxial tests, curves of modulus versus strain and damping versus strain were constructed. Curves of the most probable upper and lower bound values were also established. This is schematically illustrated on Figure 2.5.6-13.

Cyclic triaxial tests were performed on compacted silty clay and clayey silt specimens of dams and dike core materials. These controlled stress cyclic triaxial tests were conducted for an appropriate range of initial effective confining pressure, σ_{3c}, and for three initial effective principal stress ratios K_c. The results of the tests were utilized to establish the shear stress required to cause five percent strain (in five and ten cycles) as a function of the normal effective stress. The duration of the safe shutdown earthquake (SSE) determined the appropriate number of cycles, (i.e., five and ten). Typical cyclic strength characteristics are illustrated schematically on Figure 2.5.6-14.

Similar tests were also conducted on selected undisturbed samples of the foundation soil at the location of the Auxiliary Dam and Auxiliary Reservoir Separating Dike. The modulus, damping values, and cyclic strength characteristics of the filter material and the rockfill were based on

available data for similar material, as discussed in Appendix 2.5D. Appropriate variations of the values were incorporated in the analyses.

Static rockfill properties that were used in the analyses for the dams and dike described in Appendix 2.5D were based on data published in the literature. The values of these static properties were proved to be reasonable, based on the results of subsequent laboratory testing, as described in Appendix 2.5H.

The laboratory testing program for the foundation materials and linings of the channels consists of classification, grain size analysis, and Proctor compaction tests on the disturbed residual soil samples from auger borings, and triaxial shear test on the recompacted samples. A similar compaction effort was exerted on samples of modified random fill.

To further define in-situ residual soil properties, undisturbed block samples, approximately one cubic foot in volume, were obtained from the Emergency Service Water Intake and Auxiliary Reservoir Channels. A series of unconsolidated undrained, consolidated drained, and consolidated undrained triaxial shear tests were performed to develop drained and undrained shear strength parameters.

Additional bag samples of channel lining material were tested for triaxial shear strength at moisture contents in excess of those specified for field placement. In this manner, very conservative shear strength parameters were obtained for stability analysis.

The test results are presented in Appendix 2.5K.

Based on the laboratory tests, the residual soil in the channel areas was classified to be fine to medium sandy clayey silt.

A detailed study of soil shear strength and standard penetration blow counts of the standard penetration tests from the boring logs was performed by using correlation proposed by Peck, Hansen, and Thornburn (Reference 2.5.6-10). Figure 2.5.6-15 shows the blow counts of the standard penetration tests versus depth in the in-situ residual soil; blow counts and soil shear strength increase sharply with depth. The properties of in-situ residual soil in the foundation of channels are similar to those in the foundation of the Auxiliary Dam and Auxiliary Reservoir Separating Dike, as described in Appendix 2.5D.

2.5.6.5.4 Analysis Procedures

The stability of the Seismic Category I dams and dike was determined by the slip circle method and the finite element method. In establishing the stability of the Auxiliary Dam, the sliding wedge method was also used to evaluate the stability of the dam. The stability of Seismic Category I channels was determined by using the slip circle method.

2.5.6.5.4.1 Static and Pseudo-Static Stability Analysis of Dams, Dike and Channels

The static and pseudo-static stability of the dams, dike, and channel slopes was determined by a computer program that utilized the simplified Bishop slip circle method. The computer program is outlined in the ICES-LEASE-1 program user's manual (Reference 2.5.6-11).

LEASE-1 is a sub-system of ICES designed to perform stability analysis of slopes by the method of slices. The failure surfaces are assumed to be circular arcs (Reference 2.5.6-12). The computer program locates the radius that has a minimum factor of safety at each of a specified set of trial centers. Depending on the conditions to be modeled, either drained or undrained strength parameters were used.

LEASE-1 has the ability to include the effects of seismic forces through seismic commands. Horizontal and vertical seismic coefficients are applied and the factors of safety are evaluated under seismic loads. The factor of safety is defined as the ratio of the moment of the available shearing forces on the trial surface to the net moment of the driving forces.

The basic assumptions of this method of analysis are:

1. The soil behaves as a Mohr-Coulomb material.

2. The factors of safety of the cohesive component of strength and the frictional component of strength are equal.

3. The factor of safety is the same for all slices.

The factors considered in the analyses include:

1. Properties of soil on the failure surface at the base of the slice, including unit weight, cohesion, and angle of internal friction.

2. Reservoir water levels and piezometric data.

3. The inclination of the failure surface at the bottom of the slice.

4. The dynamic acceleration due to an earthquake as input as an additional static load in the pseudo-static analysis.

Sliding wedge analyses were made for the Auxiliary Dam in order to verify the sliding stability in the abutment areas where a thin horizontal layer of material with low strength exists within the weathered rock. The layer of low strength material was assumed to be in the most vulnerable location with respect to sliding.

The sliding wedge stability analysis involves an active soil wedge being mobilized against a neutral horizontal block and a passive resisting wedge. The factor of safety is calculated as the ratio of the sum of the resisting forces in the horizontal direction to the sum of the driving forces in the horizontal direction. This procedure is outlined in Navy Design Manual DM-7 (Reference 2.5.6-13).

During the initial plant design, the static and pseudo-static stability of the dams, dikes and channel slopes was determined by a computer program, ICES-LEASE 1, which utilized the simplified Bishop Slip Circle Method. STABL 5M was selected as the computer program to reanalyze the Emergency Service Water and Cooling Tower Makeup Intake Channel (Reference 2.5.6-15). STABL 5M determines the factor of safety against slope instability by the method of slices (simplified Bishop-Circular Shaped Failure Surface) which is the identical method for ICES-LEASE 1, and STABL 5M utilizes the simplified JANBU method (applicable to

General Shape Failure Surfaces) and the Spencer Method (applicable to any type failure surface). This program has been verified to comply with QA standards per the qualification evaluation (Reference 2.5.6-15).

2.5.6.5.4.2 Finite element dynamic analysis

An analytical study was made of representative sections of the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike for conditions representative of the safe shutdown earthquake (SSE) selected for the site. Each section was modeled by an appropriate finite element system (see Figures 2.5.6-16).

A full discussion of the dynamic finite element analyses, including the methodology, material properties, and the results are included in Appendix 2.5D. A brief synopsis of the dynamic finite element analysis that was performed prior to the start of construction is provided in the following paragraphs:

As a starting point for the dynamic analysis, the static stresses within the dams were calculated by using currently available static finite element analysis procedures. Non-linear material properties were used in the calculations.

The dynamic response of the dam-foundation system was computed by the finite element method in which the continuum is idealized by elements in plane strain and all components of the stress tensor are incorporated. The strength of the dam and foundation materials was determined on the basis of triaxial tests by imposing on specimens cyclic stresses which simulate those induced by an earthquake. The tests provided cyclic strength parameters representative of field conditions. Therefore, any possibility of the development of tensile stresses was accounted for by the method of analysis and the method of determination of cyclic strength parameters, i.e., the tensile stresses were not neglected.

The dynamic stresses induced within the dam based on the SSE were calculated by using the dynamic finite element computer program Quad IV. The modulus and damping values for each element were selected on the basis of the strain that would be induced in the element during the applied earthquake motion. The time history of earthquake motion that was applied to the base of the dam was appropriate to provide a satisfactory estimate of the expected frequency characteristics of the SSE.

The static finite element analyses that were performed provided value of initial normal effective stresses at any location throughout the dam. The values were then used in conjunction with data similar to those shown on Figure 2.5.6-14 in order to determine the cyclic shear stresses that are required to cause five percent strain at any location within the dam. The dynamic finite element analyses provided values of shear stresses that could be induced during earthquake motions at any location throughout the dam. The effect of vertical ground motions was also evaluated. The safety of the dam was then evaluated by comparing the shear stresses that could be induced during earthquake motions with the shear stresses that are required to cause five percent strain. The latter stresses exceeded the induced stresses by an appropriate margin; therefore the dam will have an adequate safety factor against failure during the SSE.

A typical evaluation of the failure potential along a selected plane is illustrated on Figure 2.5.6-17. As shown in the upper part of the figure, the plane represents average conditions within a finite zone of the dam. The middle part of the figure shows the distribution of the shear stresses

induced during the earthquake, (ti)N, and the shear stresses required to cause five percent strain (tf)N. The ratio (tf)N (ti)N is presented in the lower part of Figure 2.5.6-17. This ratio, which may be considered to represent a factor of safety, was required to have a minimum value of 1.1. A value of 1.1 is judged appropriate because an adequate degree of conservatism was used in interpreting the test data and in selecting the material properties for the analyses. In addition, appropriate variations of the material properties (eg. modulus and damping values, as illustrated on Figure 2.5.6-13) were incorporated in the analyses.

Some changes in design and material properties were found to be necessary during construction. The effects of these changes on the results of the dynamic finite element analyses are discussed in Appendix 2.5F.

2.5.6.5.5 Material properties

2.5.6.5.5.1 Main dam

The material properties that were used in the slip circle analyses for the main dam impervious core, transition filters, and rockfill shell are:

MAIN DAM
MATERIALS PROPERTIES

MATERIAL	WEIGHT (pcf)			STRENGTH	
	MOIST	SAT.	φ	TAN φ	C
Impervious Core	137	142	30	0.5773	200
Fine Filter	130	135	35	0.7002	0
Coarse Filter	135	140	40	0.8391	0
Rockfill	130	145	40	0.8391	0

The values listed above were determined as described in Section 2.5D.10, 2.5D.12, and 2.5D.13 of Appendix 2.5D, and Appendix 2.5F.

2.5.6.5.5.2 Auxiliary dam and auxiliary reservoir separating dike.

The material properties used in the slip circle analyses for the auxiliary dam impervious core, transition filter, and random rockfill shell and for the auxiliary reservoir separating dike impervious core and random rockfill shell are shown in the table below. The shear strengths of the random rockfill were conservatively taken to be the same as those of the impervious core.

AUXILIARY DAM AND DIKE
MATERIAL PROPERTIES

MATERIAL	WEIGHT (PCF)			STRENGTH		
	MOIST	SAT.	φ	TANφ	C	C'
Impervious Core	128	135	30	0.5773	100	300
Filter	135	140	37	0.7535	0	0
Random Rockfill	130	138	30	0.5773	0	300
Residual Soil	128	134	30	0.5773	150	150

The static (C) and dynamic (C') strength parameters were derived from tests described in Sections 2.5D.11, 2.5D.12, and 2.5D.13 of Appendix 2.5D, and Section 2.5.6.4.2.1.

The cohesion component of shear strength, C', was established by dynamic testing and is utilized with the friction angle, φ, to develop the dynamic strength of the materials during the pseudo-static analyses.

For the sliding wedge analyses of the Auxiliary Dam, the shear strength of the horizontal seams of low strength material within the rock in the abutment areas is shown on Figure 2.5.6-18.

2.5.6.5.5.3 Channels

The material properties used in the slip circle analysis for the Auxiliary Reservoir Channel and Emergency Service Water Intake and Discharge Channels are:

CHANNELS

MATERIAL PROPERTIES

MATERIAL	END OF CONSTRUCTION ANALYSIS			STATIC LONG TERM ANALYSIS			PSEUDO-STATIC ANALYSIS			RAPID DRAWDOWN ANALYSIS		
	γ pcf	psf	Ø deg	γ pcf	psf	Ø deg	γ pcf	psf	Ø deg	γ pcf	psf	Ø deg
Modified Random fill (Aux, ESW Channels)	135	1470	11.5	140	120	25	140	660	17	140	660	17
Random fill (Aux, ESW Channels)	135	1470	11.5	140	120	25	140	660	17	140	660	17
Residual+ Soil (ESW Channels)	137	420	34	138	410	29	138	385	27	138	385	27
Residual+ Soil (Aux Channel)	149	420	34	149	410	29	149	385	27	149	385	27
(Cooling* Tower and ESW Water Channel)												
Residual Soil	138	385	20	138	0	36	138	385	20	138	385	26
Rock Bedding Plane		Not Applicable			Not Applicable		165	250	19		Not Applicable	
Steep Rock Slope		Not Applicable			Not Applicable		165	5000	0		Not Applicable	

*Note: These values are for the most part conservatively extrapolated from the values of the other two channels, etc.

+The values above are based on the results of laboratory tests presented in Appendix 2.5K.

2.5.6.5.6 Results of slope stability analyses.

2.5.6.5.6.1 Static and pseudo-static evaluation of the main dam, auxiliary dam, and auxiliary separating dike.

The computed safety factors are shown on Figures 2.5.6-19, 2.5.6-20, 2.5.6-21, and 2.5.6-22. The tabulations show the range of values that were obtained for the Main Dam, Auxiliary Dam, and Auxiliary Separating Dike under static and pseudo-static loading conditions. In addition, the static factors of safety for the Auxiliary Dam, based on the sliding wedge method of analysis, are shown on Figure 2.5.6-18. The static safety factors were calculated as described above with no

consideration of seismic forces. The pseudo-static safety factors were calculated by utilizing the SSE and OBE base accelerations applied directly to the individual slip circles. The pseudo-static analysis for the OBE utilizes the static material properties in order to be consistent with conventional approaches which use earthquakes zone seismic coefficients and static properties. The pseudo-static analysis for the SSE utilizes the dynamic material properties (described in Section 2.5.6.5.5) that are realistic and compatible with actual seismic considerations.

The minimum factors of safety with and without earthquake loading are shown in Tables 2.5.6-1, 2.5.6-2, and 2.5.6-3.

The results of the static and pseudo-static stability analyses demonstrate that the slopes of the Seismic Category I reservoir structures have an adequate factor of safety under all postulated design conditions.

The analysis was performed for the main reservoir water level at Elevation 250 ft. Because dynamic effects are maximized on submerged slopes, analysis of embankment slopes for lower water levels yields higher safety factors. The higher safety factors result from the fact that imposed dynamic forces remain relatively constant, irrespective of water levels, while lower water levels maximize effective stresses and thus the shear resistance. Therefore, with the Main Reservoir at Elevation 220 ft, the safety factors will be greater than those listed on Figures 2.5.6-19, 2.5.6-20, and 2.5.6-21. The results of the analyses presented for the Auxiliary Reservoir Separating Dike on Figure 2.5.6-22 are not changed.

2.5.6.5.6.2 Dynamic finite element analysis of dams

Results of the dynamic finite element analyses are presented and discussed in Appendix 2.5D. The results indicate that adequate safety margins exist in the Main Dam, the Auxiliary Dam and the Auxiliary Reservoir Separating Dike during the SSE and OBE.

The analyses presented in Appendix 2.5D for the Main Dam and the Auxiliary Dam are for a water level at Elevation 250 ft. in the Main Reservoir. The operating water level in the Main Reservoir is Elevation 220 ft. Results of analyses presented in Appendix 2.5D indicate that the Main and Auxiliary Dams are stable and would maintain their integrity if the SSE occurs when the water level in the Main Reservoir is at Elevation 220 ft.

2.5.6.5.6.3 Stability of channel slopes

2.5.6.5.6.3.1 Emergency Service Water and Cooling Tower Makeup Channel, Emergency Service Water Intake and Discharge Channel, and Auxiliary Reservoir Channel.

Typical cross sections of the channels are shown on Figures 2.5.6-6, 2.5.6-7, 2.5.6-8, and 2.5.6-28. Four loading conditions were investigated; long term static, pseudo-static (earthquake), end of construction, and rapid drawdown. The pseudo-static analysis incorporated horizontal and vertical seismic coefficients of .15g and .10g, respectively. Water levels were varied to provide the most conservative factors of safety under seismic loading conditions.

The rapid drawdown analysis modeled an extreme condition whereby channel water levels dropped from extreme high water level to the channel floor. Except for the Emergency Service Water and Cooling Tower Makeup Channel, whereby channel water levels dropped from

extreme high level to water elevation of 204.4 feet. Utilizing a water drop from extreme high water to channel floor was considered to be conservative and unrealistic (Reference 2.5.6-15).

The strength parameters that were utilized in the long term static analysis of the emergency service water intake and discharge channel fill sections were obtained from triaxial tests on samples compacted at moisture contents four percent above optimum. Placement requirements limited field moistures to within plus or minus two percent of optimum. Hence, the strength parameters and the resulting factors of safety are very conservative.

The Auxiliary Reservoir Channel was cut entirely through hard residual soil or rock. Therefore, there were no fill sections. The Emergency Intake and Discharge Channels consisted of cut and fill sections. Both cut sections and fill sections were analyzed. Three representative cross sections conservatively modeled all channel slopes.

All channel slopes are stable under static and seismic loading. Figures 2.5.6-23, 2.5.6-24, and 2.5.6-25 present the results of the stability analyses. Stability analyses results for the Emergency Service Water and Cooling Tower Makeup Channel Slopes are contained in the qualification evaluation (Reference 2.5.6-15).

2.5.6.6 Seepage Control

2.5.6.6.1 General

Seepage control for the Main and Auxiliary Dams is provided by impervious cores and by grout curtains in the foundation rock. Graded filters are placed as protection on each side of the impervious cores.

2.5.6.6.2 Control of seepage through the dams

The criteria for the materials used in the core of the dams are that 40 percent of the material passes the No. 200 sieve and the plasticity of the placed core material is greater than 10. The very low permeability (10-8 cm/sec) of the compacted core material (see Appendix 2.5D) effectively reduces the seepage through the dams to a negligible amount.

As indicated on Figure 2.5.6-2, the transition filter zones for the Main Dam, which are comprised of fine and coarse filter zones, are founded upon weathered rock on the upstream side of the core of the dam. On the downstream side of the core, only the coarse filter is founded on weathered rock. An additional single transition filter zone, graded from the maximum size of crushed rock in the coarse filter to the minimum size of crushed rock in the fine filter, is extended to suitable rock under the fine filter zone. Wherever the single transition filter comes into contact with the downstream main dam cutoff trench face, the face is treated with concrete, as described in project specifications (Appendix 2.5I). The fill concrete treatment acts as an additional seal on the downstream cutoff trench face, as well as aiding in compaction of the filter against the rock face.

The transition filter zone and concrete on the downstream side of the main dam cutoff trench was added to preclude the possibility of piping of core materials into the weathered rock, since jointing in the hard granitic rock at the Main Dam may be developed. The piping potential in the foundation is even more remote when the grouting program described in Section 2.5.6.3 is considered.

The transition filter zone for the Auxiliary Dam, both on the upstream and downstream sides of the core, is founded on weathered rock, as indicated by the geologic mapping shown in Appendix 2.5F and by Figure 2.5.6-4. Thus the transition filter zones are founded on weathered rock on the basis of evaluation and analysis of the rock in combination with the extensive grouting program. In addition, the width of the impervious core in each of the dams effectively precludes the potential for piping in the core and foundation contact by reducing the gradient.

There are no rapid drawdown design conditions for the Main Dam or Auxiliary Separating Dike. The Auxiliary Dam may have a rapid drawdown condition on the downstream side induced by a drop in the Main Reservoir PMF water level at Elevation 238.9 ft. to Elevation 209 ft. if a hypothetical mechanistic failure of the Seismic Category I Main Dam should occur. As indicated in Appendix 2.5D, the stability analysis of the Auxiliary Dam was performed for a rapid drawdown from Elevation 250 ft. to Elevation 209 ft. The gradations of specified materials provide adequate drainage in the shell sections of the embankments. The shell materials were placed during construction such that the finer rock and random rock materials were placed near the filters and graded to the larger materials toward the outside; this assures the shortest possible drainage paths from the finer shell materials to the filter transition materials or to the more coarse rockfill materials. The rockfill material used in the Main Dam consists of granitic rocks which did not break down excessively during handling and compaction, and therefore, provides a positive drainage in the shell sections for internal embankment seepage.

2.5.6.6.3 Control of seepage through foundations and abutments

Seepage through the foundations and abutments at the Main and Auxiliary Dams is controlled by extending the impervious silty clay and clayey silt core in the cutoff trench excavated through the weathered rock to the top of sound rock, and by installation of grout curtains. At the Auxiliary Separating Dike, no cutoff trench or grouting is required since water level is the same on both sides of the dike.

Details of excavation of the cutoff trenches and backfilling with compacted impervious core material are discussed in Appendix 2.5F.

Details of installation, as well as the depth and lateral extent of the grout curtains, are presented in Section 2.5.6.3. Details, including water-test results and grout takes, are included in the final foundation report (Appendix 2.5E).

Piezometers in the Main and Auxiliary Dam and seepage measurement devices in the Main Dam were installed at locations shown on Figures 2.5.6-1 and 2.5.6-3 to monitor seepage through the dam foundations. Seepage is expected to be minimal on the basis of the very low water-test values and the extremely low grout takes, which averaged only 0.02 to 0.06 gabs per foot of curtain hole drilled. Additional information on piezometer installation and pore pressure monitoring is provided in Section 2.5.6.8.

2.5.6.7 Diversion and Closure

2.5.6.7.1 Main dam diversion and closure

A Diversion structure was provided in the Main Dam to pass the 10 year return period flood. The details of the main dam diversion system are indicated on Figure 2.5.6-26.

The diversion system consisted of corrugated metal pipe with cast-in-place reinforced concrete encasement under the entire main dam section and reinforced concrete intake and discharge structures. Details of the intake and discharge structures are shown on Figure 2.5.6-27.

The reinforced concrete diversion structure for the Main Dam was formed by using twin 10-ft. diameter galvanized corrugated metal pipes as an inner form for the reinforced concrete encasement structure.

The material for the portion of the metal corrugated pipes encased in concrete under the dam for the diversion structure is in accordance with Specification AASHO M167, "Structural Plate for Pipe, Pipe Arches, and Arches." The corrugated metal pipe extending upstream and downstream of the Main Dam complies with Specification AASHO M36 "Zinc-Coated (Galvanized) Corrugated Iron or Steel Culverts and Underdrains."

The galvanized corrugated metal pipe remained in place after construction. Since the central portion of the diversion structure under the core of the dam was completely filled with concrete to plug the pipe, there will be no flow of water through the conduit. Due to the interlocking corrugation of the plug and the concrete encasement structure, the possibility of the plug movement is minimized. The other sections of pipe are also galvanized and corrosion will be minimal. However, corrosion will not affect the integrity of the dam nor the diversion structure.

The reinforced concrete diversion structure for the Main Dam was founded upon suitable rock, was designed to Seismic Category I standards, and is capable of withstanding the effects of the most severe phenomena associated with the site. The diversion structure was constructed with construction joints as indicated on Figure 2.5.6-26. At the vertical construction joints in the reinforced concrete diversion structure, under the dam core, a 3/8 in. thick PVC waterstop was provided. At each horizontal construction joint in the reinforced concrete diversion structure under the dam core, a minimum of 3/16 in. thick mild steel waterstop was installed.

The conduits were plugged by a single concrete plug in each conduit, approximately 100 ft. long, in the central area of the dam; the plug extends the full width of the impervious core as indicated on Figure 2.5.6-26. Each plug was designed to withstand the total hydraulic head across the dam.

The concrete plug was sealed by a pressure grouting system that was installed along the length of the plug to assure that the pipe was completely filled.

The corrugated pipe acts as a key to transfer the plug loading to the reinforced concrete diversion structure.

The reinforced concrete diversion structure has two vertical to one horizontal sloping sides to minimize post construction differential settlement. The material used for backfill against the diversion structure is the same as that used in the core and filter zones. An envelope of pervious fill comprised of a 3 ft. thick layer of compacted, well graded crushed rock was provided around the diversion structure in the rockfill zone downstream of the inclined embankment filters to control seepage and to avoid concentrated loading on the structure. An envelope of a 3 ft. thick layer of compacted well graded crushed rock around the diversion structure was provided in the rockfill zone upstream of the inclined embankment filters to avoid concentrated loading on the structure. The compaction criteria for the fill around the diversion structure in the filter zones and for the crushed rock in the rockfill zone was the same as that

used for the filter zone in the embankment. The backfill material that was placed against the diversion structure was placed by using hand compaction methods when necessary. Heavy hauling equipment was not permitted to cross the conduit until the backfill reached a height of 3 ft. above the top of the structure. Above this point, machine compaction methods were used.

During preliminary preparation of the main dam site, a diversion channel was constructed immediately northeast of the existing stream bed, and the stream flow was diverted through it. Cofferdams were then constructed adjacent to the diverted creek and tied into the west abutment to protect the lower elevation work areas from periodic flooding. Next, the area underlying the conduit was mapped and the foundation was prepared. Construction of the concrete-encased conduit followed. After the conduits outside the limits of the dam were placed, upstream and downstream cofferdams were constructed across the valley. The original cofferdam was then removed and the water began flowing through the conduits. Mapping and foundation preparation along the remainder of the core trench was completed, followed by the construction of the dam. Upon completion of the dam, concrete plugs were placed in the conduit and the reservoir began to fill.

2.5.6.7.2 Auxiliary Dam Diversion

No diversion conduit was installed for the Auxiliary Dam. Diversion of the stream during the construction of the dam was performed by construction methods which ensured that the dam was constructed safely, efficiently, and met the design specification.

An upstream cofferdam was constructed at Elevation 225 ft. Water level was maintained at an acceptable level by pumping from behind the upstream cofferdam to Tom Jack Creek downstream of the core trench. When all mapping, grouting and cleaning were complete in the area between approximately Station 28+0 and 31+0, impervious backfill was placed in this portion of the core trench from the upstream cofferdam to a point downstream of the dam. The diversion channel was constructed in this backfill to approximately Elevation 220 ft. Portions of the dam on both sides of this diversion were completed to approximately Elevation 235 ft. At this time, water level behind the upstream cofferdam was controlled by pumping while the required portions of the diversion embankment are removed (impervious material in areas required to be filter material or random rockfill). The dam embankment in the area previously occupied by the diversion was then constructed to Elevation 235 ft. Construction of the dam continued with water level maintained by pumping and dam embankment.

2.5.6.8 Performance Monitoring

Instruments for monitoring the performance of the dams were installed in accordance with project specifications.

The program for periodic monitoring of instrumentation and periodic inspection of the embankments is also contained in the project specifications.

The Main Dam has settlement monuments, piezometers, and seepage monitors as shown on Figures 2.5.6-1 and 2.5.6-2. The Auxiliary Dam has settlement monitors and piezometers, as shown on Figure 2.5.6-3. The Auxiliary Separating Dike has settlement monuments, as shown on Figure 2.5.6-5.

The retaining wall including the deadmen west of the Fuel Handling Building have settlement and/or lateral movement markers as shown on Figure 3.8.4-43. The representative specimens of the retaining wall tie rods are buried in the soil backfill as shown on Figure 3.8.1-44. The frequency of monitoring settlement and lateral movement of the retaining wall is shown on Figure 3.8.4-42. The retrieval and inspection frequency of the tie rods is also shown on Figure 3.8.4-42.

2.5.6.9 Construction Notes

Several changes in construction details became necessary during construction. These changes include:

a) Borrow Area M originally proposed as a borrow source for the impervious core of the Main Dam, proved to be unsuitable since it did not meet gradation and plasticity requirements. An alternative source near the Main Dam, designated Borrow Area W, was used for the impervious core material.

b) Field changes were made in the construction of the diversion and closure system for the Auxiliary Dam (see Section 2.5.6.7).

c) Due to the deviations in moisture content requirements, portions of the original impervious core backfill of the Auxiliary Dam and the Auxiliary Separating Dike were removed and replaced with new compacted backfill.

d) The impervious liner of the emergency service water intake channel fill sections, originally placed in accordance with performance specification, was removed and replaced with new compacted backfill in accordance with specified moisture and density control requirements.

2.5.6.10 Operational Notes

This section will be provided after the reservoirs have become operational.

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APPENDIX 2.5A

Preconstruction Exploration

1. TABULATIONS OF BOREHOLE AND TEST PIT LOCATIONS

Note: The locations of all known preconstruction project borings are tabulated in this section. Boreholes drilled for Seismic Category I structures (including cancelled Units 2, 3, & 4 structures) or to sample borrow for such structures are indicated by an asterisk (*). Borehole logs for these borings are included in Section 2 of this appendix. Logs of other borings are maintained in CP&L files.

1a. P Series - Preliminary Subsurface Investigation - Plant Vicinity

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*P-1	686,303	2,011,540	150
*P-2	685,641	2,011,847	150
*P-3	685,943	2,012,509	150
*P-4	684,274	2,011,345	150
*P-5	684,706	2,012,280	151
*P-6	685,013	2,012,942	250
*P-7	685,433	2,013,849	150
*P-8	For locations see Figure 2.5.1-11	47	
*P-9	For locations see Figure 2.5.1-11	64	
*P-10	For locations see Figure 2.5.1-11	66	
*P-11	For locations see Figure 2.5.1-11	65	
*P-12	For locations see Figure 2.5.1-11	63	
*P-13	For locations see Figure 2.5.1-11	65	
*P-14	For locations see Figure 2.5.1-11	63	
*P-15	For locations see Figure 2.5.1-11	63	
*P-16B	For locations see Figure 2.5.1-11	60	
*P-17	For locations see Figure 2.5.1-11	152	
*P-18	For locations see Figure 2.5.1-11	62	
*P-19	For locations see Figure 2.5.1-11	53	
*P-20	For locations see Figure 2.5.1-11	60	
*P-21	For locations see Figure 2.5.1-11	48	
*P-43	684,929	2,012,761	154
*P-46	685,110	2,012,676	150
*P-101	685,000	2,013,389	136
*P-102	685,084	2,013,571	150
*P-103	685,168	2,013,752	103
*P-104	685,252	2,013,933	150
*P-105	685,336	2,014,115	150
*P-106	685,181	2,013,305	150
*P-107	685,265	2,013,486	250
*P-108	685,349	2,013,668	150
*P-109	685,517	2,014,030	150
*P-110	685,362	2,013,221	150
*P-111	685,447	2,013,402	152
*P-112	685,531	2,013,584	106
*P-113W	685,614	2,013,765	150

*P-114	685,699	2,013,946	150

1b. D Series - Preliminary Subsurface Investigation - Plant and Auxiliary Dam Vicinity

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*D-2	684,438	2,006,386	150
*D-3	684,116	2,006,767	150
*D-4	683,859	2,007,075	150
*D-5	683,859	2,007,575	140
*D-6	683,859	2,008,075	135
*D-7	683,859	2,008,575	150
*D-8	683,859	2,009,075	150
*D-9	683,859	2,009,575	150
*D-10	683,859	2,010,075	150
*D-11	For locations see Figure 2.5.1-11	20	
*D-12	For locations see Figure 2.5.1-11	65	
*D-13	For locations see Figure 2.5.1-11	65	
*D-14	For locations see Figure 2.5.1-11	66	
*D-15	For locations see Figure 2.5.1-11	66	
*D-16	For locations see Figure 2.5.1-11	60	
*D-17	For locations see Figure 2.5.1-11	60	
*D-18	For locations see Figure 2.5.1-11	60	
*D-19	For locations see Figure 2.5.1-11	24	
*D-20	683,559	2,008,075	113
*D-21	683,559	2,008,575	147
*D-22	683,559	2,009,075	136

1c. BP Series - Plant Design Exploration

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*BP-1	685,695	2,012,362	102
*BP-2	685,724	2,012,424	103
*BP-3	685,598	2,012,402	99
*BP-4	685,640	2,012,476	100
*BP-5	685,510	2,012,450	95
*BP-6	685,544	2,012,515	97
*BP-7	685,465	2,012,581	92
*BP-8	685,498	2,012,647	94
*BP-9	685,332	2,012,535	90
*BP-10	685,355	2,012,600	92
*BP-11	685,248	2,012,577	92
*BP-12	685,282	2,012,651	93
*BP-13	685,163	2,012,628	95
*BP-14	685,188	2,012,679	94
*BP-15	685,079	2,012,684	97
*BP-16	685,091	2,012,720	104

*BP-17	685,977	2,012,976	83
*BP-18	686,006	2,013,038	77
*BP-19	685,893	2,013,018	89
*BP-20	685,921	2,013,081	81
*BP-21	685,793	2,013,055	94
*BP-22	685,832	2,013,129	89

1c. BP Series - Plant Design Exploration (Cont'd)

	Coordinates		
Borehole Number (feet)	North	East	Depth (feet)
*BP-23	685,710	2,013,099	96
*BP-24	685,747	2,013,179	94
*BP-25	685,614	2,013,142	97
*BP-26	685,648	2,013,210	94
*BP-27	685,526	2,013,189	93
*BP-28	685,566	2,013,236	91
*BP-29	685,440	2,013,224	90
*BP-30	685,484	2,013,274	85
*BP-31	685,334	2,013,249	87
*BP-32	685,368	2,013,358	83
*BP-33	685,196	2,012,954	125
*BP-34	685,238	2,013,052	124
*BP-35	685,676	2,012,538	123
*BP-36	685,575	2,012,591	123
*BP-37	685,487	2,012,625	120
*BP-38	685,395	2,012,662	120
*BP-39	685,311	2,012,713	119
*BP-40	685,283	2,012,721	119
*BP-41	685,862	2,012,945	118
*BP-42	685,767	2,012,990	124
*BP-43	685,674	2,013,031	124
*BP-44	685,587	2,013,075	124
*BP-45	685,479	2,013,121	121
*BP-46	685,419	2,013,164	121
*BP-47	685,518	2,012,694	118
*BP-48	685,544	2,012,759	116
*BP-49	685,428	2,012,735	120
*BP-50	685,459	2,012,798	120
*BP-51	685,612	2,012,896	122
*BP-52	685,645	2,012,963	125
*BP-53	685,521	2,012,938	130
*BP-54	685,544	2,013,012	124
*BP-55	685,720	2,012,763	109
*BP-56	685,621	2,012,811	117
*BP-57	685,537	2,012,850	120
*BP-58	685,420	2,012,851	123
*BP-59	685,285	2,012,904	123
*BP-60	685,333	2,012,993	125
*BP-61	685,674	2,012,675	134
*BP-62	685,616	2,012,650	139
*BP-63	685,665	2,012,734	130
*BP-64	685,339	2,012,780	141
*BP-65	685,387	2,012,873	143
*BP-66	685,759	2,012,851	142
*BP-67	685,694	2,012,825	141

*BP-68	685,739	2,012,922	149
*BP-69	685,425	2,012,959	151
*BP-70	685,464	2,013,050	153
*BP-71	685,566	2,012,677	264
*BP-72	685,360	2,012,779	270
*BP-73	685,671	2,012,898	273
*BP-74	685,497	2,012,980	276
*BP-75	685,312	2,012,842	148
*BP-76	685,400	2,012,035	204
*BP-77	685,820	2,012,741	105
*BP-81	687,407	2,012,497	70

1c. BP Series - Plant Design Exploration (Cont'd)

Borehole Number (feet)	Coordinates North	Coordinates East	Depth (feet)
*BP-82	686,645	2,011,797	75
*BP-83	686,963	2,012,494	90
*BP-84	687,039	2,013,366	75
*BP-85	686,269	2,011,929	85
*BP-86	686,821	2,013,508	75
*BP-87	686,022	2,012,314	75
*BP-88	686,418	2,013,176	57
*BP-89	686,752	2,013,907	75
*BP-90	686,477	2,013,684	77
*BP-91	685,680	2,014,027	50
*BP-92	685,396	2,014,146	45
*BP-93	684,372	2,013,073	117
*BP-94	684,607	2,013,587	95
*BP-95	684,866	2,013,676	45
*BP-96	685,027	2,014,006	45
*BP-101	684,751	2,012,857	19
*BP-102	684,826	2,013,016	23
*BP-103	684,920	2,013,210	29
*BP-104	685,099	2,013,117	24
*BP-105	685,160	2,012,869	24
*BP-106	685,283	2,013,134	29
*BP-107	685,629	2,013,329	29
*BP-108	685,707	2,013,496	9
*BP-109	685,797	2,013,689	28
*BP-110	685,978	2,013,859	14
*BP-111	685,815	2,013,247	18
*BP-112	685,896	2,013,417	17
*BP-113	685,919	2,013,481	10
*BP-114	685,954	2,013,543	10
*BP-115	685,795	2,012,566	22
*BP-116	685,942	2,012,905	25
*BP-117	686,332	2,013,434	15
*BP-118	686,002	2,013,159	10
*BP-119	686,072	2,013,340	11
*BP-120	686,156	2,013,514	12
*BP-121	686,233	2,013,681	13
*BP-122	685,810	2,012,306	24
*BP-123	685,977	2,012,644	10

*BP-124	686,071	2,012,849	14
*BP-125	686,168	2,013,061	9
*BP-126	686,280	2,013,271	20
*BP-131	686,903	2,011,411	14
*BP-132	687,019	2,011,630	20
*BP-133	687,074	2,011,770	13
*BP-134	687,159	2,011,960	15
*BP-135	687,249	2,012,138	30
*BP-136	687,324	2,012,309	30
*BP-137	687,495	2,012,672	30
*BP-138		2,012,866	18
*BP-139	687,656	2,013,048	26
*BP-140	687,754	2,013,222	29
*BP-141	686,725	2,011,501	15
*BP-142	686,828	2,011,694	10
*BP-143	686,902	2,011,858	14
*BP-144	686,986	2,012,038	13

1c. BP Series - Plant Design Exploration (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*BP-145	687,071	2,012,221	15
*BP-146	687,149	2,012,402	19
*BP-147	687,232	2,012,582	17
*BP-148	687,319	2,012,768	18
*BP-149	687,404	2,012,950	18
*BP-150	687,491	2,013,124	17
*BP-151	687,566	2,013,298	20
*BP-152	686,565	2,011,566	13
*BP-153	686,712	2,011,949	19
*BP-154	686,803	2,012,126	15
*BP-155	686,886	2,012,319	14
*BP-156	687,057	2,012,669	18
*BP-157	687,127	2,012,840	24
*BP-158	687,217	2,013,023	24
*BP-159	687,304	2,013,209	19
*BP-160	687,386	2,013,385	21
*BP-161	687,367	2,011,672	11
*BP-162	686,464	2,011,881	16
*BP-163	686,540	2,012,024	16
*BP-164	686,613	2,012,209	20
*BP-165	686,697	2,012,393	22
*BP-166	686,782	2,012,552	23
*BP-167	686,863	2,012,756	12
*BP-168	686,950	2,012,933	24
*BP-169	686,953	2,013,152	34
*BP-170	687,123	2,013,306	34
*BP-171	687,203	2,013,476	44
*BP-172	686,179	2,011,742	18
*BP-173	686,334	2,012,120	20
*BP-174	686,428	2,012,267	10
*BP-175	686,505	2,012,478	15
*BP-176	686,597	2,012,663	11
*BP-177	686,691	2,012,833	14
*BP-178	686,770	2,013,011	29

*BP-179	686,855	2,013,197	29
*BP-180	686,939	2,013,378	29
*BP-181	687,022	2,013,571	20
*BP-182	686,005	2,011,827	12
*BP-183	686,086	2,012,012	16
*BP-184	686,176	2,012,197	30
*BP-185	686,262	2,012,374	20
*BP-186	686,345	2,012,588	20
*BP-187	686,427	2,012,739	24
*BP-188	686,510	2,012,919	20
*BP-189	686,593	2,013,101	20
*BP-190	686,681	2,013,276	28
*BP-191	686,859	2,013,650	29
*BP-192	685,825	2,011,909	13
*BP-193	685,900	2,012,088	10
*BP-194	686,088	2,012,444	17
*BP-195	686,162	2,012,636	13
*BP-196	686,254	2,012,850	16
*BP-197	686,328	2,013,000	20
*BP-198	686,512	2,013,371	28

1c. BP Series - Plant Design Exploration (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*BP-199	686,594	2,013,563	28
*BP-200	686,671	2,013,730	25
*BP-201	685,632	2,012,001	14
*BP-202	685,710	2,012,185	18
*BP-203	686,486	2,013,808	12
*BP-204	685,462	2,012,106	15
*BP-205	685,535	2,012,265	15
*BP-206	686,300	2,013,897	16
*BP-207	685,279	2,012,185	9
*BP-208	685,368	2,012,356	18
*BP-209	686,128	2,014,007	14
*BP-210	685,086	2,012,254	13
*BP-211	685,188	2,012,429	23
*BP-199	686,594	2,013,563	28
*BP-200	686,671	2,013,730	25
*BP-201	685,632	2,012,001	14
*BP-202	685,710	2,012,185	18
*BP-203	686,486	2,013,808	12
*BP-204	685,462	2,012,106	15
*BP-205	685,535	2,012,265	15
*BP-206	686,300	2,013,897	16
*BP-207	685,279	2,012,185	9
*BP-208	685,368	2,012,356	18
*BP-209	686,128	2,014,007	14
*BP-210	685,086	2,012,254	13
*BP-211	685,188	2,012,429	23
*BP-212	685,941	2,014,067	20
*BP-213	684,914	2,012,235	10

*BP-214	684,997	2,012,519	10
*BP-215	685,753	2,014,158	20
*BP-216	684,738	1,012,429	10
*BP-217	684,820	2,012,612	18
*BP-218	685,581	2,014,235	27
*BP-219	684,551	2,012,511	10
*BP-220	684,638	2,012,693	10
*BP-221	685,389	2,014,311	20
*BP-222	684,371	2,012,607	10
*BP-223	684,460	2,012,783	10
*BP-224	684,536	2,012,957	37
*BP-225	684,632	2,013,158	26
*BP-226	684,704	2,013,319	11
*BP-227	684,792	2,013,500	7
*BP-228	684,869	2,014,126	13
*BP-229	684,970	2,013,886	9
*BP-230	684,955	2,014,313	23
*BP-231	685,134	2,014,224	28
*BP-232	685,217	2,014,401	29
*BP-233	684,192	2,012,687	15
*BP-234	684,275	2,012,859	15
*BP-235	684,433	2,013,227	17
*BP-236	684,516	2,013,382	23
*BP-237	684,689	2,013,749	15
*BP-238	684,773	2,013,951	16
*BP-240	685,433	2,012,197	40

1c. BP Series - Plant Design Exploration (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*BP-241	685,453	2,012,241	40
*BP-242	685,347	2,012,247	35
*BP-243	685,360	2,012,282	35
*BP-244	685,799	2,013,374	38
*BP-245	685,824	2,013,414	37
*BP-246	685,713	2,013,418	41
*BP-247	685,751	2,013,446	38
*BP-248	684,953	2,013,082	58
*BP-249	684,995	2,013,133	63
*BP-250	684,544	2,013,302	61
*BP-251	686,960	2,012,990	50
*BP-252	686,979	2,013,142	50
*BP-253	685,617	2,013,438	42
*BP-254	685,638	2,013,499	37
*BP-255	685,248	2,012,275	30
*BP-256	685,265	2,012,347	30

1d. BC Series - Channels Explorations

Borehole Number (feet)	Coordinates North	East	Depth (feet)
BC-1	690,818	2,023,942	10
BC-2	690,628	2,023,871	30
BC-3	690,415	2,023,779	45
BC-4	690,748	2,024,356	20
BC-5	690,485	2,024,243	40
BC-6	690,236	2,024,138	45
BC-7	690,562	2,024,707	20
BC-8	690,331	2,024,596	25
BC-9	690,116	2,024,520	35
BC-10	689,988	2,024,901	30
BC-11	690,175	2,024,966	35
BC-12	690,018	2,024,903	35
BC-13	690,212	2,025,433	55
BC-14	690,048	2,025,340	65
BC-15	689,835	2,025,253	55
BC-16	690,109	2,025,820	40
BC-17	689,884	2,025,705	50
BC-18	689,665	2,025,615	71
BC-19	689,967	2,026,175	50
BC-20	689,743	2,026,081	35
BC-21	689,522	2,025,993	40
BC-22	689,813	2,026,566	45
BC-23	689,599	2,026,457	26

1d. BC Series - Channels Explorations (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
BC-24	689,360	2,026,371	45
BC-25	689,659	2,026,934	50
BC-26	689,428	2,026,805	25
BC-27	689,221	2,026,748	35
BC-28	689,516	2,027,306	50
BC-29	689,292	2,027,199	20
BC-30	689,081	2,027,098	30
BC-31	689,364	2,027,661	30
BC-32	689,159	2,027,571	15
BC-33	688,923	2,027,481	30
BC-34	688,757	2,027,851	25
BC-35	688,629	2,028,226	15
BC-36	687,072	2,017,173	10
BC-37	686,851	2,017,261	10
BC-38	686,600	2,017,348	20
BC-39	687,145	2,017,507	35
BC-40	686,971	2,017,688	35
BC-41	686,748	2,017,762	35
BC-42	687,338	2,017,924	10
BC-43	687,118	2,018,016	15
BC-44	686,887	2,018,103	20
*BC-45	687,372	2,007,956	20
*BC-46	687,414	2,008,125	35
*BC-47	687,421	2,008,332	30
*BC-48	687,524	2,008,514	15
BC-49	678,042	2,021,835	12
BC-50	678,182	2,022,096	22
BC-51	678,301	2,022,264	21
BC-52	677,698	2,022,057	12
BC-53	677,835	2,022,267	30
BC-54	677,945	2,022,472	15
BC-55	677,368	2,022,261	40
BC-56	677,493	2,022,467	38
BC-57	677,621	2,022,699	38
BC-58	677,031	2,022,499	29
BC-59	677,148	2,022,681	27
BC-60	677,276	2,022,876	36
BC-61	676,722	2,022,720	17
BC-62	676,806	2,022,885	30
BC-63	676,934	2,023,085	26
BC-64	676,355	2,022,915	13
BC-65	676,458	2,023,081	20
BC-66	676,572	2,023,306	31
BC-67	676,135	2,023,285	20
BC-68	676,244	2,023,509	38
BC-69	675,772	2,023,490	13
BC-70	675,896	2,023,708	28

*BC-71	683,816	2,012,211	40
*BC-72	683,904	2,012,391	50
*BC-73	683,989	2,012,570	60
*BC-74	684,056	2,012,748	64
BC-101	682,563	2,010,403	10
BC-102	682,782	2,010,316	20

1d BC Series - Channels Explorations (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
BC-103	682,586	2,010,774	17
BC-104	686,853	2,010,750	20
BC-105	682,715	2,011,175	25
BC-106	682,954	2,011,088	20
BC-107	682,793	2,011 582	15
BC-108	682,978	2,011,493	15
BC-109	682,937	2,011,957	15
BC-110	683,154	2,011,855	20
BC-111	683,098	2,012,316	10
BC-112	683,321	2,012,221	20
BC-113	683,416	2,012,389	20
BC-114	683,484	2,012,580	30
*BC-115	684,556	2,013,813	55
*BC-116	683,801	2,012,659	35
*BC-117	683,649	2,012,257	25
*BC-118	684,478	2,013,641	55
*BC-119	684,396	2,013,465	60
*BC-120	684,298	2,013,257	65
*BC-121	684,236	2,013,120	65
*BC-122	684,137	2,012,924	60
BC-130	691,668	2,013,761	20
BC-131	691,625	2,014,025	35
BC-132	691,582	2,014,327	50
BC-133	691,517	2,014,602	30
*BC-140	682,480	2,009,982	22
*BC-141	682,681	2,009,908	9
*BC-142	682,416	2,009,621	10
*BC-143	686,522	2,012,591	35
*BC-144	686,062	2,011,593	35
*BC-145	685,669	2,010,783	36
*BC-146	684,799	2,013,174	20
*BC-147	684,435	2,012,376	20
*BC-148	684,056	2,011,570	25
*BC-149	684,199	2,010,620	35
*BC-150	686,343	2,012,226	20
*BC-151	686,173	2,011,859	14
*BC-152	685,934	2,011,314	19
*BC-153	685,837	2,011,135	17
*BC-154	685,740	2,010,471	19
*BC-155	684,684	2,012,922	21
*BC-156	684,555	2,012,645	17
*BC-157	684,315	2,012,108	18
*BC-158	684,179	2,011,839	19

*BC-159	683,987	2,011,190	24
*BC-160	684,091	2,010,898	21
*BC-161	684,302	2,010,350	15
*BC-162	684,407	2,010,049	11
*BC-165	687,407	2,008,244	65
*BC-166	687,369	2,008,037	50
*BC-170	685,840	2,010,230	16
*BC-171	685,930	2,010,310	15

1d. BP Series - Channels Explorations (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*BC-172	686,020	2,010,490	40
*BC-173	686,070	2,010,670	19
*BC-174	686,180	2,010,850	13
*BC-175	686,260	2,011,100	60
*BC-176	686,255	2,011,360	55
*BC-177	686,240	2,011,570	49
*BC-178	686,180	2,012,300	50
*BC-179	684,405	2,010,210	30
*BC-180	684,130	2,010,530	20
*BC-181	683,990	2,010,695	39
*BC-182	683,870	2,010,830	35
*BC-183	683,740	2,010,980	33
*BC-184	683,600	2,011,200	34
*BC-185	683,510	2,011,480	40
*BC-186	683,510	2,011,780	23
*BC-187	683,600	2,012,040	36
*BC-188	683,880	2,012,850	11
*BC-189	683,970	2,013,030	15
*BC-190	684,150	2,013,270	29
*BC-191	684,185	2,013,710	65
*BC-192	684,010	2,013,800	35
*BC-193	683,900	2,013,910	40
BC-194	683,850	2,014,075	71
BC-195	683,840	2,014,370	18
BC-196	683,825	2,014,670	45
BC-197	683,830	2,014,970	35
BC-198	683,840	2,015,270	18
BC-199	683,840	2,015,570	21
BC-500	689,723	2,022,941	35
BC-501	689,351	2,022,609	30
BC-502	689,467	2,023,249	30
BC-503	689,266	2,023,102	35
BC-505	689,279	2,023,434	20
BC-521	687,753	2,024,730	65
BC-522	687,572	2,024,564	74
BC-523	687,658	2,025,189	65
BC-524	687,484	2,025,025	45
BC-525	687,300	2,024,870	55

BC-526	687,396	2,025,495	44
BC-527	687,210	2,025,330	35
BC-529	687,108	2,025,788	26
BC-530	686,953	2,025,620	30
BC-531	686,747	2,025,458	40
BC-532	686,695	2,025,905	21
BC-533	686,480	2,025,730	40
BC-534	686,861	2,026,081	30
BC-535	686,580	2,026,376	35
BC-536	686,384	2,026,208	15
BC-537	686,226	2,025,998	30
BC-538	686,311	2,026,671	30
BC-539	686,112	2,026,510	20
BC-540	685,938	2,026,399	10
BC-541	686,048	2,026,949	27
BC-542	685,854	2,026,796	24

1e. BD Series - Exploration for Separating Dikes and Saddle Dams

Borehole Number (feet)	Coordinates North	East	Depth (feet)
BD-1	685,345	2,014,831	58.0
BD-2	685,297	2,015,025	45.0
BD-3	685,250	2,015,220	43.0
BD-4	685,202	2,015,414	40.0
BD-5	685,154	2,015,608	45.0
BD-6	685,103	2,015,795	47.0
BD-7	685,063	2,016,003	91.0
BD-8	685,020	2,016,195	42.0
BD-9	684,968	2,016,389	41.0
BD-10	684,923	2,016,581	84.0
BD-11	685,081	2,014,457	31.0
BD-12	684,866	2,014,457	52.0
BD-13	684,705	2,014,383	64.0
BD-14	684,530	2,014,279	35.0
BD-15	684,351	2,014,189	74.0
*BD-16	685,457	2,008,531	45.0
*BD-17	685,430	2,008,734	35.0
*BD-18	685,386	2,008,951	22.0
*BD-19	685,350	2,009,108	25.0
*BD-20	685,314	2,009,330	30.0
BD-21	682,268	2,012,810	40.0
BD-22	682,238	2,012,977	40.0
BD-23	682,426	2,013,132	40.0
BD-24	682,492	2,013,340	35.0
BD-25	682,557	2,013,531	41.0
BD-26	682,622	2,013,702	60.0
BD-27	682,678	2,013,858	51.5
BD-28	682,774	2,014,079	51.0
BD-29	682,843	2,014,262	51.0
BD-30	682,921	2,014,450	52.0
BD-31	683,888	2,017,726	37.0

BD-32	683,734	2,017,836	43.5
BD-33	683,528	2,017,945	53.0
BD-34	683,423	2,018,011	64.9
BD-35	683,217	2,018,152	48.0
BD-36	683,039	2,018,264	57.0
BD-37	682,885	2,018,361	37.5
BD-38	681,751	2,018,445	40.5
BD-39	682,543	2,018,576	57.0
BD-40	682,321	2,018,719	40.5
BD-41	682,204	2,018,790	47.0
BD-42	681,989	2,018,912	41.0
BD-43	681,824	2,019,031	60.0
BD-44	681,269	2,019,536	50.0
BD-45	680,911	2,019,716	48.0
BD-46	680,543	2,019,898	54.0
BD-47	680,186	2,020,198	52.0
BD-56	678,445	2,021,114	30.0
BD-57	678,319	2,021,254	40.0
BD-58	678,124	2,021,392	59.0
BD-65	683,713	2,014,258	65.0
BD-66	683,526	2,014,338	69.0

1e. BD Series - Exploration for Separating Dikes and Saddle Dams (Cont'd)

	Coordinates		
Borehole Number (feet)	North	East	Depth (feet)
BD-67	683,320	2,014,395	50.0
BD-68	683,152	2,014,493	71.5
BD-101	682,471	2,004,934	16.0
BD-102	680,552	2,004,863	10.0
BD-103	680,341	2,004,837	13.0
BD-104	679,125	2,003,556	13.5
BD-105	670,055	2,004,135	8.0
BD-110	688,024	2,013,257	37.0
BD-111	688,260	2,013,358	37.0
BD-112	688,432	2,013,445	27.0
BD-113	688,706	2,013,570	36.5
BD-114	688,935	3,013,689	38.9
BD-115	689,129	2,013,809	74.0
BD-116	689,397	2,013,906	60.0
BD-117	689,617	2,014,011	50.0
BD-130	684,169	2,004,780	17.0
BD-131	681,899	2,019,704	9.0
BD-132	683,591	2,021,707	13.0
BD-133	674,954	2,018,497	14.0
BD-134	680,409	2,004,867	22.5
BD-135	680,146	2,004,810	30.0
BD-500	687,373	2,026,988	73.5
BD-501	687,240	2,027,084	53.0
BD-502	687,092	2,027,212	47.5
BD-503	686,901	2,027,378	57.0
BD-504	686,774	2,027,470	47.5
BD-505	686,488	2,027,726	37.0
BD-506	686,206	2,027,951	27.0
BD-507	685,850	2,028,210	22.0

BD-510	684,920	2,028,970	27.0
BD-511	684,765	2,029,090	43.5
BD-512	684,558	2,029,221	33.5
BD-513	684,480	2,029,270	33.0
BD-514	684,275	2,029,467	40.0
BD-515	684,421	2,029,616	34.0
BD-516	684,167	2,029,294	40.0
BD-517	684,056	2,029,142	40.0
BD-518	684,276	2,029,759	52.0
BD-519	684,140	2,029,596	40.0
BD-520	684,006	2,029,448	44.0
BD-521	684,110	2,029,865	37.0
BD-522	683,995	2,029,695	44.1
BD-525	676,849	2,024,138	44.0
BD-526	676,675	2,024,037	55.0
BD-527	676,533	2,023,911	34.0
BD-528	676,640	2,024,210	53.8
BD-529	676,474	2,024,101	60.0
BD-530	676,312	2,023,984	57.0
BD-531	676,536	2,024,373	44.0
BD-532	676,203	2,024,161	60.0
BD-533	676,372	2,024,257	53.0
BD-534	676,266	2,024,429	40.0
BD-535	676,120	2,024,604	18.0
BD-536	676,001	2,024,861	21.0
BD-537	675,838	2,025,010	20.0
BD-538	675,675	2,025,062	21.0
BD-539	675,474	2,025,110	25.0

1f. BCT Series - Cooling Towers Foundations Explorations

Borehole Number (feet)	Coordinates North	Coordinates East	Depth (feet)
BCT-1	684,130	2,011,580	18.5
BCT-2	684,220	2,011,760	13.5
BCT-3	684,300	2,011,955	4.8
BCT-4	684,300	2,011,500	18.6
BCT-5	684,385	2,011,680	50.0
BCT-6	684,470	2,011,870	9.8
BCT-7	684,490	2,011,410	25.0
BCT-8	684,570	2,011,600	8.8
BCT-9	684,670	2,011,775	13.7
BCT-10	684,660	2,011,330	6.0
BCT-11	684,750	2,011,510	40.0
BCT-12	684,830	2,011,695	6.0
BCT-13	684,850	2,011,240	17.0
BCT-14	684,940	2,011,420	25.0
BCT-15	685,020	2,011,605	10.5
BCT-16	685,030	2,011,150	44.0
BCT-17	685,115	2,011,335	50.0
BCT-18	685,205	2,011,525	14.1
BCT-19	685,210	2,011,080	39.0
BCT-20	685,300	2,011,250	18.5
BCT-21	685,385	2,011,440	9.5
BCT-22	685,390	2,010,980	29.9
BCT-23	685,475	2,011,165	55.0

BCT-24	685,560	2,011,350	9.1
BCT-25	685,510	2,014,505	50.0
BCT-26	685,595	2,014,690	9.6
BCT-27	685,680	2,014,430	33.5
BCT-28	685,765	2,014,610	9.7
BCT-29	685,855	2,014,345	45.0
BCT-30	685,945	2,014,525	9.0
BCT-31	686,040	2,014,255	13.5
BCT-32	686,120	2,014,440	13.5
BCT-33	686,200	2,014,165	20.0
BCT-34	686,310	2,014,250	4.7
BCT-35	686,410	2,014,080	22.5
BCT-36	686,495	2,014,260	13.9
BCT-37	685,590	2,013,990	55.0
BCT-38	686,685	2,014,170	14.6
BCT-39	686,860	2,014,090	7.4
BCT-40	686,940	2,014,270	26.2
BCT-41	686,770	2,014,350	15.0
BCT-42	686,580	2,014,440	15.0
BCT-43	686,395	2,014,530	8.0
BCT-44	686,210	2,014,625	10.5
BCT-45	686,020	2,014,700	11.5
BCT-46	685,850	2,014,790	33.5
BCT-47	685,680	2,014,870	28.5
BCT-48	685,495	2,014,955	23.5
BCT-49	685,435	2,014,770	9.9
BCT-50	685,325	2,014,590	30.0
BCT-51	684,040	2,011,390	4.9
BCT-52	684,210	2,011,315	4.6
BCT-53	684,395	2,011,230	5.0
BCT-54	684,575	2,011,150	8.8
BCT-55	684,765	2,011,060	33.6

1f. BCT Series - Cooling Towers Foundations Explorations (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
BCT-56	685,940	2,010,970	4.8
BCT-57	685,140	2,010,895	9.6
BCT-58	685,300	2,010,810	14.5
BCT-59	685,490	2,010,720	14.6
BCT-60	685,575	2,010,900	14.6
BCT-61	685,660	2,011,080	6.9
BCT-62	685,735	2,011,260	9.6
BCT-63	685,825	2,011,445	14.0
BCT-64	684,910	2,011,630	6.0
BCT-65	685,650	2,011,550	7.5
BCT-66	685,735	2,011,710	8.9
BCT-67	685,465	2,011,610	7.1
BCT-68	685,555	2,011,800	8.9
BCT-69	685,385	2,011,700	9.2
BCT-70	685,365	2,011,880	8.5
BCT-71	685,100	2,011,780	6.3

BCT-72	685,180	2,011,965	4.5
BCT-73	684,915	2,011,875	5.0
BCT-74	685,000	2,012,045	4.9
BCT-75	684,735	2,011,960	13.5
BCT-76	684,855	2,012,115	3.8
BCT-77	684,550	2,012,216	13.0
BCT-78	684,640	2,012,215	4.9
BCT-79	684,380	2,012,120	6.0
BCT-80	684,460	2,012,295	6.9

1g. BX Series - Auxiliary Dam Exploration

Borehole Number (feet)	Coordinates North	Coordinates East	Depth (feet)
*BX-1	684,593	2,006,535	49.0
*BX-2	684,465	2,006,688	59.0
*BX-3	684,308	2,006,562	53.0
*BX-4	684,159	2,006,432	57.0
*BX-5	684,009	2,006,296	52.0
*BX-6	683,854	2,006,181	34.5
*BX-7	683,996	2,006,980	59.0
*BX-8	683,849	2,007,450	69.0
*BX-9	683,995	2,007,953	40.0
*BX-10	683,850	2,007,955	44.5
*BX-11	683,998	2,009,458	39.0
*BX-12	683,842	2,008,403	49.0
*BX-13	684,000	2,008,950	30.0
*BX-14	683,841	2,008,950	39.0

1g. BX Series - Auxiliary Dam Exploration (Cont’d)

Borehole Number (feet)	Coordinates North	Coordinates East	Depth (feet)
*BX-15	683,833	2,009,439	54.0
*BX-16	683,850	2,009,819	59.0
*BX-17	684,564	2,006,224	59.0
BX-20	687,026	2,013,670	86.0
BX-21	687,178	2,013,864	83.0
BX-22	687,339	2,014,072	68.0
BX-23	687,481	2,014,255	55.0
BX-24	687,672	2,014,315	51.0
BX-25	687,599	2,014,407	38.0
BX-26	687,791	2,014,668	71.0
BX-27	687,937	2,014,856	32.0

BX-28	688,079	2,015,053	55.0
BX-29	687,639	2,014,463	50.0
BX-30	688,234	2,015,246	61.0
BX-31	688,530	2,015,255	50.5
BX-32	688,425	2,015,485	54.0
BX-33	688,138	2,015,487	33.0
BX-34	687,510	2,014,106	78.0
BX-35	687,342	2,014,220	71.0
BX-36	687,806	2,014,489	41.5
BX-37	687,618	2,014,654	56.0
BX-38	688,305	2,015,350	54.0
BX-39	688,246	2,015,428	61.0
BX-39A	688,246	2,015,428	21.0
*BX-40	684,200	2,006,980	38.5
*BX-41	684,115	2,006,795	87.0
*BX-42	684,050	2,006,730	86.0
*BX-43	683,970	2,006,680	75.0
*BX-44	683,940	2,006,865	75.0
*BX-45	683,850	2,006,810	75.0
*BX-46	683,870	2,006,700	70.0
*BX-47	683,670	2,006,615	65.0
*BX-48	683,470	2,006,665	11.5
*BX-49	683,340	2,006,790	12.5
*BX-50	683,280	2,006,980	45.0
*BX-51	683,220	2,007.170	23.0
*BX-52	683,175	2,007,360	20.0
*BX-53	683,125	2,007,550	45.0
*BX-54	683,085	2,007,770	9.0
*BX-55	683,010	2,007,940	9.0
*BX-56	682,970	2,008,130	40.0
*BX-57	682,895	2,008,320	14.0
*BX-58	682,720	2,008,450	25.5
*BX-59	684,055	2,006,863	75.0
*BX-60	684,010	2,006,935	75.0
*BX-61	683,980	2,006,800	75.0
*BX-62	684,370	2,006,458	70.0
*BX-63	684,255	2,006,625	68.0
*BX-64	684,295	2,006,430	75.0
*BX-65	684,175	2,006,565	75.0
*BX-66	684,220	2,006,385	80.0
*BX-67	684,120	2,006,570	73.0

1h. BF Series - Exploration for Make up Water System Structures

Borehole Number (feet)	Coordinates North	East	Depth (feet)
BF-1	661,067	2,011,275	35.5
BF-2	661,881	2,011,157	55.0
BF-3	661,946	2,003,020	81.0
BF-4	652,346	2,003,101	106.5
BF-5	652,083	2,003,066	71.0
BF-6	652,160	2,003,040	10.0
BF-7	652,280	2,030,040	10.0
BF-8	684,497	2,006,485	26.0
BF-9	684,067	2,006,139	27.0
BF-10	688,664	2,015,265	30.0
BF-11	688,294	2,015,571	30.0
BF-101	661,150	2,011,270	3.5
BF-102	661,240	2,011,260	0.5
BF-103	661,361	2,011,239	13.5
BF-104	661,491	2,011,221	13.0
BF-105	661,635	2,011,201	7.0
BF-106	661,777	2,011,179	18.0
BF-107	684,282	2,006,297	15.0
BF-108	684,160	2,006,197	15.5
BF-110	688,620	2,015,330	14.0
BF-111	688,475	2,015,436	16.5
BF-112	688,385	2,015,511	13.5
BF-113	683,265	2,008,700	10.5
BF-114	683,340	2,008,880	6.5
BF-115	683,430	2,009,060	55.0
BF-116	683,470	2,009,140	60.0
BF-117	683,505	2,009,250	65.0
BF-118	683,937	2,009,735	83.5
BF-119	683,610	2,009,420	9.0
BF-120	683,700	2,009,600	10.5
BF-121	652,595	2,002,720	24.0
BF-122	652,875	2,002,560	26.5
BF-123	653,174	2,002,473	25.0
BF-124	653,655	2,002,427	25.0
BF-125	654,039	2,002,484	9.5
BF-126	654,420	2,002,555	27.0
BF-127	654,755	2,002,775	10.0
BF-128	655,055	2,002,985	10.0
BF-129	655,338	2,003,183	4.5
BF-130	655,664	2,003,412	10.5
BF-131	655,866	2,003,554	8.5
BF-132	656,181	2,003,774	11.5
BF-133	656,540	2,004,026	12.5
BF-134	656,863	2,004,253	3.0
BF-135	657,184	2,004,402	7.0
BF-136	657,635	2,004,575	48.0
BF-137	657,945	2,004,694	22.0
BF-138	658,245	2,004,810	10.5
BF-139	658,585	2,004,940	7.5
BF-140	658,889	2,005,057	2.0
BF-141	659,105	2,005,160	15.0
BF-142	659,325	2,005,480	7.0
BF-143	659,379	2,005,900	27.5

BF-144	659,417	2,006,225	10.5
BF-145	659,466	2,006,650	17.0
BF-146	659,504	2,006,981	35.0
BF-147	659,552	2,007,395	32.5
BF-148	659,577	2,007,610	8.0

1i. X,M, and A Series - Preliminary Subsurface Investigation, Main Dam Vicinity

	Coordinates		
Borehole Number (feet)	North	East	Depth (feet)
X-1	660,958	2,010,121	90.0
X-2	661,150	2,010,470	100.0
X-3	661,388	2,010,901	97.0
M-1	664,300	2,007,820	
M-2	664,120	2,008,287	
M-3	663,941	2,008,753	
M-4	663,761	2,009,220	
M-5	663,582	2,009,687	
M-6	663,402	2,010,153	
M-7	663,223	2,010,620	
A-1	652,914	2,004,140	
A-2	652,557	2,003,790	
A-3	652,200	2,003,440	

1j. BM Series - Main Dam Exploration

	Coordinates		
Borehole Number (feet)	North	East	Depth (feet)
*BM-1	660,972	2,009,204	104.0
*BM-2	660,852	2,009,178	130.0
*BM-3	660,718	2,009,148	148.6
*BM-4	660,527	2,009,143	121.0
*BM-5	660,442	2,009,288	81.0
*BM-6	660,328	2,009,406	48.5
*BM-7	660,178	2,009,603	45.8
*BM-8	660,898	2,009,970	96.0
*BM-9	660,956	2,010,108	74.0
*BM-10	661,070	2,010,288	71.0
*BM-11	661,138	2,010,463	75.0
*BM-12	661,190	2,010,554	90.0
*BM-13	661,233	2,010,663	101.0
*BM-14	661,319	2,010,804	95.0
*BM-15	661,417	2,010,980	78.5
*BM-16	661,541	2,011,166	93.5
*BM-17	661,299	2,010,421	40.3
*BM-18	661,363	2,010,581	39.0
*BM-19	661,484	2,010,781	56.5
*BM-20	660,950	2,010,340	36.0
*BM-21	661,017	2,010,552	71.0
*BM-22	661,096	2,010,733	39.3

*BM-23	661,212	2,010,867	39.0
*BM-24	661,237	2,011,010	30.0
*BM-25	661,618	2,010,983	98.2
*BM-26	661,730	2,011,141	87.0
*BM-27	661,845	2,011,293	57.0
*BM-28	661,214	2,010,116	133.0
*BM-29	661,075	2,009,952	98.0
*BM-30	661,008	2,009,769	48.0
*BM-31	661,327	2,010,400	120.5
*BM-32	661,178	2,010,548	120.0

1j. BM Series - Main Dam Exploration (Cont'd)

	Coordinates		
Borehole Number (feet)	North	East	Depth (feet)
*BM-33	661,368	2,010,370	71.0
*BM-34	661,269	2,010,275	85.0
*BM-35	661,662	2,011,122	90.5
*BM-36	661,656	2,011,419	85.2
*BM-37	661,057	2,009,860	91.0
*BM-38	660,863	2,009,765	111.0
*BM-39	660,866	2,009,834	102.0
*BM-40	660,694	2,009,737	99.0
*BM-41	660,696	2,009,848	95.0
*BM-42	660,702	2,009,943	80.0
*BM-43	660,528	2,009,721	78.5
*BM-44	660,537	2,009,835	81.0
*BM-45	660,546	2,009,896	75.5
*BM-46	660,358	2,009,691	67.7
*BM-47	660,358	2,009,763	61.0
*BM-48	660,356	2,009,853	53.5
*BM-49	660,169	2,009,720	80.0
*BM-50	660,182	2,009,822	73.0
*BM-51	659,992	2,009,668	62.0

1k. BL Series - Exploration for Low Level Release System Pipeline

	Coordinates		
Borehole Number (feet)	North	East	Depth (feet)
BL-1	661,065	2,009,817	4.0
BL-2	661,196	2,009,819	16.5
BL-3	661,290	2,009,804	18.5
BL-4	661,354	2,009,794	19.0
BL-5	661,443	2,009,780	20.0
BL-6	661,514	2,009,768	3.0

1l. BA Series - Afterbay Dam Exploration

Borehole Number (feet)	Coordinates North	East	Depth (feet)
BA-1	652,230	2,003,316	110.0
BA-2	652,350	2,003,440	90.0
BA-3	652,524	2,003,593	82.0
BA-4	652,397	2,003,699	80.0
BA-5	652,677	2,003,770	81.0
BA-6	652,535	2,003,839	71.0
BA-7	652,802	2,003,890	104.0
BA-8	652,937	2,004,038	124.0
BA-9	653,127	2,004,084	131.0
BA-10	653,312	2,004,146	141.0
BA-11	653,521	2,004,215	133.0
BA-12	653,684	2,004,248	130.0
BA-13	653,295	2,004,299	95.0
BA-14	653,093	2,004,327	63.0
BA-15	652,896	2,004,354	59.0
BA-16	652,707	2,004,388	91.0
BA-17	652,534	2,004,468	90.0
BA-18	653,319	2,004,597	104.0
BA-19	653,408	2,004,871	92.0
BA-20	652,210	2,003,180	60.0
BA-21	652,326	2,002,858	62.0
BA-22	652,384	2,002,743	96.0
BA-23	652,432	2,002,570	43.5
BA-24	652,503	2,002,419	64.0
BA-25	653,503	2,005,159	63.0
BA-26	653,578	2,005,431	79.0

1m. BW Series - Skimmer Wall Exploration

Borehole Number (feet)	Coordinates North	East	Depth (feet)
BW-1	669,242	2,014,841	41.5
BW-2	669,077	2,015,039	41.5
BW-3	668,875	2,015,250	41.5
BW-4	668,681	2,015,465	47.0
BW-5	668,472	2,015,691	48.0
BW-6	668,192	2,015,977	57.0
BW-7	668,000	2,016,183	47.0
BW-8	667,682	2,016,532	48.0

1n. TK Series - Exploration for Norfolk Southern Railroad

(Main Line and Durham Branch Relocation)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
TK-1	661,365	2,005,222	15.0
TK-2	661,314	2,005,415	12.0
TK-3	661,034	2,006,479	17.0
TK-4	660,983	2,006,672	24.0
TK-5	660,936	2,006,867	34.0
TK-6	660,895	2,007,062	35.0
TK-7	660,863	2,007,260	29.4
TK-8	660,834	2,007,458	35.0
TK-9	660,805	2,007,656	28.5
TK-11	660,748	2,008,051	24.0
TK-12	660,720	2,008,249	38.0
TK-13	660,663	2,008,447	34.0
TK-14	660,663	2,008,645	11.0
TK-15	660,634	2,008,843	29.0
TK-16	660,606	2,009,041	20.0
TK-17	660,577	2,009,239	21.1
TK-18	660,549	2,009,437	29.0
TK-19	660,520	2,009,635	14.0
TK-20	660,267	2,011,417	20.0
TK-21	660,254	2,011,616	41.0
TK-22	660,258	2,011,791	17.0
TK-30A	662,317	1,994,473	10.0
TK-31A	663,035	1,994,691	12.0
TK-32A	663,178	1,994,735	17.0
TK-33A	664,033	1,995,013	17.0
TK-34A	664,169	1,995,076	15.0
TK-35A	664,902	1,995,593	13.0
TK-36A	665,010	1,995,696	14.0
TK-37	665,483	1,996,143	10.0
TK-38	665,591	1,996,246	6.1
TK-39A	666,536	1,997,140	19.0
TK-40A	666,681	1,997,277	22.0
TK-41A	666,826	1,997,414	17.0
TK-42A	666,971	1,997,522	18.0
TK-43A	667,719	1,998,214	14.0
TK-44A	667,888	1,998,322	14.0
TK-45	668,063	1,998,419	7.0
TK-46A	670,309	1,998,829	6.0
TK-47	670,508	1,998,848	14.5
TK-48	670,707	1,998,867	8.1
TK-49	670,906	1,998,886	7.2
TK-50A	672,001	1,998,990	8.0
TK-50AA	671,703	1,998,961	13.0
TK-51	672,399	1,999,028	8.9
TK-52	673,693	1,999,150	12.0
TK-53A	673,893	1,999,169	13.0
TK-54	674,042	2,000,183	11.0
TK-55A	675,983	1,999,367	12.0
TK-56	676,182	1,999,386	7.4
TK-57A	676,381	1,999,405	11.0
TK-58A	676,580	1,999,424	7.0

TK-59A	676,779	1,999,443	11.0
TK-60A	676,978	1,999,462	14.0
TK-61	677,177	1,999,483	8.5

1n. TK Series - Exploration for Norfolk Southern Railroad

(Main Line and Durham Branch Relocation) (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
TK-62A	679,016	2,000,610	10.0
TK-63A	679,164	2,000,744	10.0
TK-64A	680,286	2,001,657	11.5
TK-65A	681,525	2,002,013	15.0
TK-66A	681,724	2,002,034	8.5
TK-67	683,570	2,002,122	10.0
TK-68	683,768	2,002,143	15.2
TK-69A	683,165	2,002,164	13.1
TK-70	683,165	2,002,192	7.7
TK-71	684,101	2,002,530	8.8
TK-72	684,271	2,002,635	11.0
TK-73	684,433	2,002,753	9.5
TK-74	684,928	2,003,245	10.5
TK-75	685,046	2,003,406	14.5
TK-76	685,161	2,003,570	10.3
TK-77	685,276	2,003,733	7.0
TK-78A	685,391	2,003,897	12.0
TK-79	685,506	2,004,061	11.0
TK-80A	685,621	2,004,224	12.0
TK-81	685,708	2,004,347	7.0
TK-82	685,851	2,004,551	8.0
TK-83	686,868	2,005,737	9.0
TK-84	687,002	2,006,187	9.9
TK-85	687,117	2,006,351	10.0
TK-86A	687,232	2,006,514	16.0
TK-87	687,318	2,006,637	8.4
TK-88	688,037	2,007,659	9.0
TK-89A	688,152	2,007,823	14.5
TK-90	688,267	2,007,986	13.0
TK-91	688,382	2,008,150	14.5
TK-92	688,497	2,008,313	13.0
TK-93	689,576	2,009,460	6.2

1o. BR, RB, BHN, BG, SD, and SR Series - Exploration for Highway and Railroad Bridges and Embankments

Borehole Number (feet)	Coordinates North	East	Depth (feet)
BR-1	658,736	2,007,964	43.0
BR-2	658,438	2,007,945	37.0
BR-3	658,530	2,007,983	38.0
BR-4	658,645	2,007,997	40.0
BR-101	659,162	2,007,967	7.5

BR-102	658,968	2,008,003	19.5
BR-103	658,123	2,007,973	28.0
BR-104	662,614	2,005,436	10.0
BR-106	662,722	2,005,978	13.0
BR-107	662,288	2,006,317	15.0
RB-1	662,247	2,006,556	4.1
RB-2	659,964	2,001,678	18.7
RB-3	660,029	2,001,823	3.9
RB-4	659,616	2,001,386	13.6

1o. BR, RB, BHN, BG, SD, and SR Series - Exploration for Highway and Railroad Bridges and Embankments (Cont'd)

Borehole Number (feet)	Coordinates North	Coordinates East	Depth (feet)
RB-5	659,654	2,001,670	13.7
RB-6	659,710	2,011,915	9.1
BHN-1	660,511	2,011,098	15.0
BHN-2	660,409	2,011,109	25.0
BHN-3	660,306	2,011,119	25.0
BHN-4	660,207	2,011,105	25.0
BHN-5	660,108	2,011,091	20.0
BHN-6	660,561	2,011,106	10.5
BG-1	660,491	2,009,736	40.0
BG-2	660,486	2,009,767	48.5
BG-3	660,474	2,009,853	39.0
BG-4	660,469	2,009,884	23.0
BG-5	660,724	2,007,800	28.0
BG-6	660,730	2,007,761	44.0
BG-7	660,757	2,007,780	38.0
BG-8	660,763	2,007,740	59.1
BG-9	660,816	2,007,714	48.0
BG-10	660,822	2,007,714	57.1
BG-11	660,866	2,007,730	74.1
BG-12	660,872	2,007,685	76.3
BG-13	658,430	2,008,020	22.6
BG-14	658,530	2,008,020	14.0
BG-15	658,630	2,008,020	18.6
BG-16	658,730	2,008,020	19.5
BG-17	661,948	1,992,270	20.5
BG-18	661,924	1,992,336	30.0
BG-19	661,903	1,992,393	13.5
BG-20	661,880	1,992,459	25.0
SD-1	662,247	2,006,556	6.2
SD-2	662,454	2,006,339	14.8
SD-3	662,661	2,006,122	10.3
SD-4	662,868	2,005,905	10.0
SR-1A	659,615	2,008,000	20.0
SR-2A	659,815	2,007,999	12.5
SR-3A	660,014	2,007,985	13.8

1p. FB Series - Exploration for Quarry Sites

Borehole Number (feet)	Coordinates North	East	Depth (feet)
FB-1	654,925	2,002,025	76.0
FB-2	654,279	2,001,406	75.0
FB-3	655,222	2,001,589	90.0
FB-4	655,258	2,000,997	120.0
FB-11	671,441	2,018,130	58.0
FB-12	670,582	2,017,226	54.0
FB-13	671,069	2,017,801	52.0
FB-14	671,146	2,017,066	52.0
FB-15	671,738	2,017,443	77.0
FB-16	661,885	2,013,560	32.0

1p. FB Series - Exploration for Quarry Sites (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
FB-17	661,910	2,012,920	28.0
FB-18	662,260	2,013,250	42.0
FB-19	662,800	2,012,720	58.0
FB-20	662,800	2,011,850	45.5
FB-21	669.940	2,016,464	55.0
FB-22	670,408	2,016,228	60.0
FB-23	670,325	2,017,146	39.0
FB-24	670,835	2,017,146	56.5
FB-25	670,782	2,017,536	60.0
FB-26	671,056	2,017,383	80.0
FB-27	671,425	2,017,469	77.0
FB-28	671,524	2,017,864	59.0
FB-29	672,067	2,017,678	60.0
FB-30	672,150	2,017,126	60.0
FB-31	662,368	2,014,582	68.0
FB-32	663,107	2,014,593	90.0
FB-33	659,798	2,005,905	90.0
FB-34	659,847	2,006,969	82.8
FB-34A	659,847	2,006,969	42.0
FB-35	660,215	2,006,543	85.0
FB-35A	660,215	2,006,543	39.0
FB-36	660,328	2,006,045	60.0
FB-36A	660,328	2,006,045	19.0
FB-37	660,536	2,006,852	106.0
FB-37A	660,536	2,006,852	27.0

1q. BB Series - Borrow Areas Auger Borings

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*BB-1	687,395	2,016,599	50
*BB-2	687,204	2,016,401	56
*BB-3	686,802	2,015,995	51

*BB-4	686,804	2,016,594	61
*BB-5	682,603	2,006,800	44
*BB-6	682,599	2,007,399	41.5
*BB-7	682,018	2,006,782	34.7
*BB-8	682,002	2,007,400	61.0
*BB-101	687,575	2,016,197	32.0
*BB-102	687,593	2,016,393	30.0
*BB-103	687,584	2,016,604	25.0
*BB-104	687,595	2,016,808	19.0
*BB-105	687,390	2,016,014	23.0
*BB-106	687,396	2,016,202	26.0
*BB-107	687,379	2,016,383	28.0
*BB-108	687,397	2,016,393	18.0
*BB-109	687,203	2,016,010	31.0
*BB-110	687,209	2,016,193	17.0
*BB-111	687,200	2,016,600	23.0
*BB-112	687,195	2,016,792	20.0
*BB-113	687,005	2,016,007	14.0
*BB-114	686,992	2,016,193	19.0
*BB-115	686,990	2,016,409	28.0
*BB-116	686,995	2,016,591	28.0

1q. BB Series - Borrow Areas Auger Borings (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*BB-117	687,014	2,016,805	18.0
*BB-118	686,798	2,016,193	15.0
*BB-119	686,810	2,016,377	23.0
*BB-120	686,806	2,016,792	22.0
*BB-121	686,570	2,016,009	23.0
*BB-122	686,638	2,016,199	28.5
*BB-123	686,590	2,016,389	30.0
*BB-124	686,624	2,016,610	23.0
*BB-125	686,599	2,016,592	18.0
*BB-127	684,047	2,008,384	30.0
*BB-137	684,600	2,007,800	18.0
*BB-143	684,600	2,009,128	11.0
*BB-153	684,194	2,008,393	14.0
*BB-155	683,006	2,006,618	16.0
*BB-156	682,988	2,006,809	14.0
*BB-157	683,009	2,007.003	6.0
*BB-158	682,991	2,007,200	8.0
*BB-159	683,000	2,007,400	18.0
*BB-160	683,000	2,007,605	11.0
*BB-161	682,800	2,006,606	17.0
*BB-162	682,809	2,006,795	10.0
*BB-163	682,794	2,007,006	7.5
*BB-164	682,803	2,007,203	7.0
*BB-165	682,782	2,007,406	24.0
*BB-166	682,794	2,007,597	9.0
*BB-167	682,597	2,006,597	15.0
*BB-168	682,597	2,006,997	15.0
*BB-169	682,591	2,007,207	16.0
*BB-170	682,591	2,007,597	25.0
*BB-171	682,391	2,006,597	16.0
*BB-172	682,409	2,006,805	11.0

*BB-173	682,397	2,007,005	10.0
*BB-174	682,401	2,007,202	15.0
*BB-175	682,400	2,007,392	22.0
*BB-176	682,400	2,007,596	23.0
*BB-177	682,200	2,006,606	10.0
*BB-178	682,210	2,006,800	23.0
*BB-179	682,191	2,007,005	10.0
*BB-180	682,218	2,007,206	14.0
*BB-181	682,205	2,007,400	15.0
*BB-182	682,218	2,007,597	17.5
*BB-183	682,027	2,006,599	12.0
*BB-184	681,997	2,007,009	15.0
*BB-185	682,003	2,007,207	15.0
*BB-186	681,981	2,007,600	18.0
*BB-187	661,085	2,010,920	11.0
*BB-188	660,941	2,011,059	8.5
*BB-189	660,795	2,011,200	5.0
*BB-803	660,711	2,012,787	11.0
*BB-806	663,233	2,006,200	18.5
*BB-807	663,205	2,006,576	23.0
*BB-809	665,400	2,009,600	9.5
*BB-811	664,975	2,009,600	10.0
*BB-812	664,473	2,005,628	18.0
*BB-813	664,649	2,005,877	27.5
*BB-813A	664,652	2,005,865	7.0
*BB-813B	664,641	2,005,869	1.0
*BB-813C	664,645	2,005,882	6.0

1q. BB Series - Borrow Areas Auger Borings (Cont'd)

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*BB-813D	664,650	2,005,875	13.0
*BB-814	664,470	2,006,225	50.0
*BB-815	664,654	2,006,602	29.0
BB-835	658,975	2,011,775	16.5
*BB-858	665,851	2,006,007	8.0
*BB-859	665,847	2,006,646	10.0
*BB-860	665,850	2,007,197	11.0
BB-861A	655,300	2,007,200	0.5
BB-861B	655,300	2,007,200	2.0
*BB-862	665,101	2,006,660	9.0
*BB-863	664,489	2,007,247	10.0
*BB-864	664,497	2,007,747	12.0
*BB-865	664,003	2,006,246	5.0
*BB-866	663,998	2,007,253	19.0
*BB-867	664,001	2,007,747	7.0
*BB-868	663,610	2,007,278	16.0
*BB-869	663,394	2,006,250	12.0
BB-870	653,897	2,003,744	40.0
BB-871	653,580	2,004,180	45.0
BB-871A	653,580	2,004,180	30.0
BB-872	653,165	2,005,077	40.0
BB-873	653,535	2,004,145	21.0
BB-874	653,493	2,005,487	40.0
BB-875	653,622	2,006,077	40.0
BB-876	653,778	2,016,856	40.0

BB-877	652,357	2,006,369	18.0
BB-878	652,384	2,007,019	20.0
BB-879	652,929	2,007,585	40.0
BB-880	653,714	2,008,192	40.0
BB-895	657,519	2,002,802	21.0
BB-896	657,430	2,003,841	3.0
BB-896A	657,430	2,003,841	4.5
BB-902	655,638	2,002,992	13.0
BB-903	655,184	2,002,938	9.5
BB-904	654,580	2,003,000	33.0
BB-906	654,641	2,006,925	34.0
BB-907	654,510	2,007,470	39.0
BB-908	654,137	2,008,134	39.0
BB-916	653,990	2,003,573	14.3
BB-917	653,001	2,003,771	14.0
BB-918	654,012	2,003,952	33.5
BB-919	653,841	2,003,572	7.2
BB-920	653,850	2,003,971	16.0
BB-921	653,853	2,003,967	25.5
BB-922	653,845	2,004,374	9.0
BB-924	653,741	2,003,775	10.2
BB-925	653,747	2,003,996	10.0
BB-926	653,721	2,004,181	35.0
BB-927	653,709	2,004,384	30.0
BB-928	653,575	2,003,575	14.3
BB-929	653,586	2,003,777	5.0
BB-930	653,586	2,003,981	14.0
BB-931	653,584	2,004,387	30.0
BB-932	653,392	2,003,769	16.0
BB-933	653,374	2,003,982	19.0

1r. TPM, TPY, and TPZ Series - Borrow Area Test Pits

	Coordinates		
Borehole Number (feet)	North	East	Depth (feet)
*TPM-1	664,330	2,005,625	8.0
*TPM-2	664,450	2,006,475	9.0
*TPM-3A	664,029	2,006,701	8.25
*TPM-4A	663,356	2,005,556	11.0
*TPM-5A	663,847	2,006,393	5.5
*TPM-6	662,976	2,006,291	6.0
*TPY-1	687,300	2,016,152	10.0
*TPY-2A	687,166	2,016,373	12.0
*TPY-3	686,861	2,016,159	7.0
*TPY-4A	686,526	2,016,183	6.5
*TPZ-1	682,700	2,006,835	9.0
*TPZ-2	682,700	2,007,492	5.7
*TPZ-3	682,200	2,006,858	7.5
*TPZ-4	682,195	2,007,500	10.0

1s. TPA Series - Auxiliary Dam Test Pits

Borehole Number (feet)	Coordinates North	East	Depth (feet)
*TPA-1	683,800	2,007,400	10.0
*TPA-2	683,800	2,009,325	7.0

2. LOGS OF BOREHOLES AND TEST PITS

a. P Series Boreholes

APPENDIX 2.5B LABORATORY ANALYSES OF FOUNDATION MATERIALS FOR DAMS AND DIKE

APPENDIX 2.5C PRECONSTRUCTION BORROW MATERIAL TESTING

2.5C.1 INDEX PROPERTIES OF BORROW MATERIAL

This section of Appendix 2.5C discusses the index properties of the borrow materials proposed during the preconstruction stage for the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike. These index properties are for samples obtained from the test pits dug in the borrow areas.

The testing program for the evaluation of the static and dynamic engineering properties used in the embankment stability analysis is discussed in Section 2.5C.2 of this Appendix.

BORROW AREA TEST PITS

Fourteen test pits were excavated in proposed borrow areas and a total of two test trenches and six shallow borings were made in foundation soils for the Auxiliary Dam and Auxiliary Reservoir Separating Dike. Representative soil samples were obtained from each pit, four undisturbed block samples were obtained from the trenches, and six thin-wall Shelby tube samples were obtained from the borings.

The test pit logs are given in Appendix 2.5A.

TEST PIT COMPOSITE SAMPLES

An approximate 300-lb. representative sample was obtained from each test pit. Each sample contained the proper proportion of the different types of soil observed in the pit.

BORROW AREA COMPOSITE SAMPLES

Selected composite test pit samples were mixed to prepare the composite sample for each borrow area. Selection was based on inspection of the test pits, examination of the test pit composite samples, the results of grain-size analyses and liquid and plastic limit tests performed on the composite test pit samples, and the index property of samples from borings previously made in the borrow areas.

a) Main Dam Composite Sample (M)

Four of the six test pit composite samples were used to prepare the representative composite sample for the main dam borrow area. The samples from TPM4A and TPM5A were not used because they were primarily cohesionless with very low plasticity and based on grain-size analysis, were unrepresentative of the soils encountered in the other four pits or in the borings made in the borrow area. It is noted that these exploratory test pits were made to establish the extent of the borrow area suitable for use as construction material.

The percent passing the No. 200 sieve, the liquid limit, and the plasticity index obtained for each thoroughly mixed test pit composite sample are summarized below.

Composite Sample	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index
TPM1	45	29	7
TPM2	40	26	6
TPM3A	42	29	12
TPM4A	36	21	2
TPM5A	38	23	4
TPM6	42	28	4

Approximately 1000 lb. of the composite main dam borrow sample was prepared by mixing the test pit composite samples from TPM1, TPM2, TPM3A, and TPM6. Index property tests were performed on the composite samples. These tests consisted of grain-size analyses, liquid and plastic limit and specific gravity determinations, and standard compaction tests, ASTM D698-68T Method A. The results of these tests are given on page 2.5C.1-5 and summarized as follows.

Main Dam Composite Borrow Sample

Red-brown silty clayey coarse to fine sand with trace of fine gravel

Unified Soil Classification	SC
% Passing No. 200 Sieve	44
Liquid Limit	33
Plastic Limit	22
Plasticity Index	11
Standard Maximum Dry Unit Weight, lb./ft.^{3}	121.8
Standard Optimum Water Content, %	12.4
Specific Gravity	2.72

b) Auxiliary Dam Composite Sample (Z)

All four of the test pit composite samples were used to prepare the representative composite sample for the auxiliary dam borrow area Z. The percent passing the No. 200 sieve, liquid limit and plasticity index for each thoroughly mixed test pit composite sample are summarized below.

Composite Sample	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index
TPZ1	47	28	6
TPZ2	79	34	11
TPZ3	80	24	4
TPZ4	82	29	7

Approximately 1000 lb. of the composite auxiliary dam borrow sample was prepared by mixing the test pit composite samples. Index property tests were performed on the composite sample. The results of these tests are given on page 2.5C.1-7 and summarized as follows.

Auxiliary Dam Composite Borrow Sample (Z)

Brown silty clay with some coarse to fine sand with trace of fine gravel

Unified Soil Classification	CL
% Passing No. 200 Sieve	68
Liquid Limit	35
Plastic Limit	22
Plasticity Index	13
Standard Maximum Dry Unit Weight, lb./ft.^{3}	114.1
Standard Optimum Water Content, %	15.5
Specific Gravity	2.73

c) Auxiliary Dam Composite Sample (Y)

All four of the test pit composite samples were used to prepare the representative composite sample for the auxiliary dam borrow area Y. The percent passing the No. 200 sieve, liquid and plasticity index obtained for each thoroughly mixed test pit composite sample are summarized below.

Composite Sample	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index
TPY1	79	32	8
TPY2A	88	37	11
TPY3	67	29	9
TPY4A	86	33	8

Approximately 1000 lb. of the composite auxiliary dike borrow sample was prepared by mixing the test pit composite samples.

Index property tests were performed on the composite sample. The results of these tests are given on page 2.5C.1-6 and summarized as follows.

Auxiliary Dam Composite Borrow Sample (Y)

Red brown silty clay with some coarse to fine sand

Unified Soil Classification	CL
% Passing No. 200 Sieve	80
Liquid Limit	37
Plastic Limit	24
Plasticity Index	13
Standard Maximum Dry Unit Weight, lb./ft.^{3}	114.8
Standard Optimum Water Content, %	15.4
Specific Gravity	2.76

d) Plasticity Index Comparison

Plasticity index values of the composite borrow samples were estimated on the basis of visual and manual examination of the composite test pit soils and calculated as the average value of the soils obtained from the borings in each borrow area. These values and the values obtained in the laboratory are given below.

	Composite Borrow Samples Plasticity Index Values		
	Main Dam	Auxiliary Dam (Z)	Auxiliary Dam (Y)
Estimated from composite test pit soils	13	11	12
Calculated average of samples from borings	13	15	11
WMAI laboratory tests	11	13	13

The plasticity index values from laboratory tests on the composite borrow area samples are in essential agreement with the estimated and calculated average values which give confirmation that the composite soils are representative of their respective borrow areas.

TEST TRENCHES

Two test trenches were excavated with a Case 580B backhoe for the purpose of obtaining undisturbed representative block samples of the foundation soils for the Auxiliary Dam. The

trenches, identified as TPA1 and TPA2, were excavated in the foundation for the Auxiliary Dam. Each trench was inspected and logged.

The trench logs are given in Appendix 2.5A.

UNDISTURBED BLOCK SAMPLES

Four undisturbed block samples were recovered from the test trenches; three from TPA1 and one from TPA2 at the auxiliary dam site. The block samples were cut in-situ to approximately 1-ft. cubes and sealed with wax in wooden boxes.

SHELBY TUBE SAMPLES

Six 3-in.-diameter thin-wall Shelby tube samples were taken from borings located 10 ft. to 20 ft. from the test trenches. The borings were made in groups of three near trenches TPA1 and TPA2. The borings in each group were approximately two ft. apart and each sample was taken from two ft. to four ft. below ground surface which is generally above the depths at which the block samples were taken.

2.5C.2 LABORATORY TESTING PROGRAM FOR STATIC AND DYNAMIC ENGINEERING PROPERTIES

INTRODUCTION

This section presents results of laboratory investigation of the static and dynamic properties of the core material of the Main Dam (Material M), and the core material (Material Z) and foundation soils for the Auxiliary Dam and Auxiliary Reservoir Separating Dike. Results of these tests were utilized for the evaluation of the seismic stability of the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike during the preconstruction stage of engineering and design. Subsequent changes in borrow areas and material properties criteria are discussed in Appendix 2.5F, "Report on Embankments". The scope of the testing program and typical test results are presented below. Additional preconstruction test results are presented in Appendix 2.5D.

TEST PROGRAM

a) General

Tests were conducted on reconstituted specimens of composite materials M and Z, and undisturbed samples of foundation soils from the auxiliary dam area. Sampling locations are are described in Section 2.5.1.2.5, and index properties of these materials are given in Section 2.5C.1 of this Appendix.

b) Scope

The test program consisted of the following:

1) Static Tests

a) index properties

b) isotropically consolidated drained (CID) triaxial tests

c) unconsolidated undrained (UU) triaxial tests

2) Dynamic Tests

a) stress-controlled cyclic triaxial tests

b) strain-controlled cyclic triaxial tests

c) cyclic torsion tests

c) Preparation of Specimens

Material M - Static tests were performed on specimens compacted to 100 percent standard compaction at optimum water content (γ_d = 121.8 lb./ft.^{3}, ω = 12.4%). Dynamic tests were performed on specimens compacted at 100 percent standard compaction at optimum water content. A few dynamic tests were also conducted on specimens compacted at 95 percent standard compaction at optimum water content.

Material Z - Static tests were conducted on specimens compacted to 97 percent standard compaction at optimum water content (γ_d = 11.6 lb./ft.^{3}, = 15.5%). Dynamic tests were performed on specimens molded at 97 percent standard compaction and optimum water content. A few dynamic tests were also performed on specimens compacted at 95 percent and 100 percent standard compaction at optimum water content.

d) Foundation Soils in the Auxiliary Dam Area

Stress-controlled cyclic triaxial tests were conducted on undisturbed block and tube samples of foundation soils from the auxiliary dam area.

TEST RESULTS

a) Main Dam - Material M

1) Static Triaxial Tests -Results of static CID tests are summarized on pages 2.5C.2-14 and 2.5C.2-15. Results of static UU tests are presented on pages 2.5C.2-16 and 2.5C.2-17. These results are being utilized to determine parameters for static finite element analysis, namely, the modulus number K, modulus exponent n, and Poisson's ratio parameters G, F, and D (see Table 2.5C.2-1).

The incremental finite element static stress analysis is based on nonlinear, stress-dependent stress-strain behavior of soils. The following relationships are used to define the inelastic behavior.

1) Primary Loading

Tangent modulus Et for primary loading (Kondner) (References 2.5C.2-1 and 2.5C.2-2).

$$E_t = \left[1 - \frac{R_f(1 - \sin\theta)(\sigma_1 - \sigma_3)}{2c\ \cos\theta + 2\sigma_3 \sin\theta}\right]^2 E_i$$

where c and θ are the Mohr-Coulomb shear strength parameters, E_i is the initial tangent modulus, and R_f is the failure ratio or ratio between the compressive strength $(\sigma_1 - \sigma_3)_f$ and the value of the asymptotic stress difference for the hyperbolic stress-strain curve $(\sigma_1 - \sigma_3)_{ult}$.

The variation of the initial tangent modulus value with confining pressure is represented by an empirical equation (Janbu) (Reference 2.5C.2-3).

$$E_i = Kp_a\left(\frac{\sigma_3}{P_a}\right)^n$$

in which the modulus number K and exponent n are both pure numbers and P_a is the value of atmospheric pressure expressed in appropriate units.

2) Volume Changes

Kulhawy, Duncan, and Seed (Reference 2.5C.2-4) have developed a procedure for incorporating in the stress analysis the volume change characteristics of soil in terms of tangent Poisson's ratio. The initial Poisson's ratio may be expressed by:

$$u_i = G - F\ log\ \frac{\sigma_3}{P_a}$$

The value of tangent Poisson's ratio may be expressed by

$$u_t = \frac{u_i}{\left[1 - \dfrac{D\ (\sigma_1 - \sigma_3)}{Kp_a\left(\frac{\sigma_3}{P_a}\right)^n\ \ 1 - \dfrac{R_f\ (\sigma_1 - \sigma_3)\ (1 - \sin\theta)}{2c\ \cos\theta + 2\sigma_3\ \sin\theta}}\right]^2}$$

where G, F, and D are parameters.

2) Dynamic Tests

(a) Stress controlled Cyclic Triaxial Tests - A total of 31 tests were conducted on specimens compacted at 100 percent standard compaction and 10 tests on specimens compacted at 95 percent standard compaction. The tests were performed by using effective confining pressures ($\sigma_{3c)}$ of 2000, 4000, 8000, and 12000 lb./ft.^{2} and initial consolidation ratios K_c = 1, 1.5, and 2. The specimens were subjected to different ratios of cyclic deviator stress to effective confining pressures ($\sigma_d\sqrt{\sigma_{3c}}$) and the number of cycles (N) causing five percent and ten percent strains and the point where the pore pressure becomes equal to the cell pressure were determined. The results of these tests are summarized in Table 2.5C.2-2. Pages 2.5C.2-18 through 2.5C.2-20 show the relationship between

the stress ratio($\sigma_d/2\sigma_{3c}$) and the number of cycles causing five percent strain for K_c = 1, 1.5, and 2, respectively.

(b) Strain-controlled Cyclic Triaxial Tests - Four strain-controlled cyclic triaxial tests were performed on specimens compacted at 100 percent standard compaction. Results of these tests together with those from cyclic torsion tests were used for determining the dynamic shear modulus and damping for the material (see Table 2.5C.2-3).

(c) Cyclic Torsion Tests - Cyclic torsion tests have been performed using confining pressures of 500, 1000, 2000, and 4000 lb./ft.^{2}. As noted above, results of these tests have been utilized for determining the modulus and damping characteristics of the material (see Table 2.5C.2-4).

b) Material Z

1) Static Tests -Results of the static CID and UU tests on specimens compacted to 97 percent standard compaction are presented on pages 2.5C.2-21 through 2.5C.2-24. Parameters derived from the CID tests for use in static finite element analysis are given on Table 2.5C.2-5.

2) Dynamic Tests

(a) Stress-controlled cyclic triaxial tests - A total of 24 cyclic triaxial tests were conducted on specimens compacted at 95 percent, 97 percent, and 100 percent standard compaction. Confining pressure of 1250, 2500, and 5000 lb./ft.^{2} were used and initial consolidation was carried out for K_c = 1, 1.5, and 2. As in the case of material M, the number of cycles causing five percent and ten percent strain and the point where the pore pressure is equal to the cell pressure were determined. Results of tests on material compacted at 97 percent are summarized on pages 2.5C.2-25 through 2.5C.2-27 and Table 2.5C.2-6.

(b) Strain-controlled cyclic triaxial tests -Four strain-controlled cyclic triaxial tests were performed. Results of these tests are presented on Table 2.5C.2-7. Data obtained from these tests together with those from the cyclic torsion tests are being utilized for determining the shear modulus and damping characteristics of the material.

(c) Cyclic torsion tests - These series of cyclic torsion tests have been completed. Results of these tests are presented on Table 2.5C.2-8.

c) Foundation Soils in the Auxiliary Dam Area

Fourteen stress-controlled cyclic triaxial tests were conducted on undisturbed samples of foundation soils from the auxiliary dam area. Results of these tests are given on Table 2.5C.2-9.

d) Seismic Wave Velocity Measurements at Auxiliary Dam and Auxiliary Dike

1) Introduction

Seismic wave velocity measurements were made at locations along the axes of the Auxiliary Dam and Auxiliary Reservoir Separating Dike during the period 5 to 8 February 1973. The main purpose of the measurements was to determine compression-wave (P-wave) velocities (V_p), shear-wave (S-wave) velocities (V_S), and Rayleigh-wave (R-wave) velocities (V_R)of in-situ residual soil; however, the seismic wave velocity of the transitional material and the upper portion of the weathered rock were also determined. Seismic wave velocities of weathered and fractured rock and sound bedrock are shown in Section 2.5.2. Methods used to measure seismic wave velocities are discussed below.

2) Methods Used to Measure Seismic Wave Velocities

Two methods were used to measure seismic wave velocities: (1) pulse arrival measurements of compression-wave (P-wave) velocity ($V_{p)}$ and shear-wave (S-wave) velocity (V_S); and (2) steady-state vibration measurements of Rayleigh-wave (R-wave) velocities (V_R).

Pulse arrival measurements were made using a Sprengnether VS 1200 seismograph and a three-component geophone; a sledge hammer impact was used as the energy source; and Electrotech vertical geophone, located adjacent to the impact station, was used to provide zero time.

Pulse arrivals were recorded for both vertical and horizontal impacts; several records were made at each location to examine repeatability of each measurement. The three-component geophone records the propagated seismic waves in three planes at right angles.

P-wave pulse arrivals are measured by the horizontal component of the geophone that is oriented along the line between the geophone and the impact station. S-wave pulse arrivals are measured by the two components of the geophone oriented perpendicular to the line between the geophone and impact station.

To create a maximum of S-wave energy, horizontal impacts are oriented perpendicular to the direction of the measurement line; in addition, this minimizes the P-wave energy. By reversing the impact direction, the S-wave is reversed; and by comparing the two records which are symmetrical with respect to the time axis, accuracy of the interpretation is increased.

The impact-to-receiver distance is measured to an accuracy of ± 0.1 ft.; and the time of the first pulse arrival is scaled from the records to an accuracy ± 1 millisecond (msec.). Velocities are calculated by dividing the impact-to-receiver distance by the time.

Steady-state vibration measurements were made using a Heathkit audio generator (1 G-72), a Dyna Kit Mark III preamplifier, and a Goodman vibrator to generate R-waves. The velocities of the R-waves were measured using two Electrotech EV-17 vertical geophones and their response observed on a Tectronix R-5030 dual beam oscilloscope.

R-wave velocities were measured by determining the frequency required to create in-phase response of two geophones spaced at a selected distance. The frequency is varied in increments of one hertz by the audio generator. Geophone response is displayed on the oscilloscope screen, and in-phase response is determined to an accuracy ± 0.5 hz for frequency ranges of approximately 30 hz to 110 hz, and ± 5 hz above frequencies 110 hz. The distance d between geophones is measured to an accuracy of ± 0.1 ft.

The frequency of the vibrator is varied until in-phase response is obtained. At that frequency f, the distance d is a multiple of the wave length. The frequency is then progressively increased to f' which corresponds to the next higher in-phase response. The R-wave velocity is equal to V_r = d (f' - f).

3) Description of Materials

(a) Residual Soil - Residual soil includes materials which range from silty clay to silt to silty sand. As the residual soil resulted from differential weathering of the underlying bedrock, the gradation from soil to weathered rock is not clearly defined. Within the residual soil layer, there are 2-to 3-ft. thick layers of rock weathered to different degrees which are referred to as transitional material.

(b) Weathered Rock - Weathered rock includes medium hard to hard sandy to clayey siltstone, and medium to fine-grained silty sandstone. The classification used to define the weathered rock, for purposes of these seismic-wave velocity measurements, is that material which, when excavated with extreme difficulty with a backhoe, breaks down to medium hard rock fragments.

4) Results of Seismic-Wave Velocity Measurements

(a) Auxiliary Dam - Seismic velocity measurements at the site of the auxiliary dam were made at the location shown on page 2.5C.2-28.

P-wave and S-wave velocity measurements were made for impact receiver spacings of: (1) 15 ft. and 20 ft. in the residual soil layer; (2) 10 ft. and 20 ft., and 12.7 ft. and 22 ft. in the transitional material; (3) 12 ft. and 26 ft. in the weathered sandstone. Measurement locations were selected based on their representative nature; the average materials. Results of measurements are presented on Tables 2.5C.2-9 through 2.5C.2-12. Range of measurements is very narrow, indicating an excellent accuracy. For the residual soil and transitional materials, we consider these measurements are representative of the material at a depth of one-half the impact-receiver spacing for P-wave and S-wave measurements in the residual soil, and a depth of one half the wave length for R-wave measurements. For the weathered rock, we consider that the measurements are representative of the material at a depth determined as above or at a depth equal to the thickness of the layer, whichever is smaller.

The average P-wave and S-wave velocities of the residual soil are:

Impact-Receiver Spacing, ft.	Average Velocities, ft./sec. V_p	V_s
15	1505	750
20	1420	705

R-wave velocity measurements for the residual soil are:

Wave Average Velocity, Corresponding V_S

Length,ft.	ft./sec.	$V_s = V_r \times 1.1$, ft./sec.
10.2	620	680
6.1	605	665

The average P-wave and S-wave velocities of the transitional material are:

	Average Velocities, ft./sec.	
Impact-Receiver Spacing, ft.	V_p	V_s
10	1785	1000
20	2385	1335
22	3060	1380
12.7	2395	1065

The average P-wave and S-wave velocities of the weathered rock are:

	Average Velocities, ft./sec.	
Impact-Receiver Spacing, ft.	V_p	V_s
12	2400	1335
26	3250	1445

The average R-wave velocity measurements for the weathered rock are 1315 ft./sec. and 1570 ft./sec. The corresponding S-wave velocities for the weathered rock are:

V_S = 1315 x 1.1 = 1445 ft./sec.

and

V_S = 1570 x 1.1 = 1725 ft./sec.

(b) Auxiliary Reservoir Separating Dike - Seismic velocity measurements at the site of the Auxiliary Reservoir Separating Dike were made at the location shown on page 2.5C.2-29.

P-wave and S-wave velocity measurements were made for impact-receiver spacings of 25 ft. for residual soil and 20 ft. for weathered rock. Measurement locations were selected based on their representative nature; the average values measured are believed to apply to the average materials. Average velocities for residual soil are V_P = 1300 ft./sec. and V_S = 715 ft./sec. average velocities for weathered rock are V_P = 3590 ft./sec. and V_S = 2055 ft./sec. Results of measurements are presented on Table 2.5.C.2-13. We consider these measurements representative of material at a depth of approximately one-half the impact-receiver spacing.

CALCULATION OF $K_{2,max}$ ON THE BASIS OF SHEAR WAVE VELOCITIES

The equation relating $K_{2,max}$ to the shear wave velocity V_S is

$$V_s^2 \frac{\gamma}{g} = 10^3 K_{2,max} \bar{\sigma}_o 1/2$$

where V_S = shear weight velocity, ft./sec.

γ = unit weight of material, lb./ft.^{3}

g = acceleration of gravity = 32.2 ft./sec.^{2}

$\bar{\sigma}_o$ = mean effective pressure, lb./ft.^{2}

The unit weights of the materials are selected as follows:

Residual Soil	γ = 135 lb./ft.^{3}
Transitional Material	γ = 142.5 lb./ft.^{3}
Weathered Rock	γ = 150 lb./ft.^{3}

The mean effective pressure is given by the equation:

$$\bar{\sigma}_o = \frac{\sigma_v + 2\sigma_h}{3}$$

where σ_v = vertical effective stress

σ_h = horizontal effective stress

Selecting a horizontal to vertical effective stress ratio K = 0.6, the mean effective pressure is given by the equation:

$$\bar{\sigma}_o = \gamma d \, \frac{1+2 \, x \, 0.6}{3} = 0.733 \, \gamma d$$

where d is the depth below ground surface

For residual soil $\bar{\sigma}_o$ = 0.733 x 135 d and $\bar{\sigma}_o{}^{1/2}$ = 9.95 $d^{\,1/2}$

For transitional material $\bar{\sigma}_o$ = 0.733 x 142.5 d and $\bar{\sigma}_o{}^{1/2}$ = 10.2 $d^{\,1/2}$

For weathered rock $\bar{\sigma}$ = 0.733 x 150 d and $\bar{\sigma}^{1/2}$ = 10.5 $d^{1/2}$

$K_{2,\,max}$ is given by the equations below:

For the residual soil $K_{2,max} = \frac{4.20}{9.95} \, 10^{-3} \, \frac{V_s^2}{d^{\frac{1}{2}}} = 4.22 \times 10^{-4} \, V_s^2/d^{1/2}$

For transitional material $K_{2,max} = \frac{4.43}{10.2}\ 10^{-3}\ \frac{V_s^2}{d^{\frac{1}{2}}} = 4.34\ \times\ 10^{-4}\ V_s^2/d^{1/2}$

For weathered rock $K_{2,max} = \frac{4.66}{10.5}\ 10^{-3}\ \frac{V_s^2}{d^{\frac{1}{2}}} = 4.44\ \times\ 10^{-4}\ V_s^2/d^{1/2}$

The values of $K_{2,max}$ are given on Table 2.5C.2-14 for the shear wave velocities measured at the sites of the Auxiliary Dam and Auxiliary Reservoir Separating Dike.

e) Material Properties Used For Dynamic Analyses of the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike

1) Introduction

This section presents the values of material properties which are used for the dynamic analyses of the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike. These values are expected for the materials in the constructed dams and dike. Values for the Main Dam are presented on Table 2.5C.2-15 and those for the Auxiliary Dam and Auxiliary Reservoir Separating Dike are presented on Table 2.5C.2-16.

2) Basis for Selection of Material Properties for Dynamic Analyses of Main Dam

a) Unit Weights

(1) Core (Material M) - Based on laboratory tests.

(2) Fine Filter and Coarse Filter -Based on specified gradations, expected constructed relative density of 80 percent and several published (e.g., Burmister 1962) (Reference 2.5C.2-5) and unpublished (e.g., WMAI 1973) (Reference 2.5C.2-13) data regarding unit weight of granular material.

(3) Rockfill Shell -Based on in-place unit weight measurements of a rockfill at Keban Dam, Turkey (Ebasco 1972) (Reference 2.5C.2-6) with properties similar to those of the proposed rockfill.

(4) Weathered Rock - Based on laboratory tests.

b) Ratio of Horizontal Effective Stress to Vertical Effective Stress K_o

Values of ratios of horizontal effective stresses to vertical effective stresses of compacted and preconsolidated materials are available in several publications (e.g., D'Appolonia et. al., 1969, Lacroix and Horn 1973) (References 2.5C.2-7 and 2.5C.2-8). Relatively high values are selected to account for compaction of the embankment materials and preconsolidation in the weathered rock.

c) Poisson's Ratio μ

Typical values of Poisson's ratio are available in several publications (e.g., Leonards 1962, Barkan 1962) (References 2.5C.2-9 and 2.5C.2-10).

d) Shear Modulus Parameter $K_{2,max}$

(1) Core (Material M) - Based on cyclic torsion tests and cyclic triaxial tests. Tables 2.5C.2-17 and 2.5C.2-18, and pages 2.5C.2-30 and 2.5C.2-31 show the test results.

(2) Fine Filter - Based on published data (Seed and Idriss 1970) (Reference 2.5C.2-11). Fine filter will be sand with expected constructed relative density of 80 percent; see page 2.5C-72.

(3) Coarse Filter - Based on published data (Seed and Idriss 1970, Wong 1970) (References 2.5C.2-11 and 2.5C.2-12). Coarse filter will be gravel with sand having an expected constructed relative density of 80 percent.

(4) Rockfill Shell - Because the maximum shear modulus of gravel is higher than that of sand, a maximum shear modulus for rockfill higher than that of gravel is selected by extrapolation of available data; see page 2.5C.2-32.

(5) Weathered Rock - Based on seismic compression-wave velocity measurements in the weathered rock along the main dam axis; see page 2.5C.2-32.

e) Damping Ratio λ

(1) Core (Material M) - Based on cyclic torsion tests and cyclic triaxial tests. Tables 2.5C.2-17 through 2.5C.2-19 and page 2.5C.2-33 show the test results.

(2) Fine Filter - Fine filter will be sand. Average damping ratio for sands (Seed and Idriss 1970) (Reference 2.5C.2-11) is used; see page 2.5C.2-34.

(3) Coarse Filter - Coarse filter will be gravel with sand. Published data (Wong 1970) (Reference 2.5C.2-12) indicate that damping ratios of gravels and sands are similar. Average damping ratio for sands is used; see page 2.5C.2-34.

(4) Rockfill Shell and Weathered Rock - Average damping ratio for sands is used; see page 2.5C.2-34.

3) Basis for Selection of Material Properties for Dynamic Analyses of Auxiliary Dam and Auxiliary Reservoir Separating Dike

a) Unit Weights

(1) Core (Material Z) - Based on laboratory tests.

(2) Filter - Based on specified gradation, expected constructed relative density of 80 percent, and several published (e.g., Burmister 1962) (Reference 2.5C.2-5) and unpublished (e.g., WMAI 1973) (Reference 2.5C.2-13) data regarding unit weight of granular material.

(3) Random Rockfill -Based on in place unit weight measurements of a rockfill at Amos Dam, West Virginia (WMAI 1973) (Reference 2.5C.2-3), with properties

similar to those of the proposed rockfill. Unit weight of siltstone and sandstone random rockfill is selected slightly smaller than that of the granitic rockfill of the Main Dam.

(4) In-situ Soils - Based on laboratory tests.

(5) Weathered Rock - Based on laboratory tests.

b) Ratio of Horizontal Effective Stress to Vertical Effective Stress K_o

Values of horizontal effective stress to vertical effective stress of compacted and preconsolidated materials are available in several publications (D'Appolonia et. al. 1969, Lacroix and Horn 1973) (References 2.5C.2-7 and 2.5C.2-8). Relatively high values are selected to account for compaction of the embankment materials and preconsolidation in the weathered rock.

c) Poisson's Ratio

Typical values of Poisson's ratio are available in several publications (e.g., Leonards 1962, Barkan 1962) (References 2.5C.2-9 and 2.5C.2-10).

d) Shear Modulus Parameter $K_{2,max}$

(1) Core (Material Z) - Based on cyclic torsion tests and cyclic triaxial tests. Tables 2.5C.2-20 and 2.5C.2-21, and pages 2.5C.2-35 and 2.5C.2-36 show the test results.

(2) Filter -Based on published data (Seed and Idriss 1970, Wong 1970) (References 2.5.2-11 and 2.5C.2-12). Filter will be a gravelly sand with expected constructed relative density of 80 percent; see page 2.5C.2-32.

(3) Random Rockfill -Based on seismic wave velocities measured at Amos Dam, West Virginia (WMMAI 1973) (Reference 2.5C.2-13), with properties similar to those of the proposed random rockfill. At Amos Dam, P-wave, S-wave, and R-wave velocities of the rockfill lead to $K_{2,max}$ of approximately 100; the value selected for the Auxiliary Dam and Auxiliary Reservoir Separating Dike random rockfills reflects the higher compaction specified for these random rockfills than for the Amos Dam rockfill; see page 2.5C.2-32.

(4) In-situ Soil - Based on P-wave, S-wave, and R-wave velocities of the in-situ soil at several locations along the axes of the Auxiliary Dam and the Auxiliary Separating Reservoir Dike; see page 2.5C.2-32.

(5) Weathered Rock - Based on the seismic velocities in the weathered rock along the axis of the Auxiliary Dam; see page 2.5C.2-32.

e) Damping Ratio λ

(1) Core (Material Z) - Based on laboratory torsion test and cyclic triaxial test data. Tables 2.5C.2-20 through 2.5C.2-22, and page 2.5C.2-37 show the test results.

(2) Filter -Filter will be gravelly sand. Published data (Wong 1970) (Reference 2.5C.2-12) indicate that damping ratios of gravels and sands are similar. Average damping ratio for sands is used; see page 2.5C.2-34.

(3) Random Rockfill, In-situ Soils, and Weathered Rock - Average damping ratio for sands is used; see page 2.5C.2-34.

(4) Parametric studies -In the case of the core materials, the range of parametric variations was selected on the basis of laboratory test results. In the case of the filters, the range was selected on the basis of published data (Seed and Idress 1970, Wong 1970) (References 2.5C.2-11 and 2.5C.2-12). In the case of rockfill materials, the range was selected on the basis of material index properties. In the case of weathered rock the range was selected on the basis of seismic velocity measurements. For all materials the selection was made taking into account design and construction criteria. Table 2.5C.2-23 through 2.5C.2-26 show the material-property combinations used for the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike respectively.

CONCLUSION

Through the preconstruction stage of the SHNPP project, these results formed the basis for defining the static and dynamic properties of the respective materials and were used for the evaluation of seismic stability of the Main Dam, Auxiliary Dam, and the Auxiliary Reservoir Separating Dike.

Static and dynamic properties utilized in the stability analyses are further discussed in Appendix 2.5D, together with the results of the analyses.

Changes in borrow areas, and material properties criteria which became necessary after construction began are discussed in Appendix 2.5F, "Main and Auxiliary Dams and Auxiliary Reservoir Separating Dike Embankment Reports."

2.5C.3 RESULTS OF TESTS ON SAMPLES FROM BORINGS IN BORROW AREAS

The tests performed on samples from Borrow Areas Y and Z and Main Dam Borrow Area M are indicated on the matrices shown on pages 2.5C.3-2 through 2.5C.3-5. Pages 2.5C.3 6 through 2.5C.3 223 give the results of these tests. Tables 2.5C.3-1, 2 and 3 show the shrinkage factors for samples from Borrow Areas Y, Z and Main Dam Borrow Area M, respectively.

BORROW AREA Y - TESTING PROGRAM

Boring Number	Sample Number	Grain Size Analysis	Triaxial Shear Test	Proctor Compaction Test
BB101	S-1	*		
	S-2	*		
	S-6	*		
BB103	S-1	*		
	S-2	*	*	*
	S-4	*		
BB105	S-1	*	*	*
	S-2	*		
BB107	S-1	*		
	S-2	*		
BB109	S-1	*		
	S-4	*		
BB110	S-2	*	*	*
BB111	S-1	*	*	*
BB113	S-1	*		
	S-2	*	*	*
	S-3	*	*	*
BB114	S-1	*	*	*
	S-2	*		
BB116	S-1	*		
	S-2	*		
	S-4	*		
BB117	S-1	*	*	*
	S-2	*	*	*
	S-3		*	*
	S-4	*	*	*
BB119	S-1	*		
	S-2	*	*	*
BB121	S-1	*		
	S-2	*		
	S-4	*		
BB124	S-1		*	*
	S-2	*		*
	S-4	*		*
BB125	S-1	*	*	*
	S-2	*	*	*

BORROW AREA Z - TESTING PROGRAM

Boring Number	Sample Number	Grain Size Analysis	Triaxial Shear Test	Proctor Compaction Test
BB156	S-1	*	*	*
BB156	S-2	*		
BB158	S-1	*	*	*
BB158	S-2	*		
BB159	S-1	*		*
BB159	S-2		*	*
BB159	S-3	*	*	*
BB159	S-4		*	*
BB161	S-1	*		
BB161	S-4	*	*	*
BB163	S-2	*	*	*
BB166	S-1	*		
BB166	S-2	*		
BB167	S-1	*		
BB171	S-1	*		*
BB171	S-2	*		*
BB176	S-1	*		*
BB176	S-2	*		*
BB176	S-3	*	*	*
BB176	S-4	*	*	*
BB176	S-5		*	*
BB177	S-1	*		
BB177	S-2	*		
BB178	S-1			
BB178	S-2	*		
BB178	S-4	*		
BB180	S-1	*	*	*
BB180	S-2	*		
BB184	S-1	*		
BB184	S-2	*		
BB186	S-1	*		
BB186	S-2	*		

MAIN DAM BORROW AREA TESTING PROGRAM

Boring Number	Sample Number	Grain Size Analysis	Triaxial Shear Test	Proctor Compaction Test
BB806	S-1	*		
	S-2	*		*
	S-3	*		*
BB807	S-1	*		*
	S-2	*	*	*
	S-4	*	*	*
BB812	S-3	*		*
BB813	S-1	*	*	*
BB814	S-1	*		*
	S-2	*		*
	S-3	*		*
	S-4	*	*	
BB815	S-1	*		*
	S-3 & S-4	*		*
BB863	S-1	*		
	S-2	*		
BB864	S-1	*		
	S-2	*	*	*
BB865	S-1	*	*	*
BB866	S-1	*	*	*
	S-2	*		
	S-4	*		
BB867	S-1	*		
BB868	S-1	*		
	S-2	*		
BB869	S-1	*		
	S-2	*		
	S-3	*	*	*

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2.5C.2-2 "Hyperbolic Stress-Strain Formulation for Sands," R. L. Kondner, and J. S. Zelasko, Proceedings of Second Pan American Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1963.

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2.5C.2-12 "Deformation Characteristics of Gravels and Gravelly Soils Under Cyclic Loading Conditions," R. T. Wong, Ph. D. Thesis, Univ. of Calif., Berkeley, 1970.

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APPENDIX 2.5D SEISMIC STABILITY ANALYSIS OF SEISMIC CATEGORY I DAMS AND DIKE

2.5D.0 PURPOSE AND CONCLUSION

2.5D.0.1 Introduction

Appendix 2.5D describes the seismic stability analyses of the Seismic Category I dams and dike. The first seismic analysis of the SHNPP reservoir system was for the original cooling system, which included a 10,000-acre lake and afterbay; this analysis is described in the following paragraphs of Section 2.5D.0 and Sections 2.5D.1 through 2.5D.8. After the SHNPP reservoir system was redesigned for cooling tower operation, the Main Reservoir was reduced in size from 10,000 acres to approximately 4,000 acres (from NWL Elevation 250 ft. to NWL Elevation 220 ft.) and the Afterbay Reservoir was deleted. The seismic stability analysis of the reservoir following these changes is contained in Section 2.5D.9.

Appendix 2.5D describes the seismic stability analyses of three loading conditions. Loading Condition I corresponds to normal operating conditions and the occurrence of the SSE for the Elevation 250 ft. lake. Condition II involves low water conditions and the occurrence for the OBE for the Elevation 250 ft. lake. Condition III corresponds to normal operating conditions and the occurrence of the SSE for the Elevation 220 ft. lake (the present design).

The material properties and engineering properties used in the analysis of all three conditions are discussed in Sections 2.5D.10 through 2.5D.16. However, as a result of further

investigation and data developed during construction, Borrow Area M (described in Section 2.5D.10) was not used. Instead, Borrow Area W was used for the Main Dam impervious core. A description of this material and a justification for its use is given in Appendix 2.5F.

Also, there was not sufficient material for the Main Dam rockfill in the excavations for the spillway, spillway outlet, and spillway approach channel (described in Section 2.5D.12); additional Main Dam rockfill was obtained from a quarry located upstream of the dam, as described in Appendix 2.5F.

The present dimensions of the dam and dike and the discussion of construction of the embankments are contained in Appendix 2.5F. The report on foundations and grouting is in Appendix 2.5E. Preconstruction investigations are described in Appendices 2.5A, 2.5B, 2.5C, 2.5G, and 2.5H. Specifications for the dams and dikes are contained in Appendix 2.5I.

A location map showing the plant area, the Main Reservoir, the Main Dam, the Auxiliary Reservoir, the Auxiliary Dam, and the Auxiliary Separating Dike is presented in Figure 2.5D-1.

2.5D.0.2 Category I Dams: Main Dam, Auxiliary Dam, and Auxiliary Separating Dike

The Main Dam, Auxiliary Dam, and Auxiliary Separating Dike, which have heights generally smaller than one hundred feet, are normal earth and rockfill dams. The design of the Category I dams is in accord with the general practices of government agencies, such as the Army Corps of Engineers, the United States Bureau of Reclamation, and state departments of conservation.

There are several aspects of the design and construction criteria of these Category I dams which make these dams much more conservative than most existing dams.

a) The outside slopes of the dams are generally flatter than those commonly used. Dams higher than the Main Dam with similar rockfill shells of granitic rock often have outside slopes of 1.5 horizontal to 1 vertical (1.5:1). The Main Dam outside slopes are 2.0:1. Dams higher than the Auxiliary Dam and Auxiliary Reservoir Separating Dike with similar random rockfill shells built of compacted sedimentary rock often have outside slopes between 1.5:1 and 2.0:1. The Auxiliary Dam and Auxiliary Reservoir Separating Dike outside slopes are 2.5:1.

b) The rockfill shells of the Category I dams are placed in layers and compacted, whereas common practice has been to construct rockfill by dumping with or without sluicing and no compaction. Compaction of the rockfill results in a material with greater strength and lower compressibility.

c) At the site of the Category I dams, the foundation materials are excellent; they consist of rock or stiff residual soil. Unsatisfactory alluvial material has been removed.

d) Most importantly, the construction of the dam is supervised and inspected by competent personnel. Inspection is an essential part of earth and rockfill dam construction. The purpose of the inspection is to assure that: (a) on the basis of observation and measurements of the soil and rock materials encountered during construction, the design criteria remain valid; and (b) compliance with the construction criteria is achieved.

2.5D.0.3 Conventional Static and Dynamic Stability Analyses of the Category I Dams

Static and dynamic stability analyses of the Category I dams were made by Ebasco using conventional methods; see Section 2.5.6.5. The static stability of the Category I dams was investigated by using conservative static properties of the materials forming the dams and the foundation materials. Results obtained indicate that the static stability of the dams is ample.

The dynamic stability of the dams was investigated using conventional methods. Results obtained indicate that the dynamic stability of the dams is ample during the SSE.

2.5D.0.4 Finite Element Seismic Stability Analyses for the Category I Dams

2.5D.0.4.1 Procedure

Because of the importance of the Category I dams, a finite element analysis was made to evaluate the seismic stability of the dams. The procedure which has been in development since the early 1960's has provided reasonable estimates of field behavior in a number of cases where dams have been subjected to strong earthquakes. The procedure has been used for some years for evaluating the seismic stability of existing dams and for the design of proposed dams. The procedure is described in Section 2.5D.3.

2.5D.0.4.2 Design Criteria and Basic Data

2.5D.0.4.2.1 Loading Conditions

The seismic stability analysis was made using three conditions of loading. Condition I corresponds to normal operating conditions and the occurrence of the safe shutdown earthquake (SSE). Condition II corresponds to other conditions and the occurrence of the operating basis earthquake (OBE). Condition III corresponds to a lower normal water surface elevation than Condition I and the occurrence of the SSE. Sections 2.5D.1 through 2.5D.8 present the analysis of seismic stability of Category I dams for Condition I and II. Section 2.5D.9 presents analysis of seismic stability of Category I dams for Condition III. Condition I and II are defined in Section 2.5D.4.5, and Condition III is defined in Section 2.5D.9.4.

2.5D.0.4.2.2 Design Basis Earthquake

The SSE originally used for the seismic stability analysis of the Category I dams and dike is shown on Figure 2.5D-7. The accelerogram selected for the analysis is an artificial accelerogram which represents a series of earthquakes applicable to the site and which has response spectra closely enveloping the smooth response spectra which define that SSE.

In addition, the behavior of the dams and dike in response to an event prescribed by the Regulatory Guide 1.60 spectra was assessed and the details are given in Section 2.5D.18.

2.5D.0.4.2.3 Material Properties

The properties of the soil and rock materials which form the dams and the properties of the foundation materials were selected on the basis of laboratory test results, field measurements, and published and unpublished data. The material properties used in the basic set of analysis correspond to those which were expected to be obtained in the constructed dams. To account

for possible variations, these material properties were varied within a conservative range and the analyses were repeated for a conservative combination of the material properties.

2.5D.0.4.2.4 Geometry of the Dams and Compaction Criteria of the Materials

The geometry of the cross sections of the dams is shown in Figures 2.5D-2 through 2.5D-6. The core, filters, and rockfill materials are placed in layers and compacted according to specified construction criteria.

2.5D.0.4.2.4.1 Main Dam

The core of the main dam is compacted to an average degree of standard compaction* of 100 percent at a water content within plus or minus two percent of optimum. The fine and coarse filters of the main dam are compacted to an average relative density** of 75 percent, except for the upstream coarse filter which is compacted to an average relative density of 80 percent above El. 220.

2.5D.0.4.2.4.2 Auxiliary Dam

The core of the auxiliary dam is compacted to an average degree of standard compaction of 97 percent at a water content within plus or minus two percent of optimum. The filters of the auxiliary dam are compacted to an average relative density of 75 percent below El. 220 and to an average relative density of 80 percent above El. 220.

2.5D.0.4.2.4.3 Auxiliary Separating Dike

The core of the auxiliary Separating dike is compacted to an average degree of standard compaction of 97 percent below El. 220 and 100 percent above El. 220.

2.5D.0.5 Evaluation of the Seismic Stability of the Main Dam, Auxiliary Dam, and Auxiliary Separating Dike

2.5D.0.5.1 Method

The seismic stability of the dams was evaluated by comparing the shear stresses, τ_f, required to cause 5 x 10^{-2} strain at any location within the dam to the shear stresses, τ_d, induced by the SSE. The ratio, τ_f/τ_d, has been considered to represent a local factor of safety against the development of 5 x 10_{-2} strain. On the basis of correlations between the results of seismic stability evaluations by this procedure and the performance of dams which have been subjected to significant earthquake loading, it has been stated that a minimum value of the stress ratio, τ_f/τ_d, greater than approximately 1.1 provides an ample margin of safety for the dams.

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*ASTM D 698-68T, Method A, B, C, or D
**Modified from ASTM D2-49-69

2.5D.0.5.2 Results

2.5D.0.5.2.1 Main Dam

For the expected constructed material properties, the minimum local factors of safety are approximately 1.8 in the core, 1.7 in the fine filters, 1.2 in the coarse filters, and 1.5 in the rockfill shell; see Section 2.5D.5.4.7, 2.5D.5.4.8, and 2.5D.5.5. The results due to an event prescribed by the Regulatory Guide 1.60 are approximately 1.7 in the core, 1.7 in the fine filters, 1.2 in the coarse filters, and 1.5 in the rockfill shell; see Section 2.5D.18. We conclude that the Main Dam will be stable and will maintain its integrity during the SSE.

2.5D.0.5.2.2 Auxiliary Dam

For the expected constructed material properties, the minimum factors of safety are approximately 1.4 in the core, 1.3 in the filter, 1.5 in the random rockfill shell, and 1.5 in the in-situ residual soil; see Section 2.5D.6.4.6, 2.5D.6.5.4, and 2.5D.6.6. The results due to an event prescribed by the Regulatory Guide 1.60 spectra are equal to the above safety factors; see Section 2.5D.18. We conclude that the Auxiliary Dam will be stable and will maintain its integrity during the SSE.

2.5D.0.5.2.3 Auxiliary Reservoir Separating Dike

For the expected constructed material properties, the minimum factors of safety are approx. 1.7 in the core, 1.9 in the random rockfill shell, and 1.8 in the in-situ residual soil; see Section 2.5D.7.4. The results due to an event prescribed by the Regulatory Guide 1.60 spectra are equal to the above safety factors; see Section 2.5D.18. We conclude that the Auxiliary Separating Dike will be stable and will maintain its integrity during the SSE.

2.5D.1 INTRODUCTION

This Appendix presents the results of seismic finite element stability analyses of Category I dams for the Shearon Harris Nuclear Power Plant. These dams are referred to as the Main Dam, the Auxiliary Dam, and the Auxiliary Reservoir Separating Dike; their locations are shown in Figure 2.5D.1. The plant is built and operated by Carolina Power & Light Company at the Shearon Harris site in Wake County, North Carolina. Ebasco Services, New York, New York, is the architect engineer.

The evaluation of seismic stability is based on dynamic finite element analyses of the dams using equivalent linear, strain-dependent material properties.

This Appendix is divided into eighteen sections. The Category I dams analyzed in the appendix are described in Section 2.5D.2. The procedure used in the evaluation of seismic stability is outlined in Section 2.5D.3. Characteristics of the safe shutdown earthquake (SSE) and operating basis earthquake (OBE) and loading conditions I and II for which the Category I dams are analyzed in this Appendix are defined in Section 2.5D.4. Evaluation of seismic stability of the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike during the SSE (loading conditions Case I) are discussed in Section 2.5D.5, 2.5D.6, and 2.5D.7, respectively. Evaluation of seismic stability of the dams during the OBE (loading condition case II) are discussed in Section 2.5D.8. Evaluation of seismic stability of the dams for Condition III are discussed in Section 2.5D.9.

Sections 2.5D.10, 2.5D.11, 2.5D.12, and 2.5D.13 present the static and dynamic material properties of the core, filter, and rockfill materials, and of the residual soil and weathered rock. The method of evaluation of static stresses in the dams is described in Section 2.5D.14. The values of the expected constructed material properties for each zone of these dams and the parametric variations from these values are presented in Section 2.5D.15.

The procedure used for the evaluation of seismic stability of the dams is described in Section 2.5D.16. A comparison of response by various alternate lumped mass analytical models is presented in Section 2.5D.17. An assessment of the behavior of the dams to an event prescribed by the Regulatory Guide 1.60 spectra is presented in Section 2.5D.18.

2.5D.2 DESCRIPTION OF CATEGORY I DAMS

2.5D.2.1 General

The seismic stability of three Category I dams is analyzed in this appendix. They are the Main Dam, Auxiliary Dam, and Auxiliary Separating Dike which are located as shown in Figure 2.5D.1.

2.5D.2.2 Main Dam

The Main Dam is approximately 1300 ft.long with a maximum height of approximately 105 ft. Three cross sections and the longitudinal section of the Main Dam are shown in Figure 2.5D-2 and 2.5D-3. The Main Dam has a core of compacted silty clayey sand protected by two 8-ft.-thick transition filter zones and a rockfill shell on each side. The core is founded on suitable rock and the rockfill shell is founded on weathered rock. (Suitable rock and weathered rock are defined in Section 2.5D.13.1).

The rock at the foundation of the Main Dam consists of granitic gneiss; see Section 2.5D.13.

2.5D.2.3 Auxiliary Dam

The Auxiliary Dam is approximately 3903 ft. long with a maximum structural height of approximately 72 ft. Three cross sections and the longitudinal section of the Auxiliary Dam are shown in Figure 2.5D.4 and 2.5D.5. The Auxiliary Dam has a core of compacted silty clay protected by a transition filter zone and a random rockfill shell on each side. The core is founded on suitable rock. In the central portion for approximately 1400 ft. the random rockfill shell is founded on weathered rock and in the remaining portions, the random rockfill is founded on a thin layer of stiff residual soil overlying weathered rock.

The rock at the foundation of the Auxiliary Dam consists of Triassic sandstones, siltstones, claystones, and conglomerates. Residual soil consisting of silty clay and sandy clayey silt occurs as a thin layer of stiff soil overlying weathered rock; see Section 2.5D.13.

2.5D.2.4Auxiliary Separating Dike

The Auxiliary Separating Dike is approximately 1100 ft. long with a maximum height of approximately 53 ft. The maximum cross section and longitudinal section of the Auxiliary Separating Dike are shown in Figure 2.5D-6. The dike has a core of compacted silty clay protected by a random rockfill shell which is graded near the core, with the finer material placed

adjacent to the core and the coarser particles placed to the outside. The core and rockfill shell are founded on a thin layer of stiff residual soil overlying weathered rock.

The rock and residual soil at the foundation of the Auxiliary Dam are the same as those at the foundation of the Auxiliary Reservoir Separating Dike; see Section 2.5D.13.

2.5D.3 PROCEDURE USED IN SEISMIC STABILITY EVALUATION

The procedure used in evaluating the seismic stability of the Category I dams consists of the following steps:

a) Determination of the response of the dam-foundation system to the rock accelerations, including the evaluation of the induced shear stresses at various locations throughout the dam and the foundation material.

b) Representation of the irregular cycles of shear stresses induced in the dam-foundation system by an equivalent number of uniform cycles of shear stresses.

c) Determination of the static stresses existing in the dam-foundation system (prior to the rock accelerations).

d) Determination of the cyclic shear stresses required to cause strains greater than 5×10^{-2} (see Section 2.5D.5 and Section 2.5D.16) in the material for conditions representative of those existing in the dam foundation system by means of appropriate cyclic load tests on representative specimens of the materials or by correlation with data for similar materials.

e) Evaluation of the seismic stability of the dams by comparing the shear stress required to cause strain greater than 5×10^{-2} with the equivalent shear stresses induced by the rock accelerations.

This procedure was recently developed (Reference 2.5D-27 and 2.5D-28) and has provided reasonable estimates of field behavior in a number of cases (Reference 2.5D-28 and 2.5D-35). The procedure also has been used for evaluating the seismic stability of several existing dams and for the design of several proposed dams in California and other locations (Reference 2.5D-38). Most recently, it was utilized in the Virgil C. Summer Nuclear Station Near Columbia, South Carolina.

Step (1) in this procedure involves the determination of the response of the dam-foundation system to postulated base rock accelerations. These postulated base rock accelerations are represented by the artificial accelerogram described in Section 2.5D.4. The response computation is performed using the finite element method of analysis (Reference 2.5D-9 and 2.5D-11). A recently developed computer program (Reference 2.5D-14) has been used to compute the response, including induced shear stresses, for the dams at the site. This program incorporates the use of strain-dependent modulus and damping ratio for each element in the finite element representation of the dam-foundation system.

Typical response, including induced shear stresses, step (1), and a brief illustration of the procedure for representing the induced shear stresses by an equivalent number of stress cycles, step (2), are presented in Section 2.5D.16.

Determination of the static stresses, step (3), is described in Section 2.5D.14. Evaluations of the cyclic strength characteristics, step (4), of the material comprising each zone of the dams are presented in Sections 2.5D.10, 2.5D.11, 2.5D.12, and 2.5D.13.

An example of the evaluation of the seismic stability of the dams, step (5), is described in Section 2.5D.16; the results for the three dams are presented in Section 2.5D.5, 2.5D.6, and 2.5D.7.

2.5D.4 SAFE SHUTDOWN EARTHQUAKE AND LOADING CONDITIONS

2.5D.4.1 Safe Shutdown Earthquake

The safe shutdown earthquake (SSE) is evaluated and defined in FSAR Sections 2.5.2 and 3.7. For the dynamic analysis of the Category I dams, the same SSE is postulated. In addition the behavior of the dams and dike in response to an event prescribed by Regulatory Guide 1.60 spectra was assessed, see Section 2.5D.18.

A value of 0.15 gravity is assigned as the maximum horizontal ground acceleration with the corresponding maximum vertical acceleration as 0.10 g. The Category I dams are designed to remain stable assuming that the horizontal and vertical accelerations act simultaneously.

The SSE is postulated as being a shock of low magnitude occurring close to the site with a maximum duration of ten seconds. The pattern of motion should be similar to the Golden Gate (San Francisco) earthquake of 1957 or the Helena, Montana earthquake of 1935. Records from these earthquakes indicate two to three cycles of strong motion. The SSE is defined by the smooth response spectra presented on FIGURE 3.7.1-5. Using the same criteria, Dames & Moore prepared the smooth response spectrum corresponding to a structural damping ratio of 0.07 presented in Figure 2.5D-7.

2.5D.4.2 Artificial Accelerogram

The dynamic analysis requires the use of accelerograms. Two methods can be used to select the accelerograms. One method consists of scaling to the proper acceleration (ie, 0.15 g) one or several actual accelerograms selected from existing earthquake strong motion records. However, the response spectra calculated from actual accelerograms are occasionally below the smooth response spectra defining the SSE and, at some periods, may have valleys which are significantly below the smooth response spectra. This may be alleviated by using a series of actual accelerograms for the design to assure that there is no period, within the range of interest, for which the response obtained by at least one of the accelerograms of the series is significantly smaller than that indicated by the smooth response spectra. Because of the great amount of analytical work involved, another method was used to select the accelerograms for the design of the power station.

The method used for design of all Category I systems and components, including dams, was to generate a single artificial accelerogram, the response spectra of which closely envelope the smooth response spectra which define the SSE. This artificial accelerogram applicable to a damping ratio of 0.07 is presented in Figure 2.5D-8. Its duration of approximately ten seconds is equal to that of the SSE. The response-spectrum of the artificial accelerogram for 0.07 structural damping ratio is presented in Figure 2.5D-7. The accelerogram was specifically fine-tuned, so that its response spectrum for 0.07 damping has a maximum positive deviation of

seven percent above the smooth response spectrum in the range of periods of interest; ie, 0.185 sec. to 1 sec. To validate this accelerogram, a high resolution response spectral analysis was performed. The period points were spaced from 0.0025 sec. in the short period range (0.2 sec.) to 0.0125 sec. in the long period range (1 sec.). Similar artificial accelerograms were developed for damping ratios of 0.05 and 0.10, and 0.15.

Preliminary dynamic stress analysis of the Category I dams indicated that the calculated damping ratios ranged from 0.07 to 0.15; see Section 2.5D.16. Therefore, use of the artificial accelerogram presented in Figure 2.5D-8 which is applicable to a damping ratio of 0.07 for calculating the stresses induced by the SSE in the Category I dams is conservative. The conservative nature of this selection is shown in Section 2.5D.18, within the assessment of the dams and dike, to an event prescribed by the Regulatory Guide 1.60 spectra.

2.5D.4.3 Comparison Between the Artificial Accelerogram and Actual Accelerograms of Earthquakes Applicable to the Site

Envelope response spectra have been specified for design of all Category I structures for the SHNPP. These response spectra do not represent those of the earthquake motions that might be generated by any single shock, but are, however, representative of the maximum responses from a possible series of earthquakes, any one of which could be the SSE. Thus, the artificial accelerogram generated from the specified site smooth response spectra has the characteristics of the maximum motions of the series of earthquakes. As previously noted, this artificial accelerogram was utilized in the evaluation of the stresses in the Category I dams. However, the same accelerogram cannot be used in the determination of the cyclic strength, including the stress-strain characteristics of the soil and rock materials forming the dams. Theoretical analysis confirmed by laboratory testing and case histories (eg, Alaska earthquake. April 1964; San Fernando Valley earthquake, February 1971) (Reference 2.5D-35) show that cyclic strength characteristics of soil and rock are considerably affected by the number of strong motion loading cycles. The cyclic strength of soil and rock significantly decreases as the number of cycles increases because of progressive buildup of pore pressure; therefore, a conservative, but realistic, number of loading cycles are used for the determination of the cyclic strength of soil and rock.

It is expected that the actual accelerogram of any single earthquake applicable to this site representative of the SSE would not have a duration of strong motion exceeding two to three seconds and would not contain any more than four to six high peaks* corresponding to two to three cycles of strong motion, as noted in Section 2.5. For this representative SSE, we have assumed that five cycles of strong motion would occur for determining the cyclic strength characteristics of the soil and rock.

The comparison between the artificial accelerogram and the actual accelerogram of any single earthquake applicable to the site is summarized in the following table below.

COMPARISON OF ARTIFICIAL ACCELEROGRAM AND ACTUAL ACCELEROGRAM

	Duration of Strong Motions, Sec	Equivalent Number of Cycles of Strong Motions
Artificial Accelerogram	10	20

* High peaks are those with an intensity greater than two-thirds the intensity of the maximum acceleration.

Actual Accelerogram of any Single Earthquake Applicable to the Site	2 to 3	3 to 5

The stability of the Category I dams under the SSE was conservatively evaluated on the basis of the stress induced by the artificial accelerogram shown in Figure 2.5D-8 as well as a accelerogram prescribed by Regulatory Guide 1.60 spectra and of the cyclic strength and stress-strain characteristics of the soil and rock materials using five cycles of strong motion. To demonstrate the additional margin of conservatism built into the dams, the stability was also evaluated assuming ten cycles of strong motion, which is essentially twice the expected number of cycles of strong motion.

2.5D.4.4 Operating Basis Earthquake

The operating basis earthquake (OBE) is taken to have maximum horizontal and vertical accelerations equal to one-half the corresponding accelerations assigned to the SSE.

2.5D.4.5 Loading Conditions

For dynamic analyses, two loading conditions (condition I and II) comprising the design water surface elevations and applicable design earthquake are considered for each dam.

2.5D.4.5.1 Main Dam

Condition I corresponds to normal operating conditions in the Main Reservoir and the Afterbay Reservoir. The water surface elevations are shown in Figure 2.5D-2. The SSE is assumed to occur.

Condition II corresponds to the normal operating condition in the Main Reservoir and a condition which would occur if the Afterbay Dam were to fail and the water surface elevation in the Afterbay Reservoir were to drop instantaneously to Elevation 165 ft. The water surface elevations are shown in Figure 2.5D-2. The OBE is assumed to occur.

2.5D.4.5.2 Auxiliary Dam

Condition I corresponds to a high water condition one foot above the Auxiliary Dam spillway elevation in the Auxiliary Reservoir, ie Elevation 253 ft., and normal operating condition in the Main Reservoir. The water surface elevations are shown in Figure 2.5D-4. The SSE is assumed to occur.

Condition II corresponds to the normal operating condition in the Auxiliary Reservoir and a condition which would occur if the water surface elevation in the Main Reservoir were to drop instantaneously to Elevation 209 ft. The water surface elevations are shown in Figure 2.5D-4. The OBE is assumed to occur.

2.5D.4.5.3 Auxiliary Separating Dike

Condition I corresponds to a high water condition at Elevation 253 ft. in the Auxiliary Reservoir on both sides of the dike. The water surface elevation is shown in Figure 2.5D-6. The SSE is assumed to occur.

Condition II corresponds to a low water condition at Elevation 243 ft. in the Auxiliary Reservoir on both sides of the dike. The water surface elevation is shown in Figure 2.5D-6. The OBE is assumed to occur.

Conditions I and II considered for each dam in Appendix 2.5D are summarized in the following table. Condition III for the Main Dam and Auxiliary Dam is summarized in Section 2.5D.9.

LOADING CONDITIONS

Dam	Condition	Water Surface Elevation, ft.		Design Earthquake
Main Dam		Main Reservoir	Afterbay Reservoir	
	I	250	199	SSE
	II	250	165	OBE
Auxiliary Dam		Main Reservoir	Afterbay Reservoir	
	I	250	253	SSE
	II	209	250	OBE
Auxiliary Reservoir Separating Dike		Main Reservoir	Afterbay Reservoir	
	I	253	253	SSE
	II	243	243	OBE

Dynamic analyses were made for Condition I for each dam. Because a difference of water surface elevation of a few feet is of no significance, the analyses were made assuming the water surface in the Main Reservoir and Auxiliary Reservoir were at the normal water level; i.e. Elevation 250 ft. The seismic stability evaluations for Condition I are reported in Sections 2.5D.5, 2.5D.6, and 2.5D.7. The seismic stability evaluations for Condition II are reported in Section 2.5D.8.

2.5D.5 MAIN DAM, EVALUATION OF SEISMIC STABILITY

2.5D.5.1 Material Properties

The material properties used in the analyses and evaluation of the Main Dam are presented in Section 2.5D.10 for the core material, in Section 2.5D.12 for the filters and rockfill and in Section 2.5D.13 for weathered rock. The expected constructed material properties given in these appendices were used for the "basic set" analysis. To demonstrate the added margin of conservatism, these expected constructed values were varied over a reasonable range to determine the influence of such variations on the stability of the dam, as presented in Section 2.5D.15.

2.5D.5.2 Static Stress Analysis

The static stresses in the Main Dam were computed using the static finite element method of analysis described in Section 2.5D.14. These static stresses were used to determine the cyclic strength in each material of the dam.

Typical results of the static analysis are presented in Section 2.5D.14 and 2.5D.16.

2.5D.5.3 Dynamic Stress Analysis

2.5D.5.3.1 General

The stresses induced within the Main Dam during the SSE were computed using the dynamic finite element method of analysis. Three cross sections of the Main Dam were analyzed in detail. They include the maximum cross section with a height of 105 ft. (M-105), a cross section with a height of 67 ft. (M-67), and a cross section with a height of 36 ft. (M-36).

A total of eight cases were evaluated for the maximum cross section of the Main Dam. The case in which the expected constructed material properties were used was designated by Roman numeral IV with no alphabetical letter attached to it; thus, for the maximum section of the Main Dam, the case is designated M-105-IV. For the other seven cases, an alphabetical letter is used to distinguish among the various cases. The material properties used for all eight cases are summarized in Section 2.5D.15.

The responses for all cases, except Case M-105-IV, are obtained using the finite element procedure. The response for Case M-105-IV are obtained by interpolation of the computed values for Cases M-105-IV A and M-105-IV C, which is justified because of the similarity of the response of the last two cases.

For the other two cross sections of the Main Dam, Cases M-67-IV A and M-36-IV A were analyzed.

For each case, the computed responses are utilized in evaluating the stability of the dam as illustrated in Section 2.5D.16.

2.5D.5.3.2 Crest Acceleration

The computed values of maximum crest accelerations together with the predominant period of the three cross sections of the Main Dam for all cases are presented in Table 2.5D-1. The maximum acceleration computed at the crest of the maximum cross section of the Main Dam varies from approximately 0.40 g. to 0.47 g. The maximum acceleration computed at the crest of the 67-ft.-high cross section and at the crest of the 36-ft.-high cross section are 0.43 g. and 0.45 g., respectively.

The crest accelerations due to an event prescribed by the Regulatory Guide 1.60 spectra yielded maximum values differing only by 0.01 g.; namely; ranging from 0.41 g. to 0.48 g. at the crest of the maximum cross section and values of 0.43 g. and 0.44 g. for crest accelerations of the 67-ft.-high and 37 ft. high cross section, respectively.

2.5D.5.3.3 Induced Shear Stresses

The stresses induced in the dam are computed for the entire duration of the artificial accelerogram. These stresses are then converted to equivalent uniform stress cycles using the procedure summarized in Section 2.5D.16. Typical time histories of computed shear stresses are also presented in Section 2.5D.16.

2.5D.5.4 Seismic Stability Evaluation

2.5D.5.4.1 General

The stability of the dam sections during the SSE has been evaluated using the procedure described in Section 2.5D.16. In addition, the behavior of the dams and dike to an event prescribed by the Regulatory Guide 1.60 spectra have been assessed in terms of the effects on the local factors of safety as reported in this Appendix. A detailed evaluation of the effects of the Regulatory Guide 1.60 spectra was performed for the expected constructed material properties cases. The methods and details of the assessment and evaluation are given in methods and details of the assessment and evaluation are given in Section 2.5D.18. The induced stresses, τ_d, at any location within the dam are compared to the stresses, τ_f, required to cause 5 x 10^{-2} strain at the location. As discussed in Section 2.5D.16, a criterion of 5 x 10^{-2} strain is used for evaluating the stability of the dam during the SSE. This criterion has been established on the basis of correlations between the results of seismic stability evaluation by the procedure used for the present studies and the performance of earth dams which have been subject to significant earthquake loading (Reference 2.5D-28 and 2.5D-35). Case histories of earth dams which have been subjected to earthquake loading show that if the strain at any location within the dam and its foundation is smaller than 5 x 10^{-2}, the earthquake had no effect on the stability and integrity of the dam. It should not be concluded that the stability and integrity of the dam is impaired if the strain exceeds 5 x 10^{-2} at some locations within the dam and its foundation. The effect of strains exceeding 5 x 10^{-2} depend on the zone of the dam where they occur, and on the relative extent and location within a specific zone.

The ratio, τ_f/τ_d, which has been considered to represent a local factor of safety against the development of 5 x 10^{-2} strain, is then computed at each location. On the basis of the explanation given in the preceding paragraph, a minimum value of this stress ratio greater than approximately 1.1 indicates an ample margin of safety. The variation of this stress ratio is assessed within each zone of the dam to investigate the potential behavior and stability of the dam during the SSE.

A typical determination of the ratio, τ_f/τ_d, is shown in Figure 2.5D-9 along a horizontal plane at Elevation 192.5 ft. (for case M-105-IV A). The stresses induced by the ten-second duration artificial accelerogram and the stresses required to cause 5 x 10^{-2} strain in five cycles together with the values of the ratio, τ_f/τ_d, along this plane are shown in this figure. As can be noted, the value of this ratio (or of the local factor of safety) are well over unity along this plane in each zone of the dam.

Similar determinations have been made along several other horizontal planes within the dam representing the induced stresses by five cycles as well as by ten cycles; see Section 2.5D.4. The computed local factors of safety along the plane at Elevation 192.5 ft. using five cycles and ten cycles are presented in Figure 2.5C-10. The local factor of safety is reduced by

approximately 10% to 15% using the conservative assumption that the induced stresses are generated by ten cycles instead of five cycles.

The minimum values of computed stress ration, τ_f/τ_d, for all cases considered for the maximum cross section of the Main Dam are presented in Tables 2.5D-2, 2.5D-3, 2.5D-4, and 2.5D-5 using five cycles and ten cycles. The minimum values of this ratio computed within the core are listed in Table 2.5D-2; those computed in the upstream fine filter and coarse filter are presented in Tables 2.5D-3 and 2.5D-4, respectively; and those computed in the upstream rockfill shell are listed in Table 2.5D-5.

The data in these tables together with examination of Figure 2.5D-9 provide the means to assess the stability of the Main Dam during the SSE. The plots in Figure 2.5D-9 and Figures 2.5D-84 through 2.5D-88 indicate that the minimum local factor of safety is obtained over only a very small part of the plane within the zone of the dam under consideration. Within any zone, the average computed local factors of safety is somewhat higher (in the core) to significantly higher (in the filters and rockfill shell) than the minimum values listed in Tables 2.5D-2 through 2.5D-5. The data in Tables 2.5D-2 through 2.5D-5 indicate that the minimum value of τ_f/τ_d is everywhere greater than unity and in most parts of the dam it is well over unity. Thus, the stability and integrity of the dam is assured during the SSE.

The data in Tables 2.5D-2 through 2.5D-5 are presented for the various cases considered for the maximum cross section of the Main Dam. The data can be used to assess the effects of variations in material properties on the local factor of safety (and thus stability) of the dam. These effects are summarized below for each zone of the dam.

2.5D.5.4.2 Maximum Cross Section, Core

The computed minimum values of τ_f/τ_d in the core for Case M-105-IV (the basic set using expected constructed properties) range from 1.79 to 2.02 based on five cycles and from 1.57 to 1.75 based on ten cycles. Thus, there is an ample margin against the development of 5 x 10^{-2} strain anywhere in the core.

Cases M-105-IV A and M-105-IV C were evaluated to assess the influence of variations of approximately +40 percent and -20 percent in the modulus of the rockfill shell, respectively. The data in Table 2.5D-2 indicates that these variations have very little effect (a slight decrease) on the minimum local factor of safety in the core. The minimum local factor of safety decreased from 1.79 to 1.77 for both cases based on five cycles minimum, and from 1.57 to 1.53 and 1.52, respectively, for case M-105-IV A and M-105-IV C, respectively, based on ten cycles.

Case M-105-IV B was evaluated to assess the influence of increasing the modulus of the core by approximately 20 percent and decreasing the damping in the core by 10 percent. The minimum local factor of safety in the core is decreased from 1.57 to 1.40 based on ten cycles. This constitutes a reduction in the value of the minimum factor of safety of the order of 10 percent.

Case M-105-IV BC is a combination of cases M-105-IV B and M-1-5-IV C. The minimum local factor of safety based on five cycles is reduced to 1.58 ie, by approximately 12 percent (compared to 1.79 for case M-105-IV), but is still well over unity. Similarly, the minimum local factors of safety based on ten cycles decreased to 1.37 compared to 1.57 for case M-105-IV which is also well above unity.

Case M-105-IV D was evaluated to assess the influence of increasing the modulus in the rockfill shell by 40 percent and decreasing the damping in the core by 10 percent and in the filters and rockfill shell by 20 percent. These variations have practically no effect (compared to Case M-105-IV) on the minimum value of τ_f/τ_d. The minimum local factor of safety based on five cycles decreased from 1.79 to 1.77 and from 1.57 to 1.54 based on ten cycles.

Case M-105-IV E was evaluated using what is believed to be the lower bound on the modulus values in the core, filters, and rockfill shell. The minimum computed local factor of safety is increased (compared to case M-105-IV) from 1.79 to 2.20 based on five cycles and from 1.57 to 1.92 based on ten cycles, i.e., an increase of approximately 22 percent.

Thus, the minimum local factors of safety in the core, taking into account a conservative combination of material properties, is 1.58 based on five cycles (Case M-105-IV BC) and 1.37 based on ten cycles (Case M-105-IV BC).

2.5D.5.4.3 Maximum Cross Section, Fine Filters

The computed minimum values of τ_f/τ_d in the upstream fine filter are listed in Table 2.5D-3 for all cases. These values range from 1.73 to 3.30 based on five cycles and from 1.55 to 2.95 based on ten cycles for Case M-105-IV. As can be noted in Table 2.5D-3, the variations used in modulus and damping have little effect (ranging from a decrease of approximately 5 percent for Case M-105-IV B to an increase of approximately 10 percent for case M-105-IV E) on the minimum local factor of safety.

The minimum values of τ_f/τ_d within the downstream fine filter are greater than the corresponding values listed in Table 2.5D-3 (see Figure 2.5D-87 and 2.5D-88) because the water surface elevation is approximately 50 ft. lower in this part of the dam.

It should be noted that the values of τ_f/τ_d listed in Table 2.5D-3 are based on the use of filters placed at a relative density of 75 percent.

The data in Table 2.5D-3 indicate that the minimum local factor of safety in the fine filters, taking into account a conservative combination of material properties, is 1.65 based on five cycles and 1.47 based on ten cycles.

2.5D.5.4.4 Maximum Cross Section, Coarse Filters

The computed minimum values of τ_f/τ_d in the upstream coarse filter are listed in Table 2.5D-4 for all cases. These values range from 1.22 to 2.06 for five cycles and from 1.20 to 1.84 for ten cycles for Case M-105-IV.

Increasing the modulus value in the shell by approximately 40 percent (Case M-105-IV A) increases the minimum value of τ_f/τ_d (for Case M-105-IV) to values ranging between 1.46 and 2.86 for five cycles and 1.31 and 2.55 for ten cycles, ie, by approximately 10 percent to 40 percent. Decreasing the modulus value in the shell by approximately 20 percent (Case M-105-IV C) decreases the minimum value of τ_f/τ_d to values ranging between 1.27 and 1.82 for five cycles and between 1.14 and 1.63 for ten cycles, ie, by approximately 5 percent to 10 percent.

An increase in the modulus and a decrease in the damping of the core together with an increase in the modulus of the rockfill shell (Case M-105-IV B) has essentially no effect at Elevation

237.5 ft. At lower elevations, these changes result in an increase of the minimum local factor of safety ranging from approximately 10 percent to 30 percent. The minimum factor of safety varies between 1.51 and 2.73 based on five cycles and between 1.35 and 2.44 based on ten cycles.

The computed values of minimum local factors of safety for Case M-105-IV BC (combination of Cases B and C) are 1.21 based on five cycles and 1.08 based on ten cycles, i.e., approximately 5 percent to 10 percent lower than for case M-105-IV.

A decrease in the damping in the core by 10 percent and in the filters and rockfill shell by 20 percent together with an increase of the modulus in the shell by approximately 40 percent (Case M-105-IV D) has very little effect at Elevation 237.5 ft. At lower elevations, these changes result in an increase of the minimum local factor of safety ranging from approximately 10 percent to over 30 percent. The minimum local factors of safety range between 1.55 and 2.82 based on five cycles and between 1.38 and 2.51 based on ten cycles. Case M-105-IV E (using the lower bound on the modulus values) provides minimum values of τ_f/τ_d comparable to Case M-105-IV D. The minimum local factor of safety is 2.58 based on five cycles and 1.42 based on ten cycles.

2.5D.5.4.5 Maximum Cross Section, Rockfill Shells

The computed minimum values of τ_f/τ_d within the upstream rockfill shell are presented in Table 2.5D-5. For Case M-105-IV, these values range from 1.51 to 2.12 based on five cycles and from 1.35 to 1.90 based on ten cycles. As can be noted in Table 2.5D-5, increasing the modulus in the rockfill shell by 40 percent (Case M-105-IV A) lowers the least value of τ_f/τ_d to 1.3, while decreasing the modulus in the rockfill shell by 20 percent (Case M-105-IV C) raises the least values of τ_f/τ_d to 1.53.

Case M-105-IV B (modulus increases by 20 percent and damping decreases by 10 percent in the core) provides values of minimum τ_f/τ_d comparable to Case M-105-IV A. The minimum local factors of safety range from 1.36 to 1.90 based on five cycles and from 1.21 to 1.70 based on ten cycles. Case M-105-IV BC provide values of minimum τ_f/τ_d comparable to the basic set (Case M-105-IV). The minimum local factors of safety range from 1.53 to 2.22 based on five cycles and from 1.37 to 1.98 based on ten cycles. Thus, increasing the modulus by 20 percent and decreasing the damping by 10 percent in the core, increasing the modulus in the rockfill shell by 40 percent, and decreasing the damping 20 percent in the filters and rockfill shell have little effect on the computed minimum values to τ_f/τ_d.

Case M-105-IV D (similar to Case M-105-IV A, except that the damping is decreased by 10 percent in the core and 20 percent in the filters and rockfill shell) provides minimum values of τ_f/τ_d ranging from 1.24 to 1.84 based on five cycles and 1.10 to 1.64 based on ten cycles, i.e., approximately 10 percent to 20 percent less than for Case M-105-IV.

The use of the lower bound values on modulus (Case M-105-IV E) indicates minimum values of τ_f/τ_d approximately 10 percent greater than for Case M-105-IV. The minimum local factors of safety range from 1.63 to 2.50 based on five cycles and 1.46 and 2.24 based on ten cycles.

Thus, the minimum local factors of safety in the rockfill shell taking into account a conservative combination of material properties is 1.24 based on five cycles and 1.10 based on ten cycles.

2.5D.5.4.6 Influence of Vertical Component

As illustrated in Section 2.5D.16, the induced shear stresses are essentially unaffected by the vertical component of the SSE. Therefore, the minimum computed local factors of safety are essentially unaffected by the vertical component.

2.5D.5.4.7 Summary for Maximum Cross Section

The data presented in Tables 2.5D-2 through 2.5D-5 indicate that the minimum local factors of safety in each zone of the maximum cross section of the Main Dam for the expected constructed properties is as follows: (Note: See Section 2.5D.18 for safety factors from an assessment in accordance with Regulatory Guide 1.60 spectra)

MAIN DAM, MAXIMUM CROSS SECTION
MINIMUM LOCAL FACTORS OF SAFETY FOR
EXPECTED CONSTRUCTED MATERIAL PROPERTIES

	No. of Cycles	
Zone	5 cycles	10 cycle
Core	1.79	1.57
Fine Filters	1.73	1.55
Coarse Filters	1.22	1.20
Rockfill Shell	1.51	1.35

It should be noted that these minimum local factors of safety are obtained within a very small part of each zone. The values of τ_f/τ_d increase considerably in the other parts of each zone. The minimum local factors of safety in each zone of the maximum cross section of the Main Dam for conservative combinations of material properties are presented in Tables 2.5D-2 through 2.5D-5.

2.5D.5.4.8 Other Cross Sections

Two other sections (Case M-67-IV A and Case M-36-IV A) of the Main Dam were also analyzed. The minimum values of $\tau_{f/}\tau_d$ computed within each zone of these two sections are as follows: (Note: See Section 2.5D.18 for safety factors from an assessment in accordance with Regulatory Guide 1.60 spectra)

MAIN DAM, 67-FT. AND 36-FT. CROSS SECTIONS
MINIMUM LOCAL FACTORS SAFETY

	Core [a]		Filters [a]		Rockfill [a]	
Case	5 cycles	10 Cycles	5 Cycles	10 cycles	5 cycles	10 cycles
M-67-IV A	2.30	2.05	1.59	1.42	1.57	1.40
M-36-IV A	2.48	2.20	1.82	1.63	1.60	1.43

(a) minimum value occurs at Elevation 237.5 ft.

(b) minimum value occurs at Elevation 222.5 ft.

This data indicates that there is ample margin against the development of 5 x 10^{-2} strain everywhere within these two sections of the Main Dam. Comparison of this data with that corresponding to other cases studied for the maximum cross section of the Main Dam (Tables 2.5D-2 through 2.5D-5) indicate that there is an ample margin of safety within these two sections of the Main Dam taking into account a very conservative combination of material property variations and considering ten equivalent uniform cycles of shear stresses.

2.5D.5.5 Conclusions for Main Dam

2.5D.5.5.1 Maximum Cross Section

a) The minimum local factor of safety ranges from approximately 1.2 to 1.8 based on five cycles using the expected constructed values of material properties.

b) Incorporating a conservative combination of material properties, the minimum local factor of safety ranges from approximately 1.2 to 1.6 based on five cycles.

c) For the same conservative combination of material properties, and incorporating an added margin of conservatism by basing the evaluation on ten cycles of strong motion, the minimum local factors of safety range from approximately 1.1 to 1.5.

d) The minimum local factor of safety is obtained within a very small part of each zone of the dam. The local factors of safety are considerably higher in other parts of each zone.

2.5D.5.5.2 Other Cross Sections

The minimum local factors of safety in other cross sections of the Main Dam are higher than those for the maximum cross section.

2.5D.5.5.3 Stability of the Main Dam During the SSE

There is an ample margin in all cross sections of the Main Dam against the development of 5 x 10^{-2} strain. Therefore, the Main Dam will be stable and will maintain its integrity during the SSE.

2.5D.6 AUXILIARY DAM, EVALUATION OF SEISMIC STABILITY

2.5D.6.1 Material Properties

The material properties used in the analyses and evaluation of the seismic stability of the Auxiliary Dam are presented in Section 2.5D.11 for the core material, Section 2.5D.12 for the filters and the random rockfill, and Section 2.5D.13 for the residual soil and weathered rock. The cases analyzed, and the range of dynamic properties used in these analyses, are presented in Section 2.5D.15.

2.5D.6.2 Static Stress Analysis

The static stresses in the Auxiliary Dam were obtained by the procedure used for the Main Dam.

2.5D.6.3 Dynamic Stress Analysis

2.5D.6.3.1 General

Three cross sections of the Auxiliary Dam were analyzed in detail by the procedure used for the Main Dam.

These cross sections include the maximum cross section of the Auxiliary Dam (designated A-63), a cross section near the west abutment (designated A-44), and a cross section near the east abutment (designated A-24).

Five cases for the maximum cross section of the dam are evaluated in detail; these cases are designated A-63-IV, A-63-IV A, A-63-IV B, A-63-IV C, and A-63-IV AV. Three cases are evaluated for the cross section near the west abutment and one case for the cross section near the east abutment. These cases are designated A-44-IV A, A-44-IV B, A-44-IV C, and A-24-IV A, respectively.

2.5D.6.3.2 Crest Accelerations

The computed maximum crest accelerations together with the predominant periods of the three cross sections of the Auxiliary Dam for all cases are presented in Table 2.5D-1. The maximum acceleration computed at the crest of the maximum cross section of the Auxiliary Dam varies from approximately 0.39 g. to 0.51 g. The maximum acceleration computed at the crest of the cross section near the west abutment (A-44) varies from approximately 0.45 g. to 0.48 g.; that at the crest of the cross section near the east abutment (A-24) is approximately 0.45 g.

The crest accelerations due to an event prescribed by the Regulatory Guide 1.60 spectra yielded maximum values differing only by 0.01 g.; namely, ranging from 0.40 g. to 0.51 g. at the crest of the maximum section. The respective accelerations at the crest of the section near the west abutment (A-44) varied from 0.44 g. to 0.47 g.; that near the east abutment (A-24) was approximately 0.43 g.

2.5D.6.3.3 Induced Shear Stresses

The shear stresses induced in the Auxiliary Dam were computed and utilized for the evaluation of seismic stability by employing the procedure outlined in Section 2.5D.16.

2.5D.6.4 Seismic Stability Evaluation for the Maximum Cross Section

2.5D.6.4.1 General

The stability of the cross sections of the Auxiliary Dam during the SSE has been evaluated by the procedure used for the Main Dam.

The stresses, τ_d, induced in five cycles along a horizontal plane at Elevation 215 ft. within the maximum cross section of the Auxiliary Dam (case A-63-IV A) and the stresses, τ_f, required to cause 5 x 10^{-2} strain in five cycles are presented in Figure 2.5D-11. The compound values of the local factor of safety along this plane using five cycles and ten cycles are presented in Figure 2.5D-12. The computed values of the local factor of safety (the ratio τ_f/τ_d) along this plane are also presented in this figure. Similar evaluations were made along several other

horizontal planes within the three cross sections of the Auxiliary Dam. The values of the ratio τ_f/τ_d were obtained from these evaluations and used to assess the potential behavior and the stability of the Auxiliary Dam.

The minimum values of computed stress ratio, τ_f/τ_d, for all cases considered for the maximum cross section of the Auxiliary Dam are presented in Tables 2.5D-6, 2.5D-7, and 2.5D-8 for five cycles and ten cycles. The minimum values of this ratio within the core are listed in Table 2.5D-6; those for the filters are presented in Table 2.5D-7; and those computed in the random rockfill shell are listed in Table 2.5D-8.

As in the Main Dam, the minimum local factor of safety is obtained over only a very small part of the plane within the zone of the Auxiliary Dam. The local factors of safety are considerably higher in the other parts of the core and random rockfill shell; see Figure 2.5D-11 and 2.5D-12.

The computed minimum values of τ_f/τ_d in the core are listed in Table 2.5D-6 for all cases.

The computed minimum values of τ_f/τ_d in the core for Case A-63-IV (the basic set comprising the expected constructed properties) range from 1.43 to 2.13 based on five cycles and from 1.26 to 1.92 based on ten cycles. These values are obtained for a core placed at 97 percent standard compaction.

Case A-63-IV A, in which the modulus in the random rockfill shell is increased by 67 percent, provides minimum values of τ_f/τ_d that are comparable to Case A-63-IV.

Case A-63-IV B, which incorporates an increase of 25 percent in the modulus and a decrease of 10 percent in the damping for the core, indicates lower values of the ratio τ_f/τ_d than Case A-63-IV. The minimum local factor of safety for Case A-63-IV B is 1.22 based on five cycles and 1.08 based on ten cycles.

Case A-63-IV C incorporates an increase in modulus of 25 percent in the core, 33 percent in the filter, and 67 percent in the random rockfill shell together with a decrease in damping of 10 percent in the core and 20 percent in the filters and random rockfill shell. This case represents the upper bound on modulus values and the lower bound on damping values. The minimum values of τ_f/τ_d for this case are comparable to those obtained for case A-63-IV B; the least value is 1.29 based on five cycles and 1.14 based on ten cycles.

2.5D.6.4.3 Filters

The computed minimum values of τ_f/τ_d in the filters are listed in Table 2.5D-7 for all cases. These values range from 1.33 to 1.60 based on five cycles and from 1.19 to 1.42 based on ten cycles for Case A-63-IV.

The minimum local factors of safety are obtained for Case A-63-IV C; they are 1.12 based on five cycles and 1.00 based on ten cycles.

2.5D.6.4.4 Random Rockfill Shells

The computed minimum values of τ_f/τ_d in the random rockfill shell are listed in Table 2.5D-8 for all cases. These values range from 1.56 to 2.28 based on five cycles and from 1.38 to 2.10 based on ten cycles for Case A-63-IV.

The minimum local factors of safety are obtained for Case A-63-IV A; they are 1.19 based on five cycles and 1.08 based on ten cycles.

2.5D.6.4.5 Influence of Vertical Component

Case A-63-IV AV, using the same material properties as for Case A-63-IV A and applying the horizontal and vertical components of the base motion simultaneously, was evaluated to assess the influence of the vertical component. The shear stresses computed for Case A-63-IV AV are essentially equal to those computed for Case A-63-IV A. Therefore, the minimum local factors of safety are essentially unaffected by the vertical component.

2.5D.6.4.6 Summary

The data presented in Tables 2.5D-6 through 2.5D-8 indicate that the minimum local factors of safety in each zone of the maximum section of the Auxiliary Dam for the expected constructed properties is as follows:

(Note: See Section 2.5D.18 for safety factors from an assessment of the Regulatory Guide 1.60 spectra.)

AUXILIARY DAM, MAXIMUM CROSS SECTION
MINIMUM LOCAL FACTORS OF SAFETY FOR
EXPECTED CONSTRUCTED MATERIAL PROPERTIES

	No. of Cycles	
Zone	5 cycles	10 cycle
Core	1.43	1.26
Filters	1.33	1.19
Random Rockfill Shell	1.56	1.38

It should be noted that these minimum local factors of safety are obtained within a very small part of each zone. The values of τ_f/τ_d increase considerably in the other parts of each zone. The minimum local factors of safety in each zone of the maximum cross section of the Auxiliary Dam for conservative combinations of material properties are presented in Tables 2.5D 6 through 2.5D-8.

2.5D.6.5 Seismic Stability Evaluation for Cross Section A-44

2.5D.6.5.1 General

Based on the results for the maximum cross section of the Auxiliary Dam, the basic set for Section A-44 was not computed, because local factors of safety would be higher than for the other cases for which computations were made. The material properties for the core, the filters, and the random rockfill assigned to Case A-63-IV A are used in all analyses for Section A-44. The expected inplace values of the material properties for the in-situ residual soil and weathered rock are used for Case A-44-IV A. Lower bound values for the moduli of the residual soils and weathered rock are assigned for Cases A-44 IV B and A-44-IV C (see Section 2.5D.15).

2.5D.6.5.2 Accelerations Along Top of Weathered Rock

The maximum acceleration computed along the top of the weathered rock for Cases A-44-IV A and A-44-IV B ranges from 0.151 g. to 0.156 g. Thus, using the expected in-place modulus for the weathered rock, the acceleration values are essentially unmodified through this layer. For Case A-44-IV C (using the lower bound modulus for the weathered rock), the maximum acceleration along the top of the weathered rock ranges from 0.16 g. to 0.17 g. For the latter case, the acceleration values are slightly modified through the layer of weathered rock.

2.5D.6.5.3 Acceleration Along Top of In-situ Residual Soils

The maximum acceleration computed along the top of the layer of residual soils underlying the dam for Case A-44-IV A ranges from 0.21 g. to 0.23 g. For Case A-44-IV B, these values range from 0.24 g. to 0.28 g. and for Case A-44-IV C, they range from 0.25 g. to 0.29 g. Thus, the layer of residual soil amplifies the rock accelerations by approximately a factor of 1.4 to 2.0.

2.5D.6.5.4 Seismic Stability Cross Section A-44

The minimum local factors of safety for Case A-44-IV A are as follows: (Note: see Section 2.5D.18 for safety factors from an assessment of the Regulatory Guide 1.60 spectra).

AUXILIARY DAM, SECTION A-44
MINIMUM LOCAL FACTORS OF SAFETY FOR
EXPECTED CONSTRUCTED MATERIAL PROPERTIES

	No. of Cycles	
Zone	5 cycles	10 cycles
Core	1.94	1.72
Filters	1.37	1.22
Random Rockfill Shell	1.50	1.40
Residual Soil	1.50	1.30

The above values were proportioned from the other cases for the Auxiliary Dam.

The shear stresses computed in the core for Case A-44-IV B are up to approximately 30 percent higher (i.e., in some parts, the stresses are essentially equal for the two cases) than for Case A-44-IV A: in the filters, they are up to approximately 20 percent higher; in the random rockfill shell they are up to approximately 10 percent higher; and in the residual soils, they are up to approximately 10 percent higher. The stresses computed in all zones for Case A-44-IV C are essentially equal (within $\pm$5 percent) to those computed for Case A-44-IV B.

Therefore, the minimum value of τ_f/τ_d in all parts of Cross Section A-44 are well over unity for all cases, taking into account the location at which the minimum value is obtained and the change in the induced stress at the location.

2.5D.6.6 Seismic Stability Evaluation for Cross Section A-24

Case A-24-IV A was analyzed to assess the stability of this cross section. The minimum local factors of safety for this case are as follows:

(Note: see Section 2.5D.18 for safety factors from an assessment of the Regulatory Guide 1.60 spectra.)

AUXILIARY DAM, SECTION A-24
MINIMUM LOCAL FACTORS OF SAFETY FOR
EXPECTED CONSTRUCTED MATERIAL PROPERTIES

	No. of Cycles	
Zone	5 cycles	10 cycle
Core	2.25	1.99
Filters	1.59	1.42
Random Rockfill Shell	1.50	1.40
Residual Soil	1.70	1.50

2.5D.6.7 Conclusions

There is an ample margin against the development of 5 x 10^{-2} strain in the Auxiliary Dam. Therefore, the Auxiliary Dam is stable and will maintain its integrity due the SSE.

2.5D.7 AUXILIARY RESERVOIR SEPARATING DIKE, EVALUATION OF SEISMIC STABILITY

2.5D.7.1 Material Properties

The material properties used in the analyses and evaluation of the Auxiliary Separating Dike are presented in Section 2.5D.11 for the core material, Section 2.5D.12 for the random rockfill, and Section 2.5D.13 for the in-situ residual soils and weathered rock. The cases analyzed, and the range of dynamic properties used in these analyses, are presented in Section 2.5D.15.

2.5D.7.2 Static Stress Analysis

The static stresses in the sections of the Auxiliary Separating Dike were obtained by the procedure used for the Main Dam.

2.5D.7.3 Dynamic Stress Analysis

2.5D.7.3.1 General

The maximum cross section of the Auxiliary Separating Dike is analyzed in detail by the procedure used for the Main Dam. Three cases are evaluated in detail; these cases are designated D-53-IV A, D-53-IV B, and D-53-IV C.

2.5D.7.3.2 Crest Accelerations

The computed maximum crest accelerations together with the predominant period for the three cases of the Auxiliary Separating Dike are presented in Table 2.5D-1. The maximum acceleration computed at the crest of the maximum section of the Auxiliary Separating Dike varies from approximately 0.45 g to 0.52 g. The crest accelerations due to an event prescribed by the Regulatory Guide 1.60 spectra yielded the same range of values.

2.5D.7.3.3 Induced Shear Stresses

The stresses induced in the Auxiliary Separating Dike were obtained and utilized using the same procedures outlined for the Main Dam in Section 2.5D.16.

2.5D.7.4 Seismic Stability Evaluation

The stability of the maximum cross section of the Auxiliary Separating Dike during the SSE has been evaluated by the same procedure used for the Main Dam.

The stresses, τ_d, induced in five cycles along a horizontal plane at Elevation 217 ft. within the maximum cross section of the Auxiliary Separating Dike (case D-53-IV A) and the stresses, τ_f, required to cause 5 x 10^{-2} strain in five cycles are presented in Figure 2.5D-13. The computed values of the local factor of safety along this plane using five cycles and ten cycles are presented in Figure 2.5D-14. The computed values of the local factor of safety (the ratio τ_f/τ_d) along this plane are also presented in Figure 2.5D-14. Similar evaluations were made along several other horizontal planes within the maximum section of the Auxiliary Separating Dike. The values of the ratio τ_f/τ_d are obtained from these evaluations and used to assess the potential behavior and the stability of the Auxiliary Separating Dike.

The minimum values of computed stress ratio, τ_f/τ_d, for all cases considered for the maximum cross section of the Auxiliary Reservoir Separating Dike are presented in Tables 2.5D-9 and 2.5D-10 for five cycles and for ten cycles. The minimum values of this ratio computed within the core of the dam are listed in Table 2.5D-9, and those computed in the random rockfill shell are listed in Table 2.5D-10.

Based on the results for the maximum section of the Auxiliary Dam, the maximum cross section of the Auxiliary Reservoir Separating Dike was not analyzed using the basic set of material properties (Case D-53-IV). The minimum values of τ_f/τ_d for this case would be higher than those computed for the other three cases.

As in the Main Dam and Auxiliary Dam, the minimum local factor of safety is obtained over only a very small part of the plane within the zone of the dam under consideration. The local factors of safety are considerably higher in the other parts of the core and shell; see Figure 2.5D-13 and 2.5D-14.

The minimum local factors of safety in each zone of the maximum cross section of the Auxiliary Separating Dike for the expected constructed properties is as follows: (Note: see Section 2.5D.18 for safety factors from an assessment of Regulatory Guide 1.60 spectra.)

AUXILIARY RESERVOIR SEPARATING DIKE, MAXIMUM CROSS SECTION
MINIMUM LOCAL FACTORS OF SAFETY FOR
EXPECTED CONSTRUCTED MATERIAL PROPERTIES

	No. of Cycles	
Zone	5 cycles	10 cycle
Core	1.7	1.5
Random Rockfill Shells	1.9	1.7

The above values were proportioned from the other cases for the Auxiliary Reservoir Separating Dike based on the results for the Auxiliary Dam.

The minimum local factors of safety in each zone of the maximum cross section of the Auxiliary Reservoir Separating Dike for conservative combinations of material properties is presented in Tables 2.5D-9 and 2.5D-10. The minimum local factors of safety in the 5-ft. layer of residual soil underlying the Auxiliary Reservoir Separating Dike ranges from approximately 1.8 to 2.1 based on five cycles and from 1.7 to 1.9 based on ten cycles.

2.5D.7.5 Conclusion

There is an ample margin against the development of 5 x 10^{-2} strain in the Auxiliary Reservoir Separating Dike. Therefore, the Auxiliary Reservoir Separating Dike is stable and will maintain its integrity during the SSE.

2.5D.8 SEISMIC STABILITY EVALUATION FOR LOADING CONDITION II

2.5D.8.1 General

Loading Condition II pertains to the use of the accelerations associated with the operating basis earthquake (OBE) as input in the suitable rock underlying the dams. The water surface elevations in the reservoir considered for this condition are summarized in Section 2.5D.4.5.

The OBE is considered to have maximum horizontal and vertical accelerations equal to one-half the corresponding values assigned to the SSE.

The stresses induced in the dams for Condition II would be considerably lower than those presented in Section 2.5D.16 and in Section 2.5D.5, 2.5D.6, and 2.5D.7 during the SSE. The reduction in the induced stresses however, would not be as much as the reduction in the suitable rock accelerations because of the non-linear stress-strain characteristics of the soil and rock materials. Based on numerous analyses for other dams and soil deposits, it is concluded that the induced stresses for Condition II would be approximately 60 percent of those computed for Condition I.

The stability of the dams for Condition II can then be assessed based on the results obtained for Condition I, as summarized below.

2.5D.8.2 Main Dam

The water level in the Main Reservoir is assumed to be at Elevation 250 ft. The water level in the Afterbay Reservoir is assumed to drop instantaneously to Elevation 165 ft. and the OBE is assumed to occur.

The stresses induced by the OBE would be approximately 60 percent of those computed during the SSE. The water level in the main reservoir is the same as is used for Condition I. Therefore, the effective normal stresses in the upstream rockfill shell and filters would be essentially the same as for Condition I. The stresses required to cause 5×10^{-2} strain in these zones would, therefore, be the same as those used for Condition I. Because the induced stresses are decreased, the values of τ_f/τ_d listed in Tables 2.5D-3, 2.5D-4, and 2.5D-5 would be approximately 1/0.6 = 1.7 times higher for Condition II than computed for Condition I.

Because the water surface elevation in the Afterbay Reservoir is assumed to drop instantaneously to Elevation 165 ft., the phreatic line in the core would not change significantly. The effective normal stresses in the core would, therefore, be essentially the same as those used for Condition I. For the same reasons used for the upstream rockfill shell and filters, the values of τ_f/τ_d listed in Table 2.5D-2 would be approximately 1.7 times higher for Condition II than computed for Condition I.

The downstream rockfill shell and filters have a high permeability and, therefore, would no longer be fully saturated during the assumed instantaneous drop in the water level in the Afterbay Reservoir. The effective normal stresses in these zones would, therefore, increase with a corresponding increase in the stresses required to cause 5×10^{-2} strain. Even with the conservative assumption that the effective normal stresses will be equal to those used for Condition I, the values of τ_f/τ_d would be 1.7 times higher for Condition II than computed for Condition I.

2.5D.8.3 Auxiliary Dam

Condition II corresponds to the water level in the Auxiliary Reservoir at Elevation 250 ft. and the water level in the Main Reservoir dropping instantaneously to Elevation 209 ft. The OBE is assumed to occur.

For the same reasons described in the preceding section for the Main Dam, the values of τ_f/τ_d in the various zones of the Auxiliary Dam and underlying foundation layers would be at least 1.7 times higher for Condition II than presented in Section 2.5D.6 for Condition I.

2.5D.8.4 Auxiliary Reservoir Separating Dike

Condition II corresponds to a low water condition at Elevation 243 ft. in the Auxiliary Reservoir on both sides of the dike. The OBE is assumed to occur.

The effective normal stresses in the Auxiliary Separating Dike are increased because of the lowering of the water level on both sides. Therefore, the values of τ_f used in Section 2.5D.7 would also be increased. The induced stresses, τ_d, would be approximately 60 percent of those computed for Condition I. Therefore, the values of τ_f/τ_d within the core, the random rockfill shell, and the underlying residual soil would be 1.7 times the corresponding values presented in Section 2.5D.7 for Condition I.

2.5D.8.5 Conclusion

Based on the above considerations, it is concluded that the Main Dam, the Auxiliary Dam and the Auxiliary Reservoir Separating Dike each has considerable margin against the development

of 5 x 10^{-2} strain for loading Condition II. Therefore, the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike are stable and maintain their integrity during the OBE.

2.5D.9 SEISMIC STABILITY EVALUATION FOR LOADING CONDITION III

2.5D.9.1 Introduction

This section presents the results of seismic stability analyses of the Category I Main Dam and Auxiliary Dam for the Shearon Harris Nuclear Power Plant. The location of the dams is shown in Figure 2.5D-15.

The analyses presented in this section are for loading Condition III, which corresponds to elimination of the Afterbay Reservoir, and a decrease in the design water surface elevation in the Main Reservoir from Elevation 250 ft. to Elevation 220 ft.

The results of previous seismic stability analyses for the dams for loading Conditions I and II are presented in Section 2.5D.5 through 2.5D.8.

This section does not include seismic stability evaluations of the Category I Auxiliary Reservoir Separating Dike (Figure 2.5D-15) because loading Condition III is identical to loading Condition I for the Auxiliary Reservoir Separating Dike. Therefore, the seismic stability evaluation of the Auxiliary Reservoir Separating Dike for loading Condition I presented in Section 2.5D.7, is applicable for loading Condition III.

2.5D.9.2 Description of Dams

The Main Dam and Auxiliary Dam are described in Section 2.5D.2. The maximum cross sections and longitudinal sections of these dams are shown on Figure 2.5D-16 and 2.5D-17.

2.5d.9.3 Procedure Used In Seismic Stability Evaluation

Detailed dynamic analyses and evaluations of the Main Dam and Auxiliary Dam for loading Condition I have previously been made. The procedures used and results obtained for loading Condition I are described in Section 2.5D.5 through 2.5D.7.

The safe shutdown earthquake (SSE) in suitable rock beneath the dams is identical for loading Condition I and loading Condition III. The only difference between loading Condition III and loading Condition I is the change in the water surface elevations. This will result in a lower phreatic surface in the dams for loading Condition III. Because the dynamic material properties of the dams, described in previous sections, do not change, the peak shear stress τ_d of the equivalent uniform shear-stress cycles, induced in the dams by the SSE for loading Condition III, would not be significantly different from the stresses computed for loading Condition I.

The lower phreatic surface in the dams associated with loading Condition III would, however, result in significantly higher static normal effective stresses in portions of the dams as compared to loading Condition I. Therefore, the dynamic strength, i.e., cyclic stresses, τ_f, required to cause 5 x 10^{-2} strain, would significantly increase in portions of the dams for loading Condition III. Thus, in portions of the dams, the values of the local factor of safety, τ_f/τ_d, with respect to the development of 5 x 10^{-2} strain, would be higher for loading Condition III than for loading Condition I.

Based on the preceding reasoning, the following procedures were used to assess the seismic stability of the Main Dam and Auxiliary Dam for loading Condition III.

a) Values of peak shear stresses τ_d of equivalent number N of uniform shear stress cycles, induced in the dams, are obtained from the results of dynamic analyses made for loading Condition I.

b) Values of static normal effective stress in the dams are computed based on the change in position of the phreatic surface for loading Condition III as compared to loading Condition I. For this computation, a close approximation of the increase in normal effective stresses is obtained by multiplying the vertical distance between the phreatic surfaces for loading Conditions I and III by the differences in effective unit weight for the two loading conditions.

c) Values of the cyclic shear stresses τ_f required to cause 5×10^{-2} strain in five and ten cycles are computed using the values of normal effective stress obtained in step (b). Curves relating τ_f to normal effective stress are presented in Sections 2.5D.4 through 2.5D.7 for each material in the dams.

d) Values of the local factor of safety τ_f/τ_d, based on five and ten cycles, are computed from the data obtained in steps (a) and (c).

2.5D.9.4 Design Earthquake and Loading Conditions

For loading Condition III, the safe shutdown earthquake (SSE), is assumed to occur. The SSE is defined in Sections 2.5.2 and 3.7. The SSE has a maximum horizontal acceleration of 0.15 g., and a maximum vertical acceleration of 0.10 g.

The following water surface elevations apply for loading Condition III:

Main Dam - The Main Reservoir will be at the normal water level, i.e., Elevation 220 ft. The Afterbay Reservoir will be eliminated. The water surface elevation of the downstream side will be Elevation 165 ft.; see Figure 2.5D-16.

Auxiliary Dam - The Auxiliary Reservoir will be at a high water condition one foot above the Auxiliary Dam spillway elevation; i.e., Elevation 253 ft. The Main Reservoir will be at the normal water level; i.e., Elevation 220 ft. The water surface elevations are shown on Figure 2.5D-17.

Loading Condition III for each dam is summarized below:

LOADING CONDITION III

Dam	Water Surface Elevation, ft.		Design Earthquake
	Main Reservoir	Downstream Side	
Main Dam	220	165	SSE
	Main Reservoir	Auxiliary Reservoir	
Auxiliary Dam	220	253	SSE

For comparison, loading Condition I in the Main Dam pertains to water surface elevations of Elevation 250 ft. in the Main Reservoir, and Elevation 199 ft. in the Afterbay Reservoir; loading Condition I in the Auxiliary Dam pertains to water surface elevations of Elevation 250 ft. in the Main Reservoir, and Elevation 253 ft. in the Auxiliary Reservoir.

Because a difference in water surface elevation of a few feet is not significant, the seismic stability evaluation of the Auxiliary Dam for loading Condition III and for loading Condition I are made assuming the water surface in the Auxiliary Reservoir to be at the normal water level; ie, Elevation 250 ft.

2.5D.9.5 Main Dam, Evaluation Of Seismic Stability

2.5D.9.5.1 General

The procedures described in Section 2.5D.9.3 are used to assess the seismic stability of the Main Dam during loading Condition III. The maximum cross section, designated M-105 (Figure 2.5D-16), has been selected for analysis because the maximum cross section was previously found to be more critical than lower cross sections of the dam during loading Condition I.

2.5D.9.5.2 Maximum Cross Section

The case selected for analysis is M-105-IVA. As described in Section 2.5D.15, Case M-105-IVA is for the case of expected constructed material properties in the core and filters of the dam and weathered rock beneath the dam and upper bound values of shear modulus in the rockfill shells. Case M-105-IVA has been selected because the results of previous analyses of this case, for loading Condition I, have been illustrated and described in detail in Section 2.5D.5 (Figure 2.5D-9, 2.5D-10, and Section 2.5D.16).

Typical results of the seismic stability evaluation for Case M-105-IVA, for loading Condition III, are shown on Figure 2.5D-18. For comparison, the results for loading Condition I are also shown on Figure 2.5D-18.

The upper plot in Figure 2.5D-18 shows the static normal effective stresses along a typical horizontal plane. Because of the lower reservoir water levels, the static normal effective stresses in portions of the dam are higher for loading Condition III than for loading Condition I.

Values of the cyclic shear stress τ_f required to cause 5 x 10^{-2} strain in five cycles along the typical horizontal plane are shown in the middle part of Figure 2.5D-18. These values are obtained using the normal effective stresses shown in the upper part of the figure, and the

relationships between normal effective stress and τ_f defined for each material in the dam (Section 2.5D.10 for core; Section 2.5D.12 for coarse and fine filters and rockfill shells). Because of the higher static normal effective stresses, the values of τ_f are higher in portions of the dam for loading Condition III than for loading Condition I. For loading Condition III, the downstream filters and rock shells are not submerged, except for approximately 2 ft. above weathered rock. The non-submerged materials are not subject to porewater pressure increases during the earthquake motions, and have values of τ_f greater than submerged materials. Therefore, values of τ_f are not computed in the downstream filters and rockfill shells.

The middle part of Figure 2.5D-18 also shows the values of the peak shear stresses τ_d of the five equivalent uniform shear-stress cycles induced by the earthquake. As discussed in Section 2.5D.9.3 values of τ_d for loading Condition III do not differ significantly from those for loading Condition I; therefore, the values computed for loading Condition I are used for loading Condition III.

The lower part of Figure 2.5D-18 shows the variation of the local factor of safety τ_f/τ_d against the development of 5 x 10^{-2} strain in five cycles along the typical plane. Figure 2.5D-18 shows that the local factor of safety is higher in portions of the dam for loading Condition III than for loading Condition I.

Evaluations similar to those illustrated in Figure 2.5D-18 were made along several horizontal planes in the Main Dam. A summary of the computed minimum values of τ_f/τ_d obtained in the respective materials along five horizontal planes is presented in Table 2.5D-11. Minimum values obtained, based five cycles and ten cycles, are summarized. For comparison, values of τ_f/τ_d obtained for both loading Condition I and loading Condition III are shown in Table 2.5D-11. Values of τ_f/τ_d are not summarized in the downstream filters and rockfill shells for loading Condition III because these materials are not submerged, and are, therefore, stable for this loading condition.

2.5D.9.5.2.1 Core

The computed minimum values of τ_f/τ_d in the core of the Main Dam for Case M-105-IVA, loading Condition III, range from 2.32 to 2.62 based on five cycles and from 2.01 to 2.29 based on ten cycles. These values are obtained for the core constructed at 100 percent standard compaction.

The corresponding minimum values of τ_f/τ_d in the core for loading Condition I range from 1.75 to 2.22 based on five cycles, and from 1.53 to 1.92 based on ten cycles; see Table 2.5D-11. Thus, the minimum values of the local factor of safety in the core are higher for loading Condition III than for loading Condition I.

The cyclic shear stress τ_f required to cause 5 x 10^{-2} strain in the core material was found to depend not only on the static normal effective stress, but also on the value $\propto$, defined as the ratio of static shear stress to static normal effective stress; see Section 2.5D.10. The effect of $\propto$ on τf in the core is shown in Figure 2.5D-30 and 2.5D-31.

The local factors of safety in the core for loading Condition III have been computed assuming that values of $\propto$ for loading Condition III are the same as the values determined for loading Condition I. However, the values of $\propto$ could be slightly different for loading Condition III and, therefore, the minimum local factors of safety τ_f/τ_d could show a slight variation. The maximum

effect that variations in ∝ could have on values of τ_f/τ_d for loading Condition III can be determined by making the very conservative assumption that ∝ equals zero throughout the core. Using this assumption, it is found that the minimum values of τ_f/τ_d in the core would be essentially unchanged on horizontal planes at Elevation 192.5 ft. and Elevation 207.5 ft., and would be reduced by approximately 10 percent on a plane at Elevation 177.5 ft. Thus, even with these very conservative assumptions, the minimum values of τ_f/τ_d for loading Condition III would be higher than the values for loading Condition I.

2.5D.9.5.2.2 Upstream Fine Filters

The computed minimum values of τ_f/τ_d in the upstream fine filter for Case M-105-IVA, loading Condition III, range from 3.83 to 4.61 based on five cycles, and from 3.46 to 4.16 based on ten cycles. These values are obtained for filters placed at a relative density of 75 percent. Corresponding minimum values of τ_f/τ_d in the upstream fine filter for loading Condition I range from 1.70 to 4.03 based on five cycles, and from 1.52 to 3.58 based on ten cycles; see Table 2.5D-11.

2.5D.9.5.2.3 Upstream Coarse Filters

The computed minimum values of τ_f/τ_d in the upstream coarse filter for case M-105-IVA, loading Condition III, range from 2.75 to 3.38 based on five cycles and from 2.46 to 3.04 based on ten cycles. These values are obtained for filters placed at a relative density of 80 percent above Elevation 220 ft., and a relative density of 75 percent below Elevation 220 ft. Corresponding minimum values of τ_f/τ_d in the upstream coarse filter for loading Condition I range from 1.46 to 2.86 based on five cycles, and from 1.31 to 2.55 based on ten cycles; see Table 2.5D-11.

2.5D.9.5.2.4 Upstream Rockfill Shells

The computed minimum values of τ_f/τ_d in the upstream rockfill shells for Case M-105-IVA, loading Condition III, range from 1.30 to 1.88 based on five cycles, and from 1.16 to 1.68 based on ten cycles. Corresponding minimum values of $\tau_f/\tau_{5 \times 10}$ in the upstream rockfill shells for loading Conditions I range from 1.30 to 1.88 based on five cycles, and from 1.16 to 1.68 based on ten cycles; see Table 2.5D-11.

The above summary shows that there is no difference in the minimum values of τ_f/τ_d in the upstream rockfill shells between loading Conditions I and III. The minimum values of τ_f/τ_d in the upstream rockfill shells occur near the slope where the static normal effective stresses are unchanged by the difference in reservoir water level for the two loading conditions; see Figure 2.5D 18.

2.5D.9.5.2.5 Influence of Vertical Component

Based on the analyses of the maximum section of the Main Dam for loading Condition I (Section 2.5D.16), the vertical component of the SSE has an insignificant effect on the local factors of safety for loading Condition III.

2.5D.9.5.2.6 Summary of Maximum Cross Section

The computed minimum local factors of safety obtained in each zone of the Main Dam for Case M-105-IVA, loading Condition I and loading Condition III, are summarized below:

MAIN DAM, MAXIMUM CROSS SECTION
MINIMUM LOCAL FACTORS OF SAFETY
CASE M-105-IVA

	Loading Condition I		Loading Condition II	
	5 Cycles	10 Cycles	5 Cycles	10 Cycles
Core	1.75	1.53	2.32	2.01
Fine Filters	1.70	1.52	3.83	3.46
Coarse Filters	1.46	1.31	2.75	2.46
Rockfill Shells	1.30	1.16	1.30	1.16

These minimum local factors of safety are obtained in only a small portion of each zone. The values of τ_f/τ_d increase considerably in other portions of each zone.

For Case M-105-IVA, the minimum local factors of safety for loading Condition III are equal to or greater than the values obtained for loading Condition I. For other cases applicable to the maximum cross section, described in this appendix, the minimum local factors of safety for loading Condition III similarly equal or exceed the values presented in this appendix for loading Condition I.

Analyses of the maximum cross section of the Main Dam for loading Condition I were made for several conservative combinations of material properties (Section 2.5D.15). These analyses indicated ample margin against the development of 5 x 10^{-2} strain in all zones of the dam. For loading Condition III, the computed factors of safety are equal to or greater than the values for loading Condition I. Therefore, there is ample margin against the development of 5 x 10^{-2} strain in the maximum cross section for loading Condition III.

2.5D.9.5.3 Other Cross Sections

Analyses of the Main Dam for loading Condition I (Section 2.5D.5) showed that the minimum local factors of safety obtained for other cross sections were higher than those obtained for the maximum cross section. As is the case for the maximum cross section, other cross sections of the Main Dam have a lower phreatic surface for loading Condition III than for loading Condition I. Therefore, other cross sections of the Main Dam will have higher factors of safety against the development of 5 x 10^{-2} strain for loading Condition III than for loading Condition I.

2.5D.9.5.4 Conclusion

Based on the evaluations presented above, it is concluded that there is ample margin against the development of 5 x 10^{-2} strain in the Main Dam for loading Condition III. Therefore, the Main Dam will be stable and will maintain its integrity for loading Condition III.

2.5D.9.6 Auxiliary Dam, Evaluation of Seismic Stability

2.5D.9.6.1 General

The seismic stability of the Auxiliary Dam for loading Condition III has been evaluated using the procedures described in Section 2.5D.9.3. The cross section selected for analysis is the maximum cross section, designated A-63 (Figure 2.5D-17). This cross section was selected

because it was previously found (Section 2.5D.6) to be more critical than lower cross sections for loading Condition I.

2.5D.9.6.2 Maximum Cross Section

Case A-63-IVA has been selected for analysis. As described in Section 2.5D.6, Case A-63-IVA corresponds to expected constructed material properties in the core and filters of the dam and upper bound values of shear modulus in the random rockfill shells. Case A-63-IVA has been selected because the results of previous analyses for this case for loading Condition I have been illustrated in Section 2.5D.6 (Figures 2.5D-11 and 2.5D-12).

Values of equivalent uniform shear stress τ_d induced by the SSE, were obtained from the previous analysis for loading Condition I (Section 2.5D.6). Values of cyclic shear stress τ_f required to cause 5 x 10^{-2} strain were obtained using the computed values of normal effective stress (higher in portions of the dam for loading Condition III than for loading Condition I, and the relationships between τ_f and normal effective stress defined in this appendix for each material in the dam (Section 2.5D.11 for core; Section 2.5D.12 for filter and random rockfill shells).

Results of the seismic stability evaluation for Case A-63-IVA, loading Condition III, along a typical horizontal plane are shown on Figure 2.5D-19. For comparison, results obtained along this plane for loading Condition I are also shown on Figure 2.5D-19.

The minimum values of the local factor of safety τ_f/τ_d against the development of 5 x 10^{-2} strain along five horizontal planes for Case A-63-IVA are summarized in Table 2.5D-12. The values obtained for loading Conditions I and III based on five cycles and on ten cycles are presented in this table.

2.5D.9.6.2.1 Core

The computed minimum values of the local factor of safety τ_f/τ_d in the core of the Auxiliary Dam for Case A-63-IVA, loading Condition III, range from 1.47 to 2.42 based on five cycles, and from 1.31 to 2.17 based on ten cycles. These values are obtained for the core constructed at 97 percent standard compaction.

For loading Condition I, corresponding minimum values of τ_f/τ_d range from 1.46 to 2.24 based on five cycles, and from 1.29 to 2.00 based on ten cycles; see Table 2.5D-12.

The minimum local factors of safety in the core of the Auxiliary Dam are slightly higher for loading Condition III than for loading Condition I. The phreatic surface in the core, corresponding to loading Condition III, is slightly lower than the phreatic surface corresponding to loading Condition I. Therefore, the normal effective stresses in the core, and the values of τ_f, are only slightly higher for loading Condition III than for loading Condition I.

2.5D.9.6.2.2 Filters

Because the water level in the Auxiliary Reservoir is identical for loading Condition I and loading Condition III, the phreatic surface in the upstream filter of the Auxiliary Dam is essentially the same for the two loading conditions. Therefore, the minimum values of τ_f/τ_d are essentially the same for the two loading conditions. For Case A-63-IVA, the computed minimum values of τ_f/τ_d

in the upstream filter range from 1.53 to 1.75 based on five cycles, and from 1.36 to 1.56 based on ten cycles; see Table 2.5D-12.

The phreatic surface in the downstream filter for loading Condition III is essentially at the elevation of the Main Reservoir; i.e., Elevation 220 ft. for loading Condition III, and Elevation 250 ft. for loading Condition I. The computed minimum values of τ_f/τ_d in the downstream filter for Case A-63-IVA, loading Condition III, range from 2.23 to 2.34 based on five cycles, and from 2.03 to 2.12 based on ten cycles. The corresponding minimum values for loading Condition I range from 1.53 to 1.75 based on five cycles, and from 1.36 to 1.56 based on ten cycles; see Table 2.5D-12.

The minimum values of τ_f/τ_d, summarized above, pertain to the submerged portion of the filters (below the phreatic surface) , and are for filters placed at a relative density of 75 percent below Elevation 220 ft. and 80 percent above Elevation 220 ft.

2.5D.9.6.2.3 Random Rockfill Shells

The phreatic surface in the upstream random rockfill shell is essentially the same for loading Conditions I and III; the phreatic surface in the downstream random rockfill shells is lower for loading Condition III than for loading Condition I.

The computed minimum values of τ_f/τ_d in the upstream random rockfill shells for Case A-63-IVA, applicable to loading Condition I and loading Condition III, range from 1.19 to 1.70 based on five cycles, and from 1.08 to 1.57 based on ten cycles.

In the downstream random rockfill shells, the computed minimum values of τ_f/τ_d for Case A-63-IVA, loading Condition III, range from 1.64 to 1.66 based on five cycles, and from 1.40 to 1.43 based ten cycles. The minimum values of τ_f/τ_d for loading Condition I range from 1.19 to 1.70 based on five cycles, and from 1.08 to 1.57 based on ten cycles.

2.5D.9.6.2.4 Influence of Vertical Component

Based on the analyses of the maximum cross section of the Auxiliary Dam for loading Condition I (Section 2.5D.6), the vertical component of the SSE has an insignificant effect on the local factors of safety for loading Condition III.

2.5D.9.6.2.5 Summary for Maximum Cross Section

The computed minimum local factors of safety obtained in each zone of the Auxiliary Dam for Case A-63-IVA for loading Condition I and loading Condition III are summarized in the following table:

AUXILIARY DAM, MAXIMUM CROSS SECTION
MINIMUM LOCAL FACTORS OF SAFETY
CASE A-63-IVA

	Loading Condition I		Loading Condition III	
Zone	5 Cycles	10 Cycles	5 Cycles	10 Cycles
Core	1.46	1.29	1.47	1.31
Filter-Upstream	1.53	1.36	1.53	1.36

Filter-Downstream	1.53	1.36	2.23	2.03
Random Rockfill Shells-Upstream	1.19	1.08	1.19	1.08
Random Rockfill Shells-Downstream	1.19	1.08	1.64	1.40

These minimum local factors of safety are obtained within a small part of each zone. The values of τ_f/τ_d increase considerably in other parts of each zone.

For Case A-63-IVA, the minimum local factors of safety for loading Condition III are equal to or greater than for loading Condition I. For other cases applicable to the maximum cross section, described in this appendix, the minimum local factors of safety for loading Condition III similarly equal or exceed the values obtained for loading Condition I.

As described in Section 2.5D.6, analyses of the maximum cross section of the Auxiliary Dam for loading Condition I were made for several conservative combinations of material properties. These analyses indicated ample margin against the development of 5 x 10^{-2} strain in all zones of the dam. For loading Condition III, the factors of safety are equal to or greater than those for loading Condition I. Therefore, there is ample margin against the development of 5 x 10^{-2} strain in the maximum cross section during loading Condition III.

2.5D.9.6.3 Other Cross Sections

Analyses of the Auxiliary Dam during loading Condition I (Section 2.5D.6) show that the minimum local factors of safety obtained in the core filters, and random rockfill shells of other cross sections are comparable to or greater than the values obtained for the maximum cross section. Similar results would be obtained for loading Condition III.

Other cross sections of the auxiliary dam, i.e., Section A-44 and A-24, described in this appendix, include a layer of in-situ residual soil beneath the random rockfill shells. Analyses of these cross sections for loading Condition I (Section 2.5D.6) indicate ample margin against the development of 5 x 10^{-2} strain in the in-situ residual soil. For loading Condition III, the static normal effective stresses in this layer beneath the upstream shells would be unchanged from the stress determined for loading Condition I. In the layer of in-situ residual soil beneath the downstream shells, the lower phreatic surface associated with loading Condition III results in higher local factors of safety in this layer than were determined for loading Condition I.

2.5D.9.6.4 Conclusion

There is ample margin against the development of 5 x 10^{-2} strain in the Auxiliary Dam during loading Condition III. Therefore, the Auxiliary Dam is stable and will maintain its integrity during loading Condition III.

2.5D.10 PROPERTIES OF MATERIAL M

2.5D.10.1 Introduction

Material M is a composite material obtained by mixing representative composite samples from test pits in borrow area M; see Figure 2.5D-20. A test program was undertaken to determine

the physical properties and static and dynamic stress-strain characteristics of this material. Section 2.5D.10 is devoted to the discussion of test procedures, interpretation of test data, and the derivation of static and dynamic characteristics of material M.

2.5D.10.2 Origin and Preparation of Material M

2.5D.10.2.1 Geology

Soils in borrow area M are residual soils formed by weathering of the Triassic deposits consisting of sandstones, siltstones, claystones, and conglomerates. The depth and degree of weathering are variable. The in-situ residual soils used for preparing composite material M are of stiff consistency and extend to depths of approximately 5 ft. to 10 ft. (Section 2.5D.13.3).

2.5D.10.2.2 Sampling Locations

The location of borrow area M is shown in Figure 2.5D-20. Six test pits were excavated in this area to depths ranging from 5 ft. to 10 ft. and samples of different soils encountered were withdrawn for index property test. Table 2.5D-13 summarizes properties of soils taken from representative borings and test pits in this area. For boring logs, see Appendix 2.5A.

2.5D.10.2.3 Preparation of Composite Material

During excavation, the proportion of different soils in each pit was determined and a composite sample was prepared by mixing soils in the same proportion as for those in-situ. Approximately 300 lb. of material was obtained from each pit, thoroughly mixed, and a representative portion withdrawn for index property tests. The tests showed that composite samples from two test pits (TPM4A and TPM5A) were not representative of the soils in other pits or borings in the area and, therefore, were not used for preparing material M. Composite samples from the remaining four pits (TPM1, TPM2, TPM3A, and TPM6) were mixed thoroughly to obtain composite material M which was used in all further laboratory investigations.

2.5D.10.3 Physical Properties

2.5D.10.3.1 General

The following physical properties were determined:

a) index properties, including grain-size distribution, Atterberg limits, and specific gravity;

b) compaction characteristics; and

c) permeability.

2.5D.10.3.2 Index Properties

Index properties of the material are summarized in Table 2.5D-14 and the grain-size distribution curve is given in Figure 2.5D-21. On the basis of these properties, the material is described as red-brown silty clayey coarse to fine sand, with a trace of fine gravel; it is classified as SC according to Unified Soil Classification System.

2.5D.10.3.3 Compaction Characteristics

Standard and modified compaction tests were performed in accordance with ASTM D698-68T and ASTM D1557 Method A, respectively. The wet method was used; i.e., the method which consists of adjusting the water content of the specimen by wetting or drying. The dry method; i.e., the method which consists of air drying the specimen and adding water, was not used because it modifies the properties of the residual soil and compaction in the field will be done under conditions similar to those used for the wet method. Standard and modified compaction characteristics are summarized in Table 2.5D-14. The standard and modified compaction curves obtained from the wet method are shown on Figure 2.5D-21.

2.5D.10.3.4 Permeability

Two specimens were compacted at 100 percent standard compaction and optimum water content for permeability tests. The tests showed that the material is rather impervious with a permeability of 10^{-8} cm/sec.; see Table 2.5D-14.

2.5D.10.4 Static Stress-Strain Characteristics

2.5D.10.4.1 Purpose

The purpose of the static tests is to determine the stress-strain and Poisson's ratio parameters needed in a non-linear static finite element analysis. Two testing procedures have been utilized for determining these parameters in previous studies (Reference 2.5D-17). In one procedure, unconsolidated-undrained (UU) triaxial tests are conducted on partially saturated specimens. In the other procedure, isotropically consolidated-drained (CID) triaxial tests are conducted on saturated specimens. Both testing procedures have been used to determine the parameters for material M required in the static stress analysis.

2.5D.10.4.2 UU Tests

2.5D.10.4.2.1 Test Procedure

Specimens were molded at 100 percent standard compaction and optimum water content and cured. A specimen was placed inside a rubber membrane, enclosed in a mercury-filled chamber, and subjected to confining pressure in a triaxial cell, no drainage being allowed to occur. An axial deviator stress was next applied at a strain rate of 1×10^{-2}/minute. The applied stress, axial strain, and volume change were recorded.

2.5D.10.4.2.2 Results of UU Tests

The stress-strain curves and strength envelope derived from the UU tests are given in Figure 2.5D-22 and 2.5D-23, respectively.

2.5D.10.4.3 CID Tests

2.5D.10.4.3.1 Test Procedure

Four specimens were molded at 100 percent standard compaction and optimum water content. They were cured, isotropically consolidated under different confining pressures applied in

increments, and saturated under backpressure. After saturation, each specimen was tested in a triaxial cell at an average strain rate of approximately 4 x 10^{-5}/minute, allowing complete drainage. The applied deviator stress, axial deformation, and volume change were measured.

2.5D.10.4.3.2 Results of CID Tests

The stress-strain curves and strength envelope derived from the CID tests are given in Figures 2.5D-24 and 2.5D-25, respectively. These results were used to determine static strength parameters and stress-strain properties of the material; see Section 2.5D.14.4.

2.5D.10.4.4 Derivation of Parameters for Static Stress Analysis

For purposes of incremental static stress analysis, soil behavior can be represented by stress-dependent hyperbolic relationships (Reference 2.5D-16). Thus, the elastic properties (E_t and μ_t) at any point on the stress-strain curve can be conveniently related to the initial elastic properties and stress conditions through a set of parameters; different relationships are defined for primary loading and unloading or reloading (Reference 2.5D-6).

For primary loading, the initial tangent modulus E_i can be expressed as

$$E_i = Kp_a \frac{(\sigma^3)^n}{p_a} \tag{1}$$

Where σ_3 is the effective confining pressures, p_a is the atmospheric pressure, and K and n are parameters.

The tangent modulus E_t is represented by the equation

$$E_t = \left[1 - \frac{R_f\,(1-\sin\phi)\,(\sigma_1-\sigma_3)}{2c\,\cos\phi - 2\,\sigma_3\,\sin\phi}\right]^2 E_i \tag{2}$$

where R_f is the ratio of the stress difference at failure $(\sigma_1\text{-}\sigma_3)$f to the asymptotic stress difference $(\sigma_1\text{-}\sigma_3)_{ult}$ and c and θ are the Mohr-Coulomb shear strength parameters.

Similarly, the Poisson's ratio at a given point on the stress-strain curve, μ_t, can be expressed in terms of the initial Poisson's ratio, μ_i, stress conditions and three parameters, G, F, and D (Reference 2.5D-17).

$$u_i = G - F \log \frac{(\sigma_3)}{p_a} \tag{3}$$

$$\mu_t = \frac{\mu_i}{\left[1 - \dfrac{D\,(\sigma_1-\sigma_3)}{\{Kp_a\}\left\{\dfrac{\sigma_3}{(p_a)\,n}\right\}\left[1-\dfrac{R_f\,(\sigma_1-\sigma_3)(1-\sin\phi)}{2c\,\cos\phi - 2\sigma_3\,\sin\phi}\right]}\right]^2} \tag{4}$$

Using the above equations and data from the CID and UU tests, values of K, n, R_f, G, F and D were determined; see Table 2.5D-15.

2.5D.10.5 Dynamic Stress-Strain Characteristics

Laboratory investigations were carried out to determine the cyclic strength and dynamic properties of material M. The test program consisted of the following:

a) stress-controlled cyclic triaxial tests for determining the cyclic strength characteristics and shear modulus and damping ratio at high strain;

b) strain-controlled cyclic triaxial tests for determining shear modulus and damping ratio at intermediate levels of strain; and

c) cyclic torsion tests for determining the shear modulus and damping ratio at low levels of strain.

2.5D.10.5.1 Specimen Preparation

Specimens were molded at 95 percent and 100 percent standard compaction and optimum water content with a Harvard kneading compactor. For cyclic triaxial tests, specimens approximately 2.0 in. diameter and 4.0 in. high were used. For cyclic torsion tests, specimens of 1.4 in. high and 2.5 in. diameter were used. The specimens were cured, saturated under backpressure, and subjected to initial consolidation. Specimens for strain-controlled cyclic triaxial test and cyclic torsion tests were consolidated isotropically ($K_c = \frac{\bar{\sigma}_{1c}}{\bar{\sigma}_{3c}} = 1$) while those for the cyclic stress-controlled tests were consolidated under three different initial consolidation ratios, K_c = 1, 1.5, and 2.

2.5D.10.5.2 Stress-Controlled Cyclic Triaxial Tests

Thirty-three tests were conducted on specimens molded at 100 percent standard compaction and ten tests on specimens molded at 95 percent standard compaction. The tests were performed using a Modular Testing System (MTS) which applies a sinusoidally varying cyclic deviator stress of peak amplitude, $\pm\ \sigma_d$, at a frequency of one hertz. The applied deviator stress, axial deformation, and pore water pressure were measured. Loading is continued either until the specimen undergoes excessive strain or the number of cycles is high ($N \simeq 1000$).

2.5D.10.5.2.1 Results of Stress-Controlled Cyclic Triaxial Tests

Results of stress-controlled cyclic triaxial tests are summarized in Tables 2.5D-16, 2.5D-17, and 2.5D-18. For specimens compacted at 100 percent standard compaction, the relationship between the superimposed cyclic stress ratio ($\pm\sigma_d/2\bar{\sigma}_{3c}$) and the number of cycles required to cause initial liquefaction* for K_c = 1 is given in Figure 2.5D-26. Figure 2.5D-27, 2.5D-28, and 2.5D-29 present the relationship between the stress ratios and the number of cycles N required to cause 5 x 10^{-2} strain for K_c = 1, 1.5 and 2, respectively. These data were utilized for determining the cyclic strength of material M. Table 2.5D-18 presents the results of shear modulus and damping ratio determinations for several of the stress-controlled cyclic triaxial tests.

* Defined as the number of cycles causing either ±2.5 x 10-2 axial strain or a pore pressure equal to the initial effective confining pressures, whichever occurs first.

2.5D.10.5.3 Strain-Controlled Cyclic Triaxial Tests

2.5D.10.5.3.1 Test Procedure

Four tests were conducted at intermediate levels of strain (10^{-3} to 10^{-4}) on isotropically consolidated specimens by subjecting them to sinusoidally varying axial strains at a frequency of one hertz. The deformations, axial load, and pore pressures developed in the specimens were recorded. Tests were continued up to approximately 30 cycles.

2.5D.10.5.3.2 Test Results

Results of the strain-controlled cyclic triaxial tests are presented in Table 2.5D-19. To account for end effects and non-uniform strains within the specimens, the average axial strain was obtained by dividing the measured axial strain by a correction factor of 1.5 (Reference 2.5D-39). The shear modulus and damping ratio of the material was determined as outlined in Table 2.5D-19.

2.5D.10.5.4 Cyclic Torsion Tests

2.5D.10.5.4.1 Test Procedure

Specimens for cyclic torsion tests were molded at 100 percent standard compaction, saturated and isotropically consolidated under different confining pressures. The tests were conducted by using the equipment and procedure suggested by Reference 2.5D-9. After consolidation, a vibrating head was attached to the top of the specimen and a resonant frequency of the system and the corresponding amplitude were determined. Upon the completion of each cyclic stage, the vibration decay response of the system was recorded. The confining pressure on the sample was increased to a higher level and tests for shear modulus and damping were repeated.

2.5D.10.5.4.2 Results of Cyclic Torsion Tests

Results of the cyclic torsion tests are given in Table 2.5D-20. To account for non-uniform torsional strains in the specimens, the peripheral shear strains were multiplied by a factor of 0.7 to obtain the average strain. The shear modulus and damping ratios were determined for the strain range of 10^{-4} to 10^{-5}.

2.5D.10.5.5 Determination of Cyclic Strength of Material M

The cyclic strength of material M was evaluated by analyzing results of stress controlled cyclic triaxial tests presented in Table 2.5D-16 and 2.5D-17 and Figure 2.5D-26 through 2.5D-29.

Analyses of several case histories and laboratory investigations have shown (Reference 2.5D-28 and Reference 2.5D-29) that cyclic triaxial tests on isotropically consolidated specimens indicate higher dynamic strengths than the stresses required to cause failure or excessive strains in the field. Therefore, for predicting the field cyclic strength of the material from cyclic triaxial tests on isotropically consolidated samples, different correction factors are incorporated as described below. For isotropically consolidated specimens (K_c = 1), the correction factor, C_r, generally lies between 0.55 and 0.80. For anisotropically consolidated specimens (K_c = 1.5 or

2.0), the data from seed et.al. (Reference 2.5D-28) indicates that correction factors are not required to predict the field cyclic strength from cyclic triaxial tests.

2.5D.10.5.5.1 Isotropically Consolidated Specimens

It has been shown that for clean sands, the correction factor Cr is a function of relative density (Reference 2.5D-30). Because material M is a silty clayey sand, it is difficult to associate a relative density with the material at a given degree of compaction. However, through a comparison of published data (Reference 2.5D-30) and test results given on Figure 2.5D-26, an equivalent relative density can be determined for the material with respect to the relative density of a sand having the same mean grain size. It was found that for confining pressures varying from 2 k/ft.^{2} to 12 k/ft.^{2}, the behavior of material M at 100 percent standard compaction at optimum water content is comparable to that of a sand having a relative density of approximately 64 percent to 100 percent, with the higher values corresponding to lower confining pressures. The following correction factors C_r were thus obtained.

CORRECTION FACTORS FOR MATERIAL M

Confining Pressure $\bar{\sigma}_{3c}$, k/ft.$_2$	Correction Factor C_R
2	0.80
4	0.69
8	0.65
12	0.61

Using the above correction factors and the data in Figure 2.5D-27, the relationship was determined between cyclic shear stress causing 5 x 10^{-2} strain in five cycles and the effective normal stress for isotropically consolidated specimens (K_c = 1); see Figure 2.5D-30. Similarly, the relationship between the cyclic shear stress causing 5 x 10^{-2} strain in ten cycles and the normal effective stress is shown in Figure 2.5D-31.

2.5D.10.5.5.2 Anisotropically Consolidated Specimens

Cyclic tests of anisotropically consolidated samples are made in order to simulate the condition within the constructed embankment where there are initial static shear stresses on the potential failure planes. The procedures described in Reference 2.5D-28 have been used to interpret these tests to determine the cyclic strength characteristics. These procedures utilize Mohr envelope relationships to find the static stresses and the superimposed cyclic shear stresses on the plane of failure in the specimen.

The ratio of the initial static shear stress to the normal stress on the plane of failure is designated α. For the constructed embankment, $\propto$ is determined for various points in the embankment by the static finite element analysis, as described in Section 2.5D.14. For the laboratory samples, α depends primarily on the consolidation conditions (i.e., on the value of K_c) and also on the strength parameter, ϕ'. For values of K_c = 1.0., 1.5, and 2.0 (and ϕ' = 30°) the corresponding values of a are equal to 0, 0.19, and 0.345, respectively.

The cyclic strength characteristics determined from tests on anisotropically consolidated specimens are presented on Figure 2.5D-30 and 2.5D-31. Figure 2.5D-30 shows the relationship between the normal effective stress and the cyclic shear stress causing 5 x 10^{-2}

strain in five cycles; Figure 2.5D-31 shows the similar relationship for ten cycles. As can be seen in these figures, the cyclic stresses causing 5 x 10^{-2} strain are nearly equal for α = 0.19 and α = 0.345. Therefore, a single line has been drawn representing the cyclic strength characteristics for a ≥ 0.12.

2.5D.10.5.6 Dynamic Properties of Material M

Four parameters were used to define the strain-dependent dynamic properties of material M. The parameters are the shear modulus G, the damping ratio λ, the ratio of horizontal to vertical effective stresses, $\overline{K}_o$, and the Poisson's ration μ.

2.5D.10.5.6.1 Shear Modulus G

Shear modulus was determined from stress-controlled and strain-controlled cyclic triaxial tests and cyclic torsion tests for high, intermediate, and low levels of strain, respectively. Data given in Table 2.5D-19 and 2.5D-20 were utilized to determine the relationship between shear modulus and the normal effective stress $\bar{\sigma}_o$ at very low strains ($\simeq 10^{-6}$); see Figure 2.5D-32. It has been shown that this relationship can be represented by an equation (Reference 2.5D-30 and 2.5D-10).

$$G_{max} = K_{2,max}(\bar{\sigma}_o^{\frac{1}{2}}) \quad (5)$$

where G_{max} is the maximum shear modulus in k/ft.^{2}, $\bar{\sigma}_o$ is the mean normal effective stress expressed in lb./ft.^{2}, and $K_{2,\, max}$ is a parameter. As can be seen from Figure 2.5D-32, an average value of $K_{2,\, max}$ = 120 was selected for material M compacted at 100 percent standard compaction at optimum water content.

The variation of shear modulus with shear strain was also established on the basis of results of the above tests. The shear moduli are plotted versus shear strain in Figure 2.5D-33. In order to present the data at various confining pressure of 1000 lb./ft.^{2} in Figure 2.5D-33. The relationship between shear modulus and shear strain was established on the basis of the data in Figure 2.5D-33; the generalized relationship is presented in Figure 2.5D-34. Figure 2.5D-32 and 2.5D-34 define the shear modulus characteristics of material M for use in the dynamic analyses.

2.5D.10.5.6.2 Damping Ratio λ

Damping ratios were determined from stress-controlled and strain-controlled cyclic triaxial tests and cyclic torsion tests; see Tables 2.5D-18, 2.5D-19, and 2.5D-20. The variation of damping ratio with strain established from the same data is given in Figure 2.5D-35.

2.5D.10.5.6.3 Ratio of Horizontal to Vertical Effective Stresses, $\overline{K}_o$

Values of $\overline{K}_o$ were selected on the basis of published data for compacted and preconsolidated material (Reference 2.5D-5 and 2.5D-18). A value, $\overline{K}_o$ = 0.6, is selected to account for compaction of material.

2.5D.10.5.6.4 Poisson's Ratio

Typical values of Poisson's ratio for different materials are available in several publications (e.g., References 2.5D-23 and 2.5D-1). A value of $\mu = 0.35$ was selected on the basis of published data.

2.5D.11 PROPERTIES OF MATERIAL Z

2.5D.11.1 Introduction

Material Z is a composite material obtained by mixing representative composite samples from test pits in borrow area Z; see Figure 2.5D-36. A test program was undertaken to determine the physical properties and static and dynamic stress-strain characteristics of material Z. This test program was similar to that reported in Section 2.5D.10 for Material M. The presentation of test results for material Z will be similar to that for material M in Section 2.5D.10. The test procedures and methods for interpretation of test data are similar to those discussed in Section 2.5D.10 and will not be repeated in this section.

2.5D.11.2 Origin and Preparation of Material Z

2.5D.11.2.1 Geology

Soils in borrow area Z are residual soils formed from Triassic sandstones, siltstones, claystones, and conglomerates. The in-situ residual soils are of stiff consistency and extend to depths of about 5 ft. to 10 ft.

2.5D.11.2.2 Sampling Locations

The locations of borrow area Z test pits and sampled borings are shown in Figure 2.5D-36. Table 2.5D-21 summarizes properties of soils taken from representative borings and test pits in borrow area Z. For boring logs see Appendix 2.5A.

2.5D.11.2.3 Preparation of Composite Material

The same procedure was used to prepare material Z as was used for material M. Soil from all four test pits was used for the composite material.

2.5D.11.3 Physical Properties

2.5D.11.3.1 Index Properties

The index properties of Material Z are summarized in Table 2.5D-22. The grain-size distribution curve is presented in Figure 2.5D-37. On the basis of these index properties, material Z is described as brown silty clay with some coarse to fine sand and trace of fine gravel; it is classified as CL according to the Unified Soil Classification System.

2.5D.11.3.2 Compaction Characteristics

Standard and modified compaction test results are presented in Table 2.5D-22 and Figure 2.5D-37.

2.5D.11.3.3 Permeability

Tests were performed on two specimens compacted at 100 percent standard compaction and optimum water content. The measured permeability was 2 x 10^{-8} cm/sec.; see Table 2.5D-22.

2.5D.11.4 Static Stress-Strain Characteristics

2.5D.11.4.1 General

The test program for material Z was similar to that reported in Section 2.5D.10 for material M. Unconsolidated undrained (UU) triaxial tests and isotropically consolidated drained (CID) triaxial tests were performed in this test program.

2.5D.11.4.2 UU Tests

The stress-strain curves and strength envelope derived from the UU tests are given in Figure 2.5D-38 and 2.5D-39, respectively.

2.5D.11.4.3 CID Tests

The stress-strain curves and strength envelope derived from the CID tests are given in Figure 2.5D-40 and 2.5D-41, respectively.

2.5D.11.4.4 Parameters for Static Stress Analysis

The parameters K, n, R_f, G, F, and D for the static stress analysis are presented in Table 2.5D-23. These parameters were determined from the results of the UU and CID triaxial tests.

2.5D.11.5 Dynamic-Strain Characteristics

2.5D.11.5.1 General

The test program for Material Z was similar to that reported in Section 2.5D.10 for material M. The test program consisted of (1) stress-controlled cyclic triaxial tests; (2) strain-controlled cyclic triaxial tests; and (3) cyclic torsion tests.

2.5D.11.5.2 Stress-Controlled Cyclic Triaxial Tests

Results of stress-controlled cyclic triaxial tests are summarized in Tables 2.5D-24, 2.5D-25, and 2.5D-26. For specimens compacted at 97 percent standard compaction and optimum water content, the relationship between the stress ratio ($\pm\sigma_d/2\bar{\sigma}_{3c}$) and the number of cycles required to cause initial liquefaction for K_c = 1.0 is shown in Figure 2.5D-42. The relationships for $\pm\ \sigma_d/2\bar{\sigma}_{3c}$ vs. number of cycles required to cause 5 x 10^{-2} strain for K_c = 1, 1.5, and 2 are shown in Figure 2.5D-43, 2.5D-44, and 2.5D-45. Similar relationships for specimens molded at 100 percent standard compaction and optimum water content are shown in Figure 2.5D-46 (initial liquefaction) and 2.5D-47 and 2.5D-48 (5 x 10^{-2} strain for K_c = 1 and 1.5). The results of four tests made on specimens molded at approximately 98 percent compaction and 2 percent above optimum water content are included in Tables 2.5D-24 and 2.5D-25. Moduli and damping ratios determined from three stress-controlled cyclic triaxial tests on specimens at 97 percent standard compaction are given in Table 2.5D-26.

2.5D.11.5.3 Strain-Controlled Cyclic Triaxial Tests

Results of strain-controlled cyclic triaxial tests at 97 percent standard compaction are presented in Table 2.5D-27.

2.5D.11.5.4 Cyclic Torsion Tests

Results of cyclic torsion tests at 97 percent standard compaction and optimum water content are presented in Table 2.5D-28, and results at 100 percent standard compaction and optimum water content are presented in Table 2.5D-29. Table 2.5D-29 also includes results of tests conducted on specimens compacted at approximately 100 percent standard compaction and water contents 2 percent above optimum.

2.5D.11.5.5 Cyclic Strength of Material Z

Correction factors have been developed to predict the field cyclic strength of material Z from data obtained from cyclic triaxial tests at K_c = 1.0. Following the same procedure as outlined in Section 2.5D.10, it was found that the cyclic strength of material Z at 97 percent standard compaction and optimum water content is comparable to that of a clean sand at relative density of 66 percent to 100 percent. The cyclic strength of material Z at 100 percent standard compaction and optimum water content is comparable to that of a clean sand at relative density 80 percent to 100 percent. Therefore, the following correction factors were obtained.

CORRECTION FACTORS FOR MATERIAL Z

Confining Pressure $\bar{\sigma}_{3c}$, k/ft.^{2}	Correction Factor, Cr 97 Percent Standard Compaction at Optimum Water Content	Correction Factor, Cr 100 Percent Standard Compaction at Optimum Water Content
1.25	0.80	0.80
2.5	0.73	0.785
5.0	0.62	0.68

2.5D.11.5.5.1 Isotropically Consolidated Specimens

Ninety-seven percent standard compaction and optimum water content: Using the above correction factors, and the test data presented in Figure 2.5D-43, the relationship between cyclic shear stress causing 5 x 10^{-2} strain in five cycles and the normal effective stress was determined; see Figure 2.5D-49. The similar relationship for 5 x 10^{-2} strain in ten cycles is presented in Figure 2.5D-50.

One-hundred percent standard compaction and optimum water content: Using the above correction factors, and the test data in Figure 2.5D-47, similar relationships were obtained for 100 percent standard compaction. The cyclic shear stresses cuasing 5 x 10^{-2} strain in five cycles and ten cycles are shown in Figure 2.5D-51 and 2.5D-52, respectively.

2.5D.11.5.5.2 Anisotropically Consolidated Specimens

The test results from anisotropically consolidated specimens showed that the cyclic strength characteristics of material Z are not sensitive to different values of K_c (or different values of α). Therefore, single curves have been drawn defining the cyclic strength characteristics at 5 x 10^{-2} strain; see Figure 2.5D-49 (five cycles) and 2.5D-50 (ten ycles) for material Z at 97 percent standard compaction and optimum water content and Figure 2.5D-51 (five cycles) and 2.5D-52 (ten cycles) for material Z at 100 percent standard compaction and optimum water content.

2.5D.11.5.5.3 Effect of Molding Water Content on Cyclic Strength Characteristics

The effect of the molding water content on the cyclic strength of material Z was investigated by making stress-controlled cyclic triaxial tests on specimens compacted at 98 percent standard compaction and 2 percent above optimum water content. The test results are reported in Tables 2.5D-24 and 2.5D-25. The cyclic strengths determined by these tests (as indicated by the magnitude of cyclic stress and number of cycles required to cause 5 x 10^{-2} strain) are generally within the range of cyclic strength determined at 97 percent to 100 percent standard compaction and optimum water content, and, therefore, are within the expected variation of the material properties.

2.5D.11.5.6 Dynamic Properties of Material Z

2.5D.11.5.6.1 Shear Modulus G

The relationships between shear modulus, mean normal effective stress, and shear strain for material Z at 97 percent standard compaction and optimum water content are presented in Figures 2.5D-53, 2.5D-54, and 2.5D-55, based on the data in Tables 2.5D-26, 2.5D-27, and 2.5D-28. Figure 2.5D-53 describes the relationship between shear modulus and mean normal effective stress at very low strains ($\simeq 10^{-6}$). Figures 2.5D-54 and 2.5D-55 show the relationship between shear modulus and shear strain. The curves shown in Figures 2.5D-53 and 2.5D-55 define the modulus relationships for dynamic analysis. As can be seen in Figure 2.5D-53, an average value of $K_{2\,max}$ equal to 100 has been selected. The cyclic torsion test results on material Z at 100 percent standard compaction, presented in Table 2.5D-29, indicate that the shear modulus values are somewhat lower or essentially comparable to those obtained at 97 percent standard compaction. Lower values of modulus would not be expected (Reference 2.5D-10 and Reference 2.5D-31) and, therefore, it has been assumed that the shear moduli at 97 percent and 100 percent compaction are essentially the same.

2.5D.11.5.6.2 Damping Ratio λ

Damping ratios were determined for specimens compacted at 97 percent standard compaction and optimum water content from stress-controlled and strain-controlled cyclic triaxial tests and cyclic torsion tests; see Tables 2.5D-26, 2.5D-27, and 2.5D-28. The variation of damping ratio with strain established from these data is presented in Figure 2.5D-56.

For material Z at 100 percent standard compaction, damping ratios were determined by cyclic torsion tests; see Table 2.5D-29. These data indicate that the damping ratios at 100 percent standard compaction are essentially comparable to those at 97 percent compaction.

2.5D.11.5.6.3 Ratio of Horizontal to Vertical Effective Stresses, $\overline{K}_o$

Values of $\overline{K}_o$ were selected on the basis of published data for compacted and reconsolidated materials (Reference 2.5D-5 and Reference 2.5D-18). A value of $\overline{K}_o$ = 0.6, was selected to account for compaction of the material.

2.5D.11.5.6.4 Poisson's Ratio, μ

A value of μ = 0.35 was selected on the basis of published data (Reference 2.5D-23).

2.5D.12 PROPERTIES OF FILTERS AND ROCKFILLS

2.5D.12.1 Introduction

The static and dynamic properties of filters and rockfill materials for the Main Dam, Auxiliary Dam, and Auxiliary Separating Dike are described in this section. These properties were determined on the basis of the material index properties, design and construction criteria, and published and unpublished data on the properties of similar materials.

2.5D.12.2 Filter Materials

2.5D.12.2.1 Main Dam

2.5D.12.2.1.1 Proposed Design and Construction Criteria

The Main Dam will have two filters designated as fine and coarse filters, each 8-ft.-thick, on either side of the core. The specified grain-size limits and average curves for the fine and coarse filters are shown in Figure 2.5D-57. On the basis of these curves, the fine filter will be a well-graded coarse to fine sand, SW, with a mean grain-size (D_{50}) of 0.56 mm. and the coarse filter will be a well-graded sandy gravel GW with a mean grain-size (D_{50}) of 9.0 mm. The fine and coarse filters of the Main Dam will be compacted to an average relative density of 75 percent, except for the upstream coarse filter which will be compacted to an average relative density of 80 percent above Elevation 220 ft.

2.5D.12.2.1.2 Static Properties

The static properties of filter materials were selected on the basis of published data on compacted granular materials (Reference 2.5D-2; Reference 2.5D-18; and Reference 2.5D-17). Unit weights and parameters for static stress analysis are given in Table 2.5D-30.

2.5D.12.2.1.3 Dynamic Properties

The dynamic properties of the filters, viz the shear modulus G, the damping ratio λ, the ratio of horizontal to vertical effective stress K_o, the Poisson's ratio μ, and the variation of shear modulus and damping ratio with strain, were determined on the basis of published data on the behavior of granular materials.

2.5D.12.2.1.3.1 Shear Modulus G

The shear modulus G and the variation of shear modulus with strain were based on published data (Reference 2.5D-30) for the fine filter and on the data reported in References 2.5D-30 and 2.5D-42 for coarse filter; see Table 2.5D-31 and Figure 2.5D-58.

2.5D.12.2.1.3.2 Damping Ratio, λ

Work by Wong (Reference 2.5D-42) has shown that damping ratios for gravels are in the same range as sands. Therefore, the average curve for variation of damping with strain (Reference 2.5D-30) was used for both filters; see Figure 2.5D-59.

2.5D.12.2.1.3.3 Ratio of Horizontal to Vertical Effective Stress, K_o

Typical values of K_o for compacted granular materials are available in literature (Reference 2.5D-5 and Reference 2.5D-18). A value of 0.6 was selected to account for compaction and a relative density of 80 percent.

2.5D.12.2.1.3.4 Poisson's Ratio, μ

Typical values of Poisson's ratios for compacted sands are available in the literature (References 2.5D-23 and 2.5D-1). A value of μ = 0.35 was selected on the basis of the published data.

2.5D.12.2.1.4 Dynamic Strength

The cyclic strength of the filter materials for the main dam has been estimated based on published data for granular soils. Factors considered in the assessment of the cyclic strength characteristics include: (a) relative density; (b) grain size; (c) gradation; and (d) effect of initial static shear stresses.

The fine and coarse filters of the main dam are compacted to an average relative density of 75 percent except for the upstream coarse filter which is compacted to an average relative density of 80 percent above Elevation 220 ft.

The effect of mean grain size on the cyclic strength of granular soils has been studied by Lee and Fitton, Seed and Peacock, and Wong (see References 2.5D-22, 2.5D-29, and 2.5D-42). The results by Lee and Fitton indicate a substantial increase in cyclic strength as D_{50} increases from about 0.1 to 4 millimeters (mm.), and a very large increase in strength as D_{50} increases above 4 mm.; see Figure 2.5D-60. Because of the small diameter of samples tested in comparison to the maximum grain sizes, the results of Lee and Fitton may not be representative for soils with mean grain size greater than 10 mm. The data presented by Seed and Peacock (Reference 2.5D-29) indicate a similar increase in strength of the Lee and Fitton results over the grain size range considered (0.1 to 1 mm.). The results of Wong (Reference 2.5D-42) indicate only a slight increase in strength as the mean grain size increases from 0.6 to 10 mm., and a rapid increase in strength for mean grain sizes larger than 10 mm.; see Figure 2.5D-60. Because Wong's tests were made using large diameter (12 in.) samples, the results are probably more representative than Lee and Fitton's for soils having a mean grain size greater than 10 mm. Field evidence during the Alaskan earthquake of 1964 also consistently indicates

that gravelly soils with large grain size have greater cyclic strengths than do sands (Reference 2.5D-32).

The mean grain size of the fine filter is estimated to be 0.56 mm. (average) and of the coarse filter 9.0 mm. (average). Although the available data would indicate a slight to substantial increase in strength as the grain size increases, this effect has been conservatively ignored in assessing the cyclic strength characteristics of the filters. Rather, the cyclic strength of the filters has been assessed on the basis of data for Sacremento River sand (Reference 2.5D-20). The data on this particular sand (a uniformly graded fine sand with D_{50} = 0.2 mm.) has been utilized because of the comprehensive cyclic test data published, covering a wide range of confining pressures, relative densities, and cyclic stress and strain levels.

The proposed filter materials will be well-graded materials, whereas Sacramento River sand is a uniformly graded sand. Very little data is available on the cyclic strength of uniformly graded sands or gravels as compared to well-graded sands or gravels. Lee and Fitton (Reference 2.5D-22) tested a well-graded fine to coarse silty sand and concluded that there was very little, if any, difference in the strength of the well-graded sand as compared to a uniformly graded sand of the same mean grain size. Wong (Reference 2.5D-42) tested a well-graded sandy gravel and found that the strengths were lower than uniformly graded gravels at low strains ($\simeq 5 \times 10^{-2}$); however, the well-graded gravels were more resistant to the development of large strains; i.e., 10×10^{-2}.

Therefore, the published cyclic triaxial test data for Sacramento River sand provide a reasonable basis for assessing the cyclic strength characteristics of the filters. Figure 2.5D-61, prepared from this data, shows the cyclic shear stresses required to cause 5×10^{-2} strain in five and ten cycles for filter materials compacted to a relative density of 75 percent or 80 percent. The same published data also indicate that for a relative density of 85 percent, the cyclic shear stresses required to cause 5×10^{-2} strain are approximately 1.35 times greater than the cyclic shear stresses required to cause 5×10^{-2} strain for 75 percent relative density. The strength characteristics shown in Figure 2.5D-61, derived from tests on isotropically consolidated specimens (K_c = 1.0), do not include a correction factor. Because initial static shear stresses may be expected to occur in the filters (see Section 2.5D-14), the field strength will be increased and the resulting strength envelope will be close to that obtained directly from tests on isotropically consolidated samples (the effect of static shear stresses on increasing the cyclic shear strength of granular soils is described in References 2.5D-28 and 2.5D-21.

2.5D.12.2.2 Auxiliary Dam

2.5D.12.2.2.1 Proposed Design and Construction Criteria

The specified grain-size limits and an average distribution curve are shown in Figure 2.5D-62. The transition filter for the Auxiliary Dam will be a well-graded sand gravel mix with a mean grain size (D_{50}) of approximately 2.3 mm., and will be compacted to an average relative density of 75 percent below Elevation 220 ft. and to an average relative density of 80 percent above Elevation 220 ft.

2.5D.12.2.2.2 Static Properties

Static properties of the transition filter were derived by using the same procedure as for the filters of the Main Dam. These values are summarized in Table 2.5D-30.

2.5D.12.2.23 Dynamic Properties

Dynamic properties of the transition filter for the Auxiliary Dam were determined in the same manner as for the filters for the Main Dam. These properties are given in Table 2.5D-31 and Figures 2.5D-58 and 2.5D-59.

2.5D.12.2.2.4 Dynamic Strength

The dynamic strength envelope shown in Figure 2.5D-61 is also applicable to the transition filter for the Auxiliary Dam.

2.5D.12.3 Rockfill Materials

2.5D.12.3.1 Main Dam

2.5D.12.3.1.1 Proposed Design and Construction Criteria

Rockfill for the main dam was obtained from the spillway, spillway outlet and approach channel excavations. It consists of a minimum of 75 percent fresh granitic rock in a size range of 6 in. to 24 in. with a maximum particle size of 24 in. The mean grain size (D_{50}) is estimated at approximately 240 mm.

2.5D.12.3.1.2 Static Properties

The unit weights for the rockfill shell were selected on the basis of the rockfill at Keban Dam, Turkey (Reference 2.5D-8), which has properties similar to that of the proposed rockfill. Other static properties were selected on the basis of published data (Reference 2.5D-17); see Table 2.5D-32.

2.5D.12.3.1.3 Dynamic Properties

2.5D.12.3.1.3.1 Shear Modulus, G

The shear modulus for the rockfill was selected from available data on sands and gravels (Reference 2.5D-42). Since the shear modulus of gravel is higher than that of sand, a maximum shear modulus for rockfill higher than that of gravel was selected; see Table 2.5D-33. The variation in shear modulus with strain is assumed to be similar to that for sands; see Figure 2.5D-58.

2.5D.12.3.1.3.2 Damping Ratio, λ

Because the damping ratios in granular soils are not very sensitive to grain size (Reference 2.5D-42) the average relationship for sands was used; see Figure 2.5D-59.

2.5D.12.3.1.3.3 Ratio of Horizontal to Vertical Effective Stress, $\overline{K}_o$ and Poisson's Ratio, μ

Values were selected on the basis of published data (see References 2.5D-5, 2.5D-18, 2.5D-23, and 2.5D-1); see Section 2.5D.12.2.1.3 and Table 2.5D-33.

2.5D.12.3.1.4 Dynamic Strength

The dynamic strength of the rockfill has been estimated in a similar manner as for the filters. Based on the data by Lee and Fitton (Reference 2.5D-22) and Wong (Reference 2.5D-42) presented in Figure 2.5D-60, the effect of mean grain sizes larger than 10 mm. results in a rapid increase in cyclic strength. The data by Wong is utilized because it is probably more representative of the behavior of large particle size material and results in lower cyclic strengths than the data by Lee and Fitton.

In estimating the cyclic strength of the rockfill, the data in Figure 2.5D-60 by Wong has been extrapolated to mean grain size 75 mm. (approx 3 in.). The cyclic strength at 75 mm. is then compared with the strength of Sacramento River sand. By this comparison, the rockfill should be at least 65 percent stronger than Sacramento River sand. In Section 2.5D.12.2.1.4 the cyclic strength characteristics of Sacramento River sand were used to determine the cyclic strength of filter materials. Using the same procedure, the cyclic strength of compacted rockfill is estimated to be 1.65 times the strength of Sacramento River sand at 75 percent relative density. Figure 2.5D-63 presents the cyclic shear stresses required to cause 5×10^{-2} strain in five and ten cycles for rockfill at a relative density of 75 percent. As is the case in the filters, initial static shear stresses will occur in the rockfill. Therefore, the strength characteristics defined by Figure 2.5D-63 do not include a correction factor.

It is estimated that the specifications will result in a rockfill with mean grain size of the order of 240 mm. (approximately 9 in.). The larger grain size should result in strengths greater than defined by Figure 2.5D-63 which is based on a mean grain size of 75 mm.

In developing the cyclic strength characteristics, it has been assumed that pore pressures do not dissipate during the earthquake motions (i.e., the strength of the rockfill is for undrained conditions during cyclic loading). Because of the large particle size and high permeability of the rockfill, it is certain that any pore pressures would dissipate to some degree during the ground motions, and this will result in higher rockfill strengths.

2.5D.12.3.2 Auxiliary Dam and Separating Dike

2.5D.12.3.2.1 Proposed Design and Construction Criteria

The Auxiliary Dam and Auxiliary Separating Dike will have shell zones of random rockfill obtained from the excavations for the general plant area structures, channels, and the spillway. The rock will consist of weathered sandstone, siltstone, and claystone of which a minimum of 75 percent shall range in size from 1/4 in. to 24 in. The mean grain size (D_{50}) is estimated at approximately 34 mm.

2.5D.12.3.2.2 Static Properties

The unit weight of random rockfill was based on in-place unit weight measurements of a rockfill at Amos Dam, West Virginia (Reference 2.5D-40), with properties similar to those of the proposed rockfill. The unit weight of siltstone and sandstone is generally slightly smaller than that of granitic rock used for the Main Dam rockfill.

Other static properties were selected on the basis of published data used for selecting the properties for the rockfill for the Main Dam (Reference 2.5D-17); see Table 2.5D-32.

2.5D.12.3.2.3 Dynamic Properties

The shear modulus for the random rockfill has been estimated on the basis of the seismic velocity measurements made in a similar rockfill at Amos Dam, West Virginia (Reference 2.5D-40) and measurements made at Calaveras Rockfill Dam, California (Reference 2.5D-33).

At Amos Dam the rockfill zone consists of sandstone, siltstone, sand, and silt shales and up to five percent clay shales by volume. The rockfill was placed and spread by bulldozers with no additional compaction. As compacted, the rockfill has a maximum particle size of 8 in. with approximately 80 percent of the material being between 12 in. and 0.187 in. (No. 4 sieve). The in-place moist unit weight was determined as approximately 136 $lb./ft.^3$. Data from P-wave, S-wave, and R wave velocity measurements indicated a possible range in the shear modulus parameter $K_{2, max}$ of from 60 to 150 for the random rockfill at Amos Dam.

The rockfill at Calaveras Dam consists of material with a maximum particle size of 6 in., a few boulders of 12 in. size, and 25 percent fines. The rockfill has an estimated moist unit weight of 130 $lb./ft.^3$. Shear wave velocity measurements in the upstream rockfill at depths ranging from 20 ft. to 60 ft. indicated that $K_{2, max} \simeq 90$.

On the basis of the above data, $K_{2, max}$ of 90 was selected for the random rockfill; see Table 2.5D-33. The variation of shear modulus with strain, and the variation of damping ratio with strain were selected using the same procedure as for the rockfill of the Main Dam; see Figure 2.5D-58 and 2.5D-59.

The values for Poisson's ratio, μ, and ratio of horizontal to vertical effective stress, $\overline{K}_o$, were selected as described for the Main Dam rockfill.

2.5D.12.3.2.4 Dynamic Strength

The random rockfill for the Auxiliary Dam and Auxiliary Separating Dike consists of fragments of weathered sandstones, siltstones, and claystones well compacted under a vibratory roller. The softer fragments of siltstones and claystones tend to break down and the overall rockfill will be somewhat cohesive. At low confining pressures, it is reasonable to consider that the cyclic strength characteristics of the rockfill would be similar to the core material Z at 100 percent standard compaction which is derived from the same weathered rocks. At higher confining pressures, the grain-to-grain contacts will contribute a larger proportion of the strength and the material may exhibit a behavior intermediate between that of clean granular rockfill and a cohesive material such as core material Z.

The estimated cyclic strength characteristics of the random rockfill presented in Figures 2.5D-64 and 2.5D-65 are based on the preceding considerations. At low confining pressures (up to approximately 1.5 $k/ft.^2$), the results of tests on core material Z at 100 percent standard compaction (see Figures 2.5D-51 and 2.5D-52) have been used to define the cyclic stresses causing 5×10^{-2} strain in five and ten cycles. At higher confining pressures, a strength envelope intermediate between core material Z at 100 percent standard compaction and a clean granular rockfill have been estimated using the same procedure as described for the main dam rockfill, and using a relative density of 75 percent and a mean grain size of 30 mm. (approximately 1 1/4 in). (On the basis of specifications for random rockfill, the mean grain size of the random rockfill is estimated to be approximately 30 to 40 mm). Figure 2.5D-60 shows that for the mean grain

size the cyclic strength would be approximately 1.3 times higher than that for Sacramento River sand.

The strength characteristics derived from the random rockfill for five and ten cycles are shown in Figures 2.5D-64 and 2.5D-65, respectively.

2.5D.13 PROPERTIES OF FOUNDATION MATERIALS

2.5D.13.1 Introduction

The properties of the foundation materials used in the static and dynamic analyses for the Main Dam, Auxiliary Dam, and Auxiliary Separating Dike are summarized in this section. The foundation materials considered are in-situ residual soil and weathered rock. The selection of material properties used in the analyses was based on laboratory test results, field geophysical measurements, and published data on similar materials.

The in-situ residual soil and weathered rock in the main dam foundation were derived from the granite and gneiss. The in-situ residual soil and weathered rock in the auxiliary dam and auxiliary dike foundations were derived from the local Triassic sedimentary rock consisting of claystone, siltstone, sandstone, and conglomerate. Differential weathering in all of these deposits has produced variable depths and degrees of weathering.

In these analyses, suitable rock has been defined as material which has an average compressional wave velocity of 10,000 ft./sec. and requires blasting for excavation. Weathered rock is material which requires coring for sampling and can be excavated with a ripper on a Caterpillar D-8 tractor, but cannot be excavated by the blade of a Caterpillar D-8 dozer. In-situ residual soil occurs as a thin layer of stiff soil overlying weathered rock which can be sampled with a split spoon sampler and can be excavated with the blade of a Caterpillar D-8 dozer.

2.5D.13.2 Properties of Weathered Rock

2.5D.13.2.1 Main Dam

2.5D.13.2.1.1 Static Properties

Unit weights of rock samples from the Main Dam foundation area are given in Table 2.5B-3, Appendix 2.5B. The samples were taken from depths of approximately 22 $ft.^3$ to 80 ft. and have unit weights ranging from 166 $lb./ft.^3$ to 188 $lb./ft.^3$. For weathered rock at depths of approximately 0 ft. to 20 ft., a slightly lower value of the unit weight equal to 150 $lb./ft.^3$ was selected. Other parameters for static stress analysis were based on data reported in Reference 2.5D-17; see Table 2.5D-34.

2.5D.13.2.1.2 Dynamic Properties

2.5D.13.2.1.2.1 Shear Modulus

The shear modulus parameter $K_{2\,max}$ was determined for the weathered rock at the Auxiliary Dam by field seismic measurements (see Section 2.5D.13.2.2.3.1 below). A value of $K_{2\,max}$ = 700 was obtained. Results of unit weight and compressional wave velocity measurements on rock at the Main and Auxiliary Dam areas are given in Tables 2.5B-3 and 2.5B-4, respectively.

It is observed that for similar depths, the rock at the Main Dam has a higher unit weight and compression wave velocity then at the Auxiliary Dam. Therefore, it is conservative to use the same value of ($K_{2\,max}$ = 700) for weathered rock at the Main Dam as was used for the Auxiliary Dam.

2.5D.13.2.1.2.2 Variation of Shear Modulus and Damping Ratio with Shear Strain

Because of the relatively high stiffness of weathered rock, the shear strains induced by the SSE are expected to be very low (of the order of 10^{-5}). At this strain level, the reduction in shear modulus with strain is approximately 1 percent - 2 percent (Reference 2.5D-26) for typical rock and approximately 4 percent to 5 percent for sand (Reference 2.5D-30). Therefore to incorporate a measure of conservatism, the average relationship between shear modulus and shear strain for sands was used for weathered rock at the Main Dam. For the same strain level as above, the typical damping ratio of rock is low, of the order of 0.01 to 0.02 (Reference 2.5D-26). Work by Seed and Idriss (Reference 2.5D-30) and Hardin and Drenevich (Reference 2.5D-10) has shown that, for a corresponding strain level, sand has a comparable average damping ratio. Therefore, the average relationship between damping ratio and shear strain for sand (Reference 2.5D-31) was used for weathered rock; see Figure 2.5D-59.

2.5D.13.2.1.2.3 Ratio of Horizontal to Vertical Effective Stress

Values of earth pressure coefficient at rest for preconsolidated materials are available in the literature (Reference 2.5D-19). A value of $\overline{K}_o$= 0.6 was selected for weathered rock to account for preconsolidation effects.

2.5D.13.2.1.2.4 Poisson's Ratio

Typical values of Poisson's ratio are available in the literature (e.g., Reference 2.5D-19 and 2.5D-23). A value of 0.35 was selected for weathered rock.

2.5D.13.2.2 Auxiliary Dam

2.5D.13.2.2.1 General

In approximately 1400 ft. of the central portion of the Auxiliary Dam, the random rockfill shell is founded on weathered rock and in the remaining portions, the rockfill shell is founded on a thin layer of stiff in-situ residual soil overlying weathered rock. The core is founded on suitable rock.

2.5D.13.2.2.2 Static Properties

Unit weights of rock samples from depths of 18 ft. to 34 ft. from the Auxiliary Dam are given in Table 2.5B-4, Appendix 2.5B. The unit weights range between 150.7 and 177.6 lb./ft.^{3}. For weathered rock at shallower depths a slightly lower value of unit weight equal to 150 lb./ft.^{3} was selected. Other parameters for static analysis were selected based on the data reported by Kulhawy in Reference 2.5D-17; see Table 2.5D-35.

2.5D.3.2.2.3 Dynamic Properties

2.5D.13.2.2.3.1 Shear Modulus

The shear modulus parameter $K_{2\ max}$ was determined by using the field shear wave velocity measurements and procedures reported in Appendix 2.5C. For an average shear wave velocity of 2500 ft./sec. at an average depth of 15.5 ft. and unit weight of 150 lb./ft.^{3} the value of $K_{2\ max}$ is 700.

2.5D.13.2.2.3.2 Variation of Shear Modulus and Damping Ratio with Shear Strain

As in the case of weathered rock at the Main Dam, the variation in the shear modulus and damping ratio with shear strain can be represented by average relationships for sands. Therefore, the same curves were used; see Figures 2.5D-58 and 2.5D-59.

2.5D.13.2.2.3.3 Ratio of Horizontal to Vertical Effective Stress

A value equal to that for the weathered rock for the Main Dam was selected ($\bar{K}_o$ = 0.6); see Table 2.5D-35.

2.5D.13.2.2.3.4 Poisson's Ratio

Typical values of Poisson's ratio for sandstones and siltstones are available in literature (e.g., Reference 2.5D-19). A value μ = 0.35 was selected for weathered rock.

2.5D.13.2.3 Auxiliary Reservoir Separating Dike

The core and random rockfill shell of the Auxiliary Reservoir Separating Dike is founded on stiff residual soil overlying weathered rock. The in-situ properties of the weathered rock were not required for the analyses of the maximum cross section of the Auxiliary Reservoir Separating Dike because the borings indicate that at this location, the weathered rock layer is only 2 ft. to 4 ft. thick and therefore, would not significantly influence the results of static and dynamic analyses.

2.5D.13.3 Properties of In-situ Residual Soil in the Foundation of Auxiliary Dam and Auxiliary Reservoir Separating Dike

2.5D.13.3.1 General

The properties of the in-situ residual soil in the foundation of the Auxiliary Dam and Separating Dike were investigated by laboratory tests and field geophysical measurements. The locations of test pits and borings where samples were obtained and the locations where geophysical measurements were made are shown in Figures 2.5D-66 and 2.5D-74.

2.5D.13.3.2 Auxiliary Dam

2.5D.13.3.2.1 Static Properties

The unit weight of the in-situ soil in the auxiliary dam area was selected on the basis of laboratory tests on undisturbed specimens; see Table 2.5D-36. A saturated unit weight of 134

lb./ft.^{3} was selected. Other parameters for static analysis were selected on the basis of material index properties, shear strength parameters (see Table 2.5D-36) and data published in Reference 2.5D-17.

2.5D.13.3.2.2 Dynamic Properties

2.5D.13.3.2.2.1 Shear Modulus

The value of the shear modulus parameter $K_{2\,max}$ was determined on the basis of geophysical measurements made at the site (P2.5C.2-4 through 2.5C.2-7). Locations of geophysical measurements are shown in Figure 2.5D-66. Analysis of the measurements indicates that $K_{2\,max}$ has a value ranging between 86 and 108 near Sta 14+60 and between 194 and 250 near Sta 34+60. It is considered that representative values are $K_{2\,max}$ = 100 near Sta 14+60 and $K_{2\,max}$ = 190 near Sta 34+60.

2.5D.13.3.2.2.2 Variation of Shear Modulus and Damping Ratio with Shear Strain

The residual soil shows variable degrees of weathering ranging from sandy silty material near the west abutment to very stiff transitional material resembling weathered rock near the east abutment. The variation of shear modulus and damping ratio with shear strain was characterized by the same relationships as for weathered rock; see Figures 2.5D-58 and 2.5D-59.

2.5D.13.3.2.2.3 Ratio of Horizontal to Vertical Effective Stress

Typical values of this ratio are available in literature (Reference 2.5D-5 and Reference 2.5D-18). A value of $\overline{K}_o$= 0.6 was selected to account for the preconsolidation effects.

2.5D.13.3.2.2.4 Poisson's Ratio

Typical values are available in literature (e.g., References 2.5D-23 and 2.5D-1). A value of μ = 0.30 was selected.

2.5D.13.3.2.3 Cyclic Strength Characteristics

The cyclic strength characteristics of the in-situ residual soil at the auxiliary dam were determined on the basis of a series of stress-controlled cyclic triaxial tests on undisturbed samples obtained from the site.

2.5D.13.3.2.3.1 Sample Location

The undisturbed samples were obtained from test trench TPA1 (near Sta 14+60) and TPA2 (near Sta 34+60); see Figure 2.5D-66.

Undisturbed block samples, approximately 1 ft. x 1 ft. x 1 ft. cubes, were cut from the two test trenches. Undisturbed 3-in.-dia., thin-wall Shelby tube samples were also obtained from borings located 10 ft. to 20 ft. from the test trenches. A summary of the location, depth, and index properties of the samples obtained is presented in Table 2.5D-38.

2.5D.13.3.2.3.2 Sample Preparation

Specimens for testing were obtained by careful trimming of the block and Shelby tube samples. The diameter of specimens tested was approximately 2 inches. Each specimen was placed in a triaxial cell, saturated, and consolidated; isotropic consolidation (K_c = 1.0) and anisotropic consolidation (K_c = 1.5, 1.7, and 2.0) were used.

2.5D.13.3.2.3.3 Cyclic Testing Program

Stress-controlled cyclic triaxial tests were performed on 15 specimens using the Modular Testing System (MTS). The applied cyclic deviator stress, axial strain, and pore waterpressure were measured. Loading was continued either until the specimen underwent excessive strain or until approximately 1000 cycles were reached. Following cyclic testing, the liquid and plastic limits and plasticity index of each specimen were determined.

2.5D.13.3.2.3.4 Test Results

A summary of the test results, including the cyclic stress ratio, $\pm\sigma_d/2\sigma_{3c}$, and the number of cycles required to reach 2 x 10^{-2}, 5 x 10^{-2}, and 10 x 10^{-2} strain for each specimen is presented in Table 2.5D-38. As can be seen from the table, eight of the fifteen specimens strained less than 2 x 10^{-2} at 1000 cycles; nine specimens strained less than 5 x 10^{-2} at 1000 cycles. In order to assist in plotting and interpreting the data, the test results were extrapolated to estimate the number of cycles beyond the test duration (>1000 cycles) at which a given strain would occur; see Table 2.5D-38.

2.5D.13.3.2.3.5 Determination of Cyclic Strength Characteristics

The results summarized in Table 2.5D-38 are plotted in Figures 2.5D-67 through 2.5D-70. Figures 2.5D-67 and 2.5D-68 show the results for isotropically consolidated specimens (K_c = 1.0) for a strain of 2 x 10^{-2} and 5 x 10^{-2}, respectively. Figures 2.5D-69 and 2.5D-70 show similar results for anisotropically consolidated specimens (K_c = 1.5 1.7, and 2.0). Smooth curves have been drawn in these figures to define the relationship between the applied cyclic stress ratio and number of cycles. Because of the low strain attained by many of the specimens, some of the curves presented in Figures 2.5D-67 through 2.5D-70 had to be extrapolated from large numbers of cycles on the shape of similar curves determined for core materials M and Z (see Section 2.5D.10 and 2.5D.11). For a strain of 5 x 10^{-2} (Figures 2.5D-68 and 2.5D-70), curves are presented for σ_{3c} = 3 k./ft.^{2} only, as the specimens at lower confining pressures generally exhibited very small strains and extrapolation to larger strains and low numbers of cycles was not possible.

The relationship between the cyclic shear stress required to cause 5 x 10^{-2} strain in five cycles and normal effective stress is presented in Figure 2.5D-71. The similar relationship for ten cycles is presented in Figure 2.5D-72. These curves were prepared from the test data shown in Figures 2.5D-68 and 2.5D-70 and using procedures described in Section 2.5D.10. The results of the tests on isotropically consolidated samples indicated that the cyclic strengths were comparable to a clean sand having a relative density of 100 percent; therefore, a correction factor, Cr, equal to 0.80 was applied (Reference 2.5D-30).

In the evaluation of the potential for attaining 5 x 10^{-2} strain in the in-situ residual soil during the earthquake motions, the curves for $\alpha = 0$ in Figures 2.5D-71 and 2.5D-72 have been used, i.e., higher cyclic strengths indicated for $\alpha > 0$ have been ignored.

2.5D.13.3.3 Auxiliary Reservoir Separating Dike

2.5D.13.3.3.1 Static Properties

The in-situ residual soil in the Auxiliary Reservoir Separating Dike area is generally similar to that in the Auxiliary Dam area. Figure 2.5D-73 presents the index properties and compaction characteristics of a sample from test pit A. A saturated unit weight of 135 lb./ft.^{3} was used for analysis. Other parameters for static stress analysis were selected on the basis of material index properties and data published by Kulhawy (Reference 2.5D-17), see Table 2.5D-39.

2.5D.13.3.3.2 Dynamic Properties

2.5D.13.3.3.2.1 Shear Modulus

Seismic wave velocity measurements were made at locations shown in Figure 2.5D-74. The average shear wave velocity was 715 ft./sec. (Appendix 2.5C, p.2.5C.2-7). The velocity is considered to be representative of the material at depths one quarter to one half of the impact receiver spacing; $K_{2\,max}$ values ranging between approximately 60 and 90 were obtained. In the analysis $K_{2\,max}$ = 90 was used.

2.5D.13.3.3.2.2 Variation of Shear Modulus and Damping Ratio with Shear Strain

As in the case of residual soil at the Auxiliary Dam, average damping ratios for sands were used; see Figure 2.5D-59.

2.5D.13.3.3.2.3 Ratio of Horizontal to Vertical Effective Stress

Typical values are available in literature (e.g., Reference 2.5D-5 and 2.5D-18). A value of $\overline{K}_o$ = 0.60 was used; see Table 2.5D-39.

2.5D.13.3.3.2.4 Poisson's Ratio

Typical values are available in literature; (Reference 2.5D-1 and Reference 2.5D-23) (Leonards 1962, Barkan 1962). A value of $\mu = 0.30$ was selected; see Table 2.5D-39.

2.5D.13.3.3.3 Cyclic Strength Characteristics

The in-situ residual soils at the Auxiliary Reservoir Separating Dike are similar to those at the Auxiliary Dam. Therefore, the cyclic strength characteristics defined in Figures 2.5D-71 and 2.5D-72 have also been used for the dike.

2.5D.14 STATIC STRESS ANALYSIS

2.5D.14.1 Introduction

Static stress analyses were conducted to determine the normal and shear stresses in the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike. A knowledge of the initial (i.e., prior to occurrence of earthquake) static effective stress conditions is required for the evaluation of the cyclic strength of materials in the dams and dike.

2.5D.14.2 General Procedure

An incremental finite element approach is used which simulates the construction of an embankment in a series of layers. The materials in various zones of the dam are assumed to have non-linear, stress-strain characteristics. The analysis is based on the procedures developed by Kulhawy (see Reference 2.5D-17) and Duncan and Chang (see Reference 2.5D-6). The dam is divided into several horizontal layers each represented by quadrilateral elements. During any increment, appropriate values of modulus E and Poisson's ratio μ are assigned to each element. After determining the stresses, E and μ are reevaluated for the average stress conditions during the new increment and compared with the assigned values. If a significant difference is obtained, a second analysis is made with adjusted values of E and μ until a reasonable correspondence is established between the input and computed values. This process is continued until the last layer is added.

After the last layer is added, the water level is raised in one or two steps. The effect of seepage and buoyancy is taken into account. In the case of steady seepage, flownets are drawn and the computed seepage force is equally divided among the nodal points forming an element. The effect of buoyancy of stresses is evaluated for all elements below the water surfaces in the shell, core, and filters.

2.5D.14.3 Selection of Material Properties

2.5D.14.3.1 Core

Parameters for static stress analysis are derived from laboratory static triaxial tests on materials M and Z. The test results and discussion of parameters are presented in Sections 2.5D.10 and 2.5D.11. Consolidated undrained (UU) tests were made on partially saturated specimens; and, consolidated drained (CID) tests were made on saturated specimens.

2.5D.14.3.2 Filters and Rockfills

Parameters for these materials are based on index properties, design and construction specifications, and published data (Reference 2.5D-17) on dams built with similar materials; see Section 2.5D.12.

2.5D.14.3.3 In-situ Soils and Weathered Rock

The selection of parameters for static stress analysis was based on index properties, field and laboratory tests, and published data (Reference 2.5D-17); see Section 2.5D.13.

2.5D.14.4 Analysis of Dams

2.5D.14.4.1 Main Dam

The maximum cross section (M-105) was analyzed. Foundation conditions for the maximum section consist of suitable rock below the core and a layer of weathered rock below filters and rockfill shells (see Figure 2.5D-2). The weathered rock layer is incorporated in the static (and dynamic) analyses.

For the static analysis of the maximum section, the dam was divided into seven horizontal layers and the water level was raised in two steps. The finite element representation for the maximum cross section is shown in Figure 2.5D-75 and the material properties used for the analysis are given in Table 2.5D-40. Stresses were evaluated for two sets of material properties distinguished by the parameters used for the core; see Table 2.5D-41. Set M(2) in Table 2.5D-41 used the properties for the core obtained from CID test data, and Set M(3) used the properties obtained from UU test data. The normal stresses computed by the two analyses were not significantly different. The results of Set M(3) provided a more reasonable variation of the static shear stresses (Reference 2.5D-7). Set M(3) was, therefore, used for subsequent evaluations of the static stresses.

Typical results of Set M(3) are presented in Figure 2.5D-76 and 2.5D-77 which show the vertical effective stresses and shear stresses on a horizontal plane at Elevation 192.5. As can be seen, a concentration of shear and normal stresses occurs in the filters. The stress concentration can be attributed to the sharp variation in the stiffnesses of the core, filters, and the rockfill within a relatively short horizontal distance. Redistribution of stresses can be expected to occur, and the actual stresses in the filters should be lower than computed while those in the shell should be higher than those computed (Reference 2.5D-7). On this basis, average curves were drawn, shown by the heavy lines in Figure 2.5D-76 and 2.5D-77, modifying the computed stresses, but still satisfying static equilibrium (i.e., the magnitude of the total shear and normal forces on the given plane does not change). The average curves in Figure 2.5D-76 and 2.5D-77 were then used to compute ratios of shear stress τ_{xy}, to normal stress, $\bar{\sigma}$. The values of $\alpha = T_{xy}/\bar{\sigma}_y$ are shown in Figure 2.5D-78.

The two lower cross sections of the Main Dam (M-67 and M-36) have similar geometry and foundation conditions as the maximum section (M-105). Therefore, on a horizontal plane at a given distance below the crest of the dam, the static stresses in Section M-67 and M-36 would be similar to those in the maximum section, M-105. The results of the static finite element analysis of the maximum section were, therefore, utilized to determine the static stresses in Section M-67 and M-36.

2.5D.14.4.2 Auxiliary Dam

Two cross section were analyzed, viz the cross section at Sta 14+60 (A-63, maximum cross section) and the cross section at Sta 34+65 (A-44). The reasons for their selection were (a) variation in base geometry; and (b) difference in foundation conditions and relative size of the zones.

Foundation conditions for the maximum section, A-63, consist of suitable rock below the core and a thin layer (approximately 3 ft. to 4 ft. thick) of weathered rock below the filters and rockfill shells (see Figure 2.5D-4). The thin weathered rock layer would not significantly affect the static

(or dynamic) stresses in the embankment and is, therefore, neglected in the static (and the dynamic) analyses, i.e., the filters and rockfill are assumed to rest directly on suitable rock. Foundation conditions for cross section A-44 consist of suitable rock below the core and in-situ residual soil and weathered rock below filters and rockfill shells (see Figure 2.5D-5); these foundation layers are incorporated in the static (and dynamic) analyses.

The maximum cross section A-63) was divided into six layers and the water level was raised to the design elevation in a single step. Material properties selected for the analysis are given in Table 2.5D-42. Five sets of material property combinations were used; see Table 2.5D-43. Sets A-II-8, and A II-9 were similar to A-II-5 and A-II-6, except that the Poisson's ratio parameter, G, equal to 0.30 was used in the random rockfill. Set A-II-7 incorporates both sets of core test parameters corresponding to the period of construction (UU data) and period of saturation (CID data) in a single analysis. Subsequent evaluation indicated that Set A-II-7 is not valid because the processes of consolidation and pore pressure dissipation cannot be taken into account during the saturation period. Results obtained from Set A-II-9, using properties from UU tests in the core appeared to provide most reasonable values of static stresses.

One analysis was made for cross section A-44 using UU test data for the core and properties given in Table 2.5D-42 for other materials. The analysis of Section A-44 was also utilized as a basis for determining the static stresses in section A-24 which has a similar geometry and foundation.

As in the case of the main dam, the vertical and shear stresses show sharp variations in the filter zones and rockfills. Average values were obtained for the evaluation of dynamic stability in the same manner as for the Main Dam.

2.5D.14.4.3 Auxiliary Reservoir Separating Dike

The maximum cross section was analyzed. The embankment is constructed on a layer of residual soil above a thin layer (approximately 2 ft. to 4 ft. thick) of weathered rock. As for Section A-63 of the Auxiliary Dam, the weathered rock is neglected in the static (and dynamic) analyses of the maximum cross section of the dike.

The cross section analyzed had a 12-ft.-thick layer of foundation soil. Subsequent to the static analysis, the cross section design was modified so as to leave only a 5-ft.-thick layer of foundation soil, which was used in the dynamic analysis. The use of the thicker layer in the static analysis has a negligible influence on the evaluation in this case because (a) the normal stresses along vertical planes showed the same variation with depth in the foundation as in the dam, (b) the values of static shear stress in the foundation are not used in the evaluation of the cyclic strength of the foundation (see Section 2.5D.13), and (c) the foundation is relatively stiff, therefore, should not result in any significant arching and stress transfer in the embankment.

The dike was assumed to be built in six horizontal lifts, and the water level was raised to design elevation in one step and two steps. Material properties used in the analysis are given in Table 2.5D-44 and the material property variations studied are given in Table 2.5D-45. Set D(3) in Table 2.5D-45, incorporating parameters from UU tests in the core, was used to obtain the static stresses for the evaluation.

The Auxiliary Reservoir Separating Dike does not contain filters; therefore, unlike the Main and Auxiliary Dams, sharp variations in stresses were not obtained in the case of the Auxiliary Reservoir Separating Dike.

2.5D.15 PARAMETRIC STUDIES

2.5D.15.1 Introduction

Parametric studies have been conducted to assess the effects of variations in material properties on the seismic response and, in particular, on the induced stresses in the various zones of the dam.

The material properties required for a response computation are: unit weight, γ, shear modulus, G, damping ratio, λ, Poisson's ratio, μ, and the ratio of the horizontal to the vertical effective stresses, $\overline{K}_o$.

The shear modulus and the damping ratio for most soils are highly strain dependent. Therefore, the relationship between modulus and strain must also be known for a response computation. Such a relationship is most conveniently established through the use of a maximum shear modulus, G_{max}, (corresponding to very low levels of strain, of the order of $\simeq 10^{-6}$), and a reduction curve (e.g., Section 2.5D.10). Because the modulus of the foundation soils and the soils used in the construction of the dam are functions of the effective mean pressure, the maximum shear modulus for each soil type has been expressed by the parameter, $K_{2,\ max}$ (e.g., Section 2.5D.10).

The relationship between damping and strain must also be known for a response computation.

2.5D.15.2 Expected Constructed Values of Material Properties

A basic set of material properties has been established for each zone of the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike and underlying foundations. The basic set represents values for the zone expected in accordance with the design and construction criteria. This basic set, therefore, incorporates the expected constructed values of material properties.

To account for scatter in test data, some uncertainty in assigned values based on correlation with published data, and to incorporate an added margin of conservatism, the expected constructed values have been varied within a reasonable band. The bases for the variations and the combinations utilized in the analyses are presented in Sections 2.5D.15.3 and 2.5D.15.4.

It may be noted that the parameters that have the largest effects on induced stresses are the shear modulus and the damping ratio. Therefore, the emphasis in the parametric studies has been made in varying these properties. The other material properties (γ, μ, and $\overline{K}_o$) were not varied because the probable range of variation is narrow. In addition, the effects of variations in these latter parameters are adequately incorporated by the variations in the shear modulus.

2.5D.15.3 Bases for Parametric Variations

The basis for the parametric variations in each of the materials in the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike are discussed below. The combinations of these parametric variations used in the analyses are described in Section 2.5D.15.4.

2.5D.15.3.1 Core Material M

The shear modulus and damping relationships have been assigned based on a comprehensive laboratory testing program, described in Section 2.5D.10. An average value of $K_{2\ max}$, a shear modulus vs. shear strain reduction curve and a damping ratio vs. shear strain curve were selected based on the results of the test data. The average value of $K_{2\ max}$, selected is 120. For parametric studies, $K_{2\ max}$ is varied from 90 to 140 which essentially covers the variation in the experimental data. The value of $K_{2\ max}$ determines the values of maximum shear modulus, G_{max}, at very low strains ($\simeq 10^{-6}$). The shear modulus at higher strains is determined by means of the modulus reduction curve, G/G_{max} vs. γ (G is shear modulus at shear strain γ). Thus, by varying the value of $K_{2\ max}$, the values of shear modulus G at any strain γ are varied in the same proportion. Therefore, the variation in $K_{2\ max}$ effectively covers the variation in laboratory test data over the full range of shear strains.

The average relationship selected for damping ratio in Section 2.5D.10 is close to the lower bound values of damping ratios determined by laboratory tests. To provide an additional margin of conservatism, the damping ratios are varied by 10 percent below the selected average values. Higher values of damping ratio are not incorporated as these would result in lower stresses.

2.5D.15.3.2 Core Material Z

Modulus and damping ratio relationships for material Z have been assigned based on a comprehensive laboratory testing program; see Section 2.5D.11. An average value of $K_{2\ max}$ equal to 100 was selected. In parametric studies, $K_{2\ max}$ is increased to 125 which encompasses all of the test data. Lower values of $K_{2\ max}$ are not assigned because analyses of the Main Dam (with core material M) indicated that such lower values resulted in lower stresses in the core.

Parametric variation in the damping ratio consisted of reducing the selected average damping curve by 10 percent. The resulting damping ratio curve is then essentially at or below the lowest values determined by laboratory tests.

2.5D.15.3.3 Filters

As described in Section 2.5D.12, average modulus and damping ratio relationships for the coarse and fine filters of the Main Dam and transition filter of the Auxiliary Dam were determined from published data on similar sandy and gravelly soils. For the coarse and fine filters, the values of $K_{2\ max}$ selected for the basic analysis are close to upper bound values from published data for a relative density of approximately 75 percent to 80 percent. Therefore, higher values are not incorporated in the analysis. The values of $K_{2\ max}$ have been reduced in one case (from 120 to 90 in the coarse filter and from 60 to 50 in the fine filter); see Table 2.5D-46. For the transition filter of the Auxiliary Dam, the value of $K_{2\ max}$ is increased in one case from 90 to 120

which exceeds the upper bound values inferred from published data for a relative density of 75 percent to 80 percent.

For all filter material average damping ratio values were reduced by 20% which is a reasonable variation for a single material. The results of the analysis showed that a simultaneous reduction in damping ratio in all materials resulted in relatively small changes in induced stresses; the effect of changes in modulus were more significant in affecting the stresses.

2.5D.15.3.4 Rockfill

An average of $K_{2\,max}$ equal to 180 was selected for the rockfill of the Main Dam; see Section 2.5D.12. It is believed that this well-compacted granitic rockfill will be stiffer than the filter materials. Therefore, a lower bound value of $K_{2\,max}$ equal to 150 has been assigned to the rockfill, as compared to the selected value of 120 for the coarse filter of the Main Dam. To cover the possibility of a higher modulus in the rockfill, an upper bound value of $K_{2\,max}$ equal to 250 was assigned (a parametric variation of approximately 40% above the basic value). As was done for the filters, the assigned damping ratio in the rockfill was reduced by 20 percent in parametric studies.

2.5D.15.3.5 Random Rockfill

A value of $K_{2\,max}$ equal to 90 was selected for the random rockfill of the Auxiliary Dam and Auxiliary Reservoir Separating Dike based on field seismic geophysical measurements in similar rockfills of dams; see Section 2.5D.12. The compacted random rockfill may be somewhat stiffer than the rockfills in which measurements were obtained; therefore, a value of K2 max equal to 150 (67 percent greater than the basic value) was used in the parametric studies. Damping ratio was varied as described for the rockfill of the Main Dam.

2.5D.15.3.6 In-situ Residual Soil

As described in Section 2.5D.13, values of $K_{2\,max}$ in the residual soil at the Auxiliary Dam and Auxiliary Reservoir Separating Dike were determined based on geophysical measurements. The effect of increased modulus (plus 28 percent) and decreased damping ratio (minus 20 percent) was studied in parametric variations made for the Auxiliary Reservoir Separating Dike; see Table 2.5D-49.

2.5D.15.3.7 In-situ Weathered Rock

The values of $K_{2\,max}$ assigned to the weathered rock at the Main Dam and Auxiliary Dam were assigned based on field geophysical measurements; see Section 2.5D.13. In order to study the effect of modulus changes in the rock and the response of the overlying materials, the value of $K_{2\,max}$ was reduced from 700 to 250 in parametric variations for Section A-44 of the Auxiliary Dam; see Table 2.5D-48.

Damping ratio was reduced by 20 percent below the basic value in one of the parametric variations made for the Main Dam; see Table 2.5D-46.

2.5D.15.4 Material Property Combinations

Several combinations in material properties were used in the analyses of the Main Dam (maximum section M-105), Auxiliary Dam (maximum Section A-63 and A-44), and Auxiliary Reservoir Separating Dike (maximum Section D-53).

The combinations are summarized in Tables 2.5D-46 through 2.5D-49. Emphasis was placed on selecting material property combinations which would tend to increase stresses in one or more zones above the stresses computed using the properties of the basic set.

2.5D.15.4.1 Main Dam

Material property combinations used in the analysis of the Main Dam are summarized in Table 2.5D-46. Eight sets of properties, including the basic set, were used. Sets A and C were used to study the effect of higher modulus and lower modulus, respectively, in the rockfill. Set BC was analyzed to assess the effect of lower modulus in the rockfill combined with higher modulus and reduced damping ratio in the core. Set B combined increased modulus and reduced damping ratio in the core with increased modulus in the rockfill shell; this set essentially constituted upper bound modulus values for all materials. In Set E, the modulus values for all embankment materials were decreased to the lower bound values. In Set D, the lower bound damping ratio was incorporated in all embankment materials. The effect of simultaneous application of horizontal and vertical accelerations was evaluated by set AV.

2.5D.15.4.2 Auxiliary Dam, Maximum Section

Material property combinations used in the analysis of the maximum section of the Auxiliary Dam are presented in Table 2.5D-47. Five sets of properties, including the basic set, were used.

Using the results of the parametric studies for the Main Dam as a guide, sets A and B for the maximum section of the Auxiliary Dam incorporated increased moduli in the random rockfill and in the core, respectively. Set B also induced reduced damping ratio in the core. Set C combined increased moduli with decreased damping ratio in all materials. Therefore, set C represents the upper bound of modulus values and lower bound of damping values. The effect of simultaneous application of horizontal and vertical accelerations was studied in set AV.

2.5D.15.4.3 Auxiliary Dam, Cross Section A-44

Table 2.5D-48 summarizes the material property combination used for section A-44 of the Auxiliary Dam. The analyses considered the effect of modulus changes in the random rockfill, in-situ residual soil, and weathered rock.

Set A was used to evaluate the effects of an increase of the modulus of the rockfill to its upper bound value. Sets B and C were used to evaluate the effects of changes in the moduli of the in-situ residual soils and the weathered rock. The field geophysical measurements indicated a lower bound value of $K_{2\,max}$ of 60 and 250 for the residual soils and weathered rock, respectively. Set B was used to assess the effects of lowering the modulus of the residual soils to its lower bound value. Set C was used to assess the effects of decreasing simultaneously the moduli of the residual soils and the weathered rock. In both sets B and C, the upper bound value of $K_{2\,max}$ for the rockfill was used.

2.5D.15.4.4 Auxiliary Reservoir Separating Dike Maximum Section

Table 2.5D-49 summarizes the parametric variations made for the maximum section of the Auxiliary Reservoir Separating Dike. Variations in moduli and damping ratio of the core material, in-situ residual soil, and random rockfill were made. Set A evaluated the effect of increased modulus in the rockfill while set B evaluated the effect of increased modulus and decreased damping ratio in the core. Set C combined increased moduli with decreased damping ratio in all materials. Therefore, set C constitutes the upper bound on modulus values and lower bound on damping ratio. Because of the very small thickness (2 ft. to 5 ft.) of residual soil beneath the Auxiliary Reservoir Separating Dike, no additional changes in the modulus value of this soil was deemed necessary.

2.5D.16 DYNAMIC ANALYSES AND STABILITY EVALUATIONS

2.5D.16.1 Introduction

The general procedure for computing the response and evaluating the stability of an earth dam during an earthquake is briefly outlined in Section 2.5D.3. This procedure has been used in analyzing the Main Dam, Auxiliary Dam, and Auxiliary Reservoir Separating Dike. The use of this procedure is illustrated in this section for one of the cases studied. The case presented is M-105-IVA for the maximum section of the main dam using the material property combination assigned to this case; see Section 2.5D.15.

2.5D.16.2 Dynamic Analyses

The response of each section of the dam during the SSE was computed using the dynamic finite element method. The finite element representation used for the maximum cross section of the Main Dam is shown in Figure 2.5D-79. The finite element representation was extended 55 ft. and 120 ft. beyond the toes of the dam (i.e., six times the thickness of the weathered rock). In accordance with the criteria presented by Idriss (Reference 2.5D-12), this extension is sufficient so that the artificial boundaries do not affect the response of the dam.

An equivalent linear procedure is incorporated in the finite element solution so that strain-compatible values of shear modulus G, and damping ratio, λ, are used for each element. Thus, at the outset, the modulus and damping ratio values are estimated and the response is computed. The computed value of average strain in each element is then used to choose (using the applicable G vs. γ and λ vs. γ curves (see Section 2.5D.10 through 2.5D.12) new values of modulus and damping. This process is continued until a compatible solution is obtained. Normally, three to five iterations are required.

The values of strain-compatible damping ratios obtained for this case in the various zones of the dam are as follows:

DAMPING RATIOS FOR CASE M-105-IV A

Zone	Damping Ratio, λ
Core	0.14 to 0.20
Filters	0.08 to 0.15
Rockfill Shells	0.04 to 0.09

It should be noted that the damping ratio for each element in the finite element representation is selected using the average strain induced in the element and the applicable λ vs. γ curve. The damping ratios for elements in the core of the dam are based on the curve presented in Section 2.5D.10 for material M; see Figure 2.5D-35. This curve is well below the values obtained from the controlled-strain cyclic triaxial tests. Therefore, in the range of interest, the curve used for material M is a conservative representation of the test data. Accordingly, the damping ratios used in the analysis are justified although they exceed 0.15 in some parts of the core.

Comparable damping ratios are obtained for the other dams analyzed.

The strain-compatible shear modulus obtained in each element for case M-105-IV A was selected using the average strain induced in the element and the applicable $K_{2,\ max}$ and the modulus reduction curve. The values of K_2 obtained in each zone of the dam are approximately as follows:

SHEAR MODULUS PARAMETER, K_2, FOR CASE M-105-IV A

Zone	$K_{2,\ max}$	K_2 Used in Analysis
Core	120	20 to 43
Fine Filters	60	20 to 31
Coarse Filter	120	51 to 72
Rockfill Shells	250	135 to 200

The applicable value of K_2 (based on average induced strain) was then multiplied by the square root of the effective mean stress at the centroid of the element to obtain the shear modulus for that element.

The response computations provide a variety of response values including accelerations at each nodal point and stresses in each element for the entire duration of the SSE.

Typical results of the response computations are presented in Figure 2.5D-80 and 2.5D-81. Figure 2.5D-80 shows the time history of computed crest accelerations as well as the rock accelerations. The computed maximum crest acceleration is approximately 0.43 g. (i.e., the rock acceleration is amplified by a factor of approximately 2.85). The time histories of computed shear stresses at six selected points within the dam are shown in Figure 2.5D-81. These stresses are computed at the centroid of the element and, therefore, represent the stresses at a point. Two of these time histories are for stresses computed within the core of the dam; two are for stresses within the upstream filters; and two are for stresses computed in the rockfill shells. As can be noted from Figure 2.5D-81, the stresses are essentially in phase everywhere throughout the dam.

2.5D.16.5 Determination of Equivalent Uniform Peak Shear Stresses

The stresses computed within any part of the dam (see Figure 2.5D-81) vary in amplitude throughout the duration of the earthquake. The stresses induced in the dam are compared with the cyclic strength data, which are generally obtained from laboratory tests using cyclic loads of uniform peak amplitudes. Therefore, it is necessary to convert the computed time history to an equivalent number of uniform stress cycles. This is most conveniently accomplished by appropriate weighting of the time history of computed stresses as illustrated in Figure 2.5D-82.

The data in Figure 2.5D-82 illustrate the use of the procedure to convert the computed time history within a typical element in the core to twenty cycles of equivalent uniform stress. As can be noted, the ratio of the amplitude of equivalent uniform stress to the computed maximum stress is approximately 0.64. Similar computation for the time history of shear stress at other locations within the dam (including the core, filters, and rockfill shells) indicate that this ratio varies from approximately 0.64 to 0.66. An average value of 0.65 was, therefore, used to obtain the equivalent uniform peak shear stresses at all locations within the dam.

The conversion illustrated in Figure 2.5D-82 was done to obtain the equivalent uniform stress applicable to twenty cycles taking into account the entire duration (10.24 sec.) of the artificial accelerogram. However, as discussed in Section 2.5D.4, the stability of the dams under the SSE is evaluated on the basis of the stresses induced by the artificial accelerogram using five cycles of strong motion for the determination of the cyclic strength of the soil and rock materials. The equivalent uniform stresses computed by the procedure shown in Figure 2.5D-82 were considered to be applicable to five cycles. The data for the example presented in this section is based on the use of five cycles.

To provide an additional margin of conservatism, the stability was investigated using ten cycles. The results of the evaluations using five and ten cycles for all the cases considered are presented in Section 2.5D.5 for the Main Dam, in Section 2.5D.6 for the Auxiliary Dam, and in Section 2.5D.7 for the Auxiliary Reservoir Separating Dike.

2.5D.16.4 Stress Induced During Ground Motions

The stability of the dam is evaluated by comparing the stresses induced during the ground motions with the cyclic strength characteristics along selected planes within the dam. Typical variations of equivalent uniform stresses induced for five cycles during the ground motions along four planes are illustrated in Figure 2.5D-83. Similar plots have been prepared for other planes. Values of equivalent uniform stresses along horizontal planes at Elevation 237.5, 222.5, 207.5, 192.5, and 177.5 are shown in Figure 2.5D-84 through 2.5D-88.

2.5D.16.5 Seismic Stability Evaluation

The stresses required to cause 5 x 10^{-2} strain (which has been chosen as the criterion for evaluating the stability of the dams at this site (see Section 2.5D.5.4.1) along any plane within the dam are obtained by using the static stresses and the applicable cyclic strength characteristics. The static stresses are evaluated by a static finite element procedure described in Section 2.5D.15. The cyclic strength characteristics of the core materials are obtained from the results of appropriate laboratory tests as described in Section 2.5D.10 (for the core material of the Main Dam) and Section 2.5D.11 (for the core material in the Auxiliary Dam and Auxiliary Separating Dike). For the filters, rockfill, and random rockfill, the cyclic strength characteristics are selected on the basis of index properties, material type, and design and construction criteria as summarized in Section 2.5D.12.

The evaluation of the stability of the Main Dam (using the dynamic material properties assigned to case M-105-IVA) along five horizontal planes is illustrated in Figure 2.5D-84 through 2.5D-88. The upper part of each figure shows the values of initial static vertical effective stress along the plane. The middle part shows the values of stresses, τ_d, induced by the artificial accelerogram together with the stresses, τ_f, required to cause 5 x 10^{-2} strain in five cycles. Values of the ratio τ_f/τ_d are presented in the lower part of the figure. This ratio has been considered as a local

factor of safety against development of 5 x 10^{-2} strain. As can be noted from Fgiure 2.5D-84 through 2.5D-88, the values of this ratio are well over unity (i.e., there is ample safety against the development of 5 x 10^{-2} strain during the rock accelerations). The minimum local factors of safety against the development of 5 x 10^{-2} strain in the various zones of the Main Dam for case M-105-IV A are as follows:

MINIMUM LOCAL FACTORS OF SAFETY FOR 5 x 10^{-2} STRAIN
CASE M-105-IVA

		Rockfill Shell		Filters			
				Upstream		Downstream	
Plane at Elevation	Core	Upstream	Downstream	Fine	Coarse	Fine	Coarse
237.5	1.75	1.66	*	1.70	1.24	*	*
222.5	1.77	1.56	*	2.24	1.62	*	*
207.5	1.91	1.30	*	2.85	2.13	*	*
192.5	2.18	1.54	1.97	3.84	2.86	4.50	3.57
177.5	2.22	1.88	2.31	4.03	2.70	4.94	3.47

* Not applicable because above phreatic line

2.5D.16.6 Influence of Vertical Component

The results presented previously in this section were obtained using only horizontal rock accelerations. The same case was also analyzed using simultaneous horizontal and vertical rock accelerations. The artificial accelerogram of the vertical component was identical to that of the horizontal component (see Figure 2.5D-80) scaled down to a maximum vertical acceleration of 0.10 g.

The response for the dam (case M-105-IV A) during the simultaneous application of the horizontal and vertical components of the artificial accelerogram was computed using the procedure outlined earlier. These response values were then compared with those obtained by using only the horizontal component to assess the influence of the vertical component. The computed shear stresses along two typical horizontal planes are shown in Figure 2.5D-89 for the two cases. As can be noted, the stresses induced during the rock acceleration are essentially unaffected by the vertical component.

2.5D.16.7 Consideration of Tensile Stresses

The response of the dam-foundation system was computed by the finite element method in which the continuum is idealized by elements in plane strain and all components of the stress tensor are incorporated. The strength of the dam and foundation materials are determined on the basis of triaxial tests by imposing on specimens cyclic stresses which simulate that induced by the earthquake. These tests provide cyclic strength parameters applicable to the field conditions. Therefore, any possibility of development of tensile stresses is accounted for by the method of analysis and the method of determination of the cyclic strength parameters; i.e., tensile stresses are not neglected.

2.5D.17 COMPARISON OF RESPONSE BY VARIOUS ANALYTICAL MODELS

2.5D.17.1 Introduction

The response of the Category I dams during the SSE has been obtained using the finite element procedure. Strain-dependent values of modulus and damping are used in every element. A Rayleigh expression is used to formulate the damping submatrix for each element; the damping matrix for the entire system is then assembled in the usual way by adding the appropriate components of the submatrices for all elements. The response of the system is then computed by a direct integration of the equations of motion, ie, modal analysis is not used in this procedure. In fact, all the modes of the system are automatically incorporated in the solution. Because of the use of a Rayleigh expression for each submatrix, some of the higher modes of the system may be overdamped.

The comparative studies presented in this appendix have been conducted to assess the effects of the use of this procedure on response values and, in particular, the effects on induced shear stresses.

2.5D.17.2 Case Studied

An 80-ft. layer of dense sand and gravel (Figure 2.5D-90) has been analyzed using three analytical procedures. The layer is assumed to have an extensive horizontal extent; therefore, mathematical models appropriate for a semi-infinite system have been used in the analyses. The following material properties have been assigned to this layer: qm = 135 lb./ft.^{3}, K_o = 0.45, and $K_{2\,max}$ = 100. The average curves relating damping and modulus reduction to strain published for sands have been used to obtain the strain-dependent properties for this layer.

The response of this layer has been computed utilizing the artificial accelerogram (Figure 2.5D-8) that has been used in the analysis of the Category I dams.

The thickness of this layer and the material properties have been chosen so that the predominant period is within the range of interest.

2.5D.17.3 Analytical Models

In addition to the finite element model, the soil layer was represented by two other models as shown in Figure 2.5D-90. The mathematical formulation for each model is briefly described below.

2.5D.17.3.1 Finite Element Model

The finite element method is a numerical procedure by means of which the actual continuum is represented by an assemblage of elements interconnected at a finite number of nodal points. Details of the formulation of the general method are available in several recent publications (eg, References 2.5D-3, 2.5D-4, 2.5D-12, and 2.5D-36).

In earthquake response evaluations, the following set of equations are solved:

$$[M]\,\{\ddot{u}\} + [C]\,\{\dot{u}\} + [K]\,\{u\} = \{R(t)\} \qquad (6)$$

in which

[M]	=	mass matrix for the assemblage of elements,
[C]	=	damping matrix for the assemblage of elements,
[K]	=	stiffness matrix for the assemblage of elements,
u	=	nodal displacements vector (dots denote differentiation with respect to time), and
{R(t)}	=	earthquake load vector.

A detailed description for the formulation of [M], [K], and R(t) is available elsewhere.

In order to permit the use of a damping ratio for each individual element, a variable damping solution was recently developed (Reference 2.5D-14). This solution has been used in the analysis of the Category I dams and for this comparative study.

In a variable damping solution, a damping submatrix must be formulated for each individual element and then all element submatrices added, in the appropriate way, to obtain the damping matrix for the entire assemblage of elements. The following relationship is used for each element, q:

$$[c]q = \alpha_q\,[m]q + \beta_q\,[k]q \tag{7}$$

in which [c]q, [m]q, and [k]q are the damping, mass and stiffness submatrices for element q, respectively, and α_q and β_q are parameters that are functions of the damping ratio and stiffness characteristics of element q. The parameters α_q and β_q are obtained from:

$$\alpha_q = \lambda_q \cdot \omega_1 \tag{8}$$

$$\beta_q = \lambda_q/\omega_1 \tag{9}$$

The value λ_q, which represents the damping ratio for element q, is chosen based on the strain developed in the element. The parameter ω_1 is equal to the fundamental frequency of the system.

The equations of motion (equation (6)) are readily solved by a direct numerical method, such as the step-by-step method (Reference 2.5D-37). If a linear variation of acceleration is assumed over the time increment of integration, Δt, then the unknown response values at the nodal points at time, t, can be expressed in terms of the known values at time, t-Δt, as follows:

$$[u]_t = [\bar{K}]^{-1}\,\{R\}t \tag{10}$$

$$[\bar{K}] = [K] = 6[M]/\Delta t^2 = 3[C]/\Delta t \tag{11}$$

$$[\bar{R}]_t = \{R\}_t + \{A\}_t^T[M] + \{B\}_t^T\,[C] \tag{12}$$

$$\{A\}_t = \frac{6}{\Delta t^2}\{u\}_{t\text{-}\Delta t} = \frac{6}{\Delta t}\{\dot{u}\}_{t\text{-}\Delta t} + 2\{\ddot{u}\}_{t\text{-}\Delta t} \tag{13}$$

$$\{B\}_t = \frac{3}{\Delta t}\{u\}_{t}\text{-}\Delta_t + 2\{\ddot{u}\}_{t}\text{-}\Delta_t + \frac{\Delta t}{2}\{\ddot{u}\}_{t}\text{-}\Delta_t \quad (14)$$

$$\{\dot{u}\}t = \frac{3}{\Delta t}\{u\}_t - \{B\}_t \quad (15)$$

$$\{\ddot{u}\}t = \frac{6}{\Delta t^2}\{u\}_t - \{A\}_t \quad (16)$$

The stresses and strains developed in each element can then be readily computed using the values of $\{u\}_t$.

2.5D.17.3.2 Lumped-Mass Model

The lumped-mass solution is a numerical procedure by means of which a semi-infinite layer is represented by a series of sublayers. Each sublayer is then represented by a series of lumped masses interconnected by shear springs. The applicable equations of motion are identical in form to equation (6). The matrices [M], [K], and {R(t)} are readily formulated (e.g., Reference 2.5D-13). The solution of equation (6) is carried out using modal superposition. The displacements of the lumped masses are expressed in terms of the normal coordinates and mode shapes by:

$$\{u\} = [\phi]\{X\} \quad (17)$$

$[\phi]$ are the mode shapes of the system and {X} are the normal coordinates.

The mode shapes and frequencies are determined from a solution of the eigen-value problem for the undamped free vibration equations of the system (i.e., for [C] = 0 and {R(t)} = 0 in equation (6):

$$[K]\{\phi^n\} = \omega_n^2 [M]\{\phi n\} \quad (18)$$

Each column, $\{\phi^n\}$ of the matrix $[\phi]$ represents the mode shape of the nth mode of vibration whose natural circular frequency is ω_n. The normal coordinates for each mode n, are evaluated from a solution of the normal equations:

$$\ddot{X}_n + 2\lambda_n\omega_n\dot{X}_n + \omega_n^2 Xn = \frac{\{\phi n\}\, T[M]\,\{R(t)\}}{M_n} \quad (19)$$

in which λ_n = damping ratio for mode n

$M_n = \{\phi^n\}^t [M] \{\phi^n\}$ and T denotes the transpose of the vector.

The damping ratio for the layer is the weighted average of the strain-dependent damping ratio obtained for each sublayer. Generally, the same damping ratio is used for all modes, i.e., $\lambda_1 = \lambda_2 = \ldots \lambda_n$.

The lumped-mass solution can also be used to investigate directly the effects of increased damping in higher modes. The damping ratios for each mode are proportioned based on the frequencies of the mode, i.e., for mode i, the damping ratio $\lambda_i = \lambda_1 \cdot \omega_i/\omega_1$.

2.5D.17.3.3 Wave Propagation Model

The wave propagation solution is a numerical procedure whereby the soil layer is divided into a series of horizontal sublayers overlying a perfect elastic half space. Details of the formulation of this solution are available in recent publications (e.g., References 2.5D-25, 2.5D-26).

The motions within any sublayer j are assumed to satisfy the damped wave equation:

$$\rho_j \frac{\partial^2 u}{\partial t^2} = G_j \frac{\partial^2 u}{\partial z^2} + n_j \frac{\partial^2 u}{\partial z^2 \partial t} t \qquad (20)$$

in which u = u(t,z) is the horizontal displacement, t is time, z is depth within the j^{th} sublayer, and ρ_j, G_j, and n_j are the mass density, the shear modulus, and viscosity, respectively, of the j^{th} sublayer.

The steady state solution to equation (20) is:

$$u = U_j(z) \cdot \exp(i\omega t) \qquad (21)$$

where

$$U_j(z) = E_j \exp(ik_j z) + F_j \cdot \exp(-ik_j z) \qquad (22)$$

and

$$kj = \omega/Vj \qquad (23)$$

The variable V_j is the complex shear wave velocity for the material of the j^{th} sublayer. This velocity is related to ρ_j, G_j, and n_j through the relation

$$V_j^2 = (G_j + i\omega n_j)/\rho_j \qquad (24)$$

To provide a solution where energy dissipation is independent of frequency, equation (24) is restated in the form

$$V_j^2 = G_j(1 + i2\lambda_j)/\rho_j \qquad (25)$$

where λ_i is the fraction of critical damping for the the material of the j^{th} sublayer (reference 2.5D-26), which is chosen based on the strain induced in the sublayer.

The first term of equation (22) refers to a shear wave which propagates in the negative z-direction (upwards in Figure 2.5D-90, part d) with the complex amplitude E_j and the second term refers to a wave which propagates in the position z-direction (downwards in Figure 2.5D-90, part d) with the complex amplitude F_j.

The determination of the displacements u for each sublayer during the applied base motion are most conveniently carried out using Fourier techniques (Reference 2.5D-25 and 2.5D-26). Once the displacements are determined, other response values (e.g., acceleration or shear stresses) are readily obtained.

2.5D.17.4 Response Evaluation

The response of the 80-ft. layer of dense sand and gravel shown in Figure 2.5D-90, using the artificial accelerogram as input base motion (i.e.., at a depth of 80 ft.), has been evaluated by the three solutions described in the previous section.

2.5D.17.4.1 Finite Element Solution

The layer is represented by nine elements and twenty nodal points as shown in part b of Figure 2.5D-90. In order to simulate a semi-infinite system, nodal points 1 through 18 are fixed in the vertical direction and, therefore, are only permitted to move in the horizontal direction. Nodal points 19 and 20 are fixed to the base.

The damping ratios obtained for each element (based on the induced strain in the element) are shown in Figure 2.5D-91. The maximum acceleration and the maximum shear stresses computed by this solution are presented in Figure 2.5D-92.

The fundamental period of the layer computed by this procedure is approximately 0.46 sec.

2.5D.17.4.2 Lumped-Mass Solution

The layer is divided into nine sublayers, each of which is represented by two masses as shown in part c of Figure 2.5D-90. The weighted damping ratio equal to 0.12 is obtained for the entire layer. The same damping ratio is assumed for all frequencies. The maximum acceleration and the maximum shear stresses computed by this solution are presented in Figure 2.5D-92. The fundamental period of the layer computed by this procedure is approximately 0.46 sec.

2.5D.17.4.3 Wave Propagation Solution

The layer is represented by nine sublayers as shown in part d of Figure 2.5D-90. The damping ratios obtained for each sublayer are shown in Figure 2.5D-91. The maximum acceleration and the maximum shear stresses computed by this solution are presented in Figure 2.5D-92.

The fundamental period of the layer computed by this procedure is approximately 0.52 sec.

2.5D.17.4.4 Comparison of Damping Ratios

The damping ratios used in the three solutions are shown in Figure 2.5D-91. As can be noted, the finite element and wave propagation solutions which permit the use of different damping ratios throughout the layer, give essentially equal values of damping ratios. Slightly greater damping ratios are obtained in the wave propagation solution. The weighted average damping ratio obtained in the lumped-mass solution is significantly greater than the individual damping ratios in the upper 20 ft. of the layer and somewhat smaller below a depth of 30 ft. It should be noted that the damping ratio obtained for each sublayer in the lumped-mass solution (based on the strain induced in the sublayer) is comparable to the corresponding value obtained in the other two solutions.

Therefore, all three solutions provide strain-dependent damping ratios that are essentially equal in each sublayer. The lumped-mass solution, however, requires that one damping ratio be used

for the entire layer. This results, for this case, in an overestimate of damping in the upper part of the layer and a slight underestimate of the damping in the lower part of the layer.

2.5D.17.4.5 Comparison of Accelerations

The maximum acceleration computed by the three solutions is shown in Figure 2.5D-92. The three solutions provided acceleration values that are quite comparable throughout the depth of the layer. The highest maximum surface acceleration is obtained by the lumped-mass solution. This value, however, is only approximately 9 percent higher than the minimum value, which is obtained by the wave propagation solution.

2.5D.17.4.6 Comparison of Shear Stresses

The maximum shear stresses computed by the three solutions are shown in Figure 2.5D-92. As can be noted, the three solutions provide essentially identical values of shear stresses in the upper 20 ft. Below this depth, the stresses computed by the wave propagation solution are slightly lower (approximately 10 percent) than those computed by the other two solutions.

2.5D.17.5 Effects of Using Frequency Dependent Damping

A direct assessment of the effects of using frequency dependent damping can be made using the lumped-mass procedure. The results shown in Figure 2.5D-92 are obtained by a lumped-mass solution (LM1) with constant damping for all modes. Another lumped-mass solution (LM2) has also been conducted for the 80-ft. layer shown in Figure 2.5-D-90. In solution LM2, the damping for each mode is increased in proportion to the frequency of the mode.

For example, the damping ratios obtained in solutions LM1 and LM2 for the first five modes are as follows:

	Solution LM1		Solution LM2	
Mode No. (n)	ω_n	λ_n	ω_n	λ_n
1	13.7	0.12	13.7	0.117
2	39.5	0.12	40.1	0.342
3	65.1	0.12	65.9	0.562
4	90.1	0.12	91.2	0.779
5	114.2	0.12	115.6	0.987

For each solution, 16 modes are incorporated in the analysis. The damping ratio for solution LM1 is 0.12 for all modes; for solution LM2, the damping ration for modes 6 to 16 were also proportioned by the ratio $\lambda_1 \cdot \omega_n/\omega_1$ as illustrated in the above listing.

The computed maximum accelerations and maximum shear stresses for solutions LM1 and LM2 are presented in Figure 2.5D-93. The maximum surface acceleration computed by solution LM1 is approximately 20 percent greater than that computed by solution LM2. The shear stresses computed by solution LM1 are approximately 10 percent to 20 percent greater than those computed by solution LM2 in the upper 30 ft. of the layer. Below a depth of approximately 60 ft., the stresses computed by LM1 are approximately 5 percent lower.

It would appear then that the use of frequency dependent damping solution results in an underestimate in the computed maximum surface acceleration and in the shear stresses computed within a shallow depth.

2.5D.17.6 Discussion and Conclusion

The finite element solution utilized in the response studies of the Category I dams incorporates the use of different damping ratios for each individual element. The solution is not carried out by modal analysis; a direct integration process is used to solve the equations of motions. The use of a Rayleigh expression to formulate the damping submatrix for each element probably introduces somewhat increased damping in the higher modes (all modes of the system are automatically included in this solution).

The response values computed were compared with those obtained by two other solutions, ie, the wave propagation solution and the lumped-mass solution. The wave propagation solution, which also permits the use of different damping ratio for each individual sublayer, incorporates a damping term that is independent of frequency. The lumped-mass solution (LM1) also incorporates a damping term that is independent of frequency, but requires that a single damping ratio be used for the entire layer.

The comparison of response values for the three solutions (Figure 2.5D-92) indicates that the finite element solution provides values which are slightly higher than those obtained by the wave propagation solution and slightly lower than those obtained by solution LM1. This comparison leads to the conclusion, that any possible increase of damping in the higher modes of the finite element solution does not affect the response significantly.

The effects of using increased damping ratios in the higher modes has been investigated directly by the lumped-mass solution. A constant damping ratio is used for all modes in solution LM1, while the damping ratio is increased in the higher modes (based on the ratio of frequencies) in solution LM2. The results for the two solutions are summarized in Figure 2.5D-93. These results indicate that the use of frequency dependent damping reduces the stresses up to 20% within shallow depths and yields slightly higher stresses at lower depths.

2.5D.18 BEHAVIOR DURING EVENT PRESCRIBED BY REGULATORY GUIDE 1.60 SPECTRA

2.5D.18.1 Introduction

The seismic stability of the Main Dam, Auxiliary Dam and Auxiliary Separating Dike were evaluated in 1973 using an input motion an event prescribed by the smooth spectrum and accelerogram shown in Figure 2.5D-7. Details of this evaluation and the conclusions derived are described in Sections 2.5D.1 through 2.5D.8 and Section 2.5D.10 through 2.5D.17. At a meeting with the NRC in Bethesda on June 13, 1977, the staff requested that the behavior of the dams and dikes be assessed; taking into account an event prescribed by Regulatory Guide 1.60 spectra. This section presents the results of this assessment, which is based on direct comparison of the Regulatory Guide 1.60 spectra with the spectra calculated for the accelerogram used as input in 1973.

2.5D.18.2 Procedure Used

In lieu of detailed finite element analyses, a simplified procedure was utilized to make this assessment. This procedure consisted of the following steps:

a) The spectral values for the accelerogram used as input in 1973 were calculated for 0.07, 0.10 and 0.15 spectral damping ratios and compared directly to the corresponding spectra obtained from Regulatory Guide 1.60 as shown in Figures 2.5D-94 through 2.5D-96. Note that Regulatory Guide 1.60 does not provide values for 0.15 damping; these values were obtained by extrapolation using the procedure suggested by Newmark-Blume-Kapur (Reference 2.5D-24) whereby a linear relationship is established between logarithm of damping and spectral values. The selection of 0.07, 0.10 and 0.15 damping ratio was based on the fact that the damping in the shells of the dam is of the order of 0.07, in the filters of the order of 0.10, and in the core is in the order of 0.15 (Section 2.5D.16).

b) The ratio of stresses induced in various portions of the embankments by the events prescribed by Regulatory Guide 1.60 spectra and the accelerogram used as input in 1973 were then estimated by a weighting procedure, using the ratios of spectra derived from Figures 2.5D-94 through 2.5D-96 for various sections of the embankments. Lower range spectral values of the input accelerogram were utilized as a conservative estimate for proportioning the stresses induced in the embankment. For each section and for each case the spectral ratios were selected at the fundamental (first mode) period, T_l. This selection was justified because the response of the embankment is controlled mainly by the fundamental mode and to a lesser extent by the second mode of vibration. A comparison of the old and new spectra at the higher modes would indicate a reduction in stress ratios if higher modes are included.

c) The stresses induced by the earthquake ground motions were then estimated by using the ratios obtained in step (b). The ratios by which the induced stresses (as calculated in 1973) were multiplied to arrive at an estimate of the stresses induced by an event prescribed by Regulatory Guide 1.60 spectra are shown in Figures 2.5D-97, 2.5D-98, and 2.5D-99 for the cores, filters and shells, respectively.

d) New local factors of safety were then calculated using the induced stresses estimated in step (c) and the cyclic strength data presented in Section 2.5D.10 through 2.5D.13.

2.5D.18.3 Results

2.5D.18.3.1 Main Dam

The minimum local factors of safety calculated in the maximum cross section of the Main Dam (M-105) for the expected constructed material properties are the following:

	1973 Analyses		Estimated for Event Prescribed by Regulatory Guide 1.60 Spectra	
Zone	N = 5	N = 10	N = 5	N = 10
Core	1.79	1.57	1.74	1.53

Fine Filters	1.73	1.55	1.69	1.51
Coarse Filters	1.22	1.20	1.18	1.16
Rockfill Shell	1.51	1.35	1.45	1.30

where N is the number of cycles of stress applications.

The minimum local factors of safety calculated in the Main Dam, 67-ft. and 36-ft. cross sections (cases M-67-IV A and M-36-IV A) are the following:

67-ft. Section

	1973 Analyses		Estimated for Event Prescribed by Regulatory Guide 1.60 Spectra	
Zone	N = 5	N = 10	N = 5	N = 10
Core	2.30	2.05	2.37	2.11
Filters	1.59	1.42	1.62	1.45
Rockfill Shell	1.57	1.40	1.60	1.42

36-ft. Section

	1973 Analyses		Estimated for Event Prescribed by Regulatory Guide 1.60 Spectra	
Zone	N = 5	N = 10	N = 5	N = 10
Core	2.48	2.20	2.58	2.29
Filters	1.82	1.62	1.88	1.68
Rockfill Shell	1.60	1.43	1.61	1.44

Other cases of the main dam were also examined. The changes in the minimum local factors of safety for these cases were similar to those outlined above.

2.5D.18.3.2 Auxiliary Dam

The minimum local factors of safety calculated in the maximum cross section of the Auxiliary Dam (A-63) for the expected constructed material properties are the following:

	1973 Analyses		Estimated for Event Prescribed by Regulatory Guide 1.60 Spectra	
Zone	N = 5	N = 10	N = 5	N = 10
Core	1.43	1.26	1.41	1.24
Filters	1.33	1.19	1.29	1.16
Random Rockfill Shell	1.56	1.38	1.51	1.34

The minimum local factors of safety calculated in the Auxiliary Dam, 44-ft. and 24-ft. cross section (Cases A-44IV A and A-24IV A) are the following:

44-ft. Section

	1973 Analyses		Estimated for Event Prescribed by Regulatory Guide 1.60 Spectra	
Zone	N = 5	N = 10	N = 5	N = 10
Core	1.94	1.72	2.00	1.78

Filters	1.37	1.22	1.40	1.25
Random Rockfill Shell	1.50	1.40	1.53	1.43
Residual Soils	1.50	1.30	1.53	1.33

<u>24-ft. Section</u>

	1973 Analyses		Estimated for Event Prescribed by Regulatory Guide 1.60 Spectra	
Zone	N = 5	N = 10	N = 5	N = 10
Core	2.25	1.99	2.34	2.07
Filters	1.59	1.42	1.64	1.46
Random Rockfill Shell	1.50	1.40	1.50	1.40
Residual Soils	1.70	1.50	1.70	1.50

Other cases of the Auxiliary Dam were also examined. The changes in the minimum local factors of safety for these cases were similar to those outlined above.

2.5D.18.3.3 Auxiliary Separating Dike

The minimum local factors of safety calculated in the maximum cross section of the Auxiliary Reservoir Separating Dike for the expected constructed material properties are the following:

	1973 Analyses		Estimated for Event Prescribed by Regulatory Guide 1.60 Spectra	
Zone	N = 5	N = 10	N = 5	N = 10
Core	1.7	1.5	1.7	1.5
Random Rockfill Shells	1.9	1.7	1.9	1.7

2.5D.18.4 Conclusions

The results of the assessment of the behavior of the Main Dam, Auxiliary Dam and Auxiliary Separating Dike during an event prescribed by Regulatory Guide 1.60 spectra lead to the following conclusions:

a) The spectral values of the accelerogram, used as input motion of 19773, are somewhat larger (of the order of 5 to 15 percent) than the spectral values prescribed in Regulatory Guide 1.60 in the period range of approximately 0.1 to 0.2 sec. At a period of approximately 0.4 sec., the Regulatory Guide spectra are somewhat larger (of the order of 10 percent) for the lower damping ratios, but are essentially equal for damping ratios of the order of 0.15.

b) The stresses induced in the dams and the dikes during the event prescribed by Regulatory Guide 1.60 spectra are estimated to differ by less than $\pm$4 percent from those calculated in 1973.

c) The minimum local factors of safety in the dams and dikes estimated during the event prescribed by Regulatory Guide 1.60 spectra are comparable to those calculated in 1973.

d) The Main Dam, Auxiliary Dam and Auxiliary Reservoir Separating Dike will be stable, will maintain their integrity during an event prescribed by Regulatory Guide 1.60 spectra and will have a peak (zero-period) acceleration of 0.15g.

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TABLE 2.1.2-1

MINIMUM DISTANCE FROM THE SHNPP TO THE EXCLUSION AREA BOUNDARY FOR EACH MAJOR COMPASS DIRECTION

Sector	Distance (ft.)
N	6980
NNE	7000
NE	7000
ENE	7000
E	7000
ESE	7000
SE	7000
SSE	7000
S	7200
SSW	7000
SW	7000
WSW	7000
W	7000
WNW	7000
NW	6660
NNW	6640

NOTE: Distances measured from center point of the originally planned four Units.

TABLE 2.1.3-6 SUMMARY OF TOTAL POPULATION DEMAND WITHIN THE 10-MILE EPZ

Sub-Zone	Residents	Transit-Dependent	Transients	Employees	Special Facilities	Schools	Shadow Population	External Traffic	Total
A	134	4	401	519	44	0	0	0	1,102
B	1,257	42	289	0	0	0	0	0	1,588
C	2,086	69	70	0	3	0	0	0	2,228
D	346	11	224	0	0	0	0	0	581
E	45,269	1,504	1,230	1,228	261	8,889	0	0	58,376
F	22,342	743	703	789	44	7,936	0	0	32,552
G	21,463	713	824	582	407	5,002	0	0	28,991
H	3,868	128	80	0	0	0	0	0	4,076
I	963	32	0	0	0	0	0	0	995
J	1,126	37	0	57	137	0	0	0	1,357
K	688	23	440	247	0	0	0	0	1,398
L	815	27	2,767	45	0	0	0	0	3,654
M	1,753	58	2,306	0	0	285	0	0	4,402
N	851	28	2,108	0	0	0	0	0	2,987
Shadow	0	0	0	0	0	1418[1]	39,618	0	41,036
Total	102,961	3,419	11,442	3,467	896	23,530	39,618	0	185,323

NOTES:

1) Shadow Population has been reduced to 20%.
2) Special Facilities only includes medical facilities.

[1] There are two schools in the Shadow that evacuate - Deep River Elementary School in Lee County and Lafayette Elementary School in Harnett County. County emergency plans call for these facilities to be evacuated because of their close proximity to the EPZ boundary.

TABLE 2.2.2-1 MINING AND QUARRIES WITHIN A TEN MILE RADIUS OF THE SHEARON HARRIS NUCLEAR POWER PLANT

	Mines and Quarries (Figure 2.2.2-1)	County	Products
A.	Buckhorn Quarry	Wake	Inactive
B.	Holly Springs Quarry	Wake	Inactive
C.	Cherokee Brick	Chatham	Inactive
D.	Cherokee Brick	Chatham	Inactive
E.	Cherokee Brick	Chatham	Inactive
F.	Moncure Quarry	Lee	Crushed Stone
G.	Martin Marietta Aggregate	Wake	Granite

TABLE 2.2.2-2

AIRCRAFT OPERATIONS - RALEIGH-DURHAM AIRPORT

	Landings Per Year				
	Air Carrier	General Aviation	Air Taxi	Military	Total
Actual					
1976	30,826	147,861	9,365	9,568	197,620
1977	33,306	152,229	11,462	9,059	206,056
1978	34,145	154,476	13,153	7,470	209,244
1979	39,929	146,203	14,889	6,720	207,741
1980	40,225	130,079	24,382	7,487	202,173
1985	55,648	111,138	31,299	10,609	208,694
1990	124,113	83,041	67,113	8,683	283,055
1995	90,976	69,007	38,865	6,041	204,889
2000	152,817	67,325	71,434	5,103	296,679
Projected					
2005	92,100	71,300	113,800	5,000	282,200
2010	113,200	77,600	134,300	5,000	330,100
2025	200,800	99,900	232,600	5,000	538,300

Source: Raleigh-Durham Airport Authority

TABLE 2.2.3-1

DIMENSIONS OF PROPANE PLUME DOWNWIND OF 324 $FT.^3$/SEC. SOURCE, STABILITY CATEGORY F, WIND SPEED 3.3 FT./SEC.

Downwind Distance (ft.)	Plume Dispersion (std deviation) ft. σ_y	σ_z ($\sigma_z \simeq 0.5\sigma_y$)	Centerline Concentration Percent	Horizontal Distance From Plume Axis, ft. To Rich Limit y_r	To Lean Limit y_1	Vertical Distance From Plume Axis, ft. To Rich Limit z_r	To Lean Limit z_1
250	11	6	47.5	21.6	26.2	11.6	14.3
500	20	10	15.6	25.4	37.2	12.7	18.6
750	27	14	8.8	15.2	39.8	7.85	20.6
830	30	15	7.0	0	40.5	0	20.2
920	33.6	16.8	5.55	--	39.6	--	19.9
1000	36	18	4.8	--	36.6	--	18.3
1425	47.5	23.8	2.8	--	0	--	0
1640	53.4	26.7	2.21	--	--	--	--
3000	92	46	0.74	--	--	--	--

TABLE 2.2.3-2

DIMENSIONS OF PROPANE PLUME DOWNWIND OF 1000 FT.^{3}/SEC. SOURCE, STABILITY CATEGORY F, WIND SPEED 3.3 FT./SEC.

Downwind Distance (ft.)	Plume Dispersion (std deviation) ft. σ_y	σ_z ($\sigma_z \simeq 0.2\sigma_y$)	Centerline Concentration Percent	Horizontal Distance From Plume Axis, ft. To Rich Limit y_r	To Lean Limit y_1	Vertical Distance From Plume Axis, ft. To Rich Limit z_r	To Lean Limit z_1
750	27	5.4	66	47.2	68	9.44	13.6
1000	36	7.2	37.2	65.6	81.8	13.1	16.36
2000	72	14.4	9.34	54.6	112.0	10.92	22.4
2500	81	16.2	7.35	25.7	112.5	5.14	22.5
2600	83	16.75	7.0	0	112.5	0	22.5
3000	92	18.4	5.70	--	109.5	--	21.9
3500	104	20.8	4.50	--	102.0	--	20.4
4000	116	23.2	3.60	--	82.5	--	16.5
4750	131	26.2	2.80	--	0	--	0
5000	136	27.2	2.60	--	--	--	--

TABLE 2.2.3-3

BLAST AND SEISMIC PARAMETERS FOR SHOCK WAVES FROM PROPANE DETONATIONS

Yield (Ton of TNT)	Distance From Center of Detonation	Peak Over-pressure PSI	Peak Dynamic Pressure psi	Peak Reflected Pressure psi	Positive Phase Duration (msec)	Peak Acceleration Horizontal or Vertical* (g's)	Peak * Velocity (f/s)	Peak Displacement (in.)
8.9	7500	0.10	.00024	0.20	209	.002	.0027	.0001
100	5000	0.5	.006	1.01	292	.023	.0306	.0025
119	2200	1.2	.034	2.48	310	.133	.177	.0157

*Based on a conservatively assumed seismic velocity of 1000 ft./sec. and a specific gravity of 1.5.

The peak dynamic pressure, P_d, is calculated from $P_d = \frac{5(p^2)}{2(7P_o+p)}$

and the peak reflected pressure, P_r, is calculated from $P_r = 2p\,\frac{(7P_o+4p)}{(7P_o+p)}$ where p is the peak overpressure and P_o the ambient pressure.

TABLE 2.2.3-4 HAZARDOUS MATERIALS MOST FREQUENTLY SHIPPED BY RAIL

Material	Boiling Point (°C)	Vapor Pressure (mm Hg/°C)	Toxicity Hazard Rating (2) Acute Local: Inhalation	Comments
Ammonia (Anhydrous)	-33.35	10/25.7	3	Gaseous at atmospheric conditions and very toxic. Considered in control room habitability evaluation.
Caustic Soda (Sodium Hydroxide)	1390	1/739	2	Not a threat to the control room ventilation system. (3)
Liquid Propane Gas	-42.1	-	1	Not a threat considering the toxicity and the location of the railroad (1.9 miles).
Sulfuric Acid	330	1/145.8	2	Not a threat to the control room ventilation system. (3)
Chlorine	-34.5	4800/20	3, 4	Not a threat due to the probability of occurrence.
Propane	-42.1	760/-42.1	1	Not a threat considering the toxicity and the location of the railroad (1.9 miles).
Ammonium Nitrate	-	11/210	1	Not a threat to the control room ventilation system. (3)
Gasoline (Aliphatic Hydrocarbons)	150-190	-	2	Not a threat to the control room ventilation system. (3)
Phosphoric Acid	-	0.0285/20	2	Not a threat to the control room ventilation system.
Crude Oil	(Mixture of hydrocarbons)		1	Not a threat. Low inhalation toxicity index.
Methanol (Methyl Alcohol)	64.8	100/21.1	2	Not a threat considering the toxicity and the location of the railroad (1.9 miles).
Petroleum Distillate (Mineral Spirits)	150-190	-	2	Not a threat considering the toxicity and the location of the railroad (1.9 miles)
Vinyl Chloride	-13.4	2600/25	3	Considered in the control room habitability evaluation.
Butane	-0.5	1520/18.8	2	Not a threat considering the toxicity and the location of the railroad (1.9 miles).
Motor Fuel Anti-Knock Compound (Lead Tetraethyl)	198-202	1/38.4	3	Not a threat to the control room ventilation system. (3)
Butadiene (Erythrene)	-4.5	1840/21	2	Not a threat considering the toxicity and the location of the railroad (1.9 miles).
Petroleum Naphtha (Petroleum Spirits)	40-80	-	2	Not a threat considering the toxicity and the location of the railroad (1.9 miles).

(1) An Evaluation of Railroad Safety, Office of Technology Assessment, Congress of the United States, 1978.

(2) Toxicity Hazard Rating Code:

- 0 NONE: (a) No harm under any conditions; (b) Harmful only under unusual conditions or overwhelming dosage.
- 1 SLIGHT: Causes readily reversible changes which disappear after end of exposure.
- 2 MODERATE: May involve both irreversible and reversible changes not severe enough to cause death or permanent injury.
- 3 HIGH: May cause death or permanent injury after very short exposure to small quantities.
- U UNKNOWN: No information on humans considered valid by authors.

(3) Regulatory Guide 1.78 required (C.5.a) that consideration be given to those chemicals that are not gases at 100°F and normal atmospheric pressure but are liquids with vapor pressures in excess of 10 torr (10mm Hg at 0°C). The liquids excepted per this note do not fall in this category. They are not expected to be in the gaseous forms in quantities that can pose a threat.

(4) Reference SHNPP License Amendment No. 10 dated May 3, 1989 and Technical Specification change request, NLS-88-281, dated January 4, 1989, Chlorine Detection System.

TABLE 2.2.3-5

EFFECTS OF INCREASE IN PIPELINE SIZE FROM SIX INCHES TO EIGHT INCHES

a. Detonable Clouds

Model	Model Description	Line Size	Cloud Size [Volume (ft^3)] [Length (ft)]	TNT Equivalent [Tons]	Proximity of Detonation to Safety Related Structures (ft)	Peak Overpressure [psi]
Atmospheric Dispersion	Gaussian Cloud or Plume	6”	9×10^5 4750	100	5000	0.5
		8”	1.595×10^7 5750	158	4000	0.7
Gravity Layer Formation/Dispersion	Gravity Slump, Atmospheric Dispersion, Diffusion	6”	1.5×10^7 475	118	2200	1.2
		8”	1.576×10^7 4750	155	2500	<1.2

b. Missiles. The energy of the postulated missile change from 45 to 70 foot-lbs. This is still below the energy required to penetrate safety related structures. Therefore the missile hazard resulting from the change from the six inch pipe to the eight inch pipe is acceptable.
c. Fire Hazard. There is no change to the analysis resulting from the change from six inch to the eight inch pipe.
d. Seismic. Analysis is bounded by the Gravity Layer Formation/Dispersion model for six inch pipe.

TABLE 2.3.1-1

Stations Referenced for Regional Climatology and Local Meteorology

Station	Elevation (ft.)	Distance from Plant Site (mi.)	Direction from Plant Site	Climatological Region of North Carolina
Raleigh-Durham	434	19	NNE	Central Piedmont
Moncure	202	7	W	Central Piedmont
Pinehurst	548	44	SW	Southern Piedmont
Asheboro	870	54	W	Central Piedmont
Greensboro	886	69	WNW	Northern Piedmont
Charlotte	736	117	WSW	Southern Piedmont

TABLE 2.3.1-2

SITE AREA
NUMBER OF THUNDERSTORM DAYS

Month	Greensboro	Charlotte	Raleigh-Durham
January	*	1	*
February	1	1	1
March	2	2	2
April	3	3	4
May	7	6	6
June	9	8	7
July	11	10	11
August	9	7	8
September	3	3	4
October	1	1	1
November	*	1	1
December	*	*	*
Annual	47	42	46
Period of Record (years)	49	38	33

* = Indicates less than .5

TABLE 2.3.1-3

Number of Cloud-to-Ground Flashes by Season (per sq. km.)

	LOCATION		
Season	Greensboro	Charlotte	Raleigh-Durham
Winter (D, J, F)	.18	.36	.18
Spring (M, A, M)	2.19	2.00	2.19
Summer (J, J, A)	5.28	4.55	4.74
Fall (S, O, N)	.73	.91	1.10
Annual	8.56	7.65	8.38

TABLE 2.3.1-4

EXTREME WINDS AND PRECIPITATION ASSOCIATED WITH HURRICANES
RALEIGH-DURHAM AIRPORT
(1950-1978)

Storm	Date	Maximum Winds (mph)	Maximum Precipitation (inches/hr)	24-hr Precipitation (inches)
Able	31 Aug. 1952	ESE 30 G 40	1.22	3.52
Barbara	13 Aug. 1953	NE 20 G 28	Trace	Trace
Carol	30 Aug. 1954	N 18	Trace	Trace
Edna	10 Sept. 1954	N 20; NNE 16 G 25	Trace	0.01
Hazel	15 Oct. 1954	WNW 73 G 90; NW 48 G 62	1.55	4.04
Connie	11-12 Aug. 1955	E 35 G 46; NE 39 G 54; N 40	0.30; 0.25	0.68; 0.75
Diane	16-17 Aug. 1955	SE 38 G 44; ENE 32 G 53	0.48; 0.70	1.23; 4.12
Ione	19 Sept. 1955	NNE 30; G 49	0.18	0.86
Flossy	26 Sept. 1956	NNE 28; G 46; NNE 29 G 41	0.37	2.31
Helene	27 Sept. 1958	N 29 G 46	Trace	0.07
Gracie	30 Sept. 1959	SSE 25 G 37	0.64	0.78
Brenda	29 July 1960	N 24	0.47	2.60
Donna	11 Sept. 1960	N 29 G 35	0.31	1.48
Esther	20 Sept. 1961	N 17	0.15	0.15
Alma	28 Aug. 1962	NW 16	Trace	Trace
Ella	18-19 Oct. 1962	NE 22 G 32	0.00	0.00
Ginny	20-21 Oct. 1963	NNE 21 G 32; N 22 G 29	0.00	0.00
Cleo	31 Aug. 1964	NNW 15	1.12	2.95
Dora	13 Sept. 1964	NNE 25 G 38	0.31	2.36
Gladys	22 Sept. 1964	N 18 G 25	0.00	0.00
Isbell	16 Oct. 1964	NE 20 G 29	0.19	0.55
Alma	11 June 1966	NNE 23 G 32	Trace	Trace
Agnes	19-21 June 1972	14 E; 20 SE; 24 N	.03; .29; .55	.03; 1.3; 1.59
Ginger	30 Sept. - 2 Oct. 1971	32 NNW; 29 N; 14 W	.11; .34; .08	.61; 2.64; .33
Doria	26-28 Aug. 1971	20 E, 15 NNW, 9N	.53; .08; Trace	1.23; .23; Trace
Gladys	18-20 Oct. 1968	17 ESE; 15 S; 18 N	.12; .59; .01	.63; l.84; .01
Doria	9 Sept. 1967	15 N	.15	.89
Belle	8, 9 Aug. 1976	8 ENE; 13 NNW	.09; 0.0	.18; 0.00
Eloise	22-26 Sept. 1975	16 E; 14 SSE 16 SSE; 10 NW 10 WNW	.24; .32 .72; .50 .30	.95; .77 1.26; 1.02 .44

NOTE: “G” indicates “gusts to”

TABLE 2.3.1-5

SITE REGION METEOROLOGICAL EXTREMES
(month/year of occurrence) [Data period]

	Charlotte	Greensboro	Raleigh-Durham	Pinehurst	Asheboro	Moncure
Maximum Monthly Precipitation (water equivalent)	12.48 in (5/75) [1940-77]	13.26 in (9/47) [1929 77]	12.94 in (9/45) [1945-77]	13.88 in (7/59) [1951-73]	13.79 in (7/65) [1951-73]	12.55 in (7/73) [1951-73]
Maximum 24 hour Precipitation (water equivalent)	5.34 in (10/76) [1940-77]	7.49 in (9/47) [1929-77]	5.20 in (8/55) [1945-77]	7.11 in (10/54) [1951-73]	8.96 in (8/66) [1951-73]	5.14 in (8/67) [1951-73]
Minimum Monthly Precipitation (water equivalent)	Trace (10/53) [1940-77]	.13 in (9/39) [1929-77]	.23 in (4/76) [1945-77]	---	---	---
Maximum Monthly Snowfall (inches)	19.3 in (3/60) [1940-77]	22.9 in (1/66) [1929-70]	14.4 in (1/55) [1945-77]	16.0 in (12/58) [1951-73]	18.5 in (3/60) [1951-73]	14.0 in (3/60) [1951-73]
Maximum 24 hour Snowfall (inches)	12.0 in (2/69) [1940-77]	14.3 in (12/30) [1929-70]	9.3 in (3/69) [1945-77]	---	---	---
Maximum Temperature (°F)	104°F (9/54) [1940-77]	102°F (7/77) [1929-70]	105°F (7/52) [1945-77]	106°F (8/54) [1931-73]	103°F* (7/52) [1951-73]	107°F* (7/52) [1951-73]
Minimum Temperature (°F)	-5°F (1/85) [1940-85]	-8°F (1/85) [1929-85]	-9°F (1/85) [1945-85]	+3°F* (12/62) [1951-73]	-8°F 1/85 [1951-85]	-4°F (1/66) [1951-73]

*On earlier dates

TABLE 2.3.1-6

MEAN MONTHLY MAXIMUM MIXING DEPTHS (METERS ABOVE SURFACE)

Greensboro	
Month	Depth (m)
January	390
February	650
March	1130
April	1180
May	1530
June	1790
July	1490
August	1420
September	1370
October	1020
November	840
December	580

TABLE 2.3.1-7

FREQUENCY OF INVERSIONS BASED BELOW 500 FEET

Season	Percent Frequency of Inversion Occurrence at Specific Times and All Times				
	0300 GMT	1500 GMT	0000 GMT	1200 GMT	All Times
Winter	73	15	58	72	43
Spring	70	3	13	66	32
Summer	78	1	11	6	33
Fall	74	4	52	74	40

NOTE:

1. 0300 and 1500 GMT observations for the period 6/55 - 5/57.
2. 0000 and 1200 GMT observations for the period 6/57 - 5/59.
3. Observations made at Greensboro, N.C.

TABLE 2.3.2-1 WIND DISTRIBUTION BY PASQUILL STABILITY CLASSES (STAR PROGRAM)							
	A STABILITY SPEED (KTS)						
DIRECTION	0 - 3	4 - 6	7 - 10	11 - 16	17 - 21	GREATER THAN 21	TOTAL
N	0.000690	0.000354	0.000000	0.000000	0.000000	0.000000	0.001044
NNE	0.000458	0.000194	0.000000	0.000000	0.000000	0.000000	0.000652
NE	0.000295	0.000148	0.000000	0.000000	0.000000	0.000000	0.000444
ENE	0.000272	0.000171	0.000000	0.000000	0.000000	0.000000	0.000444
E	0.000423	0.000308	0.000000	0.000000	0.000000	0.000000	0.000731
ESE	0.000254	0.000137	0.000000	0.000000	0.000000	0.000000	0.000391
SE	0.000266	0.000126	0.000000	0.000000	0.000000	0.000000	0.000391
SSE	0.000217	0.000148	0.000000	0.000000	0.000000	0.000000	0.000365
S	0.000442	0.000263	0.000000	0.000000	0.000000	0.000000	0.000705
SSW	0.000445	0.000285	0.000000	0.000000	0.000000	0.000000	0.000731
SW	0.000897	0.000434	0.000000	0.000000	0.000000	0.000000	0.001331
WSW	0.000651	0.000445	0.000000	0.000000	0.000000	0.000000	0.001096
W	0.000775	0.000400	0.000000	0.000000	0.000000	0.000000	0.001174
WNW	0.000573	0.000263	0.000000	0.000000	0.000000	0.000000	0.000835
NW	0.000395	0.000205	0.000000	0.000000	0.000000	0.000000	0.000600
NNW	0.000527	0.000308	0.000000	0.000000	0.000000	0.000000	0.000835
TOTAL	0.0007581	0.004190	0.000000	0.000000	0.000000	0.000000	

RELATIVE FREQUENCY OF OCCURRENCE OF A STABILITY = 0.011770
RELATIVE FREQUENCY OF CALMS DISTRIBUTED ABOVE WITH A STABILITY = 0.006622

TABLE 2.3.2-1 WIND DISTRIBUTION BY PASQUILL STABILITY CLASSES (STAR PROGRAM)							
	B STABILITY SPEED (KTS)						
DIRECTION	0 - 3	4 - 6	7 - 10	11 - 16	17 - 21	GREATER THAN 21	TOTAL
N	0.001245	0.001781	0.001199	0.000000	0.000000	0.000000	0.004224
NNE	0.000853	0.001164	0.001039	0.000000	0.000000	0.000000	0.003056
NE	0.001054	0.001393	0.001085	0.000000	0.000000	0.000000	0.003532
ENE	0.000533	0.000890	0.000605	0.000000	0.000000	0.000000	0.002029
E	0.000594	0.000993	0.000833	0.000000	0.000000	0.000000	0.002420
ESE	0.000517	0.000788	0.000616	0.000000	0.000000	0.000000	0.001922
SE	0.000557	0.000970	0.000628	0.000000	0.000000	0.000000	0.002155
SSE	0.000576	0.000833	0.000582	0.000000	0.000000	0.000000	0.001991
S	0.001040	0.001792	0.001735	0.000000	0.000000	0.000000	0.004568
SSW	0.001387	0.001564	0.001450	0.000000	0.000000	0.000000	0.004401
SW	0.001902	0.002740	0.002443	0.000000	0.000000	0.000000	0.007085
WSW	0.001415	0.002055	0.001747	0.000000	0.000000	0.000000	0.005217
W	0.001568	0.002169	0.002158	0.000000	0.000000	0.000000	0.005895
WNW	0.001421	0.001678	0.001313	0.000000	0.000000	0.000000	0.004413
NW	0.000973	0.001370	0.000913	0.000000	0.000000	0.000000	0.003257
NNW	0.000883	0.001119	0.000902	0.000000	0.000000	0.000000	0.002984
TOTAL	0.016520	0.023301	0.019328	0.000000	0.000000	0.000000	

RELATIVE FREQUENCY OF OCCURRENCE OF B STABILITY = 0.059149
RELATIVE FREQUENCY OF CALMS DISTRIBUTED ABOVE WITH B STABILITY = 0.009168

TABLE 2.3.2-1 WIND DISTRIBUTION BY PASQUILL STABILITY CLASSES (STAR PROGRAM)							
	C STABILITY SPEED (KTS)						
DIRECTION	0 - 3	4 - 6	7 - 10	11 - 16	17 - 21	GREATER THAN 21	TOTAL
N	0.000836	0.002614	0.006256	0.001027	0.000057	0.000011	0.010803
NNE	0.000450	0.001439	0.004327	0.000696	0.000011	0.000011	0.006935
NE	0.000392	0.001564	0.005058	0.000913	0.000057	0.000000	0.007984
ENE	0.000399	0.001244	0.002923	0.000434	0.000000	0.000000	0.005000
E	0.000379	0.001279	0.003048	0.000502	0.000023	0.000000	0.005231
ESE	0.000340	0.001005	0.001712	0.000091	0.000000	0.000000	0.003149
SE	0.000307	0.001187	0.002021	0.000148	0.000000	0.000000	0.003664
SSE	0.000328	0.001085	0.002101	0.000263	0.000000	0.000000	0.003776
S	0.000578	0.002397	0.006165	0.000719	0.000034	0.000000	0.009894
SSW	0.000905	0.003334	0.006565	0.001153	0.000034	0.000000	0.011991
SW	0.001361	0.004806	0.009259	0.001347	0.000011	0.000000	0.016785
WSW	0.001003	0.003208	0.005309	0.000674	0.000011	0.000000	0.010205
W	0.000800	0.002923	0.005594	0.001005	0.000023	0.000000	0.010344
WNW	0.000638	0.002215	0.004338	0.000742	0.000046	0.000000	0.007979
NW	0.000551	0.001827	0.004236	0.000879	0.000023	0.000000	0.007515
NNW	0.000572	0.001724	0.003539	0.000639	0.000011	0.000000	0.006486
TOTAL	0.009841	0.033850	0.072449	0.011234	0.000342	0.000023	

RELATIVE FREQUENCY OF OCCURRENCE OF C STABILITY = 0.127740
RELATIVE FREQUENCY OF CALMS DISTRIBUTED ABOVE WITH C STABILITY = 0.006976

TABLE 2.3.2-1 WIND DISTRIBUTION BY PASQUILL STABILITY CLASSES (STAR PROGRAM)							
	D STABILITY SPEED (KTS)						
DIRECTION	0 - 3	4 - 6	7 - 10	11 - 16	17 - 21	GREATER THAN 21	TOTAL
N	0.001991	0.006039	0.018175	0.014716	0.001747	0.000320	0.042988
NNE	0.001685	0.005309	0.016177	0.014316	0.002066	0.000285	0.039839
NE	0.001945	0.006085	0.016668	0.011576	0.000925	0.000080	0.037279
ENE	0.001629	0.005046	0.010720	0.005994	0.000251	0.000046	0.023686
E	0.001416	0.004886	0.010686	0.003904	0.003423	0.000068	0.021304
ESE	0.001234	0.003825	0.007090	0.002580	0.000274	0.000068	0.015071
SE	0.001022	0.003539	0.008608	0.002580	0.000308	0.000034	0.016092
SSE	0.001313	0.003608	0.007615	0.003699	0.000377	0.000023	0.016634
S	0.002016	0.007078	0.019168	0.011188	0.001153	0.000091	0.040695
SSW	0.002026	0.006667	0.019648	0.013540	0.001427	0.000171	0.043480
SW	0.001947	0.006553	0.015709	0.011085	0.001267	0.000160	0.036722
WSW	0.001517	0.004053	0.005434	0.004121	0.000422	0.000080	0.015628
W	0.001496	0.004281	0.005537	0.008220	0.001553	0.000126	0.021212
WNW	0.001226	0.003128	0.005434	0.011816	0.002375	0.000217	0.024196
NW	0.001090	0.003140	0.006987	0.011519	0.002295	0.000251	0.025281
NNW	0.001153	0.003573	0.009031	0.009373	0.001336	0.000228	0.024694
TOTAL	0.024705	0.076811	0.182688	0.140230	0.018118	0.002249	

RELATIVE FREQUENCY OF OCCURRENCE OF D STABILITY = 0.444801
RELATIVE FREQUENCY OF CALMS DISTRIBUTED ABOVE WITH D STABILITY = 0.017661

TABLE 2.3.2-1 WIND DISTRIBUTION BY PASQUILL STABILITY CLASSES (STAR PROGRAM)							
	E STABILITY SPEED (KTS)						
DIRECTION	0 - 3	4 - 6	7 - 10	11 - 16	17 - 21	GREATER THAN 21	TOTAL
N	0.000000	0.003939	0.006873	0.000000	0.000000	0.000000	0.010811
NNE	0.000000	0.003265	0.003356	0.000000	0.000000	0.000000	0.006622
NE	0.000000	0.003722	0.003322	0.000000	0.000000	0.000000	0.007044
ENE	0.000000	0.002980	0.002318	0.000000	0.000000	0.000000	0.005297
E	0.000000	0.004156	0.004624	0.000000	0.000000	0.000000	0.008779
ESE	0.000000	0.003025	0.002763	0.000000	0.000000	0.000000	0.005788
SE	0.000000	0.003345	0.002340	0.000000	0.000000	0.000000	0.005685
SSE	0.000000	0.003311	0.003356	0.000000	0.000000	0.000000	0.006667
S	0.000000	0.008665	0.009944	0.000000	0.000000	0.000000	0.018609
SSW	0.000000	0.008186	0.009658	0.000000	0.000000	0.000100	0.017844
SW	0.000000	0.006633	0.005948	0.000000	0.000000	0.000100	0.012581
WSW	0.000000	0.002500	0.001895	0.000000	0.000000	0.000000	0.004395
W	0.000000	0.002375	0.004395	0.000000	0.000000	0.000000	0.006770
WNW	0.000000	0.001792	0.004978	0.000000	0.000000	0.000000	0.006770
NW	0.000000	0.001918	0.004921	0.000000	0.000000	0.000000	0.006839
NNW	0.000000	0.001998	0.005069	0.000000	0.000000	0.000000	0.007067
TOTAL	0.000000	0.061809	0.075760	0.000000	0.000000	0.000000	

RELATIVE FREQUENCY OF OCCURRENCE OF E STABILITY = 0.137570
RELATIVE FREQUENCY OF CALMS DISTRIBUTED ABOVE WITH E STABILITY = 0.000000

TABLE 2.3.2-1 WIND DISTRIBUTION BY PASQUILL STABILITY CLASSES (STAR PROGRAM)

	F STABILITY SPEED (KTS)						
DIRECTION	0 - 3	4 - 6	7 - 10	11 - 16	17 - 21	GREATER THAN 21	TOTAL
N	0.008955	0.009430	0.000000	0.000000	0.000000	0.000000	0.018385
NNE	0.006378	0.005594	0.000000	0.000000	0.000000	0.000000	0.011972
NE	0.005050	0.005137	0.000000	0.000000	0.000000	0.000000	0.010187
ENE	0.004837	0.004384	0.000000	0.000000	0.000000	0.000000	0.009221
E	0.005642	0.006051	0.000000	0.000000	0.000000	0.000000	0.011693
ESE	0.005481	0.004966	0.000000	0.000000	0.000000	0.000000	0.010447
SE	0.004368	0.003756	0.000000	0.000000	0.000000	0.000000	0.008124
SSE	0.004762	0.004738	0.000000	0.000000	0.000000	0.000000	0.009499
S	0.011355	0.012124	0.000000	0.000000	0.000000	0.000000	0.023479
SSW	0.014542	0.015127	0.000000	0.000000	0.000000	0.000100	0.029669
SW	0.012224	0.012296	0.000000	0.000000	0.000000	0.000100	0.024520
WSW	0.006704	0.005137	0.000000	0.000000	0.000000	0.000000	0.011842
W	0.006311	0.005754	0.000000	0.000000	0.000000	0.000000	0.012065
WNW	0.004924	0.004464	0.000000	0.000000	0.000000	0.000000	0.009388
NW	0.004409	0.004532	0.000000	0.000000	0.000000	0.000000	0.008942
NNW	0.004764	0.004772	0.000000	0.000000	0.000000	0.000000	0.009537
TOTAL	0.110706	0.108263	0.000000	0.000000	0.000000	0.000000	

RELATIVE FREQUENCY OF OCCURRENCE OF F STABILITY = 0.218970
RELATIVE FREQUENCY OF CALMS DISTRIBUTED ABOVE WITH F STABILITY = 0.084494

TABLE 2.3.2-1 WIND DISTRIBUTION BY PASQUILL STABILITY CLASSES (STAR PROGRAM)							
	ALL STABILITIES SPEED (KTS)						
DIRECTION	0 - 3	4 - 6	7 - 10	11 - 16	17 - 21	GREATER THAN 21	TOTAL
N	0.013149	0.024157	0.032503	0.015743	0.001804	0.000331	0.087688
NNE	0.009875	0.016965	0.024900	0.015013	0.002078	0.000297	0.069127
NE	0.009486	0.018050	0.026133	0.012490	0.000982	0.000080	0.067219
ENE	0.008073	0.014716	0.016565	0.006428	0.000251	0.000046	0.046079
E	0.008703	0.017673	0.019191	0.004407	0.000365	0.000068	0.050408
ESE	0.007822	0.013746	0.012181	0.002671	0.000274	0.000068	0.036763
SE	0.006990	0.012924	0.013597	0.002729	0.000308	0.000034	0.036581
SSE	0.007458	0.013723	0.013654	0.003962	0.000377	0.000023	0.039196
S	0.015870	0.032320	0.037013	0.011907	0.001187	0.000091	0.098389
SSW	0.018655	0.035163	0.037321	0.014693	0.001461	0.000171	0.107465
SW	0.018068	0.033462	0.033359	0.012433	0.001279	0.000160	0.098760
WSW	0.011126	0.017399	0.014385	0.004795	0.000434	0.000080	0.048218
W	0.010639	0.017901	0.017684	0.009225	0.001575	0.000126	0.057150
WNW	0.008538	0.013540	0.016063	0.012558	0.002420	0.000217	0.053336
NW	0.007277	0.012992	0.017056	0.012398	0.002318	0.000251	0.052292
NNW	0.007625	0.013494	0.018620	0.010012	0.001347	0.000228	0.051327
TOTAL	0.169353	0.308224	0.350226	0.151463	0.018461	0.002272	

RELATIVE FREQUENCY OF OCCURRENCE = 1.000000
RELATIVE FREQUENCY OF CALMS DISTRIBUTED ABOVE = 0.124920

TABLE 2.3.2-2

WIND DIRECTION PERSISTENCE DATA*

HARRIS ON-SITE METEOROLOGICAL FACILITY

JANUARY 14, 1976 TO DECEMBER 31, 1978

STABILITY CLASS A

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	43	18	11	6	4							
NNE	24	13	4	7	4	1						
NE	20	10	8	2	6							
ENE	13	3	3	3	1							
E	6	4	2	2								
ESE	11	4	1									
SE	13	1	1									
SSE	9	7		1	1							
S	19	4	2									
SSW	29	12	8	12	4							
SW	32	26	16	5	9							
WSW	33	14	14	6	7							
W	29	11	2	2	2							
WNW	36	11	11	4	8							
NW	44	21	8	6	4							
NNW	31	15	6	3	9	1						
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.6	8.5	0.0	0.0	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	7	9	0	0	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 49

*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
STABILITY CLASS C

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	67	15	5	2	1							
NNE	43	7	2	1								
NE	39	4	2									
ENE	33	8	4									
E	27	7	1									
ESE	14	2	4									
SE	15	2	3									
SSE	24	6	1	1								
S	35	11	1									
SSW	80	15	1	2								
SW	80	14	7	3								
WSW	72	9	9	1	1							
W	46	10	1	5	1							
WNW	67	7	5		1							
NW	64	13	4		1							
NNW	66	8	3	3								
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	5	0	0	0	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 17

*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
STABILITY CLASS B

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	45	11	5									
NNE	41	4	1	1	2							
NE	37	7	6									
ENE	21	6		1								
E	19	3	1									
ESE	15	4	1									
SE	14											
SSE	10	2										
S	22	6	2		1							
SSW	39	13	3	2	1							
SW	56	16	6									
WSW	62	16	3	1	2							
W	31	7	2									
WNW	53	16	4	2								
NW	48	19	4	2	2							
NNW	50	8	1									
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	6	0	0	0	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 18
*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)

WIND DIRECTION PERSISTENCE DATA*

HARRIS ON-SITE METEOROLOGICAL FACILITY

JANUARY 14, 1976 TO DECEMBER 31, 1978

STABILITY CLASS D

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	145	61	24	13	18	10	4	1				
NNE	134	54	21	11	23	4	3	3	2		1	
NE	124	46	17	14	5	5	1					1
ENE	101	31	12	7	11	2	1					
E	83	34	11	2	7	2						
ESE	85	20	11	7	4							
SE	82	28	11	5	6	1						
SSE	98	31	15	11	10	2						
S	132	48	15	5	7	1						
SSW	203	65	26	11	14		2			1		
SW	219	71	33	14	16	5						
WSW	160	61	31	16	15	2						
W	141	37	12	4	5		1					
WNW	138	44	15	11	8	2	2					
NW	137	37	13	9	14	2						
NNW	148	52	20	15	14	3	1					
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.5	8.8	11.6	14.7	17.5	20.0	24.0	28.0
MAXIMUM HOURS	1	2	3	4	7	10	13	16	18	20	24	28

NUMBER HOURS OF MISSING WIND DIRECTIONS: 88

*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)

WIND DIRECTION PERSISTENCE DATA*

HARRIS ON-SITE METEOROLOGICAL FACILITY

JANUARY 14, 1976 TO DECEMBER 31, 1978

STABILITY CLASS E

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	153	35	20	10	14	3						
NNE	127	51	24	4	13	5						
NE	124	31	17	5	10	4	1	1				
ENE	123	22	8	5	5	1	2					
E	99	21	9	4	9	2						
ESE	100	20	5	2	5							
SE	109	27	9	3	1							
SSE	147	31	18	10	10	3						
S	176	59	39	16	23	1	1		1			
SSW	226	66	41	22	24	11		1				
SW	235	41	22	11	13	2						
WSW	126	27	24	7	8	1						
W	116	25	10	3	3		1					
WNW	120	29	12	3	4	1						
NW	117	34	10	8	11	2	1					
NNW	127	33	16	5	6	4						
------	------	------	------	------	------	------	------	------	------	------	------	------
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.6	8.5	11.5	15.0	18.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	7	10	13	15	18	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 103

*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)

WIND DIRECTION PERSISTENCE DATA*

HARRIS ON-SITE METEOROLOGICAL FACILITY

JANUARY 14, 1976 TO DECEMBER 31, 1978

STABILITY CLASS F

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	113	28	14	6	6	1						
NNE	111	19	9	5	1							
NE	99	20	5	4	3							
ENE	68	19	10		3	1						
E	82	12	5	1	2							
ESE	97	15	3	1	1							
SE	82	10	1	4								
SSE	111	24	5	3	4							
S	121	45	14	9	5	1						
SSW	127	41	15	14	8	1	1					
SW	105	29	16	8	3	1						
WSW	90	19	6	5	3							
W	75	12	4	4								
WNW	70	14	3	1								
NW	82	7	3		1							
NNW	95	20	9	4	1							
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.4	9.0	11.0	0.0	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	7	10	11	0	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 51

*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
STABILITY CLASS G

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	198	55	28	11	11	2	1					
NNE	206	44	37	17	4							
NE	218	53	16	15	3							
ENE	200	35	27	6	2							
E	190	31	18	5	1							
ESE	165	27	8	3	2							
SE	140	17	10		3							
SSE	128	20	6	3	2							
S	150	30	12	3	3	1						
SSW	163	27	15	7	2							
SW	135	31	19	6	3							
WSW	161	15	6	4	6							
W	131	12	6	1	1	1						
WNW	126	9	4	1	2							
NW	125	16	11	3	1							
NNW	171	21	17	1	6	1						
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.7	8.6	11.0	0.0	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	7	10	11	0	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 67
*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
SUMMARY

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	396	168	101	50	84	30	14	2	3	1		
NNE	440	153	95	66	62	20	7	3	4	1	1	
NE	463	130	65	41	47	19	7	1	3			1
ENE	399	110	60	31	39	6	7		1			
E	375	89	58	25	22	11	1	1				
ESE	361	87	43	27	22	2						
SE	358	88	36	19	20	3	1					
SSE	394	99	53	35	41	10	2					
S	434	149	86	54	62	13	4	2	1			
SSW	451	182	94	74	93	31	9	5	2			2
SW	445	158	111	61	93	27	8	2				
WSW	410	123	77	47	62	20	8	2	1	1		
W	393	97	56	25	25	7		2				
WNW	369	83	61	30	38	17	6	1	2			
NW	366	90	68	43	54	8	8	2		1		
NNW	421	107	84	45	55	16	5	3	1	1		
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.7	8.8	11.7	14.9	17.7	20.8	24.0	29.6
MAXIMUM HOURS	1	2	3	4	7	10	13	16	19	22	24	30

NUMBER HOURS OF MISSING WIND DIRECTIONS: 371
*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
STABILITY CLASS A

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	38	22	5	8	3							
NNE	39	11	7	5	8	1						
NE	15	10	6	3	6							
ENE	12	6	4	2								
E	15	3		2	1							
ESE	12	4	3		1							
SE	6	2	1									
SSE	9	3	4									
S	23	6	4	1								
SSW	31	13	6	9	6							
SW	40	18	22	6	8	1						
WSW	40	17	8	7	6							
W	29	13	6	3								
WNW	37	16	9	7	8							
NW	33	14	10	8	2							
NNW	23	16	6	3	8	2						
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.4	8.2	0.0	0.0	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	7	9	0	0	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 13
*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
STABILITY CLASS B

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	39	11	4		1							
NNE	48	6	3	2	1							
NE	32	8	3	1	1							
ENE	21	5	1									
E	20	4	1									
ESE	15	4	1									
SE	9	2										
SSE	20	2		1								
S	18	7	1	1	1							
SSW	41	11	4	1	3							
SW	66	12	5	4								
WSW	47	14	3	2	2							
W	32	4	5									
WNW	49	18	6	3	3							
NW	44	9	4	3	1							
NNW	42	4	2									
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	6	0	0	0	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 4

*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)

WIND DIRECTION PERSISTENCE DATA*

HARRIS ON-SITE METEOROLOGICAL FACILITY

JANUARY 14, 1976 TO DECEMBER 31, 1978

STABILITY CLASS C

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	62	15	3				1					
NNE	53	6		3								
NE	42	6	2		1							
ENE	35	7										
E	29	4	1									
ESE	19	2	4									
SE	11	4	2									
SSE	22	6	4	1								
S	37	9	5	1	1							
SSW	84	12	2	3								
SW	80	15	3	4								
WSW	69	14	8	3	2							
W	51	10	2	1								
WNW	55	6	5	2								
NW	62	14	4									
NNW	65	10	2	1								
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.0	0.0	11.0	0.0	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	5	0	11	0	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 7

*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
STABILITY CLASS D

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	126	51	18	14	24	7	6	1				
NNE	119	54	31	12	27	7	3		1			1
NE	118	34	19	7	16	4	1	1	1		1	
ENE	107	36	25	5	8	2						
E	77	28	13	7	6	2						
ESE	79	25	12	3	3	3						
SE	89	28	6	7	5		1					
SSE	88	34	15	7	14	1	2					
S	122	49	19	11	4	1						
SSW	181	65	25	16	17		1	1				
SW	182	71	41	19	23	2		1				
WSW	161	60	22	18	12	3						
W	139	31	19	4	4							
WNW	128	44	15	5	6	2	1		1			
NW	126	33	11	20	10	4						
NNW	126	44	23	6	12	3	1		1			
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.5	8.6	11.3	14.5	17.7	0.0	24.0	34.0
MAXIMUM HOURS	1	2	3	4	7	10	13	15	19	0	24	34

NUMBER HOURS OF MISSING WIND DIRECTIONS: 101
*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
STABILITY CLASS E

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	124	34	9	12	15	4						
NNE	97	36	16	13	16	5						
NE	110	34	19	11	11	5	2	1				
ENE	74	20	13	10	9	3		1				
E	90	26	9	4	9	2						
ESE	71	19	11	7	5							
SE	80	26	5	4	3							
SSE	104	36	22	7	17	2						
S	144	51	37	21	28	5	1					
SSW	189	71	34	24	30	15	2	1				
SW	201	44	21	16	13	3						
WSW	131	43	21	5	13		1					
W	111	27	6	6	5		1					
WNW	100	32	14	10	2	1						
NW	82	26	13	6	15	2	1					
NNW	118	28	16	5	6	4						
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.6	8.6	11.7	15.3	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	7	10	13	16	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 32
*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
STABILITY CLASS F

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	79	19	13	8	2	1						
NNE	66	23	4	5	3							
NE	67	15	9	3	3							
ENE	50	18	5	3	1							
E	54	17	11	2	2	2						
ESE	64	15	7		3							
SE	53	15	1	1	2							
SSE	73	23	8	4	1							
S	88	36	11	11	10	1						
SSW	106	52	16	13	16	4						
SW	102	44	11	18	7	5	1					
WSW	100	34	10	6	7							
W	73	18	7	4	5							
WNW	68	9	9	3	1							
NW	77	9	4		1							
NNW	70	18	7	4	2							
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.6	8.3	11.0	0.0	0.0	0.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	7	9	11	0	0	0	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 16
*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
STABILITY CLASS G

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	65	19	18	7	6							
NNE	68	26	11	7	7	2						
NE	56	26	12	5	7	2						
ENE	68	22	13	4	9	1						
E	44	28	11	3	6	1						
ESE	41	25	15	4	6							
SE	68	26	11	5	2							
SSE	56	32	16	10	12					1		
S	82	36	17	16	12		1					
SSW	95	53	26	22	22	6						
SW	113	56	26	18	16	3						
WSW	69	50	37	28	27	9	2					
W	97	29	22	11	5	1						
WNW	92	31	16	8	9	1						
NW	90	32	15	11	3	1						
NNW	70	36	16	9	8	1						
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.6	8.5	11.6	0.0	0.0	20.0	0.0	0.0
MAXIMUM HOURS	1	2	3	4	7	10	12	0	0	20	0	0

NUMBER HOURS OF MISSING WIND DIRECTIONS: 38
*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978
SUMMARY

	NUMBER OF OCCURRENCES - WIND DIRECTION PERSISTENCE (HOURS)											
LOWER LEVEL WIND DIRECTION	1	2	3	4	5 - 7	8 - 10	11 - 13	14 - 16	17 - 19	20 - 22	23 - 25	> 25
N	197	113	67	50	79	24	15	5	3			1
NNE	199	123	69	36	76	37	8	6	2			1
NE	225	100	52	34	52	24	10	2	6		1	
ENE	195	84	55	23	48	12	6	2	1			
E	135	83	46	23	35	13	4	2				
ESE	169	76	55	29	33	4	3					
SE	184	83	30	28	27	6	1					
SSE	181	97	60	34	73	13	4		1	1		
S	231	137	85	60	93	15	13	3				
SSW	252	149	94	72	137	44	17	11	2	2	1	2
SW	332	155	111	66	108	47	15	4	4			
WSW	241	156	102	60	99	38	17	2	1			
W	319	122	61	33	48	9	2	2				1
WNW	248	92	66	38	52	23	10	2	2	1		
NW	249	97	54	54	62	15	5	3	2			
NNW	235	114	87	36	50	21	5	4	1			
------	------	------	------	------	------	------	------	------	------	------	------	
AVERAGE DURATION HOURS	1.0	2.0	3.0	4.0	5.7	8.7	11.6	14.8	17.7	20.7	23.5	32.2
MAXIMUM HOURS	1	2	3	4	7	10	13	16	19	22	24	37

NUMBER HOURS OF MISSING WIND DIRECTIONS: 374

*See last page of Table 2.3.2-2.

TABLE 2.3.2-2 (continued)
WIND DIRECTION PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978

*PERSISTENCE IS DEFINED AS A DELTA T EXISTING WITHIN A DEFINED WIND DIRECTION SECTOR AND IS NOT CONSIDERED TO BE INTERRUPTED IF IT DEPARTS FROM THAT DELTA T VALUE FOR UP TO 1 HOUR AND THEN RETURNS, OR IF THERE IS ONE HOUR OF MISSING DATA FOLLOWED BY A CONTINUED DELTA T VALUE. TWO OR MORE CONSECUTIVE HOURS OF LOST DATA ARE NOT INCLUDED IN THE PERSISTENCE DETERMINATION BUT ARE INDICATED AS "MISSING WIND DIRECTIONS".

TABLE 2.3.2-3
EXTREME WINDS AND PRECIPITATION ASSOCIATED WITH HURRICANES
RALEIGH-DURHAM AIRPORT
(1950-1978)

Storm	Date	Maximum Winds (mph)	Maximum Precipitation (in./hr.)	24-hr. Precipitation (in.)
Able	31 Aug. 1952	ESE 30 G 40	1.22	3.52
Barbara	13 Aug. 1953	NE 20 G 28	Trace	Trace
Carol	30 Aug. 1954	N 18	Trace	Trace
Edna	10 Sept. 1954	N 20; NNE 16 G 25	Trace	0.01
Hazel	15 Oct. 1954	WNW 43 G 90; NW 48 G 62	1.55	4.04
Connie	11-12 Aug. 1955	E 35 G 46; NE 39 G 54; N 40	0.30; 0.25	0.68; 0.75
Diane	16-17 Aug. 1955	SE 38 G 44; ENE 32 G 53	0.48; 0.70	1.23; 4.12
Ione	19 Sept. 1955	NNE 30; G 49	0.18	0.86
Flossy	26 Sept. 1956	NNE 28; G 46; NNE 29 G 41	0.37	2.31
Helene	27 Sept. 1958	N 29 G 46	Trace	0.07
Gracie	30 Sept. 1959	SSE 25 G 37	0.64	0.78
Brenda	29 July 1960	N 24	0.47	2.60
Donna	11 Sept. 1960	N 29 G 35	0.31	1.48
Esther	20 Sept. 1961	N 17	0.15	0.15
Alma	28 Aug. 1962	NW 16	Trace	Trace
Ella	18-19 Oct. 1962	NE 22 G 32	0.00	0.00
Ginny	20-21 Oct. 1963	NNE 21 G 32; N 22 G 29	0.00	0.00
Cleo	31 Aug. 1964	NNW 15	1.12	2.95
Dora	13 Sept. 1964	NNE 25 G 38	0.31	2.36
Gladys	22 Sept. 1964	N 18 G 25	0.00	0.00
Isbell	16 Oct. 1964	NE 20 G 29	0.19	0.55
Alma	11 June 1966	NNE 23 G 32	Trace	Trace
Doria	9 Sept. 1967	15 N	.15	.89
Gladys	18-20 Oct. 1968	17 ESE; 15 S: 18 N	.12; .59; .01	.63; 1.84; .01
Doria	26-28 Aug. 1971	20 E, 15 NNW, 9N	.53; .08; Trace	1.23; .23; Trace
Ginger	30 Sept. - 2 Oct. 1971	32 NNW; 29 N; 14 W	.11; .34; .08	.61; 2.64; .33
Agnes	19-21 June 1972	14 E; 20 SE; 24 N	.03; .29; .55	.03; 1.3; 1.59
Eloise	22-26 Sept. 1975	16 E; 14 SSE	.24; .32	.95; .77
		16 SSE; 10 NW	.72; .50	1.26; 1.02
		10 WNW	.30	.44
Belle	8, 9 Aug. 1976	8 ENE; 13 NNW	.09. 0.0	.18; 0.00

NOTE: "G" indicates "gusts to"

TABLE 2.3.2-4
RALEIGH-DURHAM NORMAL PRECIPITATION (in.) AND TEMPERATURE (°F)

	Temperature			Precipitation (in.)
Month	Maximum	Minimum	Average	Average
January	51.0	30.0	40.5	3.22
February	53.2	31.1	42.2	3.32
March	61.0	37.4	49.2	3.44
April	72.2	46.7	59.5	3.07
May	79.4	55.4	67.4	3.32
June	85.6	63.1	74.4	3.67
July	87.7	76.2	77.5	5.08
August	86.8	66.2	76.5	4.93
September	81.5	59.7	70.6	3.78
October	72.4	48.0	60.2	2.81
November	62.1	37.8	50.0	2.82
December	51.9	30.5	41.2	3.08
Average	70.4	47.8	59.1	3.55

Period: 1941-1970

TABLE 2.3.2-5

GREENSBORO NORMAL PRECIPITATION (in.) AND TEMPERATURE (°F)

	Temperature			Precipitation (in.)
Month	Maximum	Minimum	Average	Average
January	48.8	28.5	38.7	3.22
February	51.4	29.7	40.6	3.37
March	59.4	36.1	47.8	3.72
April	70.9	46.2	58.6	3.15
May	78.9	55.3	67.1	3.04
June	85.4	63.3	74.4	3.91
July	87.5	66.9	77.2	4.39
August	86.2	65.8	76.0	4.30
September	80.5	58.8	69.7	3.55
October	71.2	47.2	59.2	2.94
November	60.1	36.5	51.3	2.62
December	49.9	29.2	39.6	3.15
Average	69.2	47.0	58.1	41.36

Period: 1941-1970

TABLE 2.3.2-6

CHARLOTTE NORMAL PRECIPITATION (in.) AND TEMPERATURE (°F)

	Temperature			Precipitation (in.)
Month	Maximum	Minimum	Average	Average
January	52.1	32.1	42.1	3.51
February	54.9	33.1	44.0	3.83
March	62.2	39.0	50.6	4.52
April	72.7	48.9	60.8	3.40
May	80.2	57.4	68.8	2.90
June	86.4	65.3	75.9	3.70
July	88.3	68.7	78.5	4.57
August	87.4	67.9	77.7	3.96
September	82.0	61.9	72.0	3.46
October	73.1	50.3	61.7	2.69
November	62.4	39.6	51.0	2.74
December	52.5	32.4	42.5	3.44
Average	71.2	49.7	60.5	42.72

Period: 1941-1970

TABLE 2.3.2-7

MONCURE NORMAL PRECIPITATION (in.) AND TEMPERATURE (°F)

	Temperature			Precipitation (in.)
Month	Maximum°	Minimum°	Average°	Average*
January	51.7	25.0	38.4	3.46
February	54.0	26.9	40.5	3.71
March	61.7	34.3	48.0	4.03
April	73.2	43.6	58.4	3.53
May	80.7	52.4	66.6	3.88
June	86.9	60.4	73.7	3.89
July	90.1	65.0	77.6	6.73
August	88.7	64.4	76.6	5.48
September	83.4	57.3	70.4	4.43
October	73.7	44.7	59.2	3.21
November	64.0	33.6	48.8	3.15
December	54.2	26.8	40.5	3.40
Average	71.9	44.5	58.2	48.90

* = 1941-70

° - 1951-73

TABLE 2.3.2-8

PINEHURST NORMAL PRECIPITATION (in.) AND TEMPERATURE (°F)

	Temperature			Precipitation (in.)
Month	Maximum^O	Minimum^O	Average^O	Average*
January	52.3	31.2	41.8	3.44
February	55.5	32.8	44.1	4.00
March	63.3	39.2	51.3	4.21
April	74.9	48.6	61.8	3.67
May	82.0	56.4	69.2	3.60
June	87.7	63.7	75.7	4.80
July	90.2	67.4	78.8	6.85
August	89.2	66.7	78.0	5.60
September	83.9	60.5	72.2	4.10
October	74.0	49.5	61.8	3.33
November	63.3	39.9	51.6	3.02
December	54.3	33.3	43.8	3.32
Average	72.6	49.1	60.8	49.94

* = 1941-70

O = 1951-73

TABLE 2.3.2-9

ASHEBORO NORMAL PRECIPITATION (in.) AND TEMPERATURE (F)

	Temperature			Precipitation (in.)
Month	Maximum°	Minimum°	Average°	Average*
January	50.9	30.7	40.8	3.41
February	54.0	32.3	43.1	3.64
March	61.8	38.8	50.3	3.90
April	72.8	48.4	60.6	3.44
May	79.4	55.8	67.6	3.53
June	85.5	63.0	74.3	3.74
July	88.4	66.8	77.7	5.58
August	87.3	66.1	76.8	4.88
September	82.0	60.1	71.1	3.84
October	72.2	49.1	60.7	3.05
November	62.3	40.0	51.2	2.75
December	53.0	33.2	43.1	3.20
Average	70.8	48.7	59.8	44.87

O = 1951-73

* = 1941-70

TABLE 2.3.2-10

HARRIS ON-SITE DATA
MEAN TEMPERATURE (JANUARY 1976 - DECEMBER 1978)

Month	1976	1977	1978	Avg.
January	40.1*	29.3	34.3	34.6
February	51.2	41.7	33.8	42.2
March	55.7	55.5	48.0	53.1
April	59.0	63.0	59.4	60.5
May	66.5	68.4	65.2	66.7
June	73.1	73.7	73.7	73.5
July	77.9	81.3	76.2	78.5
August	75.5	77.6	77.3	76.8
September	69.9	72.4	72.1	71.5
October	55.2	56.6	57.2	56.3
November	43.5	52.7	54.0	50.1
December	39.6	41.1	43.8	41.5
Annual	58.9	59.4	57.9	58.7

*Data collection began on January 14th.

TABLE 2.3.2-11

HARRIS ONSITE DATA
MAXIMUM-MINIMUM TEMPERATURES (JANUARY 1976 - DECEMBER 1978)

	1976		1977		1978		Average	
Month	Max	Min	Max	Min	Max	Min	Max	Min
January	52.0*	27.8*	38.6	19.6	44.7	25.3	45.1	24.2
February	65.0	36.7	54.5	28.2	43.8	24.4	54.3	29.8
March	68.5	42.3	67.0	43.6	58.5	37.5	64.7	41.1
April	74.0	43.2	76.0	49.1	71.6	47.0	73.9	46.4
May	78.8	54.5	80.8	56.8	76.0	53.6	78.5	55.0
June	83.6	63.9	84.7	59.7	84.5	63.4	84.3	62.3
July	89.6	67.1	93.0	69.6	86.8	66.3	89.8	67.7
August	86.3	64.4	91.0	68.5	87.8	69.0	88.4	67.3
September	82.2	57.9	83.4	62.9	83.2	62.9	82.9	61.2
October	67.6	43.2	67.7	45.8	71.0	44.2	68.8	44.4
November	56.5	30.8	62.7	42.9	63.6	44.8	60.9	39.5
December	50.4	28.1	50.7	31.1	55.6	31.6	52.2	30.3
Annual	71.2	46.7	70.8	48.2	68.9	47.5	70.3	47.5

*Data collection began January 14th.

TABLE 2.3.2-12

SITE REGION METEOROLOGICAL EXTREMES
(month/year of occurrence) [Data period]

	Charlotte	Greensboro	Raleigh-Durham	Pinehurst	Asheboro	Moncure
Maximum Monthly Precipitation (water equivalent)	12.48 in. (5/75) [1940-77]	13.26 in. (9/47) [1929-77]	12.94 in. (9/45) [1945-77]	13.88 in. (7/59) [1951-73]	13.79 in. (7/65) [1951-73]	12.55 in. (7/73) [1951-73]
Maximum 24-hour Precipitation (water equivalent)	5.34 in. (10/76) [1940-77]	7.49 in. (9/47) [1929-77]	5.20 in. (8/55) [1945-77]	7.11 in. (10/54) [1951-73]	8.96 in. (8/66) [1951-73]	5.14 in. (8/67) [1951-73]
Minimum Monthly Precipitation (water equivalent)	Trace (10/53) [1940-77]	.13 in. (9/39) [1929-77]	.23 in. (4/76) [1945-77]	----	----	----
Maximum Monthly Snowfall (inches)	19.3 in. (3/60) [1940-77]	22.9 in. (1/66) [1929-70]	14.4 in. (1/55) [1945-77]	16.0 in. (12/58) [1951-73]	18.5 in. (3/60) [1951-73]	14.0 in. (3/60) [1951-73]
Maximum 24-hour Snowfall (inches)	12.0 in. (2/69) [1940-77]	14.3 in. (12/30) [1929-70]	9.3 in. (3/69) [1945-77]	----	----	----
Maximum Temperature (°F)	104°F (9/54) [1940-77]	102°F (7/77) [1929-70]	105°F (7/52) [1945-77]	106°F (8/54) [1951-73]	103°F* (7/52) [1951-73]	107°F* (7/52) [1951-73]
Minimum Temperature (°F)	-5°F (1/85) [1940-85]	-8°F (1/85) [1929-85]	-9°F (1/85) [1945-85]	+3°F* (12/62) [1951-73]	-8°F (1/85) [1951-85]	-4°F (1/66) [1951-73]

*On earlier dates

TABLE 2.3.2-13

DEWPOINT TEMPERATURES (F) AND ABSOLUTE HUMIDITY (g/m^3)

	RALEIGH-DURHAM		CHARLOTTE		GREENSBORO	
Month	Dewpoint	Absolute Humidity	Dewpoint	Absolute Humidity	Dewpoint	Absolute Humidity
January	32	4.85	32	4.85	29	4.25
February	31	6.64	32	4.85	29	4.25
March	35	5.43	36	5.64	34	5.23
April	45	7.86	46	8.27	44	7.58
May	56	11.58	56	11.58	55	11.18
June	64	15.16	64	15.16	63	14.66
July	68	17.28	67	16.73	67	16.73
August	67	16.73	67	16.73	66	16.18
September	61	13.71	61	13.71	60	13.26
October	50	9.40	50	9.40	48	8.75
November	38	6.08	39	6.31	37	5.86
December	30	4.44	32	4.85	29	4.25
Annual	48	8.75	49	9.07	47	8.45

TABLE 2.3.2-14

HARRIS ON-SITE DATA

DEWPOINT TEMPERATURE (10 METER LEVEL)

Month	1976	1977	1978	Average
January	*25.6	18.8	21.7	22.0
February	32.4	25.6	21.0	26.3
March	40.5	41.2	34.2	38.6
April	39.2	50.0	43.2	44.1
May	53.6	58.5	55.7	55.9
June	65.1	64.5	63.7	64.4
July	67.2	69.0	66.6	67.6
August	65.5	72.2	68.6	68.8
September	60.7	65.5	64.0	63.4
October	46.8	48.8	46.4	47.3
November	32.2	41.3	45.5	39.7
December	30.0	30.9	30.7	30.5
Annual	46.6	48.9	46.8	47.4

*Data collection began on January 14th.

TABLE 2.3.2-15

CHARLOTTE RELATIVE HUMIDITY (PERCENT)

Month	1:00 a.m.	7:00 a.m.	1:00 p.m.	7:00 p.m.
January	72	78	57	62
February	67	75	51	54
March	69	79	50	52
April	68	79	47	49
May	78	84	53	59
June	81	86	57	63
July	83	88	58	67
August	84	89	59	67
September	84	90	57	68
October	80	88	53	66
November	75	83	52	62
December	74	80	57	63
Annual	76	83	54	61

TABLE 2.3.2-16

GREENSBORO RELATIVE HUMIDITY (PERCENT)

Month	1:00 a.m.	7:00 a.m.	1:00 p.m.	7:00 p.m.
January	76	81	58	65
February	70	77	51	57
March	71	80	51	55
April	72	79	48	52
May	83	85	56	63
June	87	87	57	67
July	89	90	60	70
August	91	92	61	73
September	88	91	59	72
October	85	89	55	73
November	77	82	51	65
December	78	81	58	68
Annual	81	84	55	65

TABLE 2.3.2-17

RALEIGH-DURHAM RELATIVE HUMIDITY (PERCENT)

Month	1:00 a.m.	7:00 a.m.	1:00 p.m.	7:00 p.m.
January	72	78	55	63
February	67	74	48	55
March	70	79	48	55
April	73	80	44	53
May	85	87	55	67
June	87	88	57	69
July	88	90	59	72
August	90	93	61	76
September	88	93	59	78
October	85	90	54	76
November	77	83	49	65
December	75	80	56	68
Annual	80	84	54	66

TABLE 2.3.2-18

HARRIS ONSITE DATA
PRECIPITATION (in.) (JANUARY 1976 - DECEMBER 1978)

Month	1976	1977	1978	Average
January	1.29*	2.65	7.42	3.79
February	1.15	1.57	1.74	1.49
March	4.69	6.18	3.85	4.91
April	0.43	2.17	4.36	2.32
May	2.72	1.87	3.59	2.73
June	2.74	0.77	5.08	2.86
July	1.66	1.92	4.63	2.74
August	1.76	3.78	3.47	3.00
September	2.87	6.16	2.72	3.92
October	1.26	4.17	0.91	2.11
November	1.14	2.35	3.57	2.35
December	3.66	3.08	2.85	3.20
Annual	25.37	36.67	44.19	35.41

*Data collection began on January 14th.

TABLE 2.3.2-19

SHNPP ON-SITE EXTREME RAINFALL RATES

Hours	Amount (in.)	Date
1	1.92	3/21/76
2	3.69	3/21/76
3	4.00	3/21/76
6	4.41	3/21/76
12	4.41	3/21/76
24	4.41	3/21/76

TABLE 2.3.2-20

PRECIPITATION FREQUENCIES AND AMOUNTS
(1951-1960: January, April, July, October)

Station	(a) Percent of Days with Measurable Precipitation, ≥ 0.01 in.				(b) Average No. of Hourly Reports per Month with ≥ 0.01 in. in Preceding Hour				(c) Percent of the Hourly Reports with Precipitation in the Preceding Hour which were only Trace, <0.005 in.			
	J	A	J	O	J	A	J	O	J	A	J	O
Raleigh	29	31	35	22	53	50	37	41	52	46	52	57
Greensboro	32	35	37	26	59	54	33	44	52	48	54	59
Charlotte	29	34	37	23	59	51	34	39	46	43	52	55

Station	(d) Average Precipitation (in.) during Days Having ≥ 0.01 in.				(e) Average (in.) during Hours Having ≥ 0.01 in.				(f) Ratio of (b) to No. of Rainy (≥ 0.01 in.) Days per Month			
	J	A	J	O	J	A	J	O	J	A	J	O
Raleigh	.38	.42	.49	.42	.06	.08	.15	.07	5.9	5.4	3.4	6.3
Greensboro	.35	.38	.37	.39	.06	.07	.13	.07	5.8	5.2	2.9	5.6
Charlotte	.38	.40	.33	.33	.06	.08	.11	.06	6.6	5.0	2.9	5.4

TABLE 2.3.2-21

SHNPP ONSITE HOURLY PRECIPITATION OCCURRENCE*

Month	1976	1977	1978	1976-1978	Mean
January	10**	44	89	143	48
February	2	17	50	69	23
March	11	71	60	142	47
April	2	28	55	85	28
May	32	37	35	104	35
June	38	20	40	98	33
July	24	8	35	67	22
August	24	30	41	95	32
September	30	30	18	78	26
October	27	61	12	100	33
November	30	39	63	132	44
December	93	59	41	193	64
TOTAL	323	444	539	1306	435

* Number of hours with measurable precipitation.

** Data period began January 14th.

TABLE 2.3.2-22

SHNPP ONSITE FREQUENCY OF PASQUILL STABILITY CATEGORIES (PERCENT)

	A	B	C	D	E	F	G
1976	8.5	4.8	5.2	24.3	23.7	12.6	21.0
1977	6.8	4.9	7.0	27.6	22.5	12.9	18.4
1978	4.5	3.2	4.1	29.0	26.6	12.7	19.9
1976-1978	6.5	4.3	5.4	27.0	24.3	12.7	19.8

TABLE 2.3.2-23

STABILITY CLASS PERSISTENCE DATA*
HARRIS ON-SITE METEOROLOGICAL FACILITY
JANUARY 14, 1976 TO DECEMBER 31, 1978

NUMBER OF OCCURRENCES - PASQUILL CATEGORIES AND DELTA T IN DEGREES C/100 METERS

STABILITY CLASS PERSISTENCE PERIOD (HOURS)	A	B	C	D	E	F	G
	=<-1.9	-1.9 TO -1.7	-1.7 TO -1.5	-1.5 TO -0.5	-0.5 TO 1.5	1.5 TO 4.0	>4.0
1	67	358	452	568	707	504	21
2	57	111	146	332	256	204	90
3	55	55	68	180	143	131	59
4	63	34	41	108	103	87	53
5	58	16	28	80	77	79	44
6	68	12	9	53	73	53	41
7	27	4	1	51	55	27	28
8	19	1	1	51	47	21	15
9	5		1	58	33	17	35
10	1			45	37	9	26
11				20	27	6	37
12				19	23	11	58
13 - 15				23	48	5	129
16 - 19				17	19		5
20 - 24				17	7		
25 - 30				7	2		
31 - 36				7	2		
37 - 48				7	1		
49 - 72				4			
> 72							
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AVERAGE DURATION HOURS	4.0	1.8	1.7	4.2	3.7	2.7	7.6
MAXIMUM HOURS	10	8	15	62	39	15	17

TABLE 2.3.2-24

SEASONAL FREQUENCY OF PLUME LENGTHS (HOURS PER YEAR)

Wind Direction/Distance from Site (Km)		N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	CALM	*TOTAL
Winter	1	35	29	17	14	11	15	15	10	26	30	22	18	22	11	21	27	0	323
	2	4	4	3	2	1	2	2	1	3	8	4	8	8	4	5	4	0	63
	3	1	0	1	0	0	0	0	0	0	1	0	0	1	0	0	0	0	4
Spring	1	10	10	12	6	7	9	8	8	12	12	12	5	3	5	7	6	0	132
	2	2	1	1	0	0	0	1	1	1	1	2	0	0	0	1	0	0	10
	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Summer	1	12	7	9	5	8	11	7	5	13	12	10	10	5	7	2	3	0	126
	2	0	0	1	0	1	2	1	0	1	2	2	1	1	1	0	0	0	13
	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Fall	1	35	17	18	10	15	9	11	12	25	20	27	8	12	14	15	17	0	265
	2	5	2	1	1	2	1	1	3	3	2	5	1	4	3	2	2	0	38
	3	0	0	0	0	0	0	0	1	0	0	1	0	0	0	0	0	0	1
Annual	1	92	63	56	35	42	44	40	35	76	74	71	41	42	36	46	52	0	845

*Observations of calm winds were assigned with wind direction reported during the previous hour.

TABLE 2.3.2-25

JOINT OCCURRENCE FREQUENCIES FOR 10M WIND
DIRECTION AND 10M WIND SPEED BY PRECIPITATION RATE
RANGES INCLUDE LOWER END POINT, EXCLUDE UPPER END POINT

SITE=SHNP YEAR =76-78 PRECIP. RATE = .01-.20 in/hr.

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDSPD
N	0.2/ 0.00	13/ 0.05	56/ 0.24	20/ 0.08	1/ 0.00	/	/	90.2/ 0.38	5.986696
NNE	0.2/ 0.00	16/ 0.07	69/ 0.29	11/ 0.05	/	/	/	96.2/ 0.41	5.191354
NE	0.4/ 0.00	26/ 0.11	46/ 0.19	18/ 0.08	/	/	/	90.4/ 0.38	5.081858
ENE	0.2/ 0.00	12/ 0.05	27/ 0.11	6/ 0.03	/	/	/	45.2/ 0.19	5.027101
E	0.2/ 0.00	11/ 0.05	22/ 0.09	3/ 0.01	2/ 0.01	/	/	38.2/ 0.16	5.03992
ESE	0.2/ 0.00	16/ 0.07	9/ 0.04	/	/	/	/	25.2/ 0.11	3.472222
SE	0.1/ 0.00	8/ 0.03	11/ 0.05	2/ 0.01	/	/	/	21.1/0.09	4.077013
SSE	0.2/ 0.00	14/ 0.06	14/ 0.06	6/ 0.03	2/ 0.01	/	/	36.2/ 0.15	4.828038
S	0.2/ 0.00	16/ 0.07	18/ 0.08	11/ 0.05	3/ 0.01	1/ 0.00	/	49.2/0.21	6.136347
SSW	0.3/ 0.00	19/ 0.08	29/ 0.12	15/ 0.06	4/ 0.02	/	/	67.3/ 0.28	6.001733
SW	0.2/ 0.00	11/ 0.05	19/ 0.08	12/ 0.05	7/ 0.03	1/ 0.00	/	50.2/ 0.21	7.065238
WSW	0.2/ 0.00	11/ 0.05	14/ 0.06	6/ 0.03	2/ 0.01	11/ 0.05	/	44.2/ 0.19	9.602372
W	0.1/ 0.00	6/ 0.03	6/ 0.03	8/ 0.03	2/ 0.01	1/ 0.0	/	23.1/0.10	7.087661
WNW	0.1/ 0.00	8/ 0.03	8/ 0.03	7/ 0.03	1/ 0.00	2/ 0.01	/	26.1/ 0.11	7.080458
NW	0.1/ 0.00	5/ 0.02	12/ 0.05	11/ 0.05	2/ 0.01	/	/	30.1/ 0.13	6.870985
NNW	0.2/ 0.00	15/ 0.06	30/ 0.13	12/ 0.05	6/ 0.03	/	/	63.2/ 0.27	6.253559
TOTAL	3/ 0.01	207/ 0.87	390/ 1.65	148/ 0.62	32/ 0.14	16/ 0.07	0/ 0.00	796/ 3.4	5.907474

NUMBER OF BAD RECORDS: 27

TABLE 2.3.2-25 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = .21-.40 in/hr.

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDSPD
N	/	/	5/ 0.02	2/ 0.01	/	/	/	7.0/ 0.03	7.235713
NNE	/	/	5/ 0.02	2/ 0.01	/	/	/	7.0/ 0.03	6.185713
NE	/	1/ 0.00	3/ 0.01	2/ 0.01	/	/	/	6.0/ 0.03	6.383333
ENE	/	1/ 0.00	4/ 0.02	5/ 0.02	/	/	/	10.0/ 0.04	6.992499
E	/	1/ 0.00	3/ 0.01	1/ 0.00	/	/	/	5.0/ 0.02	5.784999
ESE	/	/	1/ 0.00	/	/	/	/	1.0/ 0.00	4.325
SE	/	2/ 0.01	2/0.01	1/ 0.00	/	/	/	5.0/ 0.02	4.794999
SSE	/	/	4/ 0.02	/	/	/	/	4.0/ 0.02	5.937499
S	/	1/ 0.00	2/ 0.01	/	/	/	/	3.0/ 0.01	4.108333
SSW	/	1/ 0.00	2/ 0.01	2/ 0.01	/	/	/	5.0/ 0.02	6.904999
SW	/	1/ 0.00	3/ 0.01	1/ 0.00	2/ 0.01	1/ 0.00	/	8.0/ 0.03	10.425
WSW	/	/	4/ 0.02	2/ 0.01	/	2/ 0.01	1/ 0.00	9.0/ 0.04	12.88333
W	/	/	1/ 0.00	1/ 0.00	/	/	/	2.0/ 0.01	7.599999
WNW	/	/	3/ 0.01	/	/	/	/	3.0/ 0.01	5.058333
NW	/	1/ 0.00	1/ 0.00	1/ 0.00	/	/	/	3.0/ 0.01	5.199999
NNW	/	1/ 0.00	1/ 0.00	1/ 0.00	/	/	/	3.0/ 0.01	6.041666
TOTAL	0/ 0.00	10/ 0.04	44/ 0.19	21/ 0.09	2/ 0.01	3/ 0.01	1/ 0.00	81/ 0.3	7.326542

NUMBER OF BAD RECORDS: 2

TABLE 2.3.2-25 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = .41-.60 in/hr.

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDSPD
N	/	1/ 0.00	2/ 0.01	/	/	/	/	3.0/ 0.01	5.166666
NNE	/	/	2/ 0.01	1/ 0.00	/	/	/	3.0/ 0.01	7.616666
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	2/ 0.01	/	/	/	/	2.0/ 0.01	5.924999
E	/	/	1/ 0.00	/	/	/	/	1.0/ 0.00	4.174999
ESE	/	/	/	1/ 0.00	/	/	/	1.0/ 0.00	9.499999
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	3/ 0.01	/	1/ 0.00	/	/	4.0/ 0.02	7.074999
SSW	/	/	/	2/ 0.01	/	/	/	2.0/ 0.01	10.95
SW	/	/	/	/	/	/	/	0.0/ 0.00	
WSW	/	/	/	/	/	/	/	0.0/ 0.00	
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	1/ 0.00	/	/	/	/	1.0/ 0.00	6.449999
NW	/	/	2/ 0.01	/	/	/	/	2.0/ 0.01	5.112499
NNW	/	/	1/ 0.00	/	/	/	/	1.0/ 0.00	5.899999
TOTAL	0/ 0.00	1/ 0.00	14/ 0.06	4/ 0.02	1/ 0.00	0/ 0.00	0/ 0.00	20/ 0.1	6.832499

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-25 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = .61-.80 in/hr.

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	1/ 0.00	/	/	/	/	1.0/ 0.00	6.4
SSW	/	1/ 0.00	/	/	/	/	/	1.0/ 0.00	3.499999
SW	/	/	/	/	/	/	/	0.0/ 0.00	
WSW	/	/	2/ 0.01	/	/	/	/	2.0/ 0.01	6.399999
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	/		/	/	0.0/ 0.00	
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	1/ 0.00	3/ 0.01	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	4/ 0.0	5.674999

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-25 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = .81-1.0 in/hr.

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	1/ 0.00	/	/	/	1.0/ 0.00	9.674999
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	/	/	/	/	0.0/ 0.00	
WSW	/	/	/	/	/	/	/	0.0/ 0.00	
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	/	/	/	/	0.0/ 0.00	
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.0	9.674999

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-25 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = 1.01-1.2 in/hr.

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	/	1/ 0.00	/	/	1.0/ 0.00	12.55
WSW	/	/	/	/	/	/	/	0.0/ 0.00	
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	1/ 0.00	/	/	/	1.0/ 0.00	10.9
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.00	1/ 0.00	0/ 0.00	0/ 0.00	2/ 0.0	11.725

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-25 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = 1.21-1.4 in/hr.

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	1/ 0.00	/	/	/	/	1.0/ 0.00	6.775
SW	/	/	/	/	/	/	/	0.0/ 0.00	
WSW	/	/	/	1/ 0.00	/	/	/	1.0/ 0.00	8.366666
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	1/ 0.00	/	/	/	/	1.0/ 0.00	5.775
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	2/ 0.01	1/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	3/ 0.0	6.972222

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-25 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = 1.61-1.8 in/hr.

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	/	/	1/ 0.00	/	1.0/ 0.00	22.09999
WSW	/	/	/	/	/	/	/	0.0/ 0.00	
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	/	/	/	/	0.0/ 0.00	
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.00	0/ 0.00	1/ 0.0	22.09999

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-25 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = 1.81-2.0 in/hr.

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	/	/	1/ 0.00	/	1.0/ 0.00	22.17499
WSW	/	/	/	/	/	/	/	0.0/ 0.00	
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	/	/	/	/	0.0/ 0.00	
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.00	0/ 0.00	1/ 0.0	22.17499

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-26
JOINT OCCURRENCE FREQUENCIES FOR 60M WIND
DIRECTION AND 60M WIND SPEED BY PRECIPITATION RATE
RANGES INCLUDE LOWER END POINT, EXCLUDE UPPER END POINT
SITE=SHNP YEAR =76-78 PRECIP. RATE = .01-.20 in/hr.

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.00	21/ 0.09	38/ 0.16	20/ 0.09	2/ 0.01	/	82.0/ 0.35	10.44248
NNE	/	2/ 0.01	18/ 0.08	46/ 0.20	14/ 0.06	/	/	80.0/ 0.34	9.805207
NE	/	4/ 0.02	18/ 0.08	44/ 0.19	21/ 0.09	/	/	87.0/ 0.37	0.886972
ENE	/	5/ 0.02	19/ 0.08	21/ 0.09	6/ 0.03	1/ 0.00	/	52.0/ 0.22	8.260095
E	/	7/ 0.03	12/ 0.05	12/ 0.05	4/ 0.02	/	/	35.0/ 0.15	7.205714
ESE	/	2/ 0.01	10/ 0.04	8/ 0.03	3/ 0.01	/	/	23.0/ 0.10	8.104346
SE	/	1/ 0.00	9/ 0.04	8/ 0.03	/	1/ 0.00	/	19.0/ 0.08	7.976314
SSE	/	1/ 0.00	12/ 0.05	14/ 0.06	16/ 0.07	3/ 0.01	1/ 0.00	47.0/ 0.20	11.43138
S	/	4/ 0.02	11/ 0.05	20/ 0.09	11/ 0.05	6/ 0.03	1/ 0.00	53.0/ 0.23	11.55314
SSW	/	2/ 0.01	10/ 0.04	25/ 0.11	16/ 0.07	10/ 0.04	1/ 0.00	64.0/ 0.27	12.41849
SW	/	4/ 0.02	14/ 0.06	20/ 0.09	11/ 0.05	5/ 0.02	5/ 0.02	59.0/ 0.25	11.875
WSW	/	3/ 0.01	8/ 0.03	14/ 0.06	6/ 0.03	2/ 0.01	7/ 0.03	40.0/ 0.17	13.9175
W	/	2/ 0.01	5/ 0.02	12/ 0.05	3/ 0.01	3/ 0.01	1/ 0.00	26.0/ 0.11	11.24519
WNW	/	2/ 0.01	5/ 0.02	5/ 0.02	5/ 0.02	3/ 0.01	2/ 0.01	22.0/ 0.09	12.47878
NW	/	2/ 0.01	5/ 0.02	11/ 0.05	5/ 0.02	/	/	23.0/ 0.10	9.380433
NNW	/	3/ 0.01	13/ 0.06	15/ 0.06	11/ 0.05	5/ 0.02	/	47.0/ 0.20	10.36702
TOTAL	0/ 0.00	45/ 0.19	190/ 0.81	313/ 1.33	152/ 0.65	41/ 0.17	18/ 0.08	759/ 3.2	10.52984

NUMBER OF BAD RECORDS: 64

TABLE 2.3.2-26 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = .21-.40 in/hr.

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	4/ 0.02	/	1/ 0.00	/	5.0/ 0.02	11.865
NNE	/	/	1/ 0.00	3/ 0.01	4/ 0.02	/	/	8.0/ 0.03	11.64062
NE	/	/	1/ 0.00	1/ 0.00	2/ 0.01	/	/	4.0/ 0.02	12.25
ENE	/	/	/	3/ 0.01	6/ 0.03	1/ 0.00	/	10.0/ 0.04	14.54249
E	/	/	2/ 0.01	1/ 0.00	1/ 0.00	/	/	4.0/ 0.02	8.36875
ESE	/	/	2/ 0.01	/	/	/	/	2.0/ 0.01	5.625
SE	/	/	/	3/ 0.01	1/ 0.00	/	/	4.0/ 0.02	11.95625
SSE	/	/	1/ 0.00	2/ 0.01	/	/	/	3.0/ 0.01	8.583332
S	/	/	2/ 0.01	3/ 0.01	/	/	/	5.0/ 0.02	8.594999
SSW	/	/	3/ 0.01	1/ 0.00	/	1/ 0.00	/	5.0/ 0.02	10.39
SW	/	/	/	3/ 0.01	4/ 0.02	/	2/ 0.01	9.0/ 0.04	15.70555
WSW	/	/	1/ 0.00	1/ 0.00	1/ 0.00	1/ 0.00	4/ 0.02	8.0/ 0.03	23.75624
W	/	/	/	3/ 0.01	1/ 0.00	/	/	4.0/ 0.02	10.63125
WNW	/	/	1/ 0.000	2/ 0.01	/	/	/	3.0/ 0.01	7.283333
NW	/	/	/	/	1/ 0.00	/	/	1.0/ 0.00	15.3
NNW	/	/	/	1/ 0.00	1/ 0.00	/	/	2.0/ 0.01	13.15
TOTAL	0/ 0.00	0/ 0.00	14/ 0.06	31/ 0.13	22/ 0.09	4/ 0.02	6/ 0.03	77/ 0.3	12.95422

NUMBER OF BAD RECORDS: 6

TABLE 2.3.2-26 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = .41-.60 in/hr.

LOWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	1/ 0.00	/	/	/	1.0/ 0.00	12.375
NNE	/	/	1/ 0.00	/	1/ 0.00	1/ 0.00	/	3.0/ 0.01	12.81666
NE	/	/	/	/	1/ 0.00	/	/	1.0/ 0.00	14.45
ENE	/	/	/	1/ 0.00	1/ 0.00	/	/	2.0/ 0.01	12.575
E	/	/	1/ 0.00	/	/	/	/	1.0/ 0.00	7.424999
ESE	/	/	1/ 0.00	/	/	/	/	1.0/ 0.00	6.699999
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	1/ 0.00	/	/	/	1.0/ 0.00	8.449999
S	/	/	1/ 0.00	/	1/ 0.00	1/ 0.00	/	3.0/ 0.01	12.79999
SSW	/	/	/	/	/	2/ 0.01	/	2.0/ 0.01	22.78748
SW	/	/	/	/	/	/	/	0.0/ 0.00	
WSW	/	/	/	/	/	/	/	0.0/ 0.00	
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	2/ 0.01	/	/	/	2.0/ 0.01	11.2375
NW	/	1/ 0.00	/	/	/	/	/	1.0/ 0.00	3.349999
NNW	/	/	1/ 0.00	1/ 0.00	/	/	/	2.0/ 0.01	8.549999
TOTAL	0/ 0.00	1/ 0.00	5/ 0.02	6/ 0.03	4/ 0.02	4/ 0.02	0/ 0.00	20/ 0.1	11.995

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-26 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = .61-.80 in/hr.

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	1/ 0.00	1/ 0.00	/	/	/	2.0/ 0.01	9.349999
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	/	/	/	/	0.0/ 0.00	
WSW	/	/	/	1/ 0.00	1/ 0.00	/	/	2.0/ 0.01	12.0625
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	/	/	/	/	0.0/ 0.00	
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	1/ 0.00	2/ 0.01	1/ 0.00	0/ 0.00	0/ 0.00	4/ 0.0	10.70625

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-26 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = .81-1.0 in/hr.

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	1/ 0.00	/	/	/	1.0/ 0.00	11.35
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	/	/	/	/	0.0/ 0.00	
WSW	/	/	/	/	/	/	/	0.0/ 0.00	
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	/	/	/	/	0.0/ 0.00	
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.0	11.35

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-26 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = 1.01-1.2 in/hr.

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	/	/	1/ 0.00	/	1.0/ 0.00	22.17499
WSW	/	/	/	/	/	/	/	0.0/ 0.00	
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	/	1/ 0.00	/	/	1.0/ 0.00	17.09999
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.00	1/ 0.00	0/ 0.00	2/ 0.0	19.63749

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-26 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = 1.21-1.4 in/hr.

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	1/ 0.00	/	/	/	1.0/ 0.00	10.825
WSW	/	/	/	/	1. 0.00	/	/	1.0/ 0.00	13.66666
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	1/ 0.00	/	/	/	1.0/ 0.00	9.849999
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	0/ 0.00	2/ 0.01	1/ 0.00	0/ 0.00	0/ 0.00	3/ 0.0	11.44722

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-26 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = 1.61-1.8 in/hr.

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	/	/	/	/	0.0/ 0.00	
WSW	/	/	/	/	/	/	1/ 0.00	1.0/ 0.00	33.64999
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	/	/	/	/	0.0/ 0.00	
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.00	1/ 0.0	33.64999

NUMBER OF BAD RECORDS: 0

TABLE 2.3.2-26 (Cont'd)

SITE=SHNP YEAR =76-78 PRECIP. RATE = 1.81-2.0 in/hr.

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	/	/	/	/	0.0/ 0.00	
NNE	/	/	/	/	/	/	/	0.0/ 0.00	
NE	/	/	/	/	/	/	/	0.0/ 0.00	
ENE	/	/	/	/	/	/	/	0.0/ 0.00	
E	/	/	/	/	/	/	/	0.0/ 0.00	
ESE	/	/	/	/	/	/	/	0.0/ 0.00	
SE	/	/	/	/	/	/	/	0.0/ 0.00	
SSE	/	/	/	/	/	/	/	0.0/ 0.00	
S	/	/	/	/	/	/	/	0.0/ 0.00	
SSW	/	/	/	/	/	/	/	0.0/ 0.00	
SW	/	/	/	/	/	/	/	0.0/ 0.00	
WSW	/	/	/	/	/	/	1/ 0.00	1.0/ 0.00	33.47499
W	/	/	/	/	/	/	/	0.0/ 0.00	
WNW	/	/	/	/	/	/	/	0.0/ 0.00	
NW	/	/	/	/	/	/	/	0.0/ 0.00	
NNW	/	/	/	/	/	/	/	0.0/ 0.00	
TOTAL	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	0/ 0.00	1/ 0.00	1/ 0.0	33.47499

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-3

SHNPP OPERATIONAL SENSOR ELEVATIONS

SENSORS	OPERATIONAL ELEVATIONS ABOVE TOWER BASE (METERS)
Wind	12.5 and 61.4
Relative Humidity	11.0
Solar Radiation	1.5
Ambient Temperature (2 each level)*	11.0 to 59.9
Precipitation	1.5
Barometric Pressure	1.5

* Used to calculate two differential temperature channels between these elevations.

TABLE 2.3.3-4

CHANNEL ACCURACY

The HNP meteorological channels satisfy the criteria listed below, with minor exceptions as noted. Channels based on Regulatory Guide 1.23, Rev. 0 and Regulatory Guide 1.97, Rev. 3 are base HNP commitments, while the ANSI/ANS-2.5-1984 guidance reflects industry and regulator-accepted state-of-the-art specifications.

Parameter:	Basis:	Criteria:
Wind Sensor:		
Wind Direction:	RG 1.97, Rev. 3	Range: 0 to 360° ± 5° accuracy with a deflection of 10°; Starting speed less than 1 mph; Damping ratio greater than or equal to 0.4; Delay distance less than or equal to 2 meters
Wind Speed:	RG 1.97, Rev. 3	Range: 0 to 50 mph ± 0.5 mph for speeds less than 5 mph. ± 10 percent for speeds in excess of 5 mph. Starting threshold of less than 1.0 mph. Distance constant not to exceed 2 meters (HNP Exception: Distance constant for installed instrument is 2.1 meters)
Ambient Temperature:	RG 1.23, Rev. 0	± 0.5°C for time averaged values
Differential Temperature:	RG 1.97, Rev. 3	± 0.15°C per 50 meter interval
Relative Humidity:	ANSI/ANS-2.5-1984 (invoked in lieu of original RG 1.23, Rev. 0 Dew Point basis)	Equivalent to Dew Point Accuracy of 1.5°C where relative humidity is in excess of 60% and temperature is between -30°C and +30°C.
Total Precipitation:	ANSI/ANS-2.5-1984	Resolution of 0.01 inch; ±10% of total for amounts > 0.2 inches
Solar Radiation:	ANSI/ANS-2.5-1984	Consistent with current state-of-the-art.
Barometric Pressure:	ANSI/ANS-2.5-1984	Consistent with current state-of-the-art.

TABLE 2.3.3-6

JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED

FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76

UPPER WIND LEVEL

STABILITY CLASS A

STABILITY CALCULATED FROM DIFF. TEMPERATURE

HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.2477E-01	0.2229E+00	0.3220E+00	0.1238E+00	0.0	0.0	0.6935E+00	0.8912E+01
NNE	0.0	0.1238E-01	0.1858E+00	0.3096E+00	0.1610E+00	0.0	0.0	0.6687E+00	0.9600E+01
NE	0.0	0.0	0.8669E-01	0.3467E+00	0.1486E+00	0.4954E-01	0.0	0.6316E+00	0.1133E+02
ENE	0.0	0.1238E-01	0.1486E+00	0.9907E-01	0.3715E-01	0.0	0.0	0.2972E+00	0.8112E+01
E	0.0	0.0	0.9907E-01	0.3715E-01	0.0	0.0	0.0	0.1362E+00	0.6392E+01
ESE	0.0	0.0	0.1734E+00	0.3715E-01	0.1238E-01	0.0	0.0	0.2229E+00	0.6364E+01
SE	0.0	0.0	0.6192E-01	0.0	0.0	0.0	0.0	0.6192E-01	0.5224E+01
SSE	0.0	0.1238E-01	0.1115E+00	0.3715E-01	0.1238E-01	0.0	0.0	0.1734E+00	0.7306E+01
S	0.0	0.2477E-01	0.1610E+00	0.7430E-01	0.1238E-01	0.1238E-01	0.1238E-01	0.2972E+00	0.7956E+01
SSW	0.0	0.1238E-01	0.1858E+00	0.3715E+00	0.2477E+00	0.2477E-01	0.0	0.8421E+00	0.1060E+02
SW	0.0	0.4954E-01	0.2601E+00	0.4954E+00	0.4087E+00	0.9907E-01	0.2477E-01	0.1337E+01	0.1170E+02
WSW	0.0	0.6192E-01	0.1486E+00	0.4706E+00	0.2105E+00	0.4954E-01	0.1238E-01	0.9536E+00	0.1091E+02
W	0.0	0.1238E-01	0.1362E+00	0.1486E+00	0.8669E-01	0.1238E-01	0.0	0.3963E+00	0.9358E+01
WNW	0.0	0.6192E-01	0.1610E+00	0.2353E+00	0.1858E+00	0.6192E-01	0.0	0.7059E+00	0.1068E+02
NW	0.0	0.1238E-01	0.1610E+00	0.1610E+00	0.6192E-01	0.6192E-01	0.0	0.4582E+00	0.1010E+02
NNW	0.0	0.2477E-01	0.1610E+00	0.2229E+00	0.1362E+00	0.1238E-01	0.0	0.5573E+00	0.9872E+01
TOTAL	0.0	0.3220+00	0.2464E+01	0.3368E+01	0.1845E+01	0.3839E+00	0.4954E-01	0.8433E+01	0.9026E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 265

TABLE 2.3.3-6 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
UPPER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.2477E-01	0.1238E+00	0.1362E+00	0.3715E-01	0.0	0.0	0.3220E+00	0.8394E+01
NNE	0.0	0.1238E-01	0.7430E-01	0.9907E-01	0.7430E-01	0.0	0.0	0.2601E+00	0.9710E+01
NE	0.0	0.0	0.1486E+00	0.1858E+00	0.7430E-01	0.1238E-01	0.0	0.4211E+00	0.9242E+01
ENE	0.0	0.2477E-01	0.1115E+00	0.3715E-01	0.2477E-01	0.0	0.0	0.1981E+00	0.6497E+01
E	0.0	0.1238E-01	0.1362E+00	0.2477E-01	0.0	0.0	0.0	0.1734E+00	0.5981E+01
ESE	0.0	0.1238E-01	0.8669E-01	0.2477E-01	0.0	0.0	0.0	0.1238E+00	0.6364E+01
SE	0.0	0.0	0.6192E-01	0.3715E-01	0.0	0.0	0.0	0.9907E-01	0.7037E+01
SSE	0.0	0.1238E-01	0.3715E-01	0.1238E-01	0.0	0.0	0.0	0.6192E-01	0.6518E+01
S	0.0	0.0	0.4954E-01	0.7430E-01	0.6192E-01	0.0	0.0	0.1858E+00	0.1109E+02
SSW	0.0	0.0	0.9907E-01	0.1858E+00	0.9907E-01	0.0	0.0	0.3839E+00	0.1021E+02
SW	0.0	0.0	0.1734E+00	0.2724E+00	0.1981E+00	0.2477E-01	0.1238E-01	0.6811E+00	0.1100E+02
WSW	0.0	0.2477E-01	0.6192E-01	0.2353E+00	0.1238E+00	0.0	0.2477E-01	0.4706E+00	0.1148E+02
W	0.0	0.0	0.9907E-01	0.2477E-01	0.4954E-01	0.3715E-01	0.1238E-01	0.2229E+00	0.1157E+02
WNW	0.0	0.0	0.1238E+00	0.1858E+00	0.1115E+00	0.4954E-01	0.0	0.4706E+00	0.1113E+02
NW	0.0	0.1238E-01	0.1362E+00	0.1115E+00	0.1238E+00	0.2477E-01	0.0	0.4087E+00	0.1014E+02
NNW	0.0	0.1238E-01	0.1362E+00	0.1115E+00	0.3715E-01	0.0	0.0	0.2972E+00	0.7862E+01
TOTAL	0.0	0.1486E+00	0.1659E+01	0.1759E+01	0.1015E+01	0.1486E+00	0.4954E-01	0.4780E+01	0.9014E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 4

TABLE 2.3.3-6 (Continued)

JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED

FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76

UPPER WIND LEVEL

STABILITY CLASS C

STABILITY CALCULATED FROM DIFF. TEMPERATURE

HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.4954E-01	0.1610E+00	0.1486E+00	0.3715E-01	0.0	0.0	0.3963E+00	0.7754E+01
NNE	0.0	0.1238E-01	0.1858E+00	0.1610E+00	0.0	0.0	0.0	0.3591E+00	0.7306E+01
NE	0.0	0.1238E-01	0.1238E+00	0.1362E+00	0.6192E-01	0.1238E-01	0.0	0.3467E+00	0.9252E+01
ENE	0.0	0.2477E-01	0.9907E-01	0.3715E-01	0.1238E-01	0.0	0.1238E-01	0.1858E+00	0.8826E+01
E	0.0	0.2477E-01	0.6192E-01	0.2477E-01	0.0	0.0	0.0	0.1115E+00	0.5607E+01
ESE	0.0	0.2477E-01	0.6192E-01	0.0	0.0	0.0	0.0	0.8669E-01	0.4791E+01
SE	0.0	0.0	0.6192E-01	0.2477E-01	0.0	0.0	0.0	0.8669E-01	0.6857E+01
SSE	0.0	0.1238E-01	0.6192E-01	0.6192E-01	0.2477E-01	0.0	0.0	0.1610E+00	0.8180E+01
S	0.0	0.1238E-01	0.1115E+00	0.1362E+00	0.1238E-01	0.2477E-01	0.0	0.2972E+00	0.8632E+01
SSW	0.0	0.1238E-01	0.1362E+00	0.2229E+00	0.1362E+00	0.0	0.1238E-01	0.5201E+00	0.1020E+02
SW	0.0	0.2477E-01	0.1486E+00	0.1610E+00	0.1610E+00	0.4954E-01	0.0	0.5449E+00	0.1094E+02
WSW	0.0	0.3715E-01	0.1981E+00	0.2229E+00	0.1362E+00	0.4954E-01	0.2477E-01	0.6687E+00	0.1060E+02
W	0.0	0.3715E-01	0.1610E+00	0.1486E+00	0.4954E-01	0.6192E-01	0.0	0.4582E+00	0.9786E+01
WNW	0.0	0.0	0.1115E+00	0.3715E-01	0.1115E+00	0.2477E-01	0.0	0.2848E+00	0.1176E+02
NW	0.0	0.3715E-01	0.1362E+00	0.9907E-01	0.8669E-01	0.0	0.0	0.3591E+00	0.8554E+01
NNW	0.0	0.1238E-01	0.1238E+00	0.1858E+00	0.2477E-01	0.1238E-01	0.0	0.3591E+00	0.8690E+01
TOTAL	0.0	0.3344E+00	0.1944E+01	0.1808E+01	0.8545E+00	0.2353E+00	0.4954E-01	0.5226E+01	0.8609E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 4

TABLE 2.3.3-6 (Continued)

JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
UPPER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1486E+00	0.4706E+00	0.6563E+00	0.4458E+00	0.2477E-01	0.0	0.1746E+01	0.9649E+01
NNE	0.0	0.1238E+00	0.4830E+00	0.1077E+01	0.3839E+00	0.3715E-01	0.0	0.2105E+01	0.9601E+01
NE	0.0	0.9907E-01	0.3839E+00	0.9412E+00	0.4334E+00	0.1238E-01	0.0	0.1870E+01	0.9948E+01
ENE	0.0	0.1238E+00	0.3963E+00	0.3467E+00	0.8669E-01	0.0	0.0	0.9536E+00	0.7349E+01
E	0.0	0.1610E+00	0.4334E+00	0.1610E+00	0.0	0.0	0.0	0.7554E+00	0.5855E+01
ESE	0.0	0.1238E+00	0.5573E+00	0.1486E+00	0.1115E+00	0.0	0.0	0.9412E+00	0.6714E+01
SE	0.0	0.1362E+00	0.4458E+00	0.2353E+00	0.8669E-01	0.0	0.0	0.9040E+00	0.7037E+01
SSE	0.0	0.1610E+00	0.6563E+00	0.5944E+00	0.2477E+00	0.0	0.0	0.1659E+01	0.8240E+01
S	0.1238E-01	0.2229E+00	0.5573E+00	0.5201E+00	0.1734E+00	0.1238E-01	0.0	0.1498E+01	0.7745E+01
SSW	0.0.0	0.1931E+00	0.6068E+00	0.7430E+00	0.6687E+00	0.1610E+00	0.0	0.2378E+01	0.1028E+02
SW	0.1238E-01	0.2353E+00	0.7307E+00	0.7307E+00	0.6811E+00	0.1486E+00	0.1238E-01	0.2551E+01	0.1007E+02
WSW	0.0	0.1115E+00	0.6687E+00	0.6811E+00	0.4458E+00	0.4954E-01	0.4954E-01	0.2006E+01	0.1008E+02
W	0.0	0.1362E+00	0.3963E+00	0.4458E+00	0.2229E+00	0.7430E-01	0.1238E-01	0.1288E+01	0.9472E+01
WNW	0.0	0.8669E-01	0.2353E+00	0.2105E+00	0.1486E+00	0.7430E-01	0.0	0.7554E+00	0.1002E+02
NW	0.0	0.9907E-01	0.3591E+00	0.3467E+00	0.2972E+00	0.2477E-01	0.1238E-01	0.1139E+01	0.9839E+01
NNW	0.0	0.1610E+00	0.5573E+00	0.6868E+00	0.2477E+00	0.4954E-01	0.0	0.1622E+01	0.8624E+01
TOTAL	0.2477E-01	0.2328E+01	0.7938E+01	0.8446E+01	0.4681E+01	0.6687E+00	0.8669E-01	0.2417E+02	0.8782E+01

NUMBER OF CALMS - 2

NUMBER OF BAD HOURS - 33

TABLE 2.3.3-6 (Continued)

JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED

FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76

UPPER WIND LEVEL

STABILITY CLASS E

STABILITY CALCULATED FROM DIFF. TEMPERATURE

HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.4954E-01	0.2848E+00	0.7678E+00	0.3344E+00	0.0	0.0	0.1437E+01	0.1005E+02
NNE	0.0	0.3715E-01	0.2848E+00	0.6316E+00	0.2601E+00	0.0	0.0	0.1214E+01	0.9836E+01
NE	0.0	0.6192E-01	0.3467E+00	0.8669E+00	0.2972E+00	0.0	0.0	0.1573E+01	0.9381E+01
ENE	0.0	0.3715E-01	0.1858E+00	0.3591E+00	0.1486E+00	0.0	0.0	0.7307E+00	0.9332E+01
E	0.0	0.2477E-01	0.2601E+00	0.3220E+00	0.7430E-01	0.0	0.0	0.6811E+00	0.8059E+01
ESE	0.0	0.4954E-01	0.2105E+00	0.3467E+00	0.3715E-01	0.0	0.0	0.6440E+00	0.8146E+01
SE	0.0	0.7430E-01	0.2724E+00	0.4334E+00	0.2477E-01	0.0	0.0	0.8050E+00	0.7930E+01
SSE	0.0	0.6192E-01	0.5201E+00	0.9164E+00	0.9907E-01	0.1238E-01	0.0	0.1610E+01	0.8790E+01
S	0.0	0.7430E-01	0.5944E+00	0.1548E+01	0.5820E+00	0.3715E-01	0.0	0.2836E+01	0.9870E+01
SSW	0.0	0.7430E-01	0.4582E+00	0.2229E+01	0.1065E+01	0.6192E-01	0.1238E-01	0.3901E+01	0.1078E+02
SW	0.0	0.6192E-01	0.4087E=00	0.9412E+00	0.7430E+00	0.1362E+00	0.0	0.2291E+01	0.1121E+02
WSW	0.1238E-01	0.1362E+00	0.5820E+00	0.6811E+00	0.3344E+00	0.0	0.0	0.1746E+01	0.8773E+01
W	0.0	0.3715E-01	0.1610E+00	0.4706E+00	0.2477E+00	0.1238E-01	0.0	0.9288E+00	0.1008E+02
WNW	0.0	0.4954E-01	0.2848E+00	0.4087E+00	0.2972E+00	0.2477E-01	0.0	0.1065E+01	0.1028E+02
NW	0.1238E-01	0.9907E-01	0.1981E+00	0.7307E+00	0.1858E+00	0.1238E-01	0.1238E-01	0.1251E+01	0.9554E+01
NNW	0.0	0.8669E-01	0.3467E+00	0.3591E+00	0.8669E-01	0.2477E-01	0.0	0.9040E+00	0.8118E+01
TOTAL	0.2477E-01	0.1015E+01	0.5399E+01	0.1201E+02	0.4817E+01	0.3220E+00	0.2477E-01	0.2362E-02	0.9388E+01

NUMBER OF CALMS - 2

NUMBER OF BAD HOURS - 37

TABLE 2.3.3-6 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
UPPER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1238E-01	0.2105E+00	0.4458E+00	0.2229E+00	0.0	0.0	0.8916E+00	0.1030E+02
NNE	0.0	0.3715E-01	0.1734E+00	0.4087E+00	0.1238E-01	0.0	0.0	0.6316E+00	0.8468E+01
NE	0.0	0.3715E-01	0.9907E-01	0.2848E+00	0.3715E-01	0.0	0.0	0.4582E+00	0.8786E+01
ENE	0.0	0.0	0.7430E-01	0.2353E+00	0.8669E-01	0.0	0.0	0.3963E+00	0.1025E+02
E	0.0	0.1238E-01	0.4954E-01	0.3096E+00	0.1238E-01	0.0	0.0	0.3839E+00	0.9219E+01
ESE	0.0	0.1238E-01	0.1486E+00	0.2353E+00	0.2477E-01	0.0	0.0	0.4211E+00	0.7944E+01
SE	0.0	0.1238E-01	0.1610E+00	0.2229E+00	0.0	0.0	0.0	0.3963E+00	0.7828E+01
SSE	0.0	0.2477E-01	0.1610E+00	0.2353E+00	0.1238E-01	0.0	0.0	0.4334E+00	0.7896E+01
S	0.0	0.3715E-01	0.2972E+00	0.7926E+00	0.1362E+00	0.0	0.0	0.1263E+01	0.9268E+01
SSW	0.1238E-01	0.6192E-01	0.4211E+00	0.1449E+01	0.3220E+00	0.0	0.0	0.2266E+01	0.9654E+01
SW	0.1238E-01	0.9907E-01	0.3467E+00	0.1189E+01	0.1981E+00	0.0	0.0	0.1845E+01	0.9000E+01
WSW	0.0	0.3715E-01	0.2724E+00	0.5573E+00	0.1115E+00	0.0	0.0	0.9783E+00	0.8813E+01
W	0.1238E-01	0.3715E-01	0.1981E+00	0.3096E+00	0.1362E+00	0.0	0.0	0.6935E+00	0.9078E+01
WNW	0.1238E-01	0.8669E-01	0.1238E+00	0.2848E+00	0.1238E+00	0.0	0.0	0.6316E+00	0.9174E+01
NW	0.0	0.2477E-01	0.1115E+00	0.1981E+00	0.6192E-01	0.0	0.0	0.3963E+00	0.8624E+01
NNW	0.1238E-01	0.3715E-01	0.2105E+00	0.2477E+00	0.8669E-01	0.0	0.0	0.5944E+00	0.8711E+01
TOTAL	0.6192E-01	0.5697E+00	0.3059E+01	0.7406E+01	0.1585E+01	0.0	0.0	0.1268E+02	0.8938E+01

NUMBER OF CALMS - 6

NUMBER OF BAD HOURS - 9

TABLE 2.3.3-6 (Continued)

JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
UPPER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.7430E-01	0.3591E+00	0.6192E+00	0.7430E-01	0.0	0.0	0.1127E+01	0.7940E+01
NNE	0.0	0.1238E+00	0.3467E+00	0.6811E+00	0.4954E-01	0.0	0.0	0.1201E+01	0.7847E+01
NE	0.0	0.1238E+00	0.2229E+00	0.4458E+00	0.1238E-01	0.0	0.0	0.8050E+00	0.7276E+01
ENE	01238E-01	0.1362E+00	0.2601E+00	0.2848E+00	0.1486E+00	0.0	0.0	0.8421E+00	0.8149E+01
E	0.0	0.2477E-01	0.2601E+00	0.2972E+00	0.2477E-01	0.0	0.0	0.6068E+00	0.7859E+01
ESE	0.0	0.9907E-01	0.2353E+00	0.3220E+00	0.6192E-01	0.0	0.0	0.7183E+00	0.7766E+01
SE	0.0	0.1238E+00	0.2972E+00	0.2724E+00	0.1238E-01	0.0	0.0	0.7059E+00	0.6862E+01
SSE	0.0	0.8669E-01	0.6068E+00	0.5077E+00	0.1238E-01	0.0	0.0	0.1214E+01	0.7277E+01
S	0.1238E-01	0.1610E+00	0.6811E+00	0.8050E+00	0.9907E-01	0.0	0.0	0.1759E+01	0.7775E+01
SSW	0.1238E-01	0.1858E+00	0.7926E+00	0.1424E+01	0.1734E+00	0.0	0.0	0.2588E+01	0.8367E+01
SW	0.1238E-01	0.1362E+00	0.5820E+00	0.1288E+01	0.9907E-01	0.0	0.0	0.2118E+01	0.8261E+01
WSW	0.1238E-01	0.1362E+00	0.9412E+00	0.1635E+01	0.2972E+00	0.0	0.0	0.3022E+01	0.8529E+01
W	0.1238E-01	0.1486E+00	0.4458E+00	0.6192E+00	0.6192E-01	0.0	0.0	0.1288E+01	0.7461E+01
WNW	0.1238E-01	0.1238E+00	0.5449E+00	0.3096E+00	0.3715E-01	0.0	0.0	0.1028E+01	0.6647E+01
NW	0.2477E-01	0.2105E+00	0.5820E+00	0.2105E+00	0.0	0.0	0.0	0.1028E+01	0.5408E+01
NNW	0.1238E-01	0.1734E+00	0.3715E+00	0.4087E+00	0.7430E-01	0.0	0.0	0.1040E+01	0.7140E+01
TOTAL	0.1238E+00	0.2068E+01	0.7529E+01	0.1013E+02	0.1238E+01	0.0	0.0	0.2109E+02	0.7535E+01

NUMBER OF CALMS - 10

NUMBER OF BAD HOURS - 5

TABLE 2.3.3-6 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
UPPER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.3839E+00	0.1833E+01	0.3096E+01	0.1276E+01	0.2477E-01	0.0	0.6613E+01	0.9280E+01
NNE	0.0	0.3591E+00	0.1734E+01	0.3368E+01	0.9412E+00	0.3715E-01	0.0	0.6440E+01	0.9083E+01
NE	0.0	0.3344E+00	0.1412E+01	0.3207E+01	0.1065E+01	0.8669E-01	0.0	0.6105E+01	0.9417E+01
ENE	0.1238E-01	0.3591E+00	0.1276E+01	0.1399E+01	0.5449E+00	0.0	0.1238E-01	0.3604E+01	0.8350E+01
E	0.0	0.2601E+00	0.1300E+01	0.1176E+01	0.1115E+00	0.0	00.0	0.2848E+01	0.7286E+01
ESE	0.0	0.3220E+00	0.1474E+01	0.1115E+01	0.2477E+00	0.0	0.0	0.3158E+01	0.73183+01
SE	0.0	0.3467E+00	0.1362E+01	0.1226E+01	0.1238E+00	0.0	0.0	0.3059E+01	0.7292E+01
SSE	0.0	0.3715E+00	0.2155E+01	0.2365E+01	0.4087E+00	0.1238E-01	0.0	0.5313E+01	0.8106E+01
S	0.2477E-01	0.5325E+00	0.2452E+01	0.3950E+01	0.1077E+01	0.8669E-01	0.1238E-01	0.8136E+01	0.8848E+01
SSW	0.2477E-01	0.5449E+00	0.2700E+01	0.6625E+01	0.2712E+01	0.2477E+00	0.2477E-01	0.1288E+02	0.9956E+01
SW	0.3715E-01	0.6068E+00	0.2650E+01	0.5077E+01	0.2489E+01	0.4582E+00	0.4954E-01	0.1137E+02	0.1008E+02
WSW	0.2477E-01	0.5449E+00	0.2873E+01	0.4483E+01	0.1659E+01	0.1486E+00	0.1115E+00	0.9845E+01	0.9428E+01
W	0.2477E-01	0.4087E+00	0.1598E+01	0.2167E+01	0.8545E+00	0.1981E+00	0.2477E-01	0.5276E+01	0.9148E+01
WNW	0.2477E-01	0.5201E+00	0.1585E+01	0.1672E+01	0.1015E+01	0.2353E+00	0.0	0.4941E+01	0.9573E+01
NW	0.3715E-01	0.4954E+00	0.1684E+01	0.1858E+01	0.8173E+00	0.1238E+00	0.2477E-01	0.5040E+01	0.8740E+01
NNW	0.2477E-01	0.5077E+00	0.1907E+01	0.2142E+01	0.6935E+00	0.9907E-01	0.0	0.5375E+01	0.8355E+01
TOTAL	0.2353E+00	0.6787E+01	0.2999E+02	0.4493E+02	0.1604E+02	0.1759E+01	0.2601E+00	0.1000E+03	0.9088E+01

NUMBER OF CALMS - 19

NUMBER OF BAD HOURS - 357

TABLE 2.3.3-7
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
LOWER WIND LEVEL
STABILITY CLASS A
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.4927E-01	0.4188E+00	0.3449E+00	0.1232E-01	0.0	0.0	0.8252E+00	0.7090E+01
NNE	0.0	0.0	0.2340E+00	0.3202E+00	0.0	0.0	0.0	0.5543E+00	0.7861E+01
NE	0.0	0.0	0.2340E+00	0.3326E+00	0.2463E-01	0.0	0.0	0.5912E+00	0.8095E+01
ENE	0.0	0.0	0.2463E_00	0.9853E-01	0.0	0.0	0.0	0.3449E+00	0.6689E+01
E	0.0	0.1232E-01	0.8622E-01	0.0	0.0	0.0	0.0	0.9853E-01	0.5435E+01
ESE	0.0	0.0	0.6158E-01	0.1232E-01	0.0	0.0	0.0	0.7390E-01	0.5812E+01
SE	0.0	0.0	0.9853E-01	0.1232E-01	0.0	0.0	0.0	0.1109E+00	0.5587E+01
SSE	0.0	0.1232E-01	0.1848E+00	0.1232E-01	0.1232E-01	0.0	0.0	0.2217E+00	0.6205E+01
S	0.0	0.2463E-01	0.1355E+00	0.1232E-01	0.2463E-01	0.0	0.0	0.1971E+00	0.6849E+01
SSW	0.0	0.1232E-01	0.4188E+00	0.4927E+00	0.2463E-01	0.0	0.0	0.9484E+00	0.8062E+01
SW	0.0	0.3695E-01	0.3202E+00	0.6405E+00	0.1971E+00	0.0	0.0	0.1195E+01	0.9037E+01
WSW	0.1232E-01	0.9853E-01	0.4188E+00	0.4680E+00	0.4927E-01	0.0	0.0	0.1047E+01	0.7524E+01
W	0.0	0.3695E-01	0.1848E+00	0.2217E+00	0.1232E-01	0.0	0.0	0.4557E+00	0.7210E+01
WNW	0.0	0.1232E-01	0.2463E+00	0.3326E+00	0.1109E+00	0.0	0.0	0.7021E+00	0.9014E+01
NW	0.0	0.0	0.3818E+00	0.9853E-01	0.9353E-01	0.0	0.0	0.5789E+00	0.7625E+01
NNW	0.0	0.2463E-01	0.3079E+00	0.1601E+00	0.3695E-01	0.0	0.0	0.5296E+00	0.7536E+01
TOTAL	0.1232E-01	0.3202E+00	0.3978E+01	0.3560E+01	0.6035E+00	0.0	0.0	0.8474E+01	0.7227E+01

NUMBER OF CALMS - 1

NUMBER OF BAD HOURS - 258

TABLE 2.3.3-7 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
LOWER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.0	0.2587E+00	0.9853E-01	0.0	0.0	0.0	0.3572E+00	0.6437E+01
NNE	0.0	0.1232E-01	0.1355E+00	0.7390E-01	0.0	0.0	0.0	0.2217E+00	0.6750E+01
NE	0.0	0.2463E-01	0.2463E+00	0.1355E+00	0.1232E-01	0.0	0.0	0.4188E+00	0.6493E+01
ENE	0.0	0.2463E-01	0.1355E+00	0.3695E-01	0.0	0.0	0.0	0.1971E+00	0.5852E+01
E	0.0	0.0	0.6158E-01	0.1232E-01	0.0	0.0	0.0	0.7390E-01	0.5353E+01
ESE	0.0	0.1232E-01	0.1232E+00	0.0	0.0	0.0	0.0	0.1355E+00	0.5432E+01
SE	0.0	0.1232E-01	0.3695E-01	0.2463E-01	0.0	0.0	0.0	0.7390E-01	0.6640E+01
SSE	0.0	0.0	0.3695E-01	0.1232E-01	0.0	0.0	0.0	0.4927E-01	0.6197E+01
S	0.0	0.1232E-01	0.3695E-01	0.3622E-01	0.1232E-01	0.0	0.0	0.1478E+00	0.8619E+01
SSW	0.0	0.1232E-01	0.1971E+00	0.1478E+00	0.0	0.0	0.0	0.3572E+00	0.6977E+01
SW	0.0	0.2463E-01	0.2956E+00	0.2463E+00	0.4927E-01	0.2463E-01	0.0	0.6405E+00	0.8465E+01
WSW	0.0	0.3695E-01	0.1848E+00	0.2833E+00	0.1232E-01	0.0	0.0	0.5173E+00	0.7579E+01
W	0.0	0.4927E-01	0.1109E+00	0.1478E+00	0.2463E-01	0.0	0.0	0.3326E+00	0.7405E+01
WNW	0.0	0.1232E-01	0.1478E+00	0.1724E+00	0.6158E-01	0.0	0.0	0.3941E+00	0.8627E+01
NW	0.0	0.2463E-01	0.2217E+00	0.2217E+00	0.4927E-01	0.0	0.0	0.5173E+00	0.8163E+01
NNW	0.0	0.0	0.2217E+00	0.9853E-01	0.1232E-01	0.0	0.0	0.3326E+00	0.6936E+01
TOTAL	0.0	0.2587E+00	0.2451E+01	0.1798E+01	0.2340E+00	0.2463E-01	0.0	0.4767E+01	0.6995E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 3

TABLE 2.3.3-7 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
LOWER WIND LEVEL
STABILITY CLASS C
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.2463E-01	0.2833E+00	0.2094E+00	0.0	0.0	0.0	0.5173E+00	0.6431E+01
NNE	0.0	0.2463E-01	0.2463E+00	0.2463E-01	0.0	0.0	0.0	0.2956E+00	0.5577E+01
NE	0.0	0.1232E-01	0.1355E+00	0.8622E-01	0.0	0.0	0.0	0.2340E+00	0.6989E+01
ENE	0.0	0.2463E-01	0.1109E+00	0.4927E-01	0.0	0.0	0.0	0.1848E+00	0.5899E+01
E	0.0	0.1232E-01	0.8622E-01	0.0	0.0	0.0	0.0	0.9853E-01	0.4469E+01
ESE	0.0	0.0	0.7390E-01	0.0	0.0	0.0	0.0	0.7390E-01	0.4955E+01
SE	0.0	0.2463E-01	0.6158E-01	0.3695E-01	0.0	0.0	0.0	0.1232E+00	0.5624E+01
SSE	0.0	0.3695E-01	0.8622E-01	0.3695E-01	0.0	0.0	0.0	0.1601E+00	0.5480E+01
S	0.0	0.1232E-01	0.1355E+00	0.3695E-01	0.1232E-01	0.0	0.0	0.1971E+00	0.6334E+01
SSW	0.0	0.1232E-01	0.3202E+00	0.1601E+00	0.2463E-01	0.0	0.0	0.5173E+00	0.6924E+01
SW	0.0	0.3695E-01	0.3079E+00	0.2094E+00	0.2463E-01	0.0	0.0	0.5789E+00	0.7198E+01
WSW	0.0	0.3695E-01	0.2710E+00	0.2217E+00	0.3695E-01	0.1232E-01	0.0	0.5789E+00	0.7923E+01
W	0.0	0.2463E-01	0.2587E+00	0.2094E+00	0.3695E-01	0.0	0.0	0.5296E+00	0.7711E+01
WNW	0.0	0.3695E-01	0.1601E+00	0.4927E-01	0.7390E-01	0.0	0.0	0.3202E+00	0.7986E+01
NW	0.0	0.3695E-01	0.1601E+00	0.2094E+00	0.0	0.0	0.0	0.4065E+00	0.7531E+01
NNW	0.0	0.4927E-01	0.2587E+00	0.6158E-01	0.1232E-01	0.0	0.0	0.3818E+00	0.5764E+01
TOTAL	0.0	0.4065E+00	0.2956E+01	0.1601E+01	0.2217E+00	0.1232E-01	0.0	0.5198E+01	0.6425E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 4

TABLE 2.3.3-7 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
LOWER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.2710E+00	0.1158E+01	0.6774E+00	0.2463E-01	0.0	0.0	0.2131E+01	0.6434E+01
NNE	0.0	0.2094E+00	0.1133E+01	0.2587E+00	0.0	0.0	0.0	0.1601E+01	0.5695E+01
NE	0.0	0.2833E+00	0.1244E+01	0.3695E+00	0.1232E-01	0.0	0.0	0.1909E+01	0.6043E+01
ENE	0.0	0.1109E+00	0.7390E+00	0.1478E+00	0.0	0.0	0.0	0.9977E+00	0.5388E+01
E	0.0	0.2587E+00	0.4065E+00	0.1232E-01	0.0	0.0	0.0	0.6744E+00	0.3845E+01
ESE	0.0	0.2094E+00	0.4434E+00	0.4927E-01	0.0	0.0	0.0	0.7021E+00	0.4645E+01
SE	0.0	0.2217E+00	0.6282E+00	0.1478E+00	0.0	0.0	0.0	0.9977E+00	0.5060E+01
SSE	0.0	0.3326E+00	0.9114E+00	0.2094E+00	0.0	0.0	0.0	0.1453E+01	0.5189E+01
S	0.1232E-01	0.3695E+00	0.8622E+00	0.2463E+00	0.1232E-01	0.0	0.0	0.1503E+01	0.5246E+01
SSW	0.1232E-01	0.4804E+00	0.1343E+01	0.6405E+00	0.9853E-01	0.0	0.0	0.2574E+01	0.6082E+01
SW	0.1232E-01	0.5666E+00	0.9114E+00	0.7760E+00	0.7390E-01	0.1232E-01	0.0	0.2353E+01	0.6346E+01
WSW	0.0	0.2956E+00	0.9977E+00	0.6651E+00	0.4927E-01	0.3695E-01	0.0	0.2045E+01	0.6833E+01
W	0.1232E-01	0.3572E+00	0.6528E+00	0.4188E+00	0.6158E-01	0.0	0.0	0.1503E+01	0.6213E+01
WNW	0.0	0.2710E+00	0.4311E+00	0.3449E+00	0.8622E-01	0.0	0.0	0.1133E+01	0.6531E+01
NW	0.0	0.1601E+00	0.4311E+00	0.4188E+00	0.7390E-01	0.0	0.0	0.1084E+01	0.7269E+01
NNW	0.0	0.3326E+00	0.7636E+00	0.5173E+00	0.6158E-01	0.0	0.0	0.1675E+01	0.6325E+01
TOTAL	0.4927E-01	0.4730E+01	0.1306E+02	0.5900E+01	0.5543E+00	0.4927E-01	0.0	0.2434E+02	0.5822E+01

NUMBER OF CALMS - 4

NUMBER OF BAD HOURS - 9

TABLE 2.3.3-7 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
LOWER WIND LEVEL
STABILITY CLASS E
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.3941E+00	0.8252E+00	0.2340E+00	0.0	0.0	0.0	0.1453E+01	0.4954E+01
NNE	0.2463E-01	0.6158E+00	0.7390E+00	0.4927E-01	0.0	0.0	0.0	0.1429E+01	0.3871E+01
NE	0.2463E-01	0.6035E+00	0.6405E+00	0.7390E-01	0.0	0.0	0.0	0.1343E+01	0.4202E+01
ENE	0.0	0.3449E+00	0.2833E+00	0.6158E-01	0.0	0.0	0.0	0.6897E+00	0.4006E+01
E	0.1232E-01	0.4065E+00	0.1971E+00	0.1232E-01	0.0	0.0	0.0	0.6282E+00	0.3205E+01
ESE	0.0	0.3449E+00	0.2094E+00	0.2463E-01	0.0	0.0	0.0	0.5789E+00	0.3722E+01
SE	0.1232E-01	0.4557E+00	03695E+00	0.2463E-01	0.0	0.0	0.0	0.8622E+00	0.3636E+01
SSE	0.2463E-01	0.7513E+00	0.7513E+00	0.6158E-01	0.0	0.0	0.0	0.1589E+01	0.3835E+01
S	0.3695E-01	0.1392E+01	0.1367E+01	0.2833E+00	0.1232E-01	0.0	0.0	0.3092E+01	0.4243E+01
SSW	0.3695E-01	0.1392E+01	0.1811E+0	0.3572E+00	0.2463E-01	0.0	0.0	0.3621E+01	0.4616E+01
SW	0.2463E-01	0.9114E+00	0.1022E+01	0.4804E+00	0.1232E-01	0.0	0.0	0.2451E+01	0.4902E+01
WSW	0.2463E-01	0.6282E+00	0.6405E+00	0.2217E+00	0.0	0.0	0.0	0.1515E+01	0.4532E+01
W	0.0	0.3326E+00	0.5296E+00	0.1478E+00	0.1232E-01	0.0	0.0	0.1022E+01	0.4862E+01
WNW	0.1232E-01	0.3941E+00	0.5419E+00	0.1478E+00	0.0	0.1232E-01	0.0	0.1109E+01	0.4969E+01
NW	0.0	0.2956E+00	0.6528E+00	0.1724E+00	0.3695E-01	0.0	0.0	0.1158E+01	0.5184E+01
NNW	0.2463E-01	0.5050E+00	0.5173E+00	0.9853E-01	0.1232E-01	0.0	0.0	0.1158E+01	0.4160E+01
TOTAL	0.2587E+00	0.9767E+01	0.1110E+02	0.2451E+01	0.1109E+00	0.1232E-01	0.0	0.2370E+02	0.4306E+01

NUMBER OF CALMS - 21

NUMBER OF BAD HOURS - 20

TABLE 2.3.3-7 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
LOWER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.4927E-01	0.5296E+00	0.3695E+00	0.2463E-01	0.0	0.0	0.0	0.9730E+00	0.3399E+01
NNE	0.4927E-01	0.6035E+00	0.7390E-01	0.0	0.0	0.0	0.0	0.7267E+00	0.2362E+01
NE	0.3695E-01	0.3818E+00	0.7390E-01	0.0	0.0	0.0	0.0	0.4927E+00	0.2287E+01
ENE	0.4927E-01	0.5173E+00	0.7390E-01	0.0	0.0	0.0	0.0	0.6405E+00	0.2391E+01
E	0.3695E-01	0.3818E+00	0.3695E-01	0.0	0.0	0.0	0.0	0.4557E+00	0.2064E+01
ESE	0.2463E-01	0.3695E+00	0.2463E-01	0.0	0.0	0.0	0.0	0.4188E+00	0.2095E+01
SE	0.0	0.3202E+00	0.7390E-01	0.0	0.0	0.0	0.0	0.3941E+00	0.2500E+01
SSE	0.7390E-01	0.8622E+00	0.4927E-01	0.0	0.0	0.0	0.0	0.9853E+00	0.2021E+01
S	0.8622E-01	0.1158E+01	0.3326E+00	0.0	0.0	0.0	0.0	0.1577E+01	0.2553E+01
SSW	0.9853E-01	0.1195E+01	0.4188E+00	0.0	0.0	0.0	0.0	0.1712E+01	0.2593E+01
SW	0.7390E-01	0.9730E+00	0.2217E+00	0.0	0.0	0.0	0.0	0.1269E+01	0.2572E+01
WSW	0.4927E-01	0.5789E+00	0.9853E-01	0.0	0.0	0.0	0.0	0.7267E+00	0.2285E+01
W	0.0	0.2956E+00	0.2094E+00	0.0	0.0	0.0	0.0	0.5050E+00	0.3172E+01
WNW	0.0	0.3449E+00	0.1971E+00	0.0	0.0	0.0	0.0	0.5419E+00	0.2869E+01
NW	0.0	0.2710E+00	0.1232E+00	0.0	0.0	0.0	0.0	0.3941E+00	0.2825E+01
NNW	0.4927E-01	0.5543E+00	0.1478E+00	0.1232E-01	0.0	0.0	0.0	0.7636E+00	0.2598E+01
TOTAL	0.6774E+00	0.9336E+01	0.2525E+01	0.3695E-01	0.0	0.0	0.0	0.1258E+02	0.2537E+01

NUMBER OF CALMS - 55

NUMBER OF BAD HOURS - 12

TABLE 2.3.3-7 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
LOWER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.7636E+00	0.1170E+01	0.8622E-01	0.0	0.0	0.0	0.0	0.2020E+01	0.1857E+01
NNE	0.6651E+00	0.1022E+01	0.2463E-01	0.0	0.0	0.0	0.0	0.1712E+01	0.1458E+01
NE	0.4927E+00	0.7513E+00	0.0	0.0	0.0	0.0	0.0	0.1244E+01	0.1301E+01
ENE	0.6158E+00	0.9484E+00	0.6158E-01	0.0	0.0	0.0	0.0	0.1626E+01	0.1709E+01
E	0.5173E+00	0.8006E+00	0.1232E-01	0.0	0.0	0.0	0.0	0.1330E+01	0.1542E+01
ESE	0.4311E+00	0.6651E+00	0.1232E-01	0.0	0.0	0.0	0.0	0.1109E+01	0.1438E+01
SE	0.3695E+00	0.5666E+00	0.2463E-01	0.0	0.0	0.0	0.0	0.9607E+00	0.1700E+01
SSE	0.3449E+00	0.5296E+00	0.3695E-01	0.0	0.0	0.0	0.0	0.9114E+00	0.1766E+01
S	0.6528E+00	0.9977E+00	0.1232E-01	0.0	0.0	0.0	0.0	0.1663E+01	0.1423E+01
SSW	0.5666E+00	0.8745E+00	0.0	0.0	0.0	0.0	0.0	0.1441E+01	0.1518E+01
SW	0.7021E+00	0.1084E+01	0.8622E-01	0.0	0.0	0.0	0.0	0.1872E+01	0.1786E+01
WSW	0.5296E+00	0.8129E+00	0.4927E-01	0.0	0.0	0.0	0.0	0.1392E+01	0.1531E+01
W	0.3449E+00	0.5296E+00	0.1232E-01	0.0	0.0	0.0	0.0	0.8868E+00	0.1536E+01
WNW	0.1971E+00	0.4557E+00	0.1232E-01	0.0	0.0	0.0	0.0	0.6651E+00	0.1514E+01
NW	0.3449E+00	0.5296E+00	0.2463E-01	0.0	0.0	0.0	0.0	0.8991E+00	0.1538E+01
NNW	0.4650E+00	0.7267E+00	0.2463E-01	0.0	0.0	0.0	0.0	0.1219E+01	0.1592E+01
TOTAL	0.8006E+01	0.1246E+02	0.4804E+00	0.0	0.0	0.0	0.0	0.2095E+02	0.1576E+01

NUMBER OF CALMS - 650

NUMBER OF BAD HOURS - 7

TABLE 2.3.3-7 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 PM 1/14/76 TO 11:00 PM 12/31/76
LOWER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.8129E+00	0.2439E+01	0.3399E+01	0.1589E+01	0.3695E-01	0.0	0.0	0.8277E+01	0.5072E+01
NNE	0.7390E+00	0.2488E+01	0.2587E+01	0.7267E+00	0.0	0.0	0.0	0.6540E+01	0.4341E+01
NE	0.5543E+00	0.2057E+01	0.2574E+01	0.9977E+00	0.4927E-01	0.0	0.0	0.6232E+01	0.4973E+01
ENE	0.6651E+00	0.1971E+01	0.1650E+01	0.3941E+00	0.0	0.0	0.0	0.4680E+01	0.3942E+01
E	0.5666E+00	0.1872E+01	0.8868E+00	0.3695E-01	0.0	0.0	0.0	0.3362E+01	0.2886E+01
ESE	0.4557E+00	0.1601E+01	0.9484E+00	0.8622E-01	0.0	0.0	0.0	0.3092E+01	0.3319E+01
SE	0.3818E+00	0.1601E+01	0.1293E+01	0.2463E+00	0.0	0.0	0.0	0.3523E+01	0.3799E+01
SSE	0.4434E+00	0.2525E+01	0.2057E+01	0.3326E+00	0.1232E-01	0.0	0.0	0.5370E+01	0.3845E+01
S	0.7883E+00	0.3966E+01	0.2882E+01	0.6651E+00	0.7390E-01	0.0	0.0	0.8375E+01	0.3940E+01
SSW	0.7144E+00	0.3978E+01	0.4508E+01	0.1798E+01	0.1724E+00	0.0	0.0	0.1117E+02	0.4911E+01
SW	0.8129E+00	0.3633E+01	0.3165E+01	0.2353E+01	0.3572E+00	0.3695E-01	0.0	0.1036E+02	0.5478E+01
WSW	0.6158E+00	0.2488E+01	0.2660E+01	0.1860E+01	0.1478E+00	0.4927E-01	0.0	0.7821E+01	0.5535E+01
W	0.3572E+00	0.1626E+01	0.1950E+01	0.1145E+01	0.1478E+00	0.0	0.0	0.5235E+01	0.5432E+01
WNW	0.2094E+00	0.1527E+01	0.1737E+01	0.1047E+01	0.3326E+00	0.1232E-01	0.0	0.4865E+01	0.5885E+01
NW	0.3449E+00	0.1318E+01	0.1995E+01	0.1121E+01	0.2587E+00	0.0	0.0	0.5038E+01	0.5870E+01
NNW	0.5419E+00	0.2192E+01	0.2242E+01	0.9484E+00	0.1355E+00	0.0	0.0	0.6060E+01	0.4868E+01
TOTAL	0.9004E+01	0.3728E+02	0.3654E+02	0.1535E+02	0.1724E+01	0.9853E-01	0.0	0.1000E+03	0.4797E+01

NUMBER OF CALMS - 731

NUMBER OF BAD HOURS - 313

TABLE 2.3.3-8
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/01/77 TO 11:00 PM 12/31/77
UPPER WIND LEVEL
STABILITY CLASS A
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1243E-01	0.1119E+00	0.2859E+00	0.7457E-01	0.1243E-01	0.0	0.4971E+00	0.9727E+01
NNE	0.0	0.1243E-01	0.1119E+00	0.3356E+00	0.7457E-01	0.0	0.0	0.5344E+00	0.9440E+01
NE	0.0	0.1243E-01	0.1119E+00	0.1989E+00	0.1243E-01	0.0	0.0	0.3356E+00	0.8295E+01
ENE	0.0	0.2486E-01	0.1243E-01	0.7457E-01	0.0	0.0	0.0	0.1119E+00	0.7560E+01
E	0.0	0.2486E-01	0.4971E-01	0.4971E-01	0.1243E-01	0.0	0.0	0.1367E+00	0.6887E+01
ESE	0.0	0.4971E-01	0.1119E+00	0.1243E-01	0.0	0.0	0.0	0.1740E+00	0.4963E+01
SE	0.0	0.2486E-01	0.8700E-01	0.1243E-01	0.0	0.0	0.0	0.1243E+00	0.5202E+01
SSE	0.0	0.1243E-01	0.9943E-01	0.2486E-01	0.0	0.0	0.0	0.1367E+00	0.6236E+01
S	0.0	0.1243E-01	0.7457E-01	0.1367E+00	0.1243E-01	0.0	0.0	0.2361E+00	0.8285E+01
SSW	0.0	0.0	0.9943E-01	0.3729E+00	0.2486E+00	0.4971E-01	0.0	0.7706E+00	0.1149E+02
SW	0.0	0.2486E-01	0.1864E+00	0.2486E+00	0.2486E+00	0.1616E+00	0.0	0.8700E+00	0.1230E+02
WSW	0.0	0.1243E-01	0.8700E-01	0.1864E+00	0.1864E+00	0.4971E-01	0.0	0.5220E+00	0.1171E+02
W	0.0	0.2486E-01	0.1367E+00	0.1491E+00	0.1243E+00	0.2486E-01	0.1243E-01	0.4723E+00	0.1086E+02
WNW	0.0	0.0	0.7457E-01	0.4101E+00	0.2859E+00	0.2486E-01	0.2486E-01	0.8203E+00	0.1223E+02
NW	0.0	0.1243E-01	0.8700E-01	0.2859E+00	0.2113E+00	0.0	0.0	0.5966E+00	0.1087E+02
NNW	0.0	0.1243E-01	0.6214E-01	0.3977E+00	0.1989E+00	0.1243E-01	0.0	0.6836E+00	0.1123E+02
TOTAL		0.2734E+00	0.1504E+01	0.3182E+01	0.1690E+01	0.3356E+00	0.3729E-01	0.7022E+01	0.9205E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 253

TABLE 2.3.3-8 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 PM 1/01/77 TO 11:00 PM 12/31/77
UPPER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.0	0.1740E+00	0.1740E+00	0.8700E-01	0.1243E-01	0.0	0.4474E+00	0.9472E+01
NNE	0.0	0.0	0.8700E-01	0.2113E+00	0.3729E-01	0.0	0.0	0.3356E+00	0.9103E+01
NE	0.0	0.1243E-01	0.3729E-01	0.1119E+00	0.2486E-01	0.0	0.0	0.1864E+00	0.8764E+01
ENE	0.0	0.0	0.9943E-01	0.6214E-01	0.0	0.0	0.0	0.1616E+00	0.7604E+01
E	0.0	0.0	0.1243E+00	0.6214E-01	0.0	0.0	0.0	0.1864E+00	0.7237E+01
ESE	0.0	0.0	0.9943E-01	0.1243E-01	0.0	0.0	0.0	0.1119E+00	0.6356E+01
SE	0.0	0.2486E-01	0.2486E-01	0.0	0.0	0.0	0.0	0.4971E-01	0.3547E+01
SSE	0.0	0.1243E-01	0.8700E-01	0.6214E-01	0.0	0.0	0.0	0.1616E+00	0.6623E+01
S	0.0	0.0	0.3729E-01	0.1364E+00	0.3729E-01	0.0	0.0	0.2610E+00	0.9910E+01
SSW	0.0	0.0	0.8700E-01	0.1989E+00	0.4971E-01	0.3729E-01	0.0	0.3729E+00	0.1080E+02
SW	0.0	0.2486E-01	0.8700E-01	0.2113E+00	0.1243E+00	0.3729E-01	0.0	0.4847E+00	0.1106E+02
WSW	0.0	0.4971E-01	0.2486E-01	0.1616E+00	0.2361E+00	0.3729E-01	0.2486E-01	0.5344E+00	0.1316E+02
W	0.0	0.0	0.7457E-01	0.1119E+00	0.8700E-01	0.1243E-01	0.1243E-01	0.2983E+00	0.1141E+02
WNW	0.0	0.3729E-01	0.7457E-01	0.2486E+00	0.2361E+00	0.4971E-01	0.1243E-01	0.6587E+00	0.1198E+02
NW	0.0	0.1243E-01	0.1243E+00	0.2486E+00	0.1616E+00	0.0	0.0	0.5469E+00	0.1023E+02
NNW	0.0	0.0	0.4971E-01	0.1616E+00	0.6214E-01	0.1243E-01	0.0	0.2859E+00	0.1091E+02
TOTAL	0.0	0.1740E+00	0.1293E+01	0.2225E+01	0.1143E+01	0.1989E+00	0.4971E-01	0.5083E+01	0.9259E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 9

TABLE 2.3.3-8 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 PM 1/01/77 TO 11:00 PM 12/31/77
UPPER WIND LEVEL
STABILITY CLASS C
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.0	0.2237E+00	0.2237E+00	0.1119E+00	0.0	0.0	0.5593E+00	0.9346E+01
NNE	0.0	0.1243E-01	0.1243E+00	0.1243E+00	0.9943E-01	0.2486E-01	0.0	0.3853E+00	0.1010E+02
NE	0.0	0.0	0.7457E-01	0.6214E-01	0.0	0.0	0.0	0.1367E+00	0.7101E+01
ENE	0.0	0.1243E-01	0.1491E+00	0.1740E+00	0.0	0.0	0.0	0.3356E+00	0.7911E+01
E	0.0	0.1243E-01	0.1491E+00	0.1119E+00	0.0	0.0	0.0	0.2734E+00	0.7268E+01
ESE	0.0	0.1243E-01	0.1740E+00	0.2486E-01	0.0	0.0	0.0	0.2113E+00	0.5849E+01
SE	0.0	0.2486E-01	0.1491E+00	0.3729E-01	0.0	0.0	0.0	0.2113E+00	0.5900E+01
SSE	0.0	0.4971E-01	0.9943E-01	0.1367E+00	0.1243E-01	0.0	0.0	0.2983E+00	0.7096E+01
S	0.0	0.2486E-01	0.1740E+00	0.1864E+00	0.3729E-01	0.1243E-01	0.0	0.4350E+00	0.8391E+01
SSW	0.0	0.3729E-01	0.8700E-01	0.3231E+00	0.1864E+00	0.2486E-01	0.0	0.6587E+00	0.1071E+02
SW	0.0	0.2486E-01	0.1367E+00	0.2859E+00	0.1989E+00	0.2486E-01	0.0	0.6711E+00	0.1069E+02
WSW	0.0	0.2486E-01	0.1616E+00	0.4101E+00	0.1740E+00	0.3729E-01	0.0	0.8079E+00	0.1028E+02
W	0.0	0.2486E-01	0.1119E+00	0.1243E+00	0.8700E-01	0.2486E-01	0.0	0.3729E+00	0.1013E+02
WNW	0.0	0.0	0.9943E-01	0.2610E+00	0.1243E+00	0.8700E-01	0.1243E-01	0.5841E+00	0.1227E+02
NW	0.0	0.2486E-01	0.1491E+00	0.1989E+00	0.1367E+00	0.4971E-01	0.0	0.5593E+00	0.1036E+02
NNW	0.0	0.1243E-01	0.1864E+00	0.1989E+00	0.1616E+00	0.2486E-01	0.0	0.5841E+00	0.1005E+02
TOTAL	0.0	0.2983E+00	0.2250E+01	0.2883E+01	0.1330E+01	0.3107E+00	0.1243E-01	0.7084E+01	0.8965E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 14

TABLE 2.3.3-8 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 PM 1/01/77 TO 11:00 PM 12/31/77
UPPER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1740E+00	0.7954E+00	0.9197E+00	0.2486E+00	0.0	0.0	0.2138E+01	0.8209E+01
NNE	0.0	0.1989E+00	0.9694E+00	0.1305E+01	0.3231E+00	0.0	0.0	0.2796E+01	0.8236E+01
NE	0.0	0.1367E+00	0.6711E+00	0.3604E+00	0.1618E+00	0.0	0.0	0.1330E+01	0.7316E+01
ENE	0.0	0.2237E+00	0.6090E+00	0.8700E+00	0.1491E+00	0.1243E-01	0.0	0.1864E+01	0.7810E+01
E	0.0	0.1989E+00	0.4101E+00	0.5717E+00	0.2486E-01	0.0	0.0	0.1206E+01	0.6965E+01
ESE	0.0	0.1616E+00	0.4101E+00	0.2486E+00	0.1243E-01	0.0	0.0	0.8327E+00	0.6076E+01
SE	0.0	0.1243E+00	0.5220E+00	0.4226E+00	0.9943E-01	0.2486E-01	0.0	0.1193E+01	0.7786E+01
SSE	0.0	0.1243E+00	0.4474E+00	0.6463E+00	0.2859E+00	0.1243E-01	0.0	0.1516E+01	0.9037E+01
S	0.0	0.7457E-01	0.5220E+00	0.8949E+00	0.2361E+00	0.7457E-01	0.1243E-01	0.1815E+01	0.9622E+01
SSW	0.1243E-01	0.2361E+00	0.5717E+00	0.6711E+00	0.4723E+00	0.1864E+00	0.1243E-01	0.2163E+01	0.1026E+02
SW	0.0	0.1243E+00	0.9073E+00	0.8451E+00	0.4599E+00	0.7457E-01	0.6214E-01	0.2473E+01	0.9789E+01
WSW	0.0	0.1491E+00	0.5966E+00	0.8700E+00	0.3356E+00	0.7457E-01	0.3729E-01	0.2063E+01	0.9695E+01
W	0.0	0.1367E+00	0.3231E+00	0.2983E+00	0.2610E+00	0.1119E+00	0.1243E-01	0.1143E+01	0.9991E+01
WNW	0.0	0.1243E+00	0.4101E+00	0.5517E+00	0.3107E+00	0.1989E+00	0.0	0.1616E+01	0.1056E+02
NW	0.0	0.9943E-01	0.3729E+00	0.6587E+00	0.3729E+00	0.1119E+00	0.0	0.1616E+01	0.1042E+02
NNW	0.0	0.1243E+00	0.4226E+00	0.6587E+00	0.1989E+00	0.0	0.0	0.1404E+01	0.8846E+01
TOTAL	0.1243E-01	0.2411E+01	0.8961E+01	0.1081E+02	0.3952E+01	0.8824E+00	0.1367E+00	0.2717E+02	0.8788E+01

NUMBER OF CALMS - 1

NUMBER OF BAD HOURS - 161

TABLE 2.3.3-8 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 PM 1/01/77 TO 11:00 PM 12/31/77
UPPER WIND LEVEL
STABILITY CLASS E
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1243E-01	0.2237E+00	0.6339E+00	0.1119E+00	0.0	0.0	0.9819E+00	0.9391E+01
NNE	0.0	0.2486E-01	0.1864E+00	0.8576E+00	0.1243E+00	0.0	0.0	0.1193E+01	0.9679E+01
NE	0.0	0.3729E-01	0.2486E+00	0.7830E+00	0.7457E-01	0.0	0.0	0.1143E+01	0.9013E+01
ENE	0.0	0.4971E-01	0.3480E+00	0.4971E+00	0.0	0.0	0.0	0.8949E+00	0.7819E+01
E	0.0	0.3729E-01	0.3977E+00	0.6587E+00	0.6214E-01	0.0	0.0	0.1156E+01	0.8409E+01
ESE	0.0	0.1243E-01	0.5096E+00	0.3107E+00	0.3729E-01	0.0	0.0	0.8700E+00	0.7635E+01
SE	0.0	0.6214E-01	0.2983E+00	0.2361E+00	0.0	0.0	0.0	0.5966E+00	0.6860E+01
SSE	0.0	0.2486E-01	0.2859E+00	0.9570E+00	0.1367E+00	0.0	0.0	0.1404E+01	0.9029E+01
S	0.0	0.2486E-01	0.5717E+00	0.1529E+01	0.5593E+00	0.0	0.0	0.2685E+01	0.1001E+02
SSW	0.0	0.3729E-01	0.6587E+00	0.2113E+01	0.7457E+00	0.6214E-01	0.0	0.3617E+01	0.1027E+02
SW	0.0	0.1243E-01	0.5717E+00	0.1168E+01	0.4599E+00	0.4971E-01	0.1243E-01	0.2274E+01	0.1011E+02
WSW	0.0	0.4971E-01	0.3604E+00	0.6090E+00	0.1864E+00	0.9943E-01	0.0	0.1305E+01	0.9829E+01
W	0.0	0.9943E-01	0.1740E+00	0.4350E+00	0.9943E-01	0.0	0.0	0.8079E+00	0.8877E+01
WNW	0.0	0.3729E-01	0.1740E+00	0.5966E+00	0.1740E+00	0.0	0.0	0.9819E+00	0.9777E+01
NW	0.0	0.1243E-01	0.1740E+00	0.7954E+00	0.1367E+00	0.1243E-01	0.0	0.1131E+01	0.9676E+01
NNW	0.0	0.6214E-01	0.2486E+00	0.6214E+00	0.2734E+00	0.0	0.0	0.1206E+01	0.9805E+01
TOTAL	0.0	0.5966E+00	0.5431E+01	0.1280E+02	0.3182E+01	0.2237E+00	0.1243E-01	0.2225E+02	0.9136E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 131

TABLE 2.3.3-8 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 PM 1/01/77 TO 11:00 PM 12/31/77
UPPER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.4971E-01	0.1491E+00	0.3107E+00	0.9943E-01	0.0	0.0	0.6090E+00	0.8908E+01
NNE	0.0	0.0	0.1367E+00	0.2486E+00	0.1119E+00	0.0	0.0	0.4971E+00	0.9969E+01
NE	0.0	0.4971E-01	0.1367E+00	0.3107E+00	0.4971E-01	0.0	0.0	0.5469E+00	0.8473E+01
ENE	0.0	0.4971E-01	0.1367E+00	0.3231E+00	0.6214E-01	0.0	0.0	0.5717E+00	0.8602E+01
E	0.0	0.1119E+00	0.2237E+00	0.3729E+00	0.6214E-01	0.0	0.0	0.7706E+00	0.7943E+01
ESE	0.0	0.1243E-01	0.2859E+00	0.2859E+00	0.1243E-01	0.0	0.0	0.5966E+00	0.7652E+01
SE	0.0	0.3729E-01	0.1119E+00	0.1989E+00	0.2486E-01	0.0	0.0	0.3729E+00	0.7945E+01
SSE	0.0	0.4971E-01	0.2734E+00	0.4599E+00	0.4971E-01	0.0	0.0	0.8327E+00	0.8281E+01
S	0.0	0.4971E-01	0.4599E+00	0.6463E+00	0.2361E+00	0.0	0.0	0.1392E+01	0.8899E+01
SSW	0.0	0.4971E-01	0.3480E+00	0.1206E+01	0.2983E+00	0.0	0.0	0.1902E+01	0.9682E+01
SW	0.0	0.3729E-01	0.1740E+00	0.1069E+01	0.1243E+00	0.0	0.0	0.1404E+01	0.9303E+01
WSW	0.0	0.3729E-01	0.2237E+00	0.6836E+00	0.1491E+00	0.0	0.0	0.1094E+01	0.9328E+01
W	0.0	0.0	0.2113E+00	0.4971E+00	0.1119E+00	0.0	0.0	0.8203E+00	0.9422E+01
WNW	0.0	0.1243E-01	0.1989E+00	0.3356E+00	0.4971E-01	0.0	0.0	0.5966E+00	0.8748E+01
NW	0.0	0.2486E-01	0.2113E+00	0.3231E+00	0.3729E-01	0.0	0.0	0.5966E+00	0.8135E+01
NNW	0.0	0.2486E-01	0.9943E-01	0.2983E+00	0.6214E-01	0.0	0.0	0.4847E+00	0.9178E+01
TOTAL	0.0	0.5966E+00	0.3381E+01	0.7569E+01	0.1541E+01	0.0	0.0	0.1309E+02	0.8779E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 45

TABLE 2.3.3-8 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 PM 1/01/77 TO 11:00 PM 12/31/77
UPPER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.3729E-01	0.3231E+00	0.2610E+00	0.4971E-01	0.0	0.0	0.6711E+00	0.7475E+01
NNE	0.0	0.6214E-01	0.2113E+00	0.3480E+00	0.7457E-01	0.0	0.0	0.6960E+00	0.8105E+01
NE	0.0	0.6214E-01	0.2113E+00	0.2361E+00	0.4971E-01	0.0	0.0	0.5593E+00	0.7659E+01
ENE	0.0	0.8700E-01	0.2237E+00	0.3356E+00	0.7457E-01	0.0	0.0	0.7209E+00	0.7860E+01
E	0.0	0.8700E-01	0.1243E+00	0.3107E+00	0.0	0.0	0.0	0.5220E+00	0.7812E+01
ESE	0.0	0.7457E-01	0.1491E+00	0.3356E+00	0.1243E-01	0.0	0.0	0.5717E+00	0.7606E+01
SE	0.0	0.1119E+00	0.2361E+00	0.3480E+00	0.2486E-01	0.0	0.0	0.7209E+00	0.7493E+01
SSE	0.0	0.1740E+00	0.5717E+00	0.6214E+00	0.1119E+00	0.0	0.0	0.1479E+01	0.7693E+01
S	0.0	0.9943E-01	0.5220E+00	0.6214E+00	0.9943E-01	0.0	0.0	0.1342E+01	0.7918E+01
SSW	0.1243E-01	0.1740E+00	0.5593E+00	0.1243E+01	0.9943E-01	0.0	0.0	0.2088E+01	0.8373E+01
SW	0.0	0.1367E+00	0.6587E+00	0.1056E+01	0.3729E-01	0.0	0.0	0.1889E+01	0.7831E+01
WSW	0.0	0.1616E+00	0.8327E+00	0.1305E+01	0.8700E-01	0.0	0.0	0.2386E+01	0.7823E+01
W	0.0	0.1367E+00	0.5717E+00	0.6339E+00	0.1243E-01	0.0	0.0	0.1355E+01	0.7358E+01
WNW	0.0	0.1616E+00	0.5096E+00	0.5593E+00	0.1119E+00	0.0	0.0	0.1342E+01	0.7380E+01
NW	0.0	0.1119E+00	0.5717E+00	0.2610E+00	0.0	0.0	0.0	0.9446E+00	0.6292E+01
NNW	0.0	0.1243E+00	0.4226E+00	0.4226E+00	0.4971E-01	0.0	0.0	0.1019E+01	0.7295E+01
TOTAL	0.1243E-01	0.1802E+01	0.6699E+01	0.8899E+01	0.8949E+00	0.0	0.0	0.1831E+02	0.7623E+01

NUMBER OF CALMS - 1

NUMBER OF BAD HOURS - 101

TABLE 2.3.3-8 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 PM 1/01/77 TO 11:00 PM 12/31/77
UPPER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
	0.0	0.2859E+00	0.2001E+01	0.2809E+01	0.7830E+00	0.2486E-01	0.0	0.5904E+01	0.8725E+01
NNE	0.0	0.3107E+00	0.1827E+01	0.3430E+01	0.8451E+00	0.2486E-01	0.0	0.6438E+01	0.8880E+01
NE	0.0	0.3107E+00	0.1491E+01	0.2063E+01	0.3729E+00	0.0	0.0	0.4238E+01	0.8103E+01
ENE	0.0	0.4474E+00	0.1578E+01	0.2337E+01	0.2859E+00	0.1243E-01	0.0	0.4661E+01	0.7910E+01
E	0.0	0.4723E+00	0.1479E+01	0.2138E+01	0.1616E+00	0.0	0.0	0.4251E+01	0.7668E+01
ESE	0.0	0.3231E+00	0.1740E+01	0.1230E+01	0.7457E-01	0.0	0.0	0.3368E+01	0.6955E+01
SE	0.0	0.4101E+00	0.1429E+01	0.1255E+01	0.1491E+00	0.2486E-01	0.0	0.3269E+01	0.7286E+01
SSE	0.0	0.4474E+00	0.1864E+01	0.2908E+01	0.5966E+00	0.1243E-01	0.0	0.5829E+01	0.8354E+01
S	0.0	0.2859E+00	0.2361E+01	0.4201E+01	0.1218E+01	0.8700E-01	0.1243E-01	0.8166E+01	0.9250E+01
SSW	0.2486E-01	0.5344E+00	0.2411E+01	0.6127E+01	0.2100E+01	0.3604E+00	0.1243E-01	0.1157E+02	0.9952E+01
SW	0.0	0.3853E+00	0.2722E+01	0.4884E+01	0.1653E+01	0.3480E+00	0.7457E-01	0.1007E+02	0.9764E+01
WSW	0.0	0.4847E+00	0.2287E+01	0.4226E+01	0.1355E+01	0.2983E+00	0.6214E-01	0.8712E+01	0.9543E+01
W	0.0	0.4226E+00	0.1603E+01	0.2250E+01	0.7830E+00	0.1740E+00	0.3729E-01	0.5270E+01	0.9223E+01
WNW	0.0	0.3729E+00	0.1541E+01	0.2983E+01	0.1293E+01	0.3604E+00	0.4971E-01	0.6600E+01	0.1013E+02
NW	0.0	0.2983E+00	0.1690E+01	0.2772E+01	0.1056E+01	0.1740E+00	0.0	0.5991E+01	0.9422E+01
NNW	0.0	0.3604E+00	0.1491E+01	0.2759E+01	0.1007E+01	0.4971E-01	0.0	0.5667E+01	0.9315E+01
TOTAL	0.2486E-01	0.6152E+01	0.2952E+02	0.4837E+02	0.1373E+02	0.1951E+01	0.2486E+00	0.1000E+03	0.9052E+01

NUMBER OF CALMS - 2

NUMBER OF BAD HOURS - 714

TABLE 2.3.3-9
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/77 TO 11:00 PM 12/31/77
LOWER WIND LEVEL
STABILITY CLASS A
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.2440E-01	0.3415E+00	0.1220E+00	0.1220E-01	0.0	0.0	0.5001E+00	0.6848E+01
NNE	0.0	0.1220E-01	0.2684E+00	0.1220E+00	0.0	0.0	0.0	0.4025E+00	0.6622E+01
NE	0.0	0.3659E-01	0.2196E+00	0.2440E-01	0.0	0.0	0.0	0.2806E+00	0.5902E+01
ENE	0.0	0.1220E-01	0.1220E+00	0.3659E-01	0.0	0.0	0.0	0.1708E+00	0.6070E+01
E	0.0	0.2440E-01	0.4879E-01	0.3659E-01	0.0	0.0	0.0	0.1098E+00	0.5948E+01
ESE	0.0	0.3659E-01	0.8539E-01	0.0	0.0	0.0	0.0	0.1220E+00	0.4131E+01
SE	0.0	0.6099E-01	0.8539E-01	0.0	0.0	0.0	0.0	0.1464E+00	0.4506E+01
SSE	0.0	0.1220E-01	0.9758E-01	0.1220E-01	0.0	0.0	0.0	0.1220E+00	0.5170E+01
S	0.0	0.2440E-01	0.1586E+00	0.1220E-01	0.0	0.0	0.0	0.1952E+00	0.5538E+01
SSW	0.0	0.4879E-01	0.2562E+00	0.3050E+00	0.2440E-01	0.0	0.0	0.6343E+00	0.7461E+01
SW	0.0	0.3659E-01	0.1586E+00	0.3537E+00	0.1586E+00	0.0	0.0	0.7075E+00	0.9289E+01
WSW	0.0	0.1220E-01	0.2928E+00	0.2928E+00	0.9758E-01	0.0	0.0	0.6953E+00	0.8419E+01
W	0.0	0.2440E-01	0.2318E+00	0.1342E+00	0.6099E-01	0.1220E-01	0.0	0.4635E+00	0.8208E+01
WNW	0.0	0.2440E-01	0.1708E+00	0.3781E+00	0.8539E-01	0.1220E-01	0.0	0.6709E+00	0.9240E+01
NW	0.0	0.1220E-01	0.2074E+00	0.4513E+00	0.3659E-01	0.0	0.0	0.7075E+00	0.8557E+01
NNW	0.0	0.1220E-01	0.1830E+00	0.5611E+00	0.7319E-01	0.0	0.0	0.8295E+00	0.8967E+01
TOTAL	0.0	0.4147E+00	0.2928E+01	0.2842E+01	0.5489E+00	0.2440E-01	0.0	0.6758E+01	0.6933E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 264

TABLE 2.3.3-9 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/77 TO 11:00 PM 12/31/77
LOWER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.0	0.2928E+00	0.1220E+00	0.0	0.0	0.0	0.4147E+00	0.6906E+01
NNE	0.0	0.0	0.2313E+00	0.3659E-01	0.0	0.0	0.0	0.2684E+00	0.6577E+01
NE	0.0	0.2440E-01	0.1586E+00	0.3659E-01	0.0	0.0	0.0	0.2196E+00	0.5791E+01
ENE	0.0	0.2440E-01	0.9758E-01	0.0	0.0	0.0	0.0	0.1220E+00	0.5794E+01
E	0.0	0.2440E-01	0.1220E+00	0.3659E-01	0.0	0.0	0.0	0.1830E+00	0.5497E+01
ESE	0.0	0.0	0.1098E+00	0.1220E-01	0.0	0.0	0.0	0.1220E+00	0.5653E+01
SE	0.0	0.0	0.7319E-01	0.0	0.0	0.0	0.0	0.7319E-01	0.4252E+01
SSE	0.0	0.1220E-01	0.4879E-01	0.2440E-01	0.0	0.0	0.0	0.8539E-01	0.5971E+01
S	0.0	0.2440E-01	0.1586E+00	0.4879E-01	0.0	0.0	0.0	0.2318E+00	0.6118E+01
SSW	0.0	0.1220E-01	0.2684E+00	0.6099E-01	0.2440E-01	0.0	0.0	0.3659E+00	0.6998E+01
SW	0.0	0.1220E-01	0.1830E+00	0.2074E+00	0.2440E-01	0.0	0.0	0.4269E+00	0.8086E+01
WSW	0.0	0.4879E-01	0.9758E-01	0.3415E+00	0.4879E-01	0.1220E-01	0.1220E-01	0.5611E+00	0.9523E+01
W	0.0	0.1220E-01	0.8539E-01	0.9758E-01	0.1220E-01	0.1220E-01	0.0	0.2196E+00	0.8351E+01
WNW	0.0	0.1220E-01	0.1830E+00	0.2928E+00	0.9758E-01	0.1220E-01	0.0	0.5977E+00	0.9532E+01
NW	0.0	0.2440E-01	0.2074E+00	0.3537E+00	0.4879E-01	0.0	0.0	0.6343E+00	0.8429E+01
NNW	0.0	0.0	0.1586E+00	0.1464E+00	0.3659E-01	0.0	0.0	0.3415E+00	0.8239E+01
TOTAL	0.0	0.2318E+00	0.2476E+01	0.1818E+01	0.2928E+00	0.3659E-01	0.1220E-01	0.4867E+01	0.6982E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 19

TABLE 2.3.3-9 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/77 TO 11:00 PM 12/31/77
LOWER WIND LEVEL
STABILITY CLASS C
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1220E-01	0.3537E+00	0.1952E+00	0.0	0.0	0.0	0.5611E+00	0.6748E+01
NNE	0.0	0.0	0.1708E+00	0.8539E-01	0.0	0.0	0.0	0.2562E+00	0.6783E+01
NE	0.0	0.3659E-01	0.1342E+00	0.1220E-01	0.0	0.0	0.0	0.1830E+00	0.4917E+01
ENE	0.0	0.3659E-01	0.2562E+00	0.2440E-01	0.0	0.0	0.0	0.3172E+00	0.5308E+01
E	0.0	0.7319E-01	0.2318E+00	0.1220E-01	0.0	0.0	0.0	0.3172E+00	0.4927E+01
ESE	0.0	0.1220E-01	0.1830E+00	0.1220E-01	0.0	0.0	0.0	0.2074E+00	0.5044E+01
SE	0.0	0.1220E-01	0.1830E+00	0.1220E-01	0.0	0.0	0.0	0.2074E+00	0.5155E+01
SSE	0.0	0.2440E-01	0.2806E+00	0.1220E-01	0.0	0.0	0.0	0.3172E+00	0.5365E+01
S	0.0	0.3659E-01	0.2440E+00	0.7319E-01	0.0	0.0	0.0	0.3537E+00	0.5831E+01
SSW	0.0	0.3659E-01	0.3781E+00	0.1464E+00	0.2440E-01	0.0	0.0	0.5855E+00	0.6864E+01
SW	0.0	0.4879E-01	0.3415E+00	0.2562E+00	0.0	0.0	0.0	0.6465E+00	0.7528E+01
WSW	0.0	0.0	0.3537E+00	0.3903E+00	0.2440E-01	0.0	0.0	0.7685E+00	0.8151E+01
W	0.0	0.2440E-01	0.2318E+00	0.7319E-01	0.0	0.0	0.0	0.3293E+00	0.6197E+01
WNW	0.0	0.2440E-01	0.2196E+00	0.2196E+00	0.1342E+00	0.1220E-01	0.0	0.6099E+00	0.9273E+01
NW	0.0	0.1220E-01	0.2684E+00	0.2684E+00	0.1220E+00	0.0	0.0	0.6709E+00	0.8374E+01
NNW	0.0	0.0	0.3050E+00	0.2684E+00	0.4879E-01	0.0	0.0	0.6221E+00	0.7948E+01
TOTAL	0.0	0.3903E+00	0.4135E+01	0.2061E+01	0.3537E+00	0.1220E-01	0.0	0.6953E+01	0.6526E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 14[n]

TABLE 2.3.3-9 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/77 TO 11:00 PM 12/31/77
LOWER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.2440E-01	0.4391E+00	0.1574E+01	0.3537E+00	0.0	0.0	0.0	0.2391E+01	0.5141E+01
NNE	0.2440E-01	0.6343E+00	0.1696E+01	0.1342E+00	0.0	0.0	0.0	0.2488E+01	0.4669E+01
NE	0.2440E-01	0.4757E+00	0.5977E+00	0.4879E-01	0.0	0.0	0.0	0.1147E+01	0.4004E+01
ENE	0.2440E-01	0.4147E+00	0.1110E+01	0.9758E-01	0.0	0.0	0.0	0.1647E+01	0.4875E+01
E	0.0	0.2928E+00	0.8295E+00	0.1098E+00	0.0	0.0	0.0	0.1232E+01	0.4839E+01
ESE	0.2440E-01	0.4025E+00	0.5245E+00	0.0	0.0	0.0	0.0	0.9515E+00	0.3943E+01
SE	0.0	0.2196E+00	0.7929E+00	0.1952E+00	0.1220E-01	0.0	0.0	0.1220E+01	0.5399E+01
SSE	0.0	0.3293E+00	0.9271E+00	0.2928E+00	0.1220E-01	0.0	0.0	0.1561E+01	0.5576E+01
S	0.0	0.2806E+00	0.9880E+00	0.2440E+00	0.3659E-01	0.0	0.0	0.1549E+01	0.5709E+01
SSW	0.2440E-01	0.3903E+00	0.1183E+01	0.4025E+00	0.1098E+00	0.0	0.0	0.2110E+01	0.6284E+01
SW	0.1220E-01	0.3537E+00	0.1293E+01	0.6343E+00	0.1098E+00	0.2440E-01	0.0	0.2427E+01	0.6592E+01
WSW	0.2440E-01	0.4635E+00	0.1244E+01	0.5123E+00	0.1098E+00	0.6099E-01	0.0	0.2415E+01	0.6544E+01
W	0.1220E-01	0.3415E+00	0.4879E+00	0.1952E+00	0.1098E+00	0.0	0.0	0.1147E+01	0.6241E+01
WNW	0.0	0.2806E+00	0.8539E+00	0.5245E+00	0.1830E+00	0.0	0.0	0.1842E+01	0.7095E+01
NW	0.0	0.2318E+00	0.7441E+00	0.6099E+00	0.1952E+00	0.0	0.0	0.1781E+01	0.7463E+01
NNW	0.0	0.2928E+00	0.9149E+00	0.5123E+00	0.1220E-01	0.0	0.0	0.1732E+01	0.6091E+01
TOTAL	0.1952E+00	0.5843E+01	0.1576E+02	0.4867E+01	0.8905E+00	0.8539E-01	0.0	0.2764E+02	0.5654E+01

NUMBER OF CALMS - 16

NUMBER OF BAD HOURS - 81

TABLE 2.3.3-9 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/77 TO 11:00 PM 12/31/77
LOWER WIND LEVEL
STABILITY CLASS E
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.5489E+00	0.5245E+00	0.2440E-01	0.0	0.0	0.0	0.1098E+01	0.3806E+01
NNE	0.1220E-01	0.6465E+00	0.5733E+00	0.1220E-01	0.0	0.0	0.0	0.1244E+01	0.3627E+01
NE	0.1220E-01	0.7075E+00	0.4391E+00	0.2440E-01	0.0	0.0	0.0	0.1183E+01	0.3475E+01
ENE	0.1220E-01	0.5977E+00	0.5001E+00	0.4879E-01	0.0	0.0	0.0	0.1159E+01	0.3841E+01
E	0.1220E-01	0.6221E+00	0.3415E+00	0.2440E-01	0.2440E-01	0.0	0.0	0.1025E+01	0.3538E+01
ESE	0.0	0.4757E+00	0.3293E+00	0.4879E-01	0.0	0.0	0.0	0.8539E+00	0.3696E+01
SE	0.1220E-01	0.6221E+00	0.3537E+00	0.1220E-01	0.0	0.0	0.0	0.1000E+01	0.3173E+01
SSE	0.2440E-01	0.9393E+00	0.4757E+00	0.1220E-01	0.0	0.0	0.0	0.1452E+01	0.3204E+01
S	0.2440E-01	0.1330E+01	0.9393E+00	0.1098E+00	0.0	0.0	0.0	0.2403E+01	0.3596E+01
SSW	0.2440E-01	0.1610E+01	0.1439E+01	0.1220E+00	0.1220E-01	0.0	0.0	0.3208E+01	0.3893E+01
SW	0.1220E-01	0.8417E+00	0.1061E+01	0.2684E+00	0.2440E-01	0.0	0.0	0.2208E+01	0.4802E+01
WSW	0.0	0.4269E+00	0.6587E+00	0.1586E+00	0.2440E-01	0.0	0.0	0.1269E+01	0.4943E+01
W	0.0	0.2196E+00	0.5245E+00	0.4879E-01	0.1220E-01	0.0	0.0	0.8051E+00	0.4792E+01
WNW	0.0	0.2928E+00	0.5123E+00	0.1220E+00	0.0	0.0	0.0	0.9271E+00	0.4817E+01
NW	0.0	0.3537E+00	0.8173E+00	0.8539E-01	0.0	0.0	0.0	0.1256E+01	0.4615E+01
NNW	0.0	0.3781E+00	0.8295E+00	0.1464E+00	0.1220E-01	0.0	0.0	0.1366E+01	0.4790E+01
TOTAL	0.1464E+00	0.1061E+02	0.1032E+02	0.1269E+01	0.1098E+00	0.0	0.0	0.2246E+02	0.4038E+01

NUMBER OF CALMS - 12

NUMBER OF BAD HOURS - 80

TABLE 2.3.3-9 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/77 TO 11:00 PM 12/31/77
LOWER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.2440E-01	0.5489E+00	0.1464E+00	0.0	0.0	0.0	0.0	0.7197E+00	0.2701E+01
NNE	0.2440E-01	0.5733E+00	0.3659E-01	0.0	0.0	0.0	0.0	0.6343E+00	0.2339E+01
NE	0.3659E-01	0.8295E+00	0.4679E-01	0.0	0.0	0.0	0.0	0.9149E+00	0.2200E+01
ENE	0.3659E-01	0.7319E+00	0.7319E-01	0.0	0.0	0.0	0.0	0.8417E+00	0.2191E+01
E	0.2440E-01	0.5245E+00	0.8539E-01	0.0	0.0	0.0	0.0	0.6343E+00	0.2296E+01
ESE	0.3659E-01	0.6099E+00	0.0	0.0	0.0	0.0	0.0	0.6465E+00	0.1864E+01
SE	0.2440E-01	0.5489E+00	0.2440E-01	0.0	0.0	0.0	0.0	0.5977E+00	0.1890E+01
SSE	0.3659E-01	0.8539E+00	0.6099E-01	0.0	0.0	0.0	0.0	0.9515E+00	0.1996E+01
S	0.4879E-01	0.9880E+00	0.1342E+00	0.0	0.0	0.0	0.0	0.1171E+01	0.2212E+01
SSW	0.6099E-01	0.1220E+01	0.2074E+00	0.0	0.0	0.0	0.0	0.1488E+01	0.2329E+01
SW	0.4879E-01	0.1000E+01	0.1708E+00	0.0	0.0	0.0	0.0	0.1220E+01	0.2255E+01
WSW	0.2440E-01	0.5367E+00	0.2196E+00	0.0	0.0	0.0	0.0	0.7807E+00	0.2820E+01
W	0.0	0.4635E+00	0.1830E+00	0.0	0.0	0.0	0.0	0.6465E+00	0.2888E+01
WNW	0.0	0.3659E+00	0.1220E+00	0.0	0.0	0.0	0.0	0.4879E+00	0.2722E+01
NW	0.0	0.3537E+00	0.1342E+00	0.0	0.0	0.0	0.0	0.4879E+00	0.2698E+01
NNW	0.1220E-01	0.4757E+00	0.2074E+00	0.0	0.0	0.0	0.0	0.6953E+00	0.2791E+01
TOTAL	0.4391E+00	0.1062E+02	0.1854E+01	0.0	0.0	0.0	0.0	0.1292E+02	0.2387E+01

NUMBER OF CALMS - 36

NUMBER OF BAD HOURS - 39

TABLE 2.3.3-9 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/77 TO 11:00 PM 12/31/77
LOWER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.5733E+00	0.1378E+01	0.7319E-01	0.0	0.0	0.0	0.0	0.2025E+01	0.1672E+01
NNE	0.5855E+00	0.1403E+01	0.0	0.0	0.0	0.0	0.0	0.1988E+01	0.1409E+01
NE	0.4635E+00	0.1122E+01	0.2440E-01	0.0	0.0	0.0	0.0	0.1610E+01	0.1549E+01
ENE	0.4757E+00	0.1147E+01	0.1220E-01	0.0	0.0	0.0	0.0	0.1635E+01	0.1445E+01
E	0.3659E+00	0.8783E+00	0.0	0.0	0.0	0.0	0.0	0.1244E+01	0.1275E+01
ESE	0.2562E+00	0.6221E+00	0.2440E-01	0.0	0.0	0.0	0.0	0.9027E+00	0.1566E+01
SE	0.2562E+00	0.5977E+00	0.0	0.0	0.0	0.0	0.0	0.8539E+00	0.1352E+01
SSE	0.1952E+00	0.4635E+00	0.7319E-01	0.0	0.0	0.0	0.0	0.7319E+00	0.1940E+01
S	0.3293E+00	0.7807E+00	0.1342E+00	0.3659E-01	0.0	0.0	0.0	0.1281E+01	0.2126E+01
SSW	0.2684E+00	0.6465E+00	0.4879E-01	0.0	0.0	0.0	0.0	0.9636E+00	0.1769E+01
SW	0.2806E+00	0.6709E+00	0.1220E-01	0.1220E-01	0.0	0.0	0.0	0.9758E+00	0.1680E+01
WSW	0.2074E+00	0.4757E+00	0.7319E-01	0.0	0.0	0.0	0.0	0.7563E+00	0.1841E+01
W	0.2074E+00	0.4879E+00	0.8539E-01	0.0	0.0	0.0	0.0	0.7807E+00	0.1928E+01
WNW	0.8539E-01	0.4513E+00	0.4879E-01	0.0	0.0	0.0	0.0	0.5855E+00	0.2139E+01
NW	0.1952E+00	0.4635E+00	0.4879E-01	0.1220E-01	0.0	0.0	0.0	0.7197E+00	0.2055E+01
NNW	0.3903E+00	0.9271E+00	0.3659E-01	0.0	0.0	0.0	0.0	0.1354E+01	0.1481E+01
TOTAL	0.5135E+01	0.1252E+02	0.6953E+00	0.6099E-01	0.0	0.0	0.0	0.1841E+02	0.1702E+01

NUMBER OF CALMS - 421

NUMBER OF BAD HOURS - 65

TABLE 2.3.3-9 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/77 TO 11:00 PM 12/31/77
LOWER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.6221E+00	0.2952E+01	0.3306E+01	0.8173E+00	0.1220E-01	0.0	0.0	0.7709E+01	0.4335E+01
NNE	0.6465E+00	0.3269E+01	0.2976E+01	0.3903E+00	0.0	0.0	0.0	0.7282E+01	0.3849E+01
NE	0.5367E+00	0.3232E+01	0.1622E+01	0.1464E+00	0.0	0.0	0.0	0.5538E+01	0.3219E+01
ENE	0.5489E+00	0.2964E+01	0.2171E+01	0.2074E+00	0.0	0.0	0.0	0.5892E+01	0.3590E+01
E	0.4025E+00	0.2440E+01	0.1659E+01	0.2196E+00	0.2440E-01	0.0	0.0	0.4745E+01	0.3520E+01
ESE	0.3172E+00	0.2159E+01	0.1256E+01	0.7319E-01	0.0	0.0	0.0	0.3806E+01	0.3210E+01
SE	0.2928E+00	0.2061E+01	0.1513E+01	0.2196E+00	0.1220E-01	0.0	0.0	0.4099E+01	0.3587E+01
SSE	0.2562E+00	0.2635E+01	0.1964E+01	0.3537E+00	0.1220E-01	0.0	0.0	0.5221E+01	0.3825E+01
S	0.4025E+00	0.3464E+01	0.2757E+01	0.5245E+00	0.3659E-01	0.0	0.0	0.7185E+01	0.3903E+01
SSW	0.3781E+00	0.3964E+01	0.3781E+01	0.1037E+01	0.1952E+00	0.0	0.0	0.9356E+01	0.4608E+01
SW	0.3537E+00	0.2964E+01	0.3220E+01	0.1732E+01	0.3172E+00	0.2440E-01	0.0	0.8612E+01	0.5469E+01
WSW	0.2562E+00	0.1964E+01	0.2940E+01	0.1696E+01	0.3050E+00	0.7319E-01	0.1220E-01	0.7246E+01	0.6083E+01
W	0.2196E+00	0.1574E+01	0.1830E+01	0.5489E+00	0.1952E+00	0.2440E-01	0.0	0.4391E+01	0.5175E+01
WNW	0.8539E-01	0.1452E+01	0.2110E+01	0.1537E+01	0.5001E+00	0.3659E-01	0.0	0.5721E+01	0.6651E+01
NW	0.1952E+00	0.1452E+01	0.2427E+01	0.1781E+01	0.4025E+00	0.0	0.0	0.6258E+01	0.6351E+01
NNW	0.4025E+00	0.2086E+01	0.2635E+01	0.1635E+01	0.1830E+00	0.0	0.0	0.6941E+01	0.5449E+01
TOTAL	0.5916E+01	0.4063E+02	0.3817E+02	0.1292E+02	0.2196E+01	0.1586E+00	0.1220E-01	0.1000E+01	0.4670E+01

NUMBER OF CALMS - 485

NUMBER OF BAD HOURS - 562

TABLE 2.3.3-10
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS A
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.0	0.1282E+00	0.3731E+00	0.6995E-01	0.0	0.0	0.5713E+00	0.9878E+01
NNE	0.0	0.1166E-01	0.2099E+00	0.2448E+00	0.1399E+00	0.0	0.0	0.6063E+00	0.9401E+01
NE	0.0	0.1166E-01	0.9327E-01	0.1632E+00	0.5830E-01	0.0	0.0	0.3265E+00	0.9595E+01
ENE	0.0	0.0	0.2332E-01	0.8161E-01	0.0	0.0	0.0	0.1049E+00	0.8786E+01
E	0.0	0.1166E-01	0.9327E-01	0.1166E-01	0.0	0.0	0.0	0.1166E+00	0.5684E+01
ESE	0.0	0.0	0.3498E-01	0.0	0.0	0.0	0.0	0.3498E-01	0.5630E+01
SE	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
SSE	0.0	0.0	0.1166E-01	0.2332E-01	0.0	0.0	0.0	0.3498E-01	0.6827E+01
S	0.0	0.1166E-01	0.4664E-01	0.2332E-01	0.1166E-01	0.0	0.0	0.9327E-01	0.8262E+01
SSW	0.0	0.0	0.3498E-01	0.6995E-01	0.6995E-01	0.1166E-01	0.0	0.1865E+00	0.1184E+02
SW	0.0	0.0	0.1166E+00	0.2448E+00	0.1048E+00	0.1166E-01	0.2332E-01	0.5013E+00	0.1075E+02
WSW	0.0	0.0	0.1749E+00	0.1516E+00	0.1166E-01	0.3498E-01	0.4664E-01	0.4197E+00	0.1145E+02
W	0.0	0.0	0.8161E-01	0.3498E-01	0.4664E-01	0.2332E-01	0.1166E-01	0.1982E+00	0.1197E+02
WNW	0.0	0.0	0.5838E-01	0.1399E+00	0.2565E+00	0.4664E-01	0.1166E-01	0.5130E+00	0.1380E+02
NW	0.0	0.0	0.8161E-01	0.2448E+00	0.1049E+00	0.2332E-01	0.0	0.4547E+00	0.1127E+02
NNW	0.0	0.0	0.1049E+00	0.2215E+00	0.3498E-01	0.0	0.0	0.3614E+00	0.9000E+01
TOTAL	0.0	0.4664E-01	0.1294E+01	0.2029E+01	0.9094E+00	0.1516E+00	0.9327E-01	0.4524E+01	0.9609E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 46

TABLE 2.3.3-10 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1166E-01	0.1282E+00	0.9327E-01	0.0	0.0	0.0	0.2332E+00	0.7468E+01
NNE	0.0	0.0	0.1749E+00	0.1282E+00	0.5830E-01	0.0	0.0	0.3614E+00	0.8552E+01
NE	0.0	0.1166E-01	0.1049E+00	0.1516E+00	0.3498E-01	0.0	0.0	0.3031E+00	0.8522E+01
ENE	0.0	0.0	0.5330E-01	0.4664E-01	0.0	0.0	0.0	0.1049E+00	0.7423E+01
E	0.0	0.1166E-01	0.3498E-01	0.3498E-01	0.0	0.0	0.0	0.8161E-01	0.6653E+01
ESE	0.0	0.3498E-01	0.8161E-01	0.3498E-01	0.0	0.0	0.0	0.1516E+00	0.5654E+01
SE	0.0	0.0	0.0	0.1166E-01	0.0	0.0	0.0	0.1166E-01	0.1010E+02
SSE	0.0	0.2332E-01	0.4664E-01	0.3498E-01	0.0	0.0	0.0	0.1049E+00	0.6200E+01
S	0.0	0.0	0.6995E-01	0.4664E-01	0.0	0.0	0.0	0.1166E+00	0.7067E+01
SSW	0.0	0.3498E-01	0.1166E+00	0.1166E+00	0.2332E-01	0.2332E-01	0.0	0.3143E+00	0.8434E+01
SW	0.0	0.3498E-01	0.1632E+00	0.1049E+00	0.0	0.1166E-01	0.0	0.3148E+00	0.6862E+01
WSW	0.0	0.1166E-01	0.1865E+00	0.1282E+00	0.0	0.0	0.1166E-01	0.3381E+00	0.7542E+01
W	0.0	0.1166E-01	0.3498E-01	0.2332E-01	0.1166E-01	0.1166E-01	0.0	0.9327E-01	0.9521E+01
WNW	0.0	0.1166E-01	0.1049E+00	0.8161E-01	0.6995E-01	0.2332E-01	0.1166E-01	0.3031E+00	0.1140E+02
NW	0.0	0.0	0.8161E-01	0.1166E+00	0.5830E-01	0.1166E-01	0.0	0.2682E+00	0.1048E+02
NNW	O.O	O.O	O.4664E-01	0.8161E-01	0.1166E-01	0.0	0.0	0.1399E+00	0.9046E+01
TOTAL	0.0	0.1982E+00	0.1434E+01	0.1236E+01	0.2682E+00	0.8161E-01	0.2332E-01	0.3241E+01	0.8183E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 0

TABLE 2.3.3-10 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS C
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.3498E-01	0.1399E+00	0.1166E+00	0.6995E-01	0.0	0.0	0.3614E+00	0.8307E+01
NNE	0.0	0.0	0.1399E+00	0.9327E-01	0.3493E-01	0.0	0.0	0.2682E+00	0.8083E+01
NE	0.0	0.0	0.1632E+00	0.1166E+00	0.1166E-01	0.0	0.0	0.2915E+00	0.7312E+01
ENE	0.0	0.0	0.1166E+00	0.2332E-01	0.2332E-01	0.0	0.0	0.1632E+00	0.7035E+01
E	0.0	0.0	0.1049E+00	0.3498E-01	0.0	0.0	0.0	0.1399E+00	0.6037E+01
ESE	0.0	0.2332E-01	0.6995E-01	0.0	0.0	0.0	0.0	0.9327E-01	0.4534E+01
SE	0.0	0.0	0.3498E-01	0.0	0.0	0.0	0.0	0.3498E-01	0.4860E+01
SSE	0.0	0.2332E-01	0.4664E-01	0.6995E-01	0.0	0.0	0.0	0.1399E+00	0.7357E+01
S	0.0	0.1166E-01	0.1399E+00	0.5830E-01	0.0	0.0	0.0	0.2099E+00	0.6641E+01
SSW	0.0	0.6995E-01	0.1399E+00	0.8161E-01	0.1166E-01	0.5830E-01	0.0	0.3614E+00	0.8742E+01
SW	0.0	0.4664E-01	0.2215E+00	0.1282E+00	0.2332E-01	0.3498E-01	0.3498E-01	0.4897E+00	0.9669E+01
WSW	0.0	0.4664E-01	0.1865E+00	0.1166E+00	0.0	0.1166E-01	0.1166E-01	0.3731E+00	0.7327E+01
W	0.0	0.1166E-01	0.1282E+00	0.8161E-01	0.2332E-01	0.0	0.0	0.2443E+00	0.7238E+01
WNW	0.0	0.3498E-01	0.8161E-01	0.9327E-01	0.9327E-01	0.3498E-01	0.0	0.3381E+00	0.1045E+02
NW	0.0	0.2332E-01	0.1049E+00	0.1282E+00	0.5830E-01	0.1166E-01	0.0	0.3265E+00	0.9094E+01
NNW	0.0	0.2332E-01	0.9327E-01	0.1166E+00	0.5830E-01	0.0	0.0	0.2915E+00	0.8877E+01
TOTAL	0.0	0.3498E+00	0.1912E+01	0.1859E+01	0.4081E+00	0.1516E+00	0.4664E-01	0.4127E+01	0.7598E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 0

TABLE 2.3.3-10 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1282E+00	0.1131E+01	0.1422E+01	0.3381E+00	0.6995E-01	0.0	0.3090E+01	0.8646E+01
NNE	0.0	0.1516E+00	0.1108E+01	0.1352E+01	0.2448E+00	0.2332E-01	0.0	0.2880E+01	0.8281E+01
NE	0.1166E-01	0.2332E+00	0.8395E+00	0.1038E+01	0.2332E+00	0.0	0.0	0.2355E+01	0.7921E+01
ENE	0.0	0.1632E+00	0.7812E+00	0.3148E+00	0.1049E+00	0.0	0.0	0.1364E+01	0.6609E+01
E	0.0	0.1282E+00	0.6995E+00	0.2448E+00	0.1166E-01	0.0	0.0	0.1084E+01	0.6308E+01
ESE	0.0	0.9327E-01	0.4314E+00	0.3731E+00	0.1166E-01	0.0	0.0	0.9094E+00	0.6871E+01
SE	0.0	0.1166E+00	0.3847E+00	0.1399E+00	0.1166E-01	0.0	0.0	0.6529E+00	0.6099E+01
SSE	0.0	0.9327E-01	0.4664E+00	0.2099E+00	0.1282E+00	0.2332E-01	0.4664E-01	0.9677E+00	0.8637E+01
S	0.0	0.1749E+00	0.5013E+00	0.3381E+00	0.1282E+00	0.2332E-01	0.0	0.1166E+01	0.7571E+01
SSW	0.0	0.1166E+00	0.7112E+00	0.6296E+00	0.3731E+00	0.2798E+00	0.8161E-01	0.2192E+01	0.1105E+02
SW	0.0	0.1749E+00	0.1014E+01	0.9794E+00	0.3498E+00	0.2099E+00	0.9327E-01	0.2821E+01	0.9987E+01
WSW	0.0	0.1399E+00	0.8161E+00	0.9211E+00	0.1865E+00	0.3498E-01	0.4664E-01	0.2145E+01	0.8788E+01
W	0.0	0.1049E+00	0.5830E+00	0.5946E+00	0.1516E+00	0.2332E-01	0.0	0.1457E+01	0.8136E+01
WNW	0.0	0.1166E+00	0.3731E+00	0.6646E+00	0.4547E+00	0.1166E+00	0.3948E-01	0.1761E+01	0.1077E+02
NW	0.0	0.1399E+00	0.5713E+00	0.5830E+00	0.5247E+00	0.6995E-01	0.0	0.1889E+01	0.9755E+01
NNW	0.0	0.9327E-01	0.7462E+00	0.8977E+00	0.3265E+00	0.2332E-01	0.0	0.2087E+01	0.8723E+01
TOTAL	0.1166E-01	0.2169E+01	0.1116E+02	0.1070E+02	0.3579E+01	0.8977E+00	0.3031E+00	0.2882E+02	0.8384E+01

NUMBER OF CALMS - 1

NUMBER OF BAD HOURS - 72

TABLE 2.3.3-10 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS E
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1049E+00	0.4081E+00	0.1154E+01	0.3031E+00	0.1282E+00	0.0	0.2099E+01	0.1003E+02
NNE	0.0	0.9327E-01	0.5013E+00	0.1481E+01	0.3265E+00	0.0	0.0	0.2402E+01	0.9443E+01
NE	0.0	0.1516E+00	0.5713E+00	0.1178E+01	0.3265E+00	0.0	0.0	0.2227E+01	0.8957E+01
ENE	0.0	0.8161E-01	0.4780E+00	0.5946E+00	0.1632E+00	0.4664E-01	0.0	0.1364E+01	0.8876E+01
E	0.0	0.1282E+00	0.5130E+00	0.4547E+00	0.1166E+00	0.0	0.0	0.1213E+01	0.7722E+01
ESE	0.0	0.6995E-01	0.4897E+00	0.4197E+00	0.3498E-01	0.0	0.0	0.1014E+01	0.7474E+01
SE	0.0	0.6995E-01	0.3498E+00	0.3148E+00	0.3498E-01	0.0	0.0	0.7695E+00	0.7429E+01
SSE	0.0	0.8161E-01	0.5830E+00	0.6648E+00	0.2099E+00	0.3498E-01	0.5830E-01	0.1632E+01	0.9244E+01
S	0.0	0.1166E+00	0.5596E+00	0.1178E+01	0.3964E+00	0.8161E-01	0.1166E-01	0.2343E+01	0.9705E+01
SSW	0.0	0.1049E+00	0.6296E+00	0.1481E+01	0.6296E+00	0.9327E-01	0.0	0.2938E+01	0.1012E+02
SW	0.0	0.1166E+00	0.6762E+00	0.6296E+00	0.2682E+00	0.6995E-01	0.2332E-01	0.1784E+01	0.9203E+01
WSW	0.0	0.1632E+00	0.4430E+00	0.6296E+00	0.1749E+00	0.3498E-01	0.0	0.1446E+01	0.8565E+01
W	0.0	0.9327E-01	0.3731E+00	0.5013E+00	0.1282E+00	0.1166E-01	0.0	0.1108E+01	0.8369E+01
WNW	0.0	0.5830E-01	0.3265E+00	0.6296E+00	0.1166E+00	0.0	0.0	0.1131E+01	0.8577E+01
NW	0.0	0.9327E-01	0.3381E+00	0.7695E+00	0.2099E+00	0.0	0.0	0.1411E+01	0.9029E+01
NNW	0.0	0.1399E+00	0.4314E+00	0.8511E+00	0.1632E+00	0.5830E-01	0.0	0.1644E+01	0.9017E+01
TOTAL	0.0	0.1667E+01	0.7672E+01	0.1293E+02	0.3603E+01	0.5596E+00	0.9327E-01	0.2652E+02	0.8860E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 32

TABLE 2.3.3-10 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.4664E-01	0.2332E+00	0.4547E+00	0.2798E+00	0.0	0.0	0.1014E+01	0.9809E+01
NNE	0.0	0.6995E-01	0.2099E+00	0.5363E+00	0.1865E+00	0.0	0.0	0.1003E+01	0.9264E+01
NE	0.0	0.4664E-01	0.2332E+00	0.3847E+00	0.1166E+00	0.0	0.0	0.7812E+00	0.8766E+01
ENE	0.0	0.2332E-01	0.1166E+00	0.2565E+00	0.3498E-01	0.0	0.0	0.4314E+00	0.8813E+01
E	0.0	0.3498E-01	0.2099E+00	0.3614E+00	0.5830E-01	0.0	0.0	0.6646E+00	0.8421E+01
ESE	0.0	0.4664E-01	0.2099E+00	0.3498E+00	0.5830E-01	0.0	0.0	0.6646E+00	0.8356E+01
SE	0.0	0.4664E-01	0.1749E+00	0.2798E+00	0.0	0.0	0.0	0.5013E+00	0.7413E+01
SSE	0.1166E-01	0.6995E-01	0.1865E+00	0.4430E+00	0.5830E-01	0.0	0.0	0.7695E+00	0.8545E+01
S	0.0	0.3498E-01	0.3031E+00	0.5830E+00	0.1632E+00	0.0	0.0	0.1084E+01	0.8973E+01
SSW	0.0	0.4664E-01	0.2798E+00	0.6412E+00	0.1982E+00	0.1166E-01	0.0	0.1178E+01	0.9484E+01
SW	0.0	0.2332E-01	0.2215E+00	0.9560E+00	0.2099E+00	0.0	0.0	0.1411E+01	0.9455E+01
WSW	0.0	0.4664E-01	0.2682E+00	0.5946E+00	0.1516E+00	0.0	0.0	0.1061E+01	0.8888E+01
W	0.0	0.3498E-01	0.1632E+00	0.3847E+00	0.5830E-01	0.0	0.0	0.6412E+00	0.8850E+01
WNW	0.0	0.0	0.1516E+00	0.1982E+00	0.3498E-01	0.0	0.0	0.3847E+00	0.8197E+01
NW	0.0	0.4664E-01	0.1399E+00	0.2215E+00	0.5830E-01	0.0	0.0	0.4664E+00	0.7994E+01
NNW	0.1166E-01	0.9327E-01	0.1865E+00	0.3614E+00	0.9327E-01	0.0	0.0	0.7462E+00	0.8244E+01
TOTAL	0.2332E-01	0.7112E+00	0.3288E+01	0.7007E+01	0.1761E+01	0.1166E-01	0.0	0.1280E+02	0.8717E+01

NUMBER OF CALMS - 2

NUMBER OF BAD HOURS - 9

TABLE 2.3.3-10 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1049E+00	0.3614E+00	0.3731E+00	0.5830E-01	0.0	0.0	0.8977E+00	0.7546E+01
NNE	0.0	0.4664E-01	0.4664E+00	0.3964E+00	0.6996E-01	0.0	0.0	0.9794E+00	0.7521E+01
NE	0.1166E-01	0.1516E+00	0.4897E+00	0.5480E+00	0.8161E-01	0.0	0.0	0.1282E+01	0.7522E+01
ENE	0.1166E-01	0.1865E+00	0.5013E+00	0.4780E+00	0.5830E-01	0.0	0.0	0.1236E+01	0.7016E+01
E	0.1166E-01	0.1166E-00	0.3148E+00	0.6179E+00	0.8161E-01	0.0	0.0	0.1143E+01	0.8298E+01
ESE	0.1166E-01	0.1865E+00	0.2798E+00	0.4081E+00	0.1166E-01	0.0	0.0	0.8977E+00	0.6905E+01
SE	0.1166E-01	0.1399E+00	0.2915E+00	0.3381E+00	0.5830E-01	0.0	0.0	0.8395E+00	0.7368E+01
SSE	0.0	0.1049E+00	0.2915E+00	0.5013E+00	0.3498E-01	0.0	0.0	0.9327E+00	0.7994E+01
S	0.0	0.1049E+00	0.4897E+00	0.5596E+00	0.9327E-01	0.0	0.0	0.1248E+01	0.7792E+01
SSW	0.1166E-01	0.1865E+00	0.8278E+00	0.7928E+00	0.1166E+00	0.0	0.0	0.1935E+01	0.7580E+01
SW	0.1166E-01	0.1865E+00	0.8628E+00	0.9794E+00	0.5830E-01	0.0	0.0	0.2099E+01	0.7652E+01
WSW	0.0	0.1049E+00	0.9560E+00	0.9794E+00	0.1166E+00	0.0	0.0	0.2157E+01	0.7906E+01
W	0.1166E-01	0.1516E+00	0.6995E+00	0.2448E+00	0.0	0.0	0.0	0.1108E+01	0.5835E+01
WNW	0.1166E-01	0.1282E+00	0.5480E+00	0.2448E+00	0.2332E-01	0.0	0.0	0.9560E+00	0.6542E+01
NW	0.1166E-01	0.1399E+00	0.6995E+00	0.2798E+00	0.2332E-01	0.0	0.0	0.1154E+01	0.6010E+01
NNW	0.0	0.1049E+00	0.4664E+00	0.4664E+00	0.5830E-01	0.0	0.0	0.1096E+01	0.7456E+01
TOTAL	0.1166E+00	0.2145E+01	0.8546E+01	0.8208E+01	0.9444E+00	0.0	0.0	0.1996E+02	0.7309E+01

NUMBER OF CALMS - 10

NUMBER OF BAD HOURS - 24

TABLE 2.3.3-10 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.4314E+00	0.2530E+01	0.3987E+01	0.1119E+01	0.1982E+00	0.0	0.8266E+01	0.9057E+01
NNE	0.0	0.3731E+00	0.2810E+01	0.4232E+01	0.1061E+01	0.2332E-01	0.0	0.8499E+01	0.8723E+01
NE	0.2332E-01	0.6063E+00	0.2495E+01	0.3579E+01	0.8623E+00	0.0	0.0	0.7567E+01	0.8320E+01
ENE	0.1166E-01	0.4547E+00	0.2075E+01	0.1795E+01	0.3847E+00	0.4664E-01	0.0	0.4769E+01	0.7644E+01
E	0.1166E-01	0.4314E+00	0.1970E+01	0.1761E+01	0.2682E+00	0.0	0.0	0.4442E+01	0.7501E+01
ESE	0.1166E-01	0.4547E+00	0.1597E+01	0.1586E+01	0.1166E+00	0.0	0.0	0.3766E+01	0.7186E+01
SE	0.1166E-01	0.3731E+00	0.1236E+01	0.1084E+01	0.1049E+00	0.0	0.0	0.2810E+01	0.7077E+01
SSE	0.1166E-01	0.3964E+00	0.1632E+01	0.1947E+01	0.4314E+00	0.5830E-01	0.1049E+00	0.4582E+01	0.8598E+01
S	0.0	0.4547E+00	0.2110E+01	0.2787E+01	0.7928E+00	0.1049E+00	0.1166E-01	0.6261E+01	0.8627E+01
SSW	0.1166E-01	0.5596E+00	0.2740E+01	0.3813E+01	0.1422E+01	0.4780E+00	0.8161E-01	0.9106E+01	0.9648E+01
SW	0.1166E-01	0.5830E+00	0.3276E+01	0.4022E+01	0.1014E+01	0.3381E+00	0.1749E+00	0.9421E+01	0.9160E+01
WSW	0.0	0.5130E+00	0.3031E+01	0.3521E+01	0.6412E+00	0.1166E+00	0.1166E+00	0.7940E+01	0.8540E+01
W	0.1166E-01	0.4081E+00	0.2064E+01	0.1865E+01	0.4197E+00	0.6995E-01	0.1166E-01	0.4850E+01	0.7901E+01
WNW	0.1166E-01	0.3498E+00	0.1644E+01	0.2052E+01	0.1049E+01	0.2215E+00	0.5830E-01	0.5386E+01	0.9684E+01
NW	0.1166E-01	0.4430E+00	0.2017E+01	0.2343E+01	0.1038E+01	0.1166E+00	0.0	0.5969E+01	0.8839E+01
NNW	0.1166E-01	0.4547E+00	0.2075E+01	0.2996E+01	0.7467E+00	0.8161E-01	0.0	0.6366E+01	0.8555E+01
TOTAL	0.1516E+00	0.7287E+01	0.3530E+02	0.4337E+02	0.1147E+02	0.1854E+01	0.5596E+00	0.1000E+03	0.8618E+01

NUMBER OF CALMS - 13

NUMBER OF BAD HOURS - 183

TABLE 2.3.3-11
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS A
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.0	0.3589E+00	0.2547E+00	0.0	0.0	0.0	0.6136E+00	0.7081E+01
NNE	0.0	0.0	0.3010E+00	0.2084E+00	0.0	0.0	0.0	0.5094E+00	0.7145E+01
NE	0.0	0.1158E-01	0.1968E+00	0.1042E+00	0.0	0.0	0.0	0.3126E+00	0.6775E+01
ENE	0.0	0.0	0.1273E+00	0.1158E-01	0.0	0.0	0.0	0.1389E+00	0.5923E+01
E	0.0	0.0	0.9261E-01	0.0	0.0	0.0	0.0	0.9261E-01	0.5440E+01
ESE	0.0	0.0	0.3473E-01	0.1158E-01	0.0	0.0	0.0	0.4631E-01	0.5400E+01
SE	0.0	0.1158E-01	0.0	0.0	0.0	0.0	0.0	0.1158E-01	0.2900E+01
SSE	0.0	0.0	0.3473E-01	0.0	0.0	0.0	0.0	0.3473E-01	0.5620E+01
S	0.0	0.0	0.5788E-01	0.3473E-01	0.0	0.0	0.0	0.9261E-01	0.6227E+01
SSW	0.0	0.0	0.4631E-01	0.9104E-01	0.0	0.0	0.0	0.1273E+00	0.7947E+01
SW	0.0	0.0	0.3473E+00	0.1621E+00	0.2315E-01	0.0	0.0	0.5325E+00	0.7566E+01
WSW	0.0	0.0	0.2200E+00	0.3473E-01	0.6946E-01	0.4631E-01	0.0	0.3705E+00	0.1014E+02
W	0.0	0.0	0.1737E+00	0.1158E-01	0.1158E-01	0.0	0.0	0.1968E+00	0.6175E+01
WNW	0.0	0.0	0.1042E+00	0.3589E+00	0.8104E-01	0.1158E-01	0.0	0.5557E+00	0.1024E+02
NW	0.0	0.0	0.1621E+00	0.2894E+00	0.2315E-01	0.0	0.0	0.4746E+00	0.8225E+01
NNW	0.0	0.0	0.1968E+00	0.1852E+00	0.0	0.0	0.0	0.3820E+00	0.7199E+01
TOTAL	0.0	0.2315E-01	0.2454E+01	0.1748E+01	0.2084E+00	0.5788E-01	0.0	0.4492E+01	0.6876E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 46

TABLE 2.3.3-11 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1158E-01	0.2084E+00	0.2315E-01	0.0	0.0	0.0	0.2431E+00	0.6010E+01
NNE	0.0	0.1158E-01	0.1968E+00	0.9261E-01	0.0	0.0	0.0	0.3010E+00	0.6274E+01
NE	0.0	0.1158E-01	0.2547E+00	0.4631E-01	0.0	0.0	0.0	0.3126E+00	0.5941E+01
ENE	0.0	0.0	0.1158E+00	0.0	0.0	0.0	0.0	0.1158E+00	0.5672E+01
E	0.0	0.0	0.9261E-01	0.0	0.0	0.0	0.0	0.9261E-01	0.6092E+01
ESE	0.0	0.3473E-01	0.8104E-01	0.0	0.0	0.0	0.0	0.1158E+00	0.4236E+01
SE	0.0	0.1158E-01	0.2315E-01	0.0	0.0	0.0	0.0	0.3473E-01	0.4587E+01
SSE	0.0	0.1158E-01	0.2315E-01	0.1158E-01	0.0	0.0	0.0	0.4631E-01	0.4545E+01
S	0.0	0.3473E-01	0.1389E+00	0.0	0.0	0.0	0.0	0.1737E+00	0.5331E+01
SSW	0.0	0.2315E-01	0.2431E+00	0.2315E-01	0.0	0.0	0.0	0.2894E+00	0.5599E+01
SW	0.0	0.1153E-01	0.2431E+00	0.2315E-01	0.2315E-01	0.0	0.0	0.3010E+00	0.6108E+01
WSW	0.0	0.3473E-01	0.2778E+00	0.2315E-01	0.1158E-01	0.0	0.0	0.3473E+00	0.5743E+01
W	0.0	0.1158E-01	0.4631E-01	0.0	0.3473E-01	0.0	0.0	0.9261E-01	0.8305E+01
WNW	0.0	0.2315E-01	0.1158E+00	0.1158E+00	0.1158E-01	0.1158E-01	0.0	0.2778E+00	0.7820E+01
NW	0.0	0.0	0.1158E+00	0.1158E+00	0.2315E-01	0.0	0.0	0.2547E+00	0.8115E+01
NNW	0.0	0.0	0.1158E+00	0.9261E-01	0.0	0.0	0.0	0.2084E+00	0.7174E+01
TOTAL	0.0	0.2315E+00	0.2292E+01	0.5673E+00	0.1042E+00	0.1158E-01	0.0	0.3207E+01	0.6097E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 1

TABLE 2.3.3-11 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS C
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.2315E-01	0.2200E+00	0.1042E+00	0.0	0.0	0.0	0.3473E+00	0.6455E+01
NNE	0.0	0.2315E-01	0.2431E+00	0.2315E-01	0.0	0.0	0.0	0.2894E+00	0.5244E+01
NE	0.0	0.2315E-01	0.1968E+00	0.2315E-01	0.0	0.0	0.0	0.2431E+00	0.5257E+01
ENE	0.0	0.1158E-01	0.2084E+00	0.2315E-01	0.0	0.0	0.0	0.2431E+00	0.5280E+01
E	0.0	0.0	0.1042E+00	0.0	0.0	0.0	0.0	0.1042E+00	0.4913E+01
ESE	0.0	0.3473E-01	0.4631E-01	0.0	0.0	0.0	0.0	0.8104E-01	0.4116E+01
SE	0.0	0.2315E-01	0.3473E-01	0.0	0.0	0.0	0.0	0.5788E-01	0.3518E+01
SSE	0.0	0.1158E-01	0.6946E-01	0.1158E-01	0.0	0.0	0.0	0.9261E-01	0.5126E+01
S	0.0	0.3473E-01	0.1505E+00	0.1158E-01	0.0	0.0	0.0	0.1968E+00	0.4988E+01
SSW	0.0	0.6846E-01	0.2547E+00	0.3473E-01	0.0	0.0	0.0	0.3589E+00	0.5148E+01
SW	0.0	0.1042E+00	0.3126E+00	0.3473E-01	0.6946E-01	0.0	0.0	0.5210E+00	0.6309E+01
WSW	0.0	0.3473E-01	0.2547E+00	0.2315E-01	0.3473E-01	0.2315E-01	0.0	0.3705E+00	0.7670E+01
W	0.0	0.0	0.1968E+00	0.1158E-01	0.0	0.0	0.0	0.2084E+00	0.5224E+01
WNW	0.0	0.3473E-01	0.1737E+00	0.1158E+00	0.3473E-01	0.0	0.0	0.3589E+00	0.7433E+01
NW	0.0	0.3473E-01	0.1158E+00	0.1158E+00	0.1158E-01	0.0	0.0	0.2778E+00	0.6929E+01
NNW	0.0	0.3473E-01	0.1737E+00	0.1389E+00	0.0	0.0	0.0	0.3473E+00	0.6664E+01
TOTAL	0.0	0.4978E+00	0.2755E+01	0.6715E+00	0.1505E+00	0.2315E-01	0.0	0.4098E+01	0.5642E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 0

TABLE 2.3.3-11 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.4862E+00	0.2211E+01	0.5557E+00	0.5788E-01	0.0	0.0	0.3311E+01	0.5649E+01
NNE	0.0	0.7062E+00	0.2165E+01	0.2431E+00	0.0	0.0	0.0	0.3114E+01	0.4838E+01
NE	0.0	0.4862E+00	0.1424E+01	0.2200E+00	0.1158E-01	0.0	0.0	0.2142E+01	0.4988E+01
ENE	0.0	0.4631E+00	0.7178E+00	0.1273E+00	0.0	0.0	0.0	0.1308E+01	0.4556E+01
E	0.0	0.2547E+00	0.7062E+00	0.4631E-01	0.0	0.0	0.0	0.1007E+01	0.4414E+01
ESE	0.0	0.1621E+00	0.6830E+00	0.2315E-01	0.0	0.0	0.0	0.8683E+00	0.4774E+01
SE	0.0	0.1737E+00	0.4399E+00	0.3473E-01	0.0	0.0	0.0	0.6483E+00	0.4594E+01
SSE	0.0	0.1852E+00	0.6483E+00	0.6946E-01	0.3473E-01	0.0	0.0	0.9377E+00	0.5209E+01
S	0.0	0.2663E+00	0.7293E+00	0.8104E-01	0.3473E-01	0.0	0.0	0.1111E+01	0.5181E+01
SSW	0.0	0.5441E+00	0.1227E+01	0.4283E+00	0.1852E+00	0.0	0.0	0.2385E+01	0.6126E+01
SW	0.0	0.3936E+00	0.1505E+01	0.7525E+00	0.1158E+00	0.0	0.0	0.2767E+01	0.6460E+01
WSW	0.0	0.2084E+00	0.1343E+01	0.4168E+00	0.1273E+00	0.5788E-01	0.0	0.2153E+01	0.6888E+01
W	0.0	0.2431E+00	0.7872E+00	0.2663E+00	0.1158E=01	0.0	0.0	0.1308E+01	0.5619E+01
WNW	0.0	0.1852E+00	0.8451E+00	0.6020E+00	0.9261E-01	0.1158E-01	0.0	0.1737E+01	0.7020E+01
NW	0.0	0.1852E+00	0.1019E+01	0.6251E+00	0.6948E-01	0.0	0.0	0.1899E+01	0.6689E+01
NNW	0.0	0.4283E+00	0.1169E+01	0.6715E+00	0.3473E-01	0.0	0.0	0.2304E+01	0.6208E+01
TOTAL	0.0	0.5372E+01	0.1762E+02	0.5163E+01	0.7756E+00	0.6946E-01	0.0	0.2900E+02	0.5576E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 39

TABLE 2.3.3-11 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS E
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.8104E+00	0.1412E+01	0.2084E+00	0.3473E-01	0.0	0.0	0.2466E+01	0.4813E+01
NNE	0.1158E-01	0.1111E+01	0.1459E+01	0.5788E-01	0.0	0.0	0.0	0.2639E+01	0.3938E+01
NE	0.0	0.9146E+00	0.9146E+00	0.1389E+00	0.0	0.0	0.0	0.1968E+01	0.4172E+01
ENE	0.0	0.8219E+00	0.4515E+00	0.1621E+00	0.1158E-01	0.0	0.0	0.1447E+01	0.4146E+01
E	0.0	0.6367E+00	0.5210E+00	0.8104E-01	0.0	0.0	0.0	0.1239E+01	0.3888E+01
ESE	0.0	0.5441E+00	0.3010E+00	0.3473E-01	0.0	0.0	0.0	0.8798E+00	0.3305E+01
SE	0.0	0.5210E+00	0.3241E+00	0.1158E-01	0.0	0.0	0.0	0.8567E+00	0.3287E+01
SSE	0.1158E-01	0.9146E+00	0.5673E+00	0.2315E-01	0.3473E-01	0.0	0.0	0.1551E+01	0.3691E+01
S	0.1158E-01	0.1366E+01	0.8798E+00	0.2315E+00	0.4631E-01	0.1158E-01	0.0	0.2547E+01	0.4278E+01
SSW	0.1158E-01	0.1169E+01	0.1262E+01	0.2084E+00	0.3473E-01	0.0	0.0	0.2686E+01	0.4331E+01
SW	0.0	0.6830E+00	0.7988E+00	0.2778E+00	0.2315E-01	0.0	0.0	0.1783E+01	0.4921E+01
WSW	0.0	0.4168E+00	0.6251E+00	0.1273E+00	0.3473E-01	0.0	0.0	0.1204E+01	0.5001E+01
W	0.0	0.4515E+00	0.4978E+00	0.1273E+00	0.0	0.0	0.0	0.1077E+01	0.4347E+01
WNW	0.0	0.4631E+00	0.4862E+00	0.9261E-01	0.0	0.0	0.0	0.1042E+01	0.4132E+01
NW	0.0	0.5557E+00	0.9261E+00	0.9261E-01	0.0	0.0	0.0	0.1574E+01	0.4411E+01
NNW	0.0	0.5788E+00	0.8104E+00	0.1505E+00	0.5788E-01	0.0	0.0	0.1598E+01	0.4688E+01
TOTAL	0.4631E-01	0.1196E+02	0.1224E+02	0.2026E+01	0.2778E+00	0.1158E-01	0.0	0.2656E+02	0.4209E+01

NUMBER OF CALMS - 4

NUMBER OF BAD HOURS - 13

TABLE 2.3.3-11 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.1158E-01	0.8683E+00	0.5673E+00	0.0	0.0	0.0	0.0	0.1447E+01	0.3120E+01
NNE	0.1158E-01	0.8683E+00	0.3010E+00	0.0	0.0	0.0	0.0	0.1181E+01	0.2794E+01
NE	0.0	0.7293E+00	0.4631E-01	0.0	0.0	0.0	0.0	0.7756E+00	0.2423E+01
ENE	0.0	0.6830E+00	0.3473E-01	0.0	0.0	0.0	0.0	0.7178E+00	0.2479E+01
E	0.0	0.4978E+00	0.5788E-01	0.0	0.0	0.0	0.0	0.5557E+00	0.2342E+01
ESE	0.0	0.6483E+00	0.9261E-01	0.0	0.0	0.0	0.0	0.7409E+00	0.2209E+01
SE	0.0	0.4283E+00	0.2315E-01	0.0	0.0	0.0	0.0	0.4515E+00	0.2116E+01
SSE	0.0	0.7988E+00	0.1158E-01	0.0	0.0	0.0	0.0	0.8104E+00	0.2071E+01
S	0.1158E-01	0.1100E+01	0.1369E+00	0.0	0.0	0.0	0.0	0.1250E+01	0.2341E+01
SSW	0.1158E-01	0.1146E+01	0.2663E+00	0.0	0.0	0.0	0.0	0.1424E+01	0.2699E+01
SW	0.0	0.5094E+00	0.2663E+00	0.0	0.0	0.0	0.0	0.7756E+00	0.3051E+01
WSW	0.0	0.4746E+00	0.1852E+00	0.0	0.0	0.0	0.0	0.6599E+00	0.2851E+01
W	0.0	0.3473E+00	0.1273E+00	0.0	0.0	0.0	0.0	0.4746E+00	0.2807E+01
WNW	0.0	0.2663E+00	0.6946E-01	0.0	0.0	0.0	0.0	0.3357E+00	0.2833E+01
NW	0.0	0.3010E+00	0.1158E+00	0.0	0.0	0.0	0.0	0.4168E+00	0.2822E+01
NNW	0.0	0.4862E+00	0.2315E+00	0.0	0.0	0.0	0.0	0.7178E+00	0.2896E+01
TOTAL	0.4631E-01	0.1015E+02	0.2535E+01	0.0	0.0	0.0	0.0	0.1273E+02	0.2616E+01

NUMBER OF CALMS - 4

NUMBER OF BAD HOURS - 7

TABLE 2.3.3-11 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.2431E+00	0.2142E+01	0.1042E+00	0.0	0.0	0.0	0.0	0.2489E+01	0.1793E+01
NNE	0.2084E+00	0.1899E+01	0.5788E-01	0.0	0.0	0.0	0.0	0.2165E+01	0.1627E+01
NE	0.2315E+00	0.2107E+01	0.3473E-01	0.0	0.0	0.0	0.0	0.2373E+01	0.1661E+01
ENE	0.1621E+00	0.1470E+01	0.4631E-01	0.0	0.0	0.0	0.0	0.1679E+01	0.1545E+01
E	0.1621E+00	0.1447E+01	0.2315E-01	0.0	0.0	0.0	0.0	0.1632E+01	0.1504E+01
ESE	0.1273E+00	0.1153E+01	0.0	0.0	0.0	0.0	0.0	0.1285E+01	0.1488E+01
SE	0.9261E-01	0.7988E+00	0.3473E-01	0.0	0.0	0.0	0.0	0.9261E+00	0.1456E+01
SSE	0.1042E+00	0.8683E+00	0.0	0.0	0.0	0.0	0.0	0.9724E+00	0.1420E+01
S	0.9261E-01	0.3104E+00	0.0	0.0	0.0	0.0	0.0	0.9030E+00	0.1371E+01
SSW	0.1273E+00	0.1088E+01	0.2315E-01	0.0	0.0	0.0	0.0	0.1239E+01	0.1584E+01
SW	0.6946E-01	0.5788E+00	0.2315E-01	0.0	0.0	0.0	0.0	0.6715E+00	0.1615E+01
WSW	0.8104E-01	0.6599E+00	0.2315E-01	0.0	0.0	0.0	0.0	0.7641E+00	0.1603E+01
W	0.0	0.4746E+00	0.0	0.0	0.0	0.0	0.0	0.4746E+00	0.1359E+01
WNW	0.2315E-01	0.4978E+00	0.2315E-01	0.0	0.0	0.0	0.0	0.5441E+00	0.1608E+01
NW	0.6946E-01	0.5441E+00	0.1158E-01	0.0	0.0	0.0	0.0	0.6251E+00	0.1509E+01
NNW	0.1158E+00	0.1007E+01	0.4631E-01	0.0	0.0	0.0	0.0	0.1169E+01	0.1661E+01
TOTAL	0.1910E+01	0.1755E+02	0.4515E+00	0.0	0.0	0.0	0.0	0.1991E+02	0.1550E+01

NUMBER OF CALMS - 165

NUMBER OF BAD HOURS - 16

TABLE 2.3.3-11 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 1/1/78 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.2547E+00	0.4341E+01	0.5082E+01	0.1146E+01	0.9261E-01	0.0	0.0	0.1092E+02	0.4420E+01
NNE	0.2315E+00	0.4619E+01	0.4723E+01	0.6251E+00	0.0	0.0	0.0	0.1020E+02	0.3904E+01
NE	0.2315E+00	0.4283E+01	0.3068E+01	0.5325E+00	0.1158E-01	0.0	0.0	0.8127E+01	0.3747E+01
ENE	0.1621E+00	0.3450E+01	0.1702E+01	0.3241E+00	0.1158E-01	0.0	0.0	0.5649E+01	0.3434E+01
E	0.1621E+00	0.2836E+01	0.1598E+01	0.1273E+00	0.0	0.0	0.0	0.4723E+01	0.3147E+01
ESE	0.1273E+00	0.2582E+01	0.1239E+01	0.6946E-01	0.0	0.0	0.0	0.4017E+01	0.2953E+01
SE	0.9261E-01	0.1968E+01	0.8798E+00	0.4631E-01	0.0	0.0	0.0	0.2987E+01	0.2888E+01
SSE	0.1158E+00	0.2790E+01	0.1354E+01	0.1158E+00	0.6946E-01	0.0	0.0	0.4445E+01	0.3317E+01
S	0.1158E+00	0.3612E+01	0.2095E+01	0.3589E+00	0.8104E-01	0.1158E-01	0.0	0.6275E+01	0.3750E+01
SSW	0.1505E+00	0.4040E+01	0.3323E+01	0.7756E+00	0.2200E+00	0.0	0.0	0.8509E+01	0.4336E+01
SW	0.6946E-01	0.2281E+01	0.3496E+01	0.1250E+01	0.2547E+00	0.0	0.0	0.7351E+01	0.5375E+01
WSW	0.8104E-01	0.1829E+01	0.2929E+01	0.6251E+00	0.2778E+00	0.1273E+00	0.0	0.5869E+01	0.5601E+01
W	0.0	0.1528E+01	0.1829E+01	0.4168E+00	0.5788E-01	0.0	0.0	0.3832E+01	0.4457E+01
WNW	0.2315E-01	0.1470E+01	0.1818E+01	0.1285E+01	0.2200E+00	0.3473E-01	0.0	0.4851E+01	0.5969E+01
NW	0.6946E-01	0.1621E+01	0.2466E+01	0.1239E+01	0.1273E+00	0.0	0.0	0.5522E+01	0.5420E+01
NNW	0.1158E+00	0.2535E+01	0.2744E+01	0.1239E+01	0.9261E-01	0.0	0.0	0.6726E+01	0.4868E+01
TOTAL	0.2303E+01	0.4579E+02	0.4034E+02	0.1018E+02	0.1517E+01	0.1737E+00	0.0	0.1000E+03	0.4302E+01

NUMBER OF CALMS - 173

NUMBER OF BAD HOURS - 122

TABLE 2.3.3-12
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS A
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1215E-01	0.1539E+00	0.3280E+00	0.8908E-01	0.4049E-02	0.0	0.5871E+00	0.9463E+01
NNE	0.0	0.1215E-01	0.1701E+00	0.2956E+00	0.1255E+00	0.0	0.0	0.6033E+00	0.9484E+01
NE	0.0	0.8098E-02	0.9717E-01	0.2348E+00	0.7288E-01	0.1620E-01	0.0	0.4292E+00	0.1010E+02
ENE	0.0	0.1215E-01	0.6073E-01	0.8503E-01	0.1215E-01	0.0	0.0	0.1701E+00	0.8138E+01
E	0.0	0.1215E-01	0.8098E-01	0.3239E-01	0.4049E-02	0.0	0.0	0.1296E+00	0.6341E+01
ESE	0.0	0.1620E-01	0.1053E+00	0.1620E-01	0.4049E-02	0.0	0.0	0.1417E+00	0.5741E+01
SE	0.0	0.8098E-02	0.4859E-01	0.4049E-02	0.0	0.0	0.0	0.6073E-01	0.5209E+01
SSE	0.0	0.8098E-02	0.7288E-01	0.2834E-01	0.4049E-02	0.0	0.0	0.1134E+00	0.6835E+01
S	0.0	0.1620E-01	0.9312E-01	0.7693E-01	0.1215E-01	0.4049E-02	0.4049E-02	0.2065E+00	0.8127E+01
SSW	0.0	0.4049E-02	0.1053E+00	0.2672E+00	0.1862E+00	0.2834E-01	0.0	0.5911E+00	0.1111E+02
SW	0.0	0.2834E-01	0.1862E+00	0.3280E+00	0.2510E+00	0.8908E-01	0.1620E-01	0.8989E+00	0.1171E+02
WSW	0.0	0.2429E-01	0.1377E+00	0.2672E+00	0.1336E+00	0.4454E-01	0.2024E-01	0.6276E+00	0.1125E+02
W	0.0	0.1215E-01	0.1174E+00	0.1093E+00	0.8503E-01	0.2024E-01	0.8098E-02	0.3523E+00	0.1052E+02
WNW	0.0	0.1620E-01	0.9717E-01	0.2591E+00	0.2429E+00	0.4454E-01	0.1215E-01	0.6721E+00	0.1212E+02
NW	0.0	0.8098E-02	0.1093E+00	0.2308E+00	0.1255E+00	0.2834E-01	0.0	0.5021E+00	0.1077E+02
NNW	0.0	0.1215E-01	0.1093E+00	0.2794E+00	0.1215E+00	0.8098E-02	0.0	0.5304E+00	0.1024E+02
TOTAL	0.0	0.2105E+00	0.1745E+01	0.2842E+01	0.1470E+01	0.2875E+00	0.6073E-01	0.6616E+01	0.9197E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 564

TABLE 2.3.3-12 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1215E-01	0.1417E+00	0.1336E+00	0.4049E-01	0.4049E-02	0.0	0.3320E+00	0.8641E+01
NNE	0.0	0.4049E-02	0.1134E+00	0.1458E+00	0.5668E-01	0.0	0.0	0.3199E+00	0.9048E+01
NE	0.0	0.8098E-02	0.9717E-01	0.1498E+00	0.4454E-01	0.4049E-02	0.0	0.3037E+00	0.8897E+01
ENE	0.0	0.8098E-02	0.8908E-01	0.4859E-01	0.8098E-02	0.0	0.0	0.1539E+00	0.7095E+01
E	0.0	0.8098E-02	0.9717E-01	0.4049E-01	0.0	0.0	0.0	0.1458E+00	0.6635E+01
ESE	0.0	0.1620E-01	0.8908E-01	0.2429E-01	0.0	0.0	0.0	0.1296E+00	0.6073E+01
SE	0.0	0.8098E-02	0.2834E-01	0.1620E-01	0.0	0.0	0.0	0.5264E-01	0.6199E+01
SSE	0.0	0.1620E-01	0.5668E-01	0.3644E-01	0.0	0.0	0.0	0.1093E+00	0.6463E+01
S	0.0	0.0	0.5264E-01	0.1012E+00	0.3239E-01	0.0	0.0	0.1862E+00	0.9677E+01
SSW	0.0	0.1215E-01	0.1012E+00	0.1660E+00	0.5668E-01	0.2024E-01	0.0	0.3563E+00	0.9864E+01
SW	0.0	0.2024E-01	0.1417E+00	0.1943E+00	0.1053E+00	0.2429E-01	0.4049E-02	0.4899E+00	0.1009E+02
WSW	0.0	0.2834E-01	0.9312E-01	0.1741E+00	0.1174E+00	0.1215E-01	0.2024E-01	0.4454E+00	0.1110E+02
W	0.0	0.4049E-02	0.6883E-01	0.5264E-01	0.4859E-01	0.2024E-01	0.8098E-02	0.2024E+00	0.1117E+02
WNW	0.0	0.1620E-01	0.1012E+00	0.1701E+00	0.1377E+00	0.4049E-01	0.8098E-02	0.4737E+00	0.1157E+02
NW	0.0	0.8098E-02	0.1134E+00	0.1579E+00	0.1134E+00	0.1215E-01	0.0	0.4049E+00	0.1026E+02
NNW	0.0	0.4049E-02	0.7693E-01	0.1174E+00	0.3644E-01	0.4049E-02	0.0	0.2389E+00	0.9290E+01
TOTAL	0.0	0.1741E+00	0.1462E+01	0.1729E+01	0.7976E+00	0.1417E+00	0.4049E-01	0.4344E+01	0.8879E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 13

TABLE 2.3.3-12 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS C
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.2834E-01	0.1741E+00	0.1620E+00	0.7288E-01	0.0	0.0	0.4373E+00	0.8576E+01
NNE	0.0	0.8098E-02	0.1498E+00	0.1255E+00	0.4454E-01	0.8098E-02	0.0	0.3361E+00	0.8566E+01
NE	0.0	0.4049E-02	0.1215E+00	0.1053E+00	0.2429E-01	0.4049E-02	0.0	0.2591E+00	0.8125E+01
ENE	0.0	0.1215E-01	0.1215E+00	0.7693E-01	0.1215E-01	0.0	0.4049E-02	0.2267E+00	0.7937E+01
E	0.0	0.1215E-01	0.1053E+00	0.5668E-01	0.0	0.0	0.0	0.1741E+00	0.6577E+01
ESE	0.0	0.2024E-01	0.1012E+00	0.8098E-02	0.0	0.0	0.0	0.1296E+00	0.5289E+01
SE	0.0	0.8098E-02	0.8098E-01	0.2024E-01	0.0	0.0	0.0	0.1093E+00	0.6033E+01
SSE	0.0	0.2834E-01	0.6883E-01	0.8908E-01	0.1215E-01	0.0	0.0	0.1984E+00	0.7448E+01
S	0.0	0.1620E-01	0.1417E+00	0.1255E+00	0.1620E-01	0.1215E-01	0.0	0.3118E+00	0.8057E+01
SSW	0.0	0.4049E-01	0.1215E+00	0.2065E+00	0.1093E+00	0.2834E-01	0.4049E-02	0.5102E+00	0.1006E+02
SW	0.0	0.3239E-01	0.1701E+00	0.1903E+00	0.1255E+00	0.3644E-01	0.1215E-01	0.5668E+00	0.1046E+02
WSW	0.0	0.3644E-01	0.1822E+00	0.2470E+00	0.1012E+00	0.3239E-01	0.1215E-01	0.6114E+00	0.9767E+01
W	0.0	0.2429E-01	0.1336E+00	0.1174E+00	0.5264E-01	0.2834E-01	0.0	0.3563E+00	0.9294E+01
WNW	0.0	0.1215E-01	0.9717E-01	0.1296E+00	0.1093E+00	0.4859E-01	0.4049E-02	0.4008E+00	0.1162E+02
NW	0.0	0.2834E-01	0.1296E+00	0.1417E+00	0.9312E-01	0.2024E-01	0.0	0.4130E+00	0.9501E+01
NNW	0.0	0.1620E-01	0.1336E+00	0.1660E+00	0.8098E-01	0.1215E-01	0.0	0.4089E+00	0.9367E+01
TOTAL	0.0	0.3280E+00	0.2033E+01	0.1968E+01	0.8543E+00	0.2308E+00	0.3644E-01	0.5450E+01	0.8542E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 18

TABLE 2.3.3-12 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.1498E+00	0.8057E+00	0.1008E+01	0.3442E+00	0.3239E-01	0.0	0.2340E+01	0.8760E+01
NNE	0.0	0.1579E+00	0.8584E+00	0.1247E+01	0.3158E+00	0.2024E-01	0.0	0.2599E+01	0.8614E+01
NE	0.0	0.1579E+00	0.6357E+00	0.7855E+00	0.2753E+00	0.4049E-02	0.0	0.1858E+01	0.8447E+01
ENE	0.4049E-02	0.1701E+00	0.5992E+00	0.5061E+00	0.1134E+00	0.4049E-02	0.0	0.1397E+01	0.7298E+01
E	0.4049E-02	0.1620E+00	0.5183E+00	0.3239E+00	0.1215E-01	0.0	0.0	0.1020E+01	0.6452E+01
ESE	0.0	0.1255E+00	0.4656E+00	0.2591E+00	0.4454E-01	0.0	0.0	0.8948E+00	0.6576E+01
SE	0.0	0.1255E+00	0.4494E+00	0.2632E+00	0.6478E-01	0.8098E-02	0.0	0.9110E+00	0.7123E+01
SSE	0.0	0.1255E+00	0.5223E+00	0.4778E+00	0.2186E+00	0.1215E-01	0.1620E-01	0.1373E+01	0.8624E+01
S	0.0	0.1579E+00	0.5264E+00	0.5790E+00	0.1782E+00	0.3644E-01	0.4049E-02	0.1482E+01	0.8446E+01
SSW	0.4049E-02	0.1822E+00	0.6316E+00	0.6802E+00	0.5021E+00	0.2105E+00	0.3239E-01	0.2243E+01	0.1054E+02
SW	0.4049E-02	0.1782E+00	0.8867E+00	0.8543E+00	0.4940E+00	0.1458E+00	0.5668E-01	0.2620E+01	0.9951E+01
WSW	0.0	0.1336E+00	0.6964E+00	0.8260E+00	0.3199E+00	0.5264E-01	0.4454E-01	0.2073E+01	0.9489E+01
W	0.0	0.1255E+00	0.4373E+00	0.4494E+00	0.2105E+00	0.6883E-01	0.8098E-02	0.1300E+01	0.9100E+01
WNW	0.0	0.1093E+00	0.3401E+00	0.4859E+00	0.3077E+00	0.1296E+00	0.1215E-01	0.1385E+01	0.1055E+02
NW	0.0	0.1134E+00	0.4373E+00	0.5304E+00	0.4008E+00	0.6883E-01	0.4049E-02	0.1555E+01	0.9999E+01
NNW	0.0	0.1255E+00	0.5790E+00	0.7248E+00	0.2591E+00	0.2429E-01	0.0	0.1713E+01	0.8725E+01
TOTAL	0.1620E-01	0.2300E+01	0.9389E+01	0.1000E+02	0.4061E+01	0.8179E+00	0.1782E+00	0.2676E+02	0.8668E+01

NUMBER OF CALMS - 4

NUMBER OF BAD HOURS - 266

TABLE 2.3.3-12 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS E
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.5668E-01	0.3077E+00	0.8584E+00	0.2510E+00	0.4454E-01	0.0	0.1518E+01	0.9900E+01
NNE	0.0	0.5265E-01	0.3280E+00	0.1000E+01	0.2389E+00	0.0	0.0	0.1620E+01	0.9596E+01
NE	0.0	0.8503E-01	0.3927E+00	0.9474E+00	0.2348E+00	0.0	0.0	0.1660E+01	0.9101E+01
ENE	0.0	0.5668E-01	0.3401E+00	0.4859E+00	0.1053E+00	0.1620E-01	0.0	0.1004E+01	0.8678E+01
E	0.0	0.6478E-01	0.3927E+00	0.4778E+00	0.8503E-01	0.0	0.0	0.1020E+01	0.8049E+01
ESE	0.0	0.4454E-01	0.4049E+00	0.3604E+00	0.3644E-01	0.0	0.0	0.8462E+00	0.7695E+01
SE	0.0	0.6883E-01	0.3077E+00	0.3280E+00	0.2024E-01	0.0	0.0	0.7248E+00	0.7458E+01
SSE	0.0	0.5668E-01	0.4656E+00	0.8422E+00	0.1498E+00	0.1620E-01	0.2024E-01	0.1551E+01	0.9027E+01
S	0.0	0.7288E-01	0.5749E+00	0.1413E+01	0.5102E+00	0.4049E-01	0.4049E-02	0.2616E+01	0.9863E+01
SSW	0.0	0.7288E-01	0.5830E+00	0.1931E+01	0.8098E+00	0.7288E-01	0.4049E-02	0.3474E+01	0.1041E+02
SW	0.0	0.6478E-01	0.5547E+00	0.9070E+00	0.4859E+00	0.8503E-01	0.1215E-01	0.2109E+01	0.1023E+02
WSW	0.4049E-02	0.1174E+00	0.4616E+00	0.6397E+00	0.2308E+00	0.4454E-01	0.0	0.1498E+01	0.9003E+01
W	0.0	0.7693E-01	0.2389E+00	0.4697E+00	0.1579E+00	0.8098E-02	0.0	0.9515E+00	0.9055E+01
WNW	0.0	0.4859E-01	0.2632E+00	0.5466E+00	0.1943E+00	0.8098E-02	0.0	0.1061E+01	0.9499E+01
NW	0.0	0.6883E-01	0.2389E+00	0.7652E+00	0.1782E+00	0.8098E-02	0.4049E-02	0.1263E+01	0.9386E+01
NNW	0.4049E-02	0.9717E-01	0.3442E+00	0.6154E+00	0.1741E+00	0.2834E-01	0.0	0.1263E+01	0.9052E+01
TOTAL	0.8098E-02	0.1105E+01	0.6199E+01	0.1259E+02	0.3863E+01	0.3725E+00	0.4454E-01	0.2418E+02	0.9126E+01

NUMBER OF CALMS - 2

NUMBER OF BAD HOURS - 200

TABLE 2.3.3-12 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.3644E-01	0.1984E+00	0.4049E+00	0.2024E+00	0.0	0.0	0.8422E+00	0.9765E+01
NNE	0.0	0.3644E-01	0.1741E+00	0.4008E+00	0.1053E+00	0.0	0.0	0.7167E+00	0.9194E+01
NE	0.4049E-02	0.4454E-01	0.1579E+00	0.3280E+00	0.6883E-01	0.0	0.0	0.6033E+00	0.8684E+01
ENE	0.0	0.2429E-01	0.1093E+00	0.2713E+00	0.6073E-01	0.0	0.0	0.4656E+00	0.9128E+01
E	0.4049E-02	0.5264E-01	0.1620E+00	0.3482E+00	0.4454E-01	0.0	0.0	0.6114E+00	0.8388E+01
ESE	0.0	0.2429E-01	0.2146E+00	0.2915E+00	0.3239E-01	0.0	0.0	0.5628E+00	0.8013E+01
SE	0.0	0.3239E-01	0.1498E+00	0.2348E+00	0.8098E-02	0.0	0.0	0.4251E+00	0.7692E+01
SSE	0.4049E-02	0.4859E-01	0.2065E+00	0.3806E+00	0.4049E-01	0.0	0.0	0.6802E+00	0.8303E+01
S	0.0	0.4049E-01	0.3523E+00	0.6721E+00	0.1782E+00	0.0	0.0	0.1243E+01	0.9044E+01
SSW	0.4049E-02	0.5264E-01	0.3482E+00	0.1089E+01	0.2713E+00	0.4049E-02	0.0	0.1769E+01	0.9624E+01
SW	0.4049E-02	0.5264E-01	0.2470E+00	0.1069E+01	0.1782E+00	0.0	0.0	0.1551E+01	0.9234E+01
WSW	0.4049E-02	0.4049E-01	0.2551E+00	0.6114E+00	0.1377E+00	0.0	0.0	0.1049E+01	0.9015E+01
W	0.0	0.2429E-01	0.1903E+00	0.3968E+00	0.1012E+00	0.0	0.0	0.7126E+00	0.9136E+01
WNW	0.0	0.3239E-01	0.1579E+00	0.2713E+00	0.6883E-01	0.0	0.0	0.5304E+00	0.8772E+01
NW	0.0	0.3239E-01	0.1539E+00	0.2470E+00	0.5264E-01	0.0	0.0	0.4859E+00	0.8218E+01
NNW	0.4049E-02	0.5264E-01	0.1660E+00	0.3037E+00	0.8098E-01	0.0	0.0	0.6073E+00	0.8636E+01
TOTAL	0.2834E-01	0.6276E+00	0.3243E+01	0.7320E+01	0.1632E+01	0.4049E-02	0.0	0.1286E+02	0.8803E+01

NUMBER OF CALMS - 7

NUMBER OF BAD HOURS - 63

TABLE 2.3.3-12 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.7288E-01	0.3482E+00	0.4170E+00	0.6073E-01	0.0	0.0	0.8989E+00	0.7690E+01
NNE	0.0	0.7693E-01	0.3442E+00	0.4737E+00	0.6478E-01	0.0	0.0	0.9596E+00	0.7792E+01
NE	0.0	0.1134E+00	0.3118E+00	0.4130E+00	0.4859E-01	0.0	0.0	0.8867E+00	0.7477E+01
ENE	0.8098E-02	0.1377E+00	0.3320E+00	0.3685E+00	0.9312E-01	0.0	0.0	0.9393E+00	0.7559E+01
E	0.0	0.7693E-01	0.2348E+00	0.4130E+00	0.3644E-01	0.0	0.0	0.7612E+00	0.8075E+01
ESE	0.0	0.1215E+00	0.2227E+00	0.3563E+00	0.2834E-01	0.0	0.0	0.7288E+00	0.7362E+01
SE	0.8098E-02	0.1255E+00	0.2753E+00	0.3199E+00	0.3239E-01	0.0	0.0	0.7612E+00	0.7252E+01
SSE	0.4049E-02	0.1215E+00	0.4859E+00	0.5426E+00	0.5264E-01	0.0	0.0	0.1207E+01	0.7637E+01
S	0.8098E-02	0.1215E+00	0.5628E+00	0.6600E+00	0.9717E-01	0.0	0.0	0.1450E+01	0.7824E+01
SSW	0.8098E-02	0.1822E+00	0.7288E+00	0.1146E+01	0.1296E+00	0.0	0.0	0.2195E+01	0.8128E+01
SW	0.8098E-02	0.1539E+00	0.7045E+00	0.1105E+01	0.6478E-01	0.0	0.0	0.2037E+01	0.7913E+01
WSW	0.8098E-02	0.1336E+00	0.9110E+00	0.1300E+01	0.1660E+00	0.0	0.0	0.2518E+01	0.8124E+01
W	0.8098E-02	0.1458E+00	0.5749E+00	0.4940E+00	0.2429E-01	0.0	0.0	0.1247E+01	0.6925E+01
WNW	0.8098E-02	0.1377E+00	0.5345E+00	0.3685E+00	0.5668E-01	0.0	0.0	0.1105E+01	0.6907E+01
NW	0.8098E-02	0.1539E+00	0.6195E+00	0.2510E+00	0.8098E-02	0.0	0.0	0.1041E+01	0.5903E+01
NNW	0.8098E-02	0.1336E+00	0.4211E+00	0.4332E+00	0.6073E-01	0.0	0.0	0.1057E+01	0.7304E+01
TOTAL	0.8503E-01	0.2008E+01	0.7612E+01	0.9061E+01	0.1024E+01	0.0	0.0	0.1979E+02	0.7492E+01

NUMBER OF CALMS - 21

NUMBER OF BAD HOURS - 130

TABLE 2.3.3-12 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
UPPER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.3685E+00	0.2130E+01	0.3312E+01	0.1061E+01	0.8503E-01	0.0	0.6956E+01	0.9035E+01
NNE	0.0	0.3482E+00	0.2138E+01	0.3689E+01	0.9515E+00	0.2834E-01	0.0	0.7154E+01	0.8875E+01
NE	0.4049E-02	0.4211E+00	0.1814E+01	0.2964E+01	0.7693E+00	0.2834E-01	0.0	0.6000E+01	0.8635E+01
ENE	0.1215E-01	0.4211E+00	0.1652E+01	0.1842E+01	0.4049E+00	0.2024E-01	0.4049E-02	0.4357E+01	0.7928E+01
E	0.8098E-02	0.3887E+00	0.1591E+01	0.1692E+01	0.1822E+00	0.0	0.0	0.3863E+01	0.7509E+01
ESE	0.0	0.3685E+00	0.1603E+01	0.1316E+01	0.1458E+00	0.0	0.0	0.3433E+01	0.7152E+01
SE	0.8098E-02	0.3765E+00	0.1340E+01	0.1186E+01	0.1255E+00	0.8098E-02	0.0	0.3045E+01	0.7221E+01
SSE	0.8098E-02	0.4049E+00	0.1879E+01	0.2397E+01	0.4778E+00	0.2834E-01	0.3644E-01	0.5231E+01	0.8346E+01
S	0.8098E-02	0.4251E+00	0.2304E+01	0.3628E+01	0.1024E+01	0.9312E-01	0.1215E-01	0.7495E+01	0.8926E+01
SSW	0.1620E-01	0.5466E+00	0.2620E+01	0.5486E+01	0.2065E+01	0.3644E+00	0.4049E-01	0.1114E+02	0.9867E+01
SW	0.1619E-01	0.5305E+00	0.2891E+01	0.4648E+01	0.1705E+01	0.3806E+00	0.1012E+00	0.1027E+02	0.9686E+01
WSW	0.1619E-01	0.5143E+00	0.2737E+01	0.4065E+01	0.1207E+01	0.1862E+00	0.9717E-01	0.8823E+01	0.9187E+01
W	0.8098E-02	0.4130E+00	0.1761E+01	0.2089E+01	0.6802E+00	0.1458E+00	0.2429E-01	0.5122E+01	0.8763E+01
WNW	0.8098E-02	0.3725E+00	0.1591E+01	0.2231E+01	0.1117E+01	0.2713E+00	0.3644E-01	0.5628E+01	0.9824E+01
NW	0.8098E-02	0.4130E+00	0.1802E+01	0.2324E+01	0.9717E+00	0.1377E+00	0.8098E-02	0.5664E+01	0.9012E+01
NNW	0.1620E-01	0.4413E+00	0.1830E+01	0.2640E+01	0.8138E+00	0.7693E-01	0.0	0.5818E+01	0.8737E+01
TOTAL	0.1377E+00	0.6753E+01	0.3168E+02	0.4551E+02	0.1370E+02	0.1854E+01	0.3604E+00	0.1000E+03	0.8913E+01

NUMBER OF CALMS - 34

NUMBER OF BAD HOURS - 1254

TABLE 2.3.3-13
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS A
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.2404E-01	0.3727E+00	0.2404E+00	0.8014E-02	0.0	0.0	0.6452E+00	0.7025E+01
NNE	0.0	0.4007E-02	0.2685E+00	0.2164E+00	0.0	0.0	0.0	0.4889E+00	0.7267E+01
NE	0.0	0.1603E-01	0.2164E+00	0.1523E+00	0.8014E-02	0.0	0.0	0.3927E+00	0.7216E+01
ENE	0.0	0.4007E-02	0.1643E+01	0.4809E-01	0.0	0.0	0.0	0.2164E+00	0.6358E+01
E	0.0	0.1202E-01	0.7614E-01	0.1202E-01	0.0	0.0	0.0	0.1002E+00	0.5621E+01
ESE	0.0	0.1202E-01	0.6011E-01	0.8014E-02	0.0	0.0	0.0	0.8014E-01	0.4889E+01
SE	0.0	0.2404E-01	0.6011E-01	0.4007E-02	0.0	0.0	0.0	0.8816E-01	0.4875E+01
SSE	0.0	0.8014E-02	0.1042E+00	0.8014E-02	0.4007E-02	0.0	0.0	0.1242E+00	0.5815E+01
S	0.0	0.1603E-01	0.1162E+00	0.2004E-01	0.8014E-02	0.0	0.0	0.1603E+00	0.6220E+01
SSW	0.0	0.2004E-01	0.2364E+00	0.2885E+00	0.1603E-01	0.0	0.0	0.5610E+00	0.7830E+01
SW	0.0	0.2404E-01	0.2765E+00	0.3807E+00	0.1242E+00	0.0	0.0	0.8054E+00	0.8773E+01
WSW	0.4007E-02	0.3606E-01	0.3086E+00	0.2605E+00	0.7213E-01	0.1603E-01	0.0	0.6973E+00	0.8303E+01
W	0.0	0.2004E-01	0.1964E+00	0.1202E+00	0.2805E-01	0.4007E-02	0.0	0.3687E+00	0.7431E+01
WNW	0.0	0.1202E-01	0.1723E+00	0.3566E+00	0.9217E-01	0.8014E-02	0.0	0.6412E+00	0.9461E+01
NW	0.0	0.4007E-02	0.2484E+00	0.2805E+00	0.5209E-01	0.0	0.0	0.5851E+00	0.8164E+01
NNW	0.0	0.1202E-01	0.2284E+00	0.3005E+00	0.3606E-01	0.0	0.0	0.5770E+00	0.8134E+01
TOTAL	0.4007E-02	0.2484E+00	0.3106E+01	0.2697E+01	0.4488E+00	0.2805E-01	0.0	0.6532E+01	0.7086E+01

NUMBER OF CALMS - 1

NUMBER OF BAD HOURS - 568

TABLE 2.3.3-13 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.4007E-02	0.2525E+00	0.8014E-01	0.0	0.0	0.0	0.3366E+00	0.6520E+01
NNE	0.0	0.8014E-02	0.1883E+00	0.6812E-01	0.0	0.0	0.0	0.2645E+00	0.6505E+01
NE	0.0	0.2004E-01	0.2204E+00	0.7213E-01	0.4007E-02	0.0	0.0	0.3166E+00	0.6144E+01
ENE	0.0	0.1603E-01	0.1162E+00	0.1202E-01	0.0	0.0	0.0	0.1443E+00	0.5786E+01
E	0.0	0.8014E-02	0.9217E-01	0.1603E-01	0.0	0.0	0.0	0.1162E+00	0.5632E+01
ESE	0.0	0.1603E-01	0.1042E+00	0.4007E-02	0.0	0.0	0.0	0.1242E+00	0.5117E+01
SE	0.0	0.8014E-02	0.4408E-01	0.8014E-02	0.0	0.0	0.0	0.6011E-01	0.5274E+01
SSE	0.0	0.8014E-02	0.3606E-01	0.1603E-01	0.0	0.0	0.0	0.6011E-01	0.5651E+01
S	0.0	0.2404E-01	0.1122E+00	0.4408E-01	0.4007E-02	0.0	0.0	0.1843E+00	0.6514E+01
SSW	0.0	0.1603E-01	0.2364E+00	0.7614E-01	0.8014E-02	0.0	0.0	0.3366E+00	0.6574E+01
SW	0.0	0.1603E-01	0.2404E+00	0.1563E+00	0.3206E-01	0.8014E-02	0.0	0.4528E+00	0.7805E+01
WSW	0.0	0.4007E-01	0.1883E+00	0.2124E+00	0.2404E-01	0.4007E-02	0.4007E-02	0.4729E+00	0.7870E+01
W	0.0	0.2404E-01	0.8014E-01	0.8014E-01	0.2404E-01	0.4007E-02	0.0	0.2124E+00	0.7862E+01
WNW	0.0	0.1603E-01	0.1483E+00	0.1923E+00	0.5610E-01	0.8014E-02	0.0	0.4208E+00	0.8865E+01
NW	0.0	0.1603E-01	0.1803E+00	0.2284E+00	0.4007E-01	0.0	0.0	0.4648E+00	0.8273E+01
NNW	0.0	0.0	0.1643E+00	0.1122E+00	0.1603E-01	0.0	0.0	0.2925E+00	0.7494E+01
TOTAL	0.0	0.2404E+00	0.2404E+01	0.1378E+01	0.2084E+00	0.2404E-01	0.4007E-02	0.4260E+01	0.6743E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 23

TABLE 2.3.3-13 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS C
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.2004E-01	0.2845E+00	0.1683E+00	0.0	0.0	0.0	0.4729E+00	0.6561E+01
NNE	0.0	0.1603E-01	0.2204E+00	0.4408E-01	0.0	0.0	0.0	0.2805E+00	0.5820E+01
NE	0.0	0.2404E-01	0.1563E+00	0.4007E-01	0.0	0.0	0.0	0.2204E+00	0.5763E+01
ENE	0.0	0.2404E-01	0.1923E+00	0.3206E-01	0.0	0.0	0.0	0.2484E+00	0.5442E+01
E	0.0	0.2805E-01	0.1403E+00	0.4007E-02	0.0	0.0	0.0	0.1723E+00	0.4839E+01
ESE	0.0	0.1603E-01	0.1002E+00	0.4007E-02	0.0	0.0	0.0	0.1202E+00	0.4809E+01
SE	0.0	0.2004E-01	0.9217E-01	0.1603E-01	0.0	0.0	0.0	0.1282E+00	0.5046E+01
SSE	0.0	0.2404E-01	0.1443E+00	0.2004E-01	0.0	0.0	0.0	0.1883E+00	0.5356E+01
S	0.0	0.2805E-01	0.1763E+00	0.4007E-01	0.4007E-02	0.0	0.0	0.2484E+00	0.5730E+01
SSW	0.0	0.4007E-01	0.3166E+00	0.1122E+00	0.1603E-01	0.0	0.0	0.4849E+00	0.6445E+01
SW	0.0	0.6412E-01	0.3206E+00	0.1643E+00	0.3206E-01	0.0	0.0	0.5810E+00	0.7043E+01
WSW	0.0	0.2404E-01	0.2925E+00	0.2084E+00	0.3206E-01	0.1202E-01	0.0	0.5690E+00	0.7967E+01
W	0.0	0.1603E-01	0.2284E+00	0.9617E-01	0.1202E-01	0.0	0.0	0.3526E+00	0.6738E+01
WNW	0.0	0.3206E-01	0.1843E+00	0.1282E+00	0.8014E-01	0.4007E-02	0.0	0.4288E+00	0.8427E+01
NW	0.0	0.2805E-01	0.1803E+00	0.1964E+00	0.4408E-01	0.0	0.0	0.4488E+00	0.7816E+01
NNW	0.0	0.2805E-01	0.2444E+00	0.1563E+00	0.2004E-01	0.0	0.0	0.4488E+00	0.7000E+01
TOTAL	0.0	0.4328E+00	0.3274E+01	0.1431E+01	0.2404E+00	0.1603E-01	0.0	0.5394E+01	0.6300E+01

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 18

TABLE 2.3.3-13 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.8014E-02	0.4007E+00	0.1659E+01	0.5290E+00	0.2805E-01	0.0	0.0	0.2625E+01	5706E+01
NNE	0.8014E-02	0.5209E+00	0.1675E+01	0.2124E+00	0.0	0.0	0.0	0.2416E+01	4967E+01
NE	0.8014E-02	0.4168E+00	0.1094E+01	0.2124E+00	0.8014E-02	0.0	0.0	0.1739E+01	5157E+01
ENE	0.8014E-02	0.3326E+00	0.8535E+00	0.1242E+00	0.0	0.0	0.0	0.1318E+01	4892E+01
E	0.0	0.2685E+00	0.6492E+00	0.5610E-01	0.0	0.0	0.0	0.9738E+00	4462E+01
ESE	0.0	0.2565E+00	0.5530E+00	0.2404E-01	0.0	0.0	0.0	0.8335E+00	4435E+01
SE	0.0	0.2044E+00	0.6171E+00	0.1242E+00	0.4007E-02	0.0	0.0	0.9497E+00	5093E+01
SSE	0.0	0.28053+00	0.8255E+00	0.1883E+00	0.1603E-01	0.0	0.0	0.1310E+01	5345E+01
S	0.8014E-02	0.3045E+00	0.8575E+00	0.1883E+00	0.2805E-01	0.0	0.0	0.1386E+01	5398E+01
SSW	0.8014E-02	0.4729E+00	0.1250E+01	0.4889E+00	0.1322E+00	0.0	0.0	0.2352E+01	0.6156E+01
SW	0.8014E-02	0.4368E+00	0.1242E+01	0.7213E+00	0.1002E+00	0.1202E-01	0.0	0.2521E+01	0.6467E+01
WSW	0.8014E-02	0.3206E+00	0.1198E+01	0.5290E+00	0.9617E-01	0.5209E-01	0.0	0.2204E+01	0.6748E+01
W	0.8014E-02	0.3126E+00	0.6452E+00	0.2925E+00	0.6011E-01	0.0	0.0	0.1318E+01	0.6016E+01
WNW	0.0	0.2444E+00	0.7133E+00	0.4929E+00	0.1202E+00	0.4007E-02	0.0	0.1575E+01	0.6934E+01
NW	0.0	0.1923E+00	0.7373E+00	0.5530E+00	0.1122E+00	0.0	0.0	0.1595E+01	0.7101E+01
NNW	0.8014E-02	0.3526E+00	0.9537E+00	0.5690E+00	0.3606E-01	0.0	0.0	0.1919E+01	0.6207E+01
TOTAL	0.8014E-01	0.5318E+01	0.1552E+02	0.5306E+01	0.7413E+00	0.6812E-01	0.0	0.2704E+02	0.5693E+01

NUMBER OF CALMS - 20

NUMBER OF BAD HOURS - 129

TABLE 2.3.3-13 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS E
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.1202E-01	0.5891E+00	0.9297E+00	0.1563E+00	0.1202-01	0.0	0.0	0.1699E-01	0.4637E+01
NNE	0.1202E-01	0.7974E+00	0.9337E+00	0.4007E-01	0.0	0.0	0.0	0.1783E+01	0.3850E+01
NE	0.1202E-01	0.7453E+00	0.6692E+00	0.8014E-01	0.0	0.0	0.0	0.1507E+01	0.4001E+01
ENE	0.1202E-01	0.5931E+00	0.4127E+00	0.9217E-01	0.4007E-02	0.0	0.0	0.1114E+01	0.4013E+01
E	0.8014E-02	0.5570E+00	0.3566E+00	0.4007E-01	0.8014E-02	0.0	0.0	0.9697E+00	0.3625E+01
ESE	0.4007E-02	0.4568E+00	0.2805E+00	0.3606E-01	0.0	0.0	0.0	0.7774E+00	0.3548E+01
SE	0.8014E-02	0.5330E+00	0.3486E+00	0.1603E-01	0.0	0.0	0.0	0.9056E+00	0.3353E+01
SSE	0.1202E-01	0.8696E+00	0.5971E+00	0.3206E-01	0.1202E-01	0.0	0.0	0.1523E+01	0.3588E+01
S	0.2004E-01	0.1362E+01	0.1058E+01	0.2084E+00	0.2004E-01	0.4007E-02	0.0	0.2673E+01	0.4064E+01
SSW	0.2004E-01	0.1386E+01	0.1499E+01	0.2284E+00	0.2404E-01	0.0	0.0	0.3158E+01	0.4291E+01
SW	0.1202E-01	0.8095E+00	0.9577E+00	0.3406E+00	0.2004E-01	0.0	0.0	0.2140E+01	0.4874E+01
WSW	0.8014E-02	0.4889E+00	0.6412E+00	0.1683E+00	0.2004E-01	0.0	0.0	0.1326E+01	0.4810E+01
W	0.0	0.3366E+00	0.5169E+00	0.1082E+00	0.8014E-02	0.0	0.0	0.9697E+00	0.4645E+01
WNW	0.0	0.3847E+00	0.5129E+00	0.1202E+00	0.0	0.4007E+02	0.0	0.1022E+01	0.4628E+01
NW	0.0	0.4047E+00	0.8014E+00	0.1162E+00	0.1202E-01	0.0	0.0	0.1334E+01	0.4692E+01
NNW	0.8014E-02	0.4889E+00	0.7213E+00	0.1322E+00	0.2805E-01	0.0	0.0	0.1378E+01	0.4579E+01
TOTAL	0.1483E+00	0.1080E+02	0.1124E+02	0.1915E+01	0.1683E+00	0.8014E-02	0.0	0.2428E+02	0.4200E+01

NUMBER OF CALMS - 37

NUMBER OF BAD HOURS - 113

TABLE 2.3.3-13 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.2805E-01	0.6532E+00	0.3647E+00	0.8014E-02	0.0	0.0	0.0	0.1054E+01	0.3208E+01
NNE	0.2805E-01	0.6852E+00	0.1403E+00	0.0	0.0	0.0	0.0	0.8535E+00	0.2568E+01
NE	0.2805E-01	0.6492E+00	0.5610E-01	0.0	0.0	0.0	0.0	0.7333E+00	0.2303E+01
ENE	0.2805E-01	0.6452E+00	0.6011E-01	0.0	0.0	0.0	0.0	0.7333E+00	0.2347E+01
E	0.2004E-01	0.4688E+00	0.6011E-01	0.0	0.0	0.0	0.0	0.5490E+00	0.2253E+01
ESE	0.2404E-01	0.5450E+00	0.4007E-01	0.0	0.0	0.0	0.0	0.6091E+00	0.2066E+01
SE	0.2004E-01	0.4328E+00	0.4007E-01	0.0	0.0	0.0	0.0	0.4929E+00	0.2130E+01
SSE	0.3206E-01	0.8375E+00	0.4007E-01	0.0	0.0	0.0	0.0	0.9096E+00	0.2028E+01
S	0.4408E-01	0.1082E+01	0.2004E+00	0.0	0.0	0.0	0.0	0.1326E+01	0.2384E+01
SSW	0.4809E-01	0.1186E+01	0.2965E+00	0.0	0.0	0.0	0.0	0.1531E+01	0.2544E+01
SW	0.3206E-01	0.8215E+00	0.2204E+00	0.0	0.0	0.0	0.0	0.1074E+01	0.2579E+01
WSW	0.2404E-01	0.5290E+00	0.1683E+00	0.0	0.0	0.0	0.0	0.7213E+00	0.2661E+01
W	0.4007E-02	0.3687E+00	0.1723E+00	0.0	0.0	0.0	0.0	0.5450E+00	0.2950E+01
WNW	0.0	0.3246E+00	0.1282E+00	0.0	0.0	0.0	0.0	0.4528E+00	0.2808E+01
NW	0.0	0.3086E+00	0.1242E+00	0.0	0.0	0.0	0.0	0.4328E+00	0.2777E+01
NNW	0.2004E-01	0.5049E+00	0.1964E+00	0.4007E-02	0.0	0.0	0.0	0.7253E+00	0.2764E+01
TOTAL	0.3807E+00	0.1004E+02	0.2308E+01	0.1202E-01	0.0	0.0	0.0	0.1274E+02	0.2517E+01

NUMBER OF CALMS - 95

NUMBER OF BAD HOURS - 58

TABLE 2.3.3-13 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.5490E+00	0.1575E+01	0.8816E-01	0.0	0.0	0.0	0.0	0.2212E+01	0.1774E+01
NNE	0.5049E+00	0.1451E+01	0.2805E-01	0.0	0.0	0.0	0.0	0.1984E+01	0.1520E+01
NE	0.4688E+00	0.1342E+01	0.2004E-01	0.0	0.0	0.0	0.0	0.1831E+01	0.1566E+01
ENE	0.4168E+00	0.1194E+01	0.4007E-01	0.0	0.0	0.0	0.0	0.1651E+01	0.1558E+01
E	0.3687E+00	0.1050E+01	0.1202E-01	0.0	0.0	0.0	0.0	0.1431E+01	0.1451E+01
ESE	0.2885E+00	0.8215E+00	0.1202E-01	0.0	0.0	0.0	0.0	0.1122E+01	0.1495E+01
SE	0.2324E+00	0.6572E+00	0.2004E-01	0.0	0.0	0.0	0.0	0.9096E+00	0.1495E+01
SSE	0.2204E+00	0.6251E+00	0.3606E-01	0.0	0.0	0.0	0.0	0.8816E+00	0.1655E+01
S	0.3005E+00	0.8616E+00	0.4809E-01	0.1202E-01	0.0	0.0	0.0	0.1222E+01	0.1646E+01
SSW	0.3045E+00	0.8736E+00	0.2404E-01	0.0	0.0	0.0	0.0	0.1202E+01	0.1610E+01
SW	0.2725E+00	0.7734E+00	0.4007E-01	0.4007E-02	0.0	0.0	0.0	0.1090E+01	0.1713E+01
WSW	0.2284E+00	0.6492E+00	0.4809E-01	0.0	0.0	0.0	0.0	0.9257E+00	0.1636E+01
W	0.1763E+00	0.4969E+00	0.3206E-01	0.0	0.0	0.0	0.0	0.7053E+00	0.1621E+01
WNW	0.1282E+00	0.4688E+00	0.2805E-01	0.0	0.0	0.0	0.0	0.6251E+00	0.1755E+01
NW	0.1803E+00	0.5129E+00	0.2805E-01	0.4007E-02	0.0	0.0	0.0	0.7253E+00	0.1691E+01
NNW	0.3126E+00	0.8896E+00	0.3606E-01	0.0	0.0	0.0	0.0	0.1238E+01	0.1581E+01
TOTAL	0.4953E+01	0.1424E+02	0.5410E+00	0.2004E-01	0.0	0.0	0.0	0.1976E+02	0.1610E+01

NUMBER OF CALMS - 1236

NUMBER OF BAD HOURS - 88

TABLE 2.3.3-13 (Continued)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 4:00 AM 1/14/76 TO 11:00 PM 12/31/78
LOWER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFF. TEMPERATURE
HARRIS ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.5971E+00	0.3266E+01	0.3951E+01	0.1182E+01	0.4809E-01	0.0	0.0	0.9044E+01	0.4584E+01
NNE	0.5530E+00	0.3482E+01	0.3454E+01	0.5810E+00	0.0	0.0	0.0	0.8071E+01	0.3998E+01
NE	0.5169E+00	0.3214E+01	0.2432E+01	0.5570E+00	0.2004E-01	0.0	0.0	0.6740E+01	0.3971E+01
ENE	0.4648E+00	0.2809E+01	0.1839E+01	0.3086E+00	0.4007E-02	0.0	0.0	0.5426E+01	0.3623E+01
E	0.3967E+00	0.2392E+01	0.1386E+01	0.1282E+00	0.8014E-02	0.0	0.0	0.4312E+01	0.3222E+01
ESE	0.3166E+00	0.2124E+01	0.1150E+01	0.7614E-01	0.0	0.0	0.0	0.3667E+01	0.3135E+01
SE	0.2605E+00	0.1879E+01	0.1222E+01	0.1683E+00	0.4007E-02	0.0	0.0	0.3534E+01	0.3439E+01
SSE	0.2645E+00	0.2653E+01	0.1783E+01	0.2645E+00	0.3206E-01	0.0	0.0	0.4997E+01	0.3670E+01
S	0.3727E+00	0.3679E+01	0.2569E+01	0.5129E+00	0.6412E-01	0.4007E-02	0.0	0.7201E+01	0.3869E+01
SSW	0.3807E+00	0.3995E+01	0.3859E+01	0.1194E+01	0.1964E+00	0.0	0.0	0.9625E+01	0.4634E+01
SW	0.3246E+00	0.2945E+01	0.3298E+01	0.1767E+01	0.3086E+00	0.2004E-01	0.0	0.8664E+01	0.5444E+01
WSW	0.2725E+00	0.2088E+01	0.2845E+01	0.1378E+01	0.2444E+00	0.8415E-01	0.4007E-02	0.6916E+01	0.5745E+01
W	0.1883E+00	0.1575E+01	0.1871E+01	0.6973E+00	0.1322E+00	0.8014E-02	0.0	0.4472E+01	0.5048E+01
WNW	0.1282E+00	0.1483E+01	0.1887E+01	0.1290E+01	0.3486E+00	0.2805E-01	0.0	0.5165E+01	0.6195E+01
NW	0.1803E+00	0.1467E+01	0.2300E+01	0.1378E+01	0.2605E+00	0.0	0.0	0.5586E+01	0.5890E+01
NNW	0.3486E+00	0.2276E+01	0.2545E+01	0.1274E+01	0.1362E+00	0.0	0.0	0.6580E+01	0.5068E+01
TOTAL	0.5566E+01	0.4133E+02	0.3839E+02	0.1276E+02	0.1807E+01	0.1443E+00	0.4007E-02	0.1000E+03	0.4578E+01

NUMBER OF CALMS - 1389

NUMBER OF BAD HOURS - 997

TABLE 2.3.3-14
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
UPPER WIND LEVEL
STABILITY CLASS A
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.0	0.16	0.26	0.12	0.02	0.0	0.57	9.81
NNE	0.0	0.01	0.16	0.35	0.11	0.0	0.0	0.64	9.52
NE	0.0	0.0	0.12	0.16	0.04	0.0	0.0	0.32	8.85
ENE	0.0	0.01	0.20	0.22	0.01	0.0	0.0	0.45	7.94
E	0.0	0.02	0.13	0.21	0.04	0.0	0.0	0.40	8.15
ESE	0.0	0.02	0.07	0.07	0.04	0.0	0.0	0.20	7.78
SE	0.0	0.01	0.09	0.01	0.02	0.0	0.0	0.14	6.72
SSE	0.0	0.0	0.12	0.02	0.0	0.0	0.0	0.14	5.65
S	0.0	0.02	0.25	0.20	0.12	0.0	0.0	0.59	8.77
SSW	0.0	0.02	0.26	0.39	0.21	0.01	0.0	0.90	10.03
SW	0.0	0.01	0.21	0.26	0.21	0.05	0.0	0.74	10.63
WSW	0.0	0.01	0.19	0.39	0.14	0.04	0.0	0.77	9.91
W	0.0	0.02	0.14	0.28	0.15	0.02	0.0	0.62	10.31
WNW	0.0	0.0	0.13	0.21	0.09	0.05	0.01	0.49	10.65
NW	0.0	0.02	0.12	0.26	0.06	0.01	0.0	0.47	9.14
NNW	0.0	0.0	0.16	0.21	0.20	0.06	0.01	0.64	11.40
TOTAL	0.0	0.20	2.52	3.52	1.55	0.26	0.0	8.07	9.08

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 219

TABLE 2.3.3-14 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
UPPER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.0	0.11	0.22	0.09	0.01	0.0	0.44	10.01
NNE	0.0	0.01	0.06	0.15	0.05	0.0	0.0	0.27	9.57
NE	0.0	0.01	0.08	0.11	0.01	0.0	0.0	0.21	8.00
ENE	0.0	0.02	0.06	0.11	0.04	0.0	0.0	0.22	8.49
E	0.0	0.0	0.09	0.05	0.01	0.0	0.0	0.15	7.09
ESE	0.0	0.01	0.05	0.01	0.0	0.0	0.0	0.07	5.63
SE	0.0	0.0	0.05	0.0	0.0	0.0	0.0	0.05	5.01
SSE	0.0	0.0	0.12	0.05	0.0	0.01	0.0	0.18	7.49
S	0.0	0.0	0.14	0.02	0.07	0.0	0.0	0.24	8.64
SSW	0.0	0.0	0.11	0.04	0.09	0.02	0.0	0.26	11.16
SW	0.0	0.01	0.14	0.09	0.06	0.08	0.0	0.39	10.90
WSW	0.0	0.01	0.02	0.11	0.05	0.0	0.0	0.19	10.56
W	0.0	0.0	0.04	0.05	0.05	0.01	0.01	0.15	13.14
WNW	0.0	0.01	0.06	0.14	0.01	0.04	0.01	0.27	11.25
NW	0.0	0.0	0.11	0.14	0.07	0.01	0.0	0.33	9.89
NNW	0.0	0.01	0.05	0.07	0.07	0.01	0.0	0.21	11.51
TOTAL	0.0	0.11	1.27	1.35	0.67	0.20	0.02	3.63	9.27

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 1

TABLE 2.3.3-14 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
UPPER WIND LEVEL
STABILITY CLASS C
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.04	0.13	0.22	0.06	0.01	0.0	0.46	8.65
NNE	0.0	0.0	0.07	0.18	0.07	0.0	0.0	0.32	9.38
NE	0.0	0.0	0.11	0.14	0.0	0.0	0.0	0.25	7.52
ENE	0.0	0.02	0.11	0.09	0.01	0.0	0.0	0.24	7.73
E	0.0	0.01	0.05	0.04	0.0	0.0	0.0	0.09	7.16
ESE	0.0	0.0	0.02	0.01	0.0	0.0	0.0	0.04	6.36
SE	0.0	0.01	0.11	0.05	0.0	0.0	0.0	0.16	6.09
SSE	0.0	0.0	0.09	0.09	0.0	0.01	0.0	0.20	8.22
S	0.0	0.01	0.14	0.08	0.05	0.01	0.0	0.29	8.82
SSW	0.0	0.01	0.15	0.08	0.11	0.04	0.0	0.39	9.93
SW	0.0	0.02	0.08	0.16	0.06	0.04	0.0	0.37	10.77
WSW	0.0	0.01	0.11	0.11	0.07	0.01	0.0	0.31	9.79
W	0.0	0.02	0.05	0.14	0.02	0.02	0.01	0.27	10.68
WNW	0.0	0.05	0.05	0.09	0.11	0.02	0.0	0.32	10.78
NW	0.0	0.0	0.08	0.21	0.06	0.0	0.0	0.35	9.69
NNW	0.0	0.0	0.08	0.15	0.08	0.01	0.0	0.33	10.61
TOTAL	0.0	0.21	1.43	1.86	0.70	0.18	0.01	4.38	8.89

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 0

TABLE 2.3.3-14 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
UPPER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.13	0.95	1.71	0.55	0.11	0.0	3.45	9.45
NNE	0.0	0.11	0.86	1.53	0.99	0.04	0.0	3.52	10.07
NE	0.0	0.16	0.65	0.82	0.22	0.0	0.0	1.86	8.30
ENE	0.0	0.07	0.48	0.74	0.09	0.04	0.0	1.43	8.40
E	0.0	0.11	0.39	0.24	0.07	0.02	0.01	0.84	7.91
ESE	0.0	0.18	0.45	0.31	0.06	0.01	0.01	1.01	7.07
SE	0.0	0.06	0.54	0.42	0.14	0.04	0.0	1.20	8.24
SSE	0.0	0.09	0.68	0.49	0.33	0.07	0.0	1.67	8.95
S	0.0	0.11	0.52	0.73	0.24	0.05	0.01	1.65	9.11
SSW	0.0	0.15	0.55	0.58	0.61	0.11	0.0	2.00	10.30
SW	0.0	0.19	0.65	0.81	0.44	0.16	0.0	2.25	10.02
WSW	0.0	0.09	0.54	0.68	0.24	0.06	0.0	1.61	9.10
W	0.0	0.12	0.33	0.25	0.18	0.11	0.0	0.98	9.54
WNW	0.0	0.07	0.22	0.44	0.19	0.11	0.0	1.02	10.45
NW	0.0	0.11	0.42	0.80	0.29	0.0	0.0	1.63	9.14
NNW	0.0	0.14	0.53	1.04	0.57	0.09	0.0	2.37	9.94
TOTAL	0.0	1.88	8.78	11.59	5.21	1.00	0.04	28.50	9.12

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 33

TABLE 2.3.3-14 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
UPPER WIND LEVEL
STABILITY CLASS E
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.11	0.38	1.08	0.31	0.0	0.0	1.87	9.34
NNE	0.0	0.04	0.31	0.99	0.34	0.01	0.0	1.68	9.93
NE	0.0	0.02	0.32	0.75	0.07	0.0	0.0	1.17	8.79
ENE	0.0	0.08	0.25	0.61	0.06	0.0	0.0	1.00	8.47
E	0.0	0.04	0.53	0.46	0.14	0.0	0.0	1.17	8.11
ESE	0.0	0.02	0.39	0.37	0.13	0.0	0.0	0.91	8.43
SE	0.0	0.02	0.52	0.68	0.21	0.08	0.01	1.53	9.81
SSE	0.0	0.02	0.49	0.99	0.44	0.04	0.0	1.98	10.12
S	0.0	0.07	0.57	1.46	0.61	0.0	0.0	2.71	9.80
SSW	0.0	0.05	0.72	1.59	0.81	0.07	0.01	3.25	10.32
SW	0.0	0.19	0.60	0.81	0.49	0.09	0.0	2.19	9.65
WSW	0.0	0.09	0.49	0.60	0.16	0.04	0.01	1.40	8.79
W	0.0	0.07	0.25	0.54	0.12	0.0	0.0	0.98	8.67
WNW	0.0	0.01	0.18	0.53	0.37	0.0	0.0	1.08	10.81
NW	0.0	0.02	0.33	0.39	0.20	0.01	0.0	0.95	9.31
NNW	0.0	0.04	0.29	0.79	0.32	0.0	0.0	1.44	9.98
TOTAL	0.0	0.90	6.61	12.65	4.78	0.34	0.04	25.32	9.39

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 3

TABLE 2.3.3-14 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
UPPER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.05	0.12	0.57	0.19	0.0	0.0	0.92	9.87
NNE	0.0	0.04	0.20	0.57	0.12	0.0	0.0	0.92	9.44
NE	0.0	0.01	0.25	0.41	0.08	0.0	0.0	0.75	8.82
ENE	0.0	0.01	0.15	0.35	0.11	0.0	0.0	0.62	9.89
E	0.0	0.04	0.19	0.41	0.06	0.0	0.0	0.70	8.74
ESE	0.0	0.05	0.20	0.29	0.01	0.0	0.0	0.55	7.47
SE	0.0	0.01	0.25	0.34	0.05	0.0	0.0	0.65	8.21
SSE	0.0	0.04	0.27	0.53	0.04	0.0	0.0	0.87	8.53
S	0.0	0.04	0.32	0.73	0.18	0.0	0.0	1.26	9.32
SSW	0.0	0.05	0.25	0.67	0.21	0.0	0.0	1.18	9.52
SW	0.0	0.05	0.16	0.38	0.05	0.0	0.0	0.64	8.30
WSW	0.0	0.01	0.26	0.32	0.02	0.0	0.0	0.61	8.13
W	0.0	0.02	0.18	0.15	0.04	0.0	0.0	0.39	7.64
WNW	0.0	0.02	0.15	0.13	0.04	0.0	0.0	0.34	7.85
NW	0.0	0.06	0.21	0.18	0.04	0.0	0.0	0.48	7.32
NNW	0.0	0.05	0.28	0.41	0.12	0.0	0.0	0.86	8.50
TOTAL	0.0	0.53	3.44	6.44	1.33	0.0	0.0	11.74	8.60

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 3

TABLE 2.3.3-14 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
UPPER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
UPPER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.08	0.37	0.44	0.12	0.0	0.0	1.00	7.97
NNE	0.0	0.11	0.32	0.57	0.04	0.0	0.0	1.02	7.85
NE	0.0	0.14	0.42	0.45	0.16	0.0	0.0	1.18	7.75
ENE	0.0	0.08	0.28	0.27	0.12	0.0	0.0	0.75	8.28
E	0.0	0.08	0.27	0.34	0.12	0.0	0.0	0.81	8.09
ESE	0.01	0.15	0.38	0.25	0.0	0.0	0.0	0.79	6.31
SE	0.0	0.11	0.25	0.29	0.02	0.0	0.0	0.67	6.99
SSE	0.0	0.07	0.28	0.58	0.06	0.0	0.0	0.99	8.24
S	0.01	0.15	0.33	0.54	0.05	0.0	0.0	1.08	7.79
SSW	0.0	0.08	0.62	1.17	0.04	0.0	0.0	1.91	8.08
SW	0.0	0.14	0.54	0.84	0.08	0.0	0.0	1.60	7.94
WSW	0.01	0.24	0.78	0.94	0.09	0.0	0.0	2.60	7.59
W	0.01	0.21	0.65	0.44	0.01	0.0	0.0	1.32	6.42
WNW	0.01	0.21	0.35	0.40	0.02	0.0	0.0	1.00	6.51
NW	0.01	0.22	0.52	0.44	0.0	0.0	0.0	1.19	6.49
NNW	0.0	0.12	0.41	0.41	0.04	0.0	0.0	0.98	7.38
TOTAL	0.07	2.20	6.77	8.35	0.97	0.0	0.0	18.36	7.48

NUMBER OF CALMS - 6

NUMBER OF BAD HOURS - 12

TABLE 2.3.3-14 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
UPPER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

UPPER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.40	2.21	4.50	1.44	0.15	0.0	8.71	9.31
NNE	0.0	0.31	1.98	4.34	1.71	0.05	0.0	8.38	9.62
NE	0.0	0.35	1.94	2.85	0.59	0.0	0.0	5.74	8.34
ENE	0.0	0.31	1.53	2.40	0.44	0.04	0.0	4.71	8.52
E	0.0	0.29	1.65	1.74	0.44	0.02	0.01	4.16	8.12
ESE	0.01	0.44	1.55	1.31	0.24	0.01	0.01	3.57	7.32
SE	0.0	0.22	1.80	1.80	0.45	0.12	0.01	4.41	8.43
SSE	0.0	0.22	2.06	2.76	0.86	0.13	0.0	6.03	9.02
S	0.01	0.40	2.26	3.77	1.31	0.06	0.01	7.82	9.15
SSW	0.0	0.37	2.66	4.51	2.09	0.25	0.01	9.88	9.77
SW	0.0	0.61	2.39	3.36	1.39	0.42	0.0	8.18	9.51
WSW	0.01	0.47	2.39	3.15	0.78	0.14	0.01	6.95	8.67
W	0.01	0.47	1.63	1.85	0.57	0.16	0.02	4.71	8.62
WNW	0.01	0.38	1.14	1.94	0.82	0.21	0.02	4.54	9.57
NW	0.01	0.44	1.79	2.41	0.72	0.04	0.0	5.41	8.51
NNW	0.0	0.35	1.81	3.09	1.39	0.18	0.0	6.82	9.62
TOTAL	0.07	6.03	30.82	45.78	15.21	1.98	0.12	100.00	9.03

NUMBER OF CALMS - 6

NUMBER OF BAD HOURS - 271

TABLE 2.3.3-15
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
LOWER WIND LEVEL
STABILITY CLASS A
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.0	0.40	0.22	0.01	0.0	0.0	0.64	7.12
NNE	0.0	0.0	0.39	0.19	0.0	0.0	0.0	0.58	6.99
NE	0.0	0.01	0.22	0.04	0.0	0.0	0.0	0.27	5.92
ENE	0.0	0.04	0.32	0.08	0.0	0.0	0.0	0.44	5.97
E	0.0	0.04	0.31	0.06	0.0	0.0	0.0	0.40	5.81
ESE	0.0	0.04	0.18	0.05	0.0	0.0	0.0	0.26	5.77
SE	0.0	0.02	0.12	0.02	0.0	0.0	0.0	0.17	4.81
SSE	0.0	0.02	0.11	0.0	0.0	0.0	0.0	0.13	3.99
S	0.0	0.02	0.37	0.09	0.0	0.0	0.0	0.49	6.11
SSW	0.01	0.07	0.54	0.34	0.0	0.0	0.0	0.97	6.60
SW	0.0	0.05	0.41	0.25	0.02	0.0	0.0	0.73	7.20
WSW	0.0	0.02	0.45	0.26	0.05	0.0	0.0	0.78	7.26
W	0.0	0.01	0.31	0.15	0.02	0.0	0.0	0.50	7.32
WNW	0.0	0.01	0.27	0.18	0.05	0.01	0.0	0.52	7.86
NW	0.0	0.01	0.39	0.09	0.0	0.0	0.0	0.50	5.98
NNW	0.0	0.02	0.27	0.27	0.08	0.0	0.0	0.65	8.19
TOTAL	0.01	0.39	5.07	2.31	0.24	0.01	0.0	8.03	6.43

NUMBER OF CALMS - 1

NUMBER OF BAD HOURS - 226

TABLE 2.3.3-15 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
LOWER WIND LEVEL
STABILITY CLASS B
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.01	0.21	0.20	0.0	0.0	0.0	0.43	7.43
NNE	0.0	0.04	0.09	0.09	0.0	0.0	0.0	0.22	6.81
NE	0.0	0.0	0.17	0.05	0.0	0.0	0.0	0.21	6.36
ENE	0.0	0.04	0.15	0.04	0.0	0.0	0.0	0.22	5.48
E	0.0	0.0	0.11	0.04	0.0	0.0	0.0	0.14	5.69
ESE	0.0	0.04	0.05	0.01	0.0	0.0	0.0	0.09	4.22
SE	0.0	0.06	0.02	0.0	0.0	0.0	0.0	0.08	3.24
SSE	0.0	0.05	0.12	0.0	0.01	0.0	0.0	0.18	4.82
S	0.0	0.02	0.08	0.09	0.0	0.0	0.0	0.20	6.78
SSW	0.0	0.02	0.12	0.08	0.0	0.0	0.0	0.22	6.53
SW	0.0	0.02	0.20	0.11	0.08	0.0	0.0	0.41	8.15
WSW	0.0	0.02	0.09	0.06	0.02	0.0	0.0	0.20	7.21
W	0.0	0.01	0.07	0.06	0.04	0.0	0.0	0.18	8.71
WNW	0.0	0.01	0.13	0.08	0.05	0.0	0.0	0.27	8.47
NW	0.0	0.04	0.09	0.15	0.0	0.0	0.0	0.28	7.73
NNW	0.0	0.0	0.15	0.08	0.04	0.0	0.0	0.27	7.62
TOTAL	0.0	0.38	1.87	1.15	0.24	0.0	0.0	3.63	6.58

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 2

TABLE 2.3.3-15 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
LOWER WIND LEVEL
STABILITY CLASS C
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.06	0.30	0.07	0.0	0.0	0.0	0.43	6.09
NNE	0.0	0.02	0.21	0.08	0.0	0.0	0.0	0.32	6.81
NE	0.0	0.04	0.24	0.0	0.0	0.0	0.0	0.27	4.92
ENE	0.0	0.04	0.15	0.05	0.0	0.0	0.0	0.24	5.63
E	0.0	0.04	0.05	0.02	0.0	0.0	0.0	0.11	4.78
ESE	0.0	0.02	0.04	0.0	0.0	0.0	0.0	0.06	4.18
SE	0.0	0.01	0.08	0.0	0.0	0.0	0.0	0.09	5.41
SSE	0.0	0.06	0.15	0.04	0.0	0.0	0.0	0.25	5.28
S	0.0	0.01	0.07	0.09	0.0	0.0	0.0	0.27	6.39
SSW	0.0	0.08	0.20	0.11	0.0	0.0	0.0	0.39	5.90
SW	0.0	0.06	0.14	0.17	0.02	0.0	0.0	0.39	7.34
WSW	0.0	0.02	0.18	0.06	0.04	0.0	0.0	0.30	7.40
W	0.0	0.05	0.12	0.07	0.04	0.0	0.0	0.27	7.45
WNW	0.0	0.0	0.09	0.08	0.06	0.0	0.0	0.24	9.28
NW	0.0	0.01	0.15	0.22	0.0	0.0	0.0	0.39	7.37
NNW	0.0	0.0	0.21	0.13	0.02	0.0	0.0	0.37	7.73
TOTAL	0.0	0.52	2.49	1.20	0.18	0.0	0.0	4.38	6.37

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 2

TABLE 2.3.3-15 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
LOWER WIND LEVEL
STABILITY CLASS D
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.39	2.15	0.73	0.04	0.0	0.0	3.31	6.00
NNE	0.0	0.49	2.31	0.79	0.01	0.0	0.0	3.60	5.77
NE	0.0	0.51	1.14	0.30	0.0	0.0	0.0	1.94	5.18
ENE	0.0	0.38	0.96	0.09	0.01	0.0	0.0	1.44	4.94
E	0.0	0.30	0.45	0.09	0.04	0.0	0.0	0.88	4.90
ESE	0.0	0.31	0.50	0.11	0.04	0.0	0.0	0.95	5.02
SE	0.0	0.24	0.83	0.19	0.0	0.0	0.0	1.25	5.32
SSE	0.0	0.21	0.99	0.30	0.02	0.0	0.0	1.53	5.89
S	0.0	0.39	1.03	0.20	0.05	0.0	0.0	1.67	5.52
SSW	0.0	0.52	0.98	0.52	0.04	0.0	0.0	2.06	5.81
SW	0.0	0.34	1.12	0.75	0.13	0.0	0.0	2.34	6.66
WSW	0.0	0.30	0.90	0.32	0.08	0.0	0.0	1.60	6.31
W	0.0	0.27	0.44	0.24	0.07	0.0	0.0	1.02	6.21
WNW	0.0	0.14	0.49	0.30	0.12	0.0	0.0	1.04	7.09
NW	0.0	0.19	0.80	0.41	0.01	0.0	0.0	1.42	6.21
NNW	0.0	0.22	1.40	0.79	0.13	0.0	0.0	2.54	6.90
TOTAL	0.0	5.20	16.49	6.13	0.78	0.0	0.0	28.60	5.86

NUMBER OF CALMS - 0

NUMBER OF BAD HOURS - 36

TABLE 2.3.3-15 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
LOWER WIND LEVEL
STABILITY CLASS E
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.0	0.62	1.04	0.07	0.0	0.0	0.0	1.73	4.25
NNE	0.01	1.12	1.02	0.12	0.0	0.0	0.0	2.27	3.94
NE	0.0	0.76	0.46	0.04	0.0	0.0	0.0	1.25	3.46
ENE	0.0	0.72	0.52	0.01	0.0	0.0	0.0	1.25	3.30
E	0.0	0.50	0.38	0.04	0.0	0.0	0.0	0.91	3.62
ESE	0.0	0.56	0.18	0.08	0.0	0.0	0.0	0.82	3.50
SE	0.0	0.84	0.71	0.12	0.06	0.0	0.0	1.73	4.20
SSE	0.01	0.97	0.89	0.30	0.01	0.0	0.0	2.18	4.54
S	0.01	1.28	1.11	0.26	0.0	0.0	0.0	2.66	4.03
SSW	0.01	1.47	1.55	0.24	0.01	0.0	0.0	3.28	4.14
SW	0.0	0.67	0.73	0.41	0.04	0.0	0.0	1.86	5.18
WSW	0.0	0.59	0.49	0.05	0.04	0.0	0.0	1.16	4.03
W	0.0	0.36	0.41	0.07	0.0	0.0	0.0	0.84	4.14
WNW	0.0	0.38	0.39	0.24	0.0	0.0	0.0	1.01	5.03
NW	0.0	0.59	0.47	0.12	0.01	0.0	0.0	1.20	4.28
NNW	0.0	0.47	0.45	0.30	0.01	0.0	0.0	1.23	5.04
TOTAL	0.05	11.90	10.81	2.45	0.18	0.0	0.0	25.38	4.17

NUMBER OF CALMS - 4

NUMBER OF BAD HOURS - 8

TABLE 2.3.3-15 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
LOWER WIND LEVEL
STABILITY CLASS F
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.02	0.94	0.36	0.0	0.0	0.0	0.0	1.31	2.90
NNE	0.04	1.04	0.21	0.0	0.0	0.0	0.0	1.29	2.63
NE	0.02	0.94	0.06	0.0	0.0	0.0	0.0	1.02	2.48
ENE	0.02	0.80	0.06	0.0	0.0	0.0	0.0	0.89	2.40
E	0.02	0.67	0.08	0.01	0.0	0.0	0.0	0.79	2.29
ESE	0.01	0.47	0.05	0.0	0.0	0.0	0.0	0.53	2.22
SE	0.01	0.46	0.06	0.0	0.0	0.0	0.0	0.53	2.33
SSE	0.02	0.79	0.11	0.0	0.0	0.0	0.0	0.92	2.18
S	0.02	0.88	0.21	0.02	0.0	0.0	0.0	1.14	2.70
SSW	0.02	0.75	0.22	0.01	0.0	0.0	0.0	1.01	2.71
SW	0.0	0.36	0.06	0.0	0.0	0.0	0.0	0.41	2.39
WSW	0.0	0.28	0.02	0.01	0.0	0.0	0.0	0.32	2.40
W	0.0	0.16	0.02	0.0	0.0	0.0	0.0	0.19	2.37
WNW	0.0	0.32	0.09	0.0	0.0	0.0	0.0	0.41	2.44
NW	0.0	0.26	0.07	0.0	0.0	0.0	0.0	0.33	2.62
NNW	0.0	0.43	0.15	0.0	0.0	0.0	0.0	0.58	2.82
TOTAL	0.22	9.55	1.85	0.06	0.0	0.0	0.0	11.68	2.49

NUMBER OF CALMS - 19

NUMBER OF BAD HOURS - 13

TABLE 2.3.3-15 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
LOWER WIND LEVEL
STABILITY CLASS G
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

		SPEED CLASS (MPH)							
LOWER WIND DIRECTION	CALM	0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.34	1.67	0.09	0.0	0.0	0.0	0.0	2.11	2.01
NNE	0.32	1.52	0.05	0.0	0.0	0.0	0.0	1.88	1.66
NE	0.37	1.79	0.06	0.0	0.0	0.0	0.0	2.21	1.82
ENE	0.31	1.46	0.01	0.01	0.0	0.0	0.0	1.79	1.65
E	0.25	1.17	0.0	0.0	0.0	0.0	0.0	1.42	1.38
ESE	0.19	0.90	0.01	0.0	0.0	0.0	0.0	1.10	1.29
SE	0.19	0.89	0.01	0.0	0.0	0.0	0.0	1.09	1.44
SSE	0.18	0.82	0.0	0.0	0.0	0.0	0.0	0.99	1.25
S	0.19	0.88	0.0	0.01	0.0	0.0	0.0	1.08	1.36
SSW	0.19	0.92	0.01	0.0	0.0	0.0	0.0	1.12	1.58
SW	0.13	0.62	0.01	0.0	0.0	0.0	0.0	0.76	1.49
WSW	0.08	0.40	0.01	0.0	0.0	0.0	0.0	0.50	1.38
W	0.06	0.36	0.02	0.0	0.0	0.0	0.0	0.44	1.65
WNW	0.09	0.44	0.01	0.0	0.0	0.0	0.0	0.54	1.48
NW	0.0	0.32	0.01	0.0	0.0	0.0	0.0	0.33	1.56
NNW	0.15	0.71	0.07	0.0	0.0	0.0	0.0	0.94	1.66
TOTAL	3.04	14.84	0.39	0.02	0.0	0.0	0.0	18.30	1.54

NUMBER OF CALMS - 257

NUMBER OF BAD HOURS - 25

TABLE 2.3.3-15 (CONTINUED)
JOINT PERCENTAGE FREQUENCIES OF WIND DIRECTION AND SPEED
FOR THE PERIOD 12:00 AM 2/1/79 TO 11:00 PM 1/31/80
LOWER WIND LEVEL
SUMMARY
STABILITY CALCULATED FROM DIFFERENTIAL TEMPERATURE
H DIGAMET ON-SITE METEOROLOGICAL FACILITY

LOWER WIND DIRECTION	CALM	SPEED CLASS (MPH) 0.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25.0	> 25.0	TOTAL	AVG. WIND SPEED
N	0.37	3.68	4.56	1.30	0.05	0.0	0.0	9.96	4.68
NNE	0.37	4.23	4.29	1.28	0.01	0.0	0.0	10.17	4.42
NE	0.39	4.04	2.34	0.41	0.0	0.0	0.0	7.19	3.61
ENE	0.33	3.47	2.18	0.28	0.01	0.0	0.0	6.27	3.53
E	0.27	2.71	1.37	0.26	0.04	0.0	0.0	4.65	3.34
ESE	0.20	2.33	0.99	0.25	0.04	0.0	0.0	3.81	3.35
SE	0.20	2.52	1.83	0.33	0.06	0.0	0.0	4.95	3.80
SSE	0.21	2.92	2.37	0.63	0.05	0.0	0.0	6.18	4.11
S	0.22	3.48	2.97	0.78	0.05	0.0	0.0	7.50	4.15
SSW	0.24	3.84	3.63	1.30	0.05	0.0	0.0	9.06	4.51
SW	0.13	2.12	2.69	1.68	0.30	0.0	0.0	6.91	5.71
WSW	0.08	1.65	2.14	0.76	0.22	0.0	0.0	4.85	5.33
W	0.06	1.22	1.40	0.59	0.17	0.0	0.0	3.43	5.36
WNW	0.09	1.30	1.48	0.88	0.27	0.01	0.0	4.04	5.76
NW	0.0	1.42	2.00	1.01	0.02	0.0	0.0	4.45	5.25
NNW	0.15	1.86	2.71	1.57	0.28	0.0	0.0	6.58	5.75
TOTAL	3.33	42.78	38.96	13.32	1.61	0.01	0.0	100.00	4.53

NUMBER OF CALMS - 281

NUMBER OF BAD HOURS - 312

TABLE 2.3.3-16
JOINT OCCURRENCE FREQUENCIES FOR
LOW WIND DIRECTION AND LOW WIND SPEED
RANGES INCLUDE LOWER END POINT, EXCLUDE UPPER END POINT
SITE=SHNP YEAR=76-78 MONTH=01 SUMMARY OVER ALL STABILITY
LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	6.0/ 0.33	49/ 2.67	74/ 4.03	6/ 0.33	/	/	/	135.0/ 7.34	3.96136
NNE	5.2/ 0.28	43/ 2.34	48/ 2.61	6/ 0.33	/	/	/	102.2/ 5.56	3.92621
NE	4.3/ 0.23	35/ 1.90	13/ 0.71	4/ 0.22	/	/	/	56.3/ 3.06	3.01784
ENE	4.4/ 0.24	36/ 1.96	14/ 0.76	/	/	/	/	54.4/ 2.96	2.70496
E	3.8/ 0.21	31/ 1.69	15/ 0.82	/	/	/	/	49.8/ 2.71	2.52861
ESE	3.9/ 0.21	32/ 1.74	16/ 0.87	/	/	/	/	51.9/ 2.82	2.61994
SE	2.9/ 0.16	24/ 1.31	14/ 0.76	/	/	/	/	40.9/ 2.23	3.08802
SSE	3.9/ 0.21	32/ 1.74	28/ 1.52	5/ 0.27	4/ 0.22	/	/	72.9/ 3.97	4.39197
S	5.0/ 0.27	41/ 2.23	29/ 1.58	9/ 0.49	6/ 0.33	1/ 0.05	/	91.0/ 4.95	4.72509
SSW	5.6/ 0.30	46/ 2.50	37/ 2.01	16/ 0.87	5/ 0.27	/	/	109.6/ 5.96	4.47202
SW	7.2/ 0.39	59/ 3.21	54/ 2.94	36/ 1.96	3/ 0.16	/	/	159.2/ 8.66	5.00738
WSW	6.2/ 0.34	51/ 2.77	55/ 2.99	36/ 1.96	12/ 0.65	13/ 0.71	/	173.2/ 9.42	6.89535
W	7.9/ 0.43	65/ 3.54	56/ 3.05	17/ 0.92	8/ 0.44	/	/	153.9/ 8.37	4.62037
WNW	6.0/ 0.33	49/ 2.67	78/ 4.24	91/ 4.95	20/ 1.09	3/ 0.16	/	247.0/13.44	7.34757
NW	3.5/ 0.19	29/ 1.58	104/ 5.66	56/ 3.05	17/ 0.92	/	/	209.5/11.40	6.61317
NNW	4.4/ 0.24	36/ 1.96	63/ 3.43	26/ 1.41	2/ 0.11	/	/	131.4/ 7.15	5.24309
TOTAL	80.0/ 4.35	658/35.80	698/37.98	308/16.76	77. 4.19	17/ 0.92	/	1838/ 100	5.15417

NUMBER OF BAD RECORDS: 67

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=A
LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	6/ 0.33	/	/	/	/	7.0/ 0.38	5.65357
NNE	/	/	3/ 0.16	/	/	/	/	3.0/ 0.16	6.70000
NE	/	/	/	/	/	/	/	/	
ENE	/	/	/	/	/	/	/	/	
E	/	/	/	/	/	/	/	/	
ESE	/	/	/	/	/	/	/	/	
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	/	/	/	/	/	
S	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.75000
SSW	/	1/ 0.05	2/ 0.11	3/ 0.16	/	/	/	6.0/ 0.33	7.37083
SW	/	/	3/ 0.16	4/ 0.22	1/ 0.05	/	/	8.0/ 0.44	8.56875
WSW	/	1/ 0.05	4/ 0.22	11/ 0.60	6/ 0.33	4/ 0.22	/	26.0/ 1.41	11.91442
W	/	/	1/ 0.05	5/ 0.27	2/ 0.11	/	/	8.0/ 0.44	10.42500
WNW	/	/	3/ 0.16	28/ 1.52	7/ 0.38	1/ 0.05	/	39.0/ 2.12	11.06987
NW	/	/	9/ 0.49	13/ 0.71	3/ 0.16	/	/	25.0/ 1.36	8.66200
NNW	/	1/ 0.05	6/ 0.33	3/ 0.16	1/ 0.05	/	/	11.0/ 0.60	7.19697
TOTAL	/	4/ 0.22	38/ 2.07	67/ 3.65	20/ 1.09	5/ 0.27	/	134.0/ 7.29	9.70012

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	6/ 0.33	/	/	/	/	6.0/ 0.33	5.56250
NNE	/	/	2/ 0.11	/	/	/	/	2.0/ 0.11	4.63750
NE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.77500
ENE	/	/	2/ 0.11	/	/	/	/	2.0/ 0.11	5.03750
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	3.77500
ESE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	3.12500
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	/	/	/	/	/	
S	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	9.72500
SSW	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.93333
SW	/	/	4/ 0.22	3/ 0.16	/	/	/	7.0/ 0.38	7.19643
WSW	/	/	4/ 0.22	7/ 0.38	1/ 0.05	/	/	12.0/ 0.65	8.98750
W	/	/	1/ 0.05	1/ 0.05	2/ 0.11	/	/	4.0/ 0.22	11.33125
WNW	/	1/ 0.05	5/ 0.27	18/ 0.98	2/ 0.11	1/ 0.05	/	27.0/ 1.47	9.81574
NW	/	/	7/ 0.38	13/ 0.71	/	/	/	20.0/ 1.09	8.46625
NNW	/	/	2/ 0.11	4/ 0.22	/	/	/	6.0/ 0.33	7.98333
TOTAL	/	2/ 0.11	36/ 1.96	47/ 2.56	5/ 0.27	1/ 0.05	/	91.0/ 4.95	8.42701

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	6/ 0.33	/	/	/	/	6.0/ 0.33	4.95417
NNE	/	/	6/ 0.33	/	/	/	/	6.0/ 0.33	5.32083
NE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.32500
ENE	/	/	2/ 0.11	/	/	/	/	2.0/ 0.11	6.66250
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.55000
ESE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	3.47500
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.17500
SSE	/	/	2/ 0.11	/	/	/	/	2.0/ 0.11	4.23750
S	/	3/ 0.16	2/ 0.11	1/ 0.05	/	/	/	6.0/ 0.33	4.38750
SSW	/	/	4/ 0.22	2/ 0.11	/	/	/	6.0/ 0.33	5.97917
SW	/	3/ 0.16	4/ 0.22	3/ 0.16	/	/	/	10.0/ 0.54	5.77500
WSW	/	/	1/ 0.05	6/ 0.33	/	2/ 0.11	/	9.0/ 0.49	11.66666
W	/	/	1/ 0.05	4/ 0.22	/	/	/	5.0/ 0.27	8.13500
WNW	/	/	5/ 0.27	13/ 0.71	3/ 0.16	/	/	21.0/ 1.14	9.80714
NW	/	/	4/ 0.22	9/ 0.49	2/ 0.11	/	/	15.0/ 0.82	8.95333
NNW	/	/	5/ 0.27	6/ 0.33	/	/	/	11.0/ 0.60	7.96136
TOTAL	/	7/ 0.38	45/ 2.45	44/ 2.39	5/ 0.27	2/ 0.11	/	103.0/ 5.60	7.71286

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.3/ 0.02	13/ 0.71	38/ 2.07	5/ 0.27	/	/	/	56.3/ 3.06	4.54788
NNE	0.3/ 0.02	13/ 0.71	30/ 1.63	6/ 0.33	/	/	/	49.3/ 2.68	5.06973
NE	0.3/ 0.02	16/ 0.87	8/ 0.44	4/ 0.22	/	/	/	28.3/ 1.54	3.82553
ENE	0.2/ 0.01	9/ 0.49	9/ 0.49	/	/	/	/	18.2/ 0.99	3.52335
E	0.1/ 0.01	6/ 0.33	10/ 0.54	/	/	/	/	16.1/ 0.88	3.82298
ESE	0.2/ 0.01	9/ 0.49	13/ 0.71	/	/	/	/	22.2/ 1.21	3.70833
SE	0.2/ 0.01	7/ 0.38	9/ 0.49	/	/	/	/	16.2/ 0.88	4.09105
SSE	0.2/ 0.01	9/ 0.49	17/ 0.92	4/ 0.22	3/ 0.16	/	/	33.2/ 1.81	5.92545
S	0.1/ 0.01	4/ 0.22	11/ 0.60	6/ 0.33	4/ 0.22	1/ 0.05	/	26.1/ 1.42	7.66842
SSW	0.2/ 0.01	7/ 0.38	15/ 0.82	6/ 0.33	4/ 0.22	/	/	32.2/ 1.75	5.95807
SW	0.2/ 0.01	8/ 0.44	24/ 1.31	13/ 0.71	1/ 0.05	/	/	46.2/ 2.51	6.25216
WSW	0.1/ 0.01	4/ 0.22	26/ 1.41	11/ 0.60	5/ 0.27	7/ 0.38	/	53.1/ 2.89	8.67632
W	0.2/ 0.01	11/ 0.60	24/ 1.31	7/ 0.38	4/ 0.22	/	/	46.2/ 2.51	6.13312
WNW	0.2/ 0.01	10/0.54	39/ 2.12	29/ 1.58	8/ 0.44	1/ 0.05	/	87.2/ 4.74	7.57999
NW	0.0/ 0.00	2/ 0.11	33/ 1.80	17/ 0.92	11/ 0.60	/	/	63.0/ 3.43	7.84537
NNW	0.2/ 0.01	10/ 0.54	17/ 0.92	13/ 0.71	1/ 0.05	/	/	41.2/ 2.24	6.03823
TOTAL	3.0/ 0.16	138/ 7.51	323/17.57	121/ 6.58	41/ 2.23	9/ 0.49	/	635.0/34.55	6.16403

NUMBER OF BAD RECORDS: 22

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.3/ 0.02	13/ 0.71	17/ 0.92	1/ 0.05	/	/	/	31.3/ 1.70	4.21446
NNE	0.1/ 0.01	6/ 0.33	7/ 0.38	/	/	/	/	13.1/ 0.71	3.64949
NE	0.1/ 0.01	6/ 0.33	3/ 0.16	/	/	/	/	9.1/ 0.50	3.23809
ENE	0.2/ 0.01	9/ 0.49	/	/	/	/	/	9.2/ 0.50	2.54619
E	0.2/ 0.01	9/ 0.49	3/ 0.16	/	/	/	/	12.2/ 0.66	2.62910
ESE	0.1/ 0.01	3/ 0.16	2/ 0.11	/	/	/	/	5.1/ 0.28	2.93137
SE	0.1/ 0.01	4/ 0.22	4/ 0.22	/	/	/	/	8.1/ 0.44	3.49074
SSE	0.1/ 0.01	5/ 0.27	9/ 0.49	1/ 0.05	1/ 0.05	/	/	16.1/ 0.88	5.17391
S	0.2/ 0.01	11/ 0.60	14/ 0.76	1/ 0.05	2/ 0.11	/	/	28.2/ 1.53	4.97163
SSW	0.1/ 0.01	7/ 0.38	15/ 0.82	5/ 0.27	1/ 0.05	/	/	28.1/ 1.53	5.56939
SW	0.2/ 0.01	9/ 0.49	18/ 0.98	13/ 0.71	1/ 0.05	/	/	41.2/ 2.24	6.12257
WSW	0.2/ 0.01	11/ 0.60	13/ 0.71	1/ 0.05	/	/	/	25.2/ 1.37	4.23710
W	0.2/ 0.01	9/ 0.49	20/ 1.09	/	/	/	/	29.2/ 1.59	4.09418
WNW	0.4/ 0.02	17/ 0.92	23/ 1.25	3/ 0.16	/	/	/	43.4/ 2.36	4.24482
NW	0.3/ 0.02	13/ 0.71	48/ 2.61	4/ 0.22	1/ 0.05	/	/	66.3/ 3.61	4.96191
NNW	0.2/ 0.01	9/ 0.49	33/ 1.80	/	/	/	/	42.2/ 2.30	4.39218
TOTAL	3.0/ 0.16	141/ 7.67	229/12.46	29/ 1.58	6/ 0.33	/	/	408.0/22.20	4.57117

NUMBER OF BAD RECORDS: 8

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.1/ 0.01	4/ 0.22	1/ 0.05	/	/	/	/	5.1/ 0.28	2.53921
NNE	0.3/ 0.02	10/ 0.54	/	/	/	/	/	10.3/ 0.56	1.74636
NE	0.1/ 0.01	4/ 0.22	/	/	/	/	/	4.1/ 0.22	1.49085
ENE	0.3/ 0.02	8/ 0.44	1/ 0.05	/	/	/	/	9.3/ 0.51	2.18817
E	0.1/ 0.01	4/ 0.22	/	/	/	/	/	4.1/ 0.22	1.52134
ESE	0.2/ 0.01	7/ 0.38	1/ 0.05	/	/	/	/	8.2/ 0.45	2.11280
SE	0.2/ 0.01	5/ 0.27	/	/	/	/	/	5.2/ 0.28	1.82211
SSE	0.2/ 0.01	7/ 0.38	/	/	/	/	/	7.2/ 0.39	1.41319
S	0.4/ 0.02	11/ 0.60	1/ 0.05	/	/	/	/	12.4/ 0.67	1.91129
SSW	0.3/ 0.02	9/ 0.49	/	/	/	/	/	9.3/ 0.51	2.04973
SW	0.4/ 0.02	14/ 0.76	1/ 0.05	/	/	/	/	15.4/ 0.84	2.03409
WSW	0.7/ 0.04	21/ 1.14	5/ 0.27	/	/	/	/	26.7/ 1.45	2.49532
W	0.8/ 0.04	26/ 1.41	8/ 0.44	/	/	/	/	34.8/ 1.89	2.74928
WNW	0.4/ 0.02	13/ 0.71	3/ 0.16	/	/	/	/	16.4/ 0.89	2.73780
NW	0.2/ 0.01	6/ 0.33	3/ 0.16	/	/	/	/	9.2/ 0.50	2.97283
NNW	0.3/ 0.02	8/ 0.44	/	/	/	/	/	8.3/ 0.45	2.37500
TOTAL	5.0/ 0.27	157/ 8.54	24/ 1.31	/	/	/	/	186.0/10.12	2.30625

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	5.9/ 0.32	18/ 0.98	/	/	/	/	/	23.9/ 1.30	1.14697
NNE	4.6/ 0.25	14/ 0.76	/	/	/	/	/	18.6/ 1.01	1.10887
NE	3.0/ 0.16	9/ 0.49	/	/	/	/	/	12.0/ 0.65	0.97500
ENE	3.3/ 0.18	10/ 0.54	/	/	/	/	/	13.3/ 0.72	1.07049
E	4.0/ 0.22	12/ 0.65	/	/	/	/	/	16.0/ 0.87	1.04062
ESE	3.6/ 0.20	11/ 0.60	/	/	/	/	/	14.6/ 0.79	0.92979
SE	2.6/ 0.14	8/ 0.44	/	/	/	/	/	10.6/ 0.58	1.52830
SSE	3.6/ 0.20	11/ 0.60	/	/	/	/	/	14.6/ 0.79	1.20034
S	4.0/ 0.22	12/ 0.65	/	/	/	/	/	16.0/ 0.87	1.00312
SSW	7.3/ 0.40	22/ 1.20	/	/	/	/	/	29.3/ 1.59	1.11220
SW	8.3/ 0.45	25/ 1.36	/	/	/	/	/	33.3/ 1.81	1.23986
WSW	4.6/ 0.25	14/ 0.76	2/ 0.11	/	/	/	/	20.6/ 1.12	1.47087
W	6.3/ 0.34	19/ 1.03	1/ 0.05	/	/	/	/	26.3/ 1.43	1.40779
WNW	2.6/ 0.14	8/ 0.44	/	/	/	/	/	10.6/ 0.58	1.21698
NW	2.6/ 0.14	8/ 0.44	/	/	/	/	/	10.6/ 0.58	1.04009
NNW	2.6/ 0.14	8/ 0.44	/	/	/	/	/	10.6/ 0.58	1.45283
TOTAL	69.0/ 3.75	209/11.37	3/ 0.16	/	/	/	/	281.0/15.29	1.19039

NUMBER OF BAD RECORDS: 24

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	9.7/ 0.50	52/ 2.66	68/ 4.50	59/ 3.02	1/ 0.05	/	/	209.7/10.72	5.58123
NNE	11.4/ 0.58	61/ 3.12	48/ 2.45	16/ 0.82	/	/	/	136.4/ 6.97	3.84237
NE	8.0/ 0.41	43/ 2.20	12/ 0.61	3/ 0.15	1/ 0.05	/	/	67.0/ 3.43	2.99552
ENE	5.4/ 0.28	29/ 1.48	11/ 0.56	2/ 0.10	/	/	/	47.4/ 2.42	2.90172
E	7.1/ 0.36	38/ 1.94	12/ 0.61	/	/	/	/	57.1/ 2.92	2.38135
ESE	6.7/ 0.34	36/ 1.84	24/ 1.23	1/ 0.05	/	/	/	67.7/ 3.46	3.06382
SE	4.7/ 0.24	25/ 1.28	20/ 1.02	3/ 0.15	1/ 0.05	/	/	53.7/ 2.75	3.70950
SSE	4.9/ 0.25	26/ 1.33	26/ 1.33	9/ 0.46	1/ 0.05	/	/	66.9/ 3.42	4.28475
S	12.1/ 0.62	65/ 3.32	42/ 2.15	15/ 0.77	4/ 0.20	/	/	138.1/ 7.06	4.14826
SSW	9.5/ 0.49	51/ 2.61	86/ 4.40	42/ 2.15	11/ 0.56	/	/	199.5/10.20	5.86867
SW	9.5/ 0.49	51/ 2.61	66/ 3.37	79/ 4.04	16/ 0.82	/	/	221.5/11.32	6.70011
WSW	6.9/ 0.35	37/ 1.89	55/ 2.81	46/ 2.35	7/ 0.36	1/0.05	/	152.9/ 7.82	6.25956
W	4.7/ 0.24	25/ 1.28	18/ 0.92	21/ 1.07	2/ 0.10	/	/	70.7/ 3.61	5.50306
WNW	6.9/ 0.35	37/ 1.89	40/ 2.04	42/ 2.15	7/ 0.36	/	/	132.9/ 6.79	6.07669
NW	6.0/ 0.31	32/ 1.64	58/ 2.97	69/ 3.53	15/ 0.77	/	/	180.0/ 9.20	7.09509
NNW	7.5/ 0.38	40/ 2.04	62/ 3.17	41/ 2.10	4/ 0.20	/	/	154.5/ 7.90	5.57362
TOTAL	121.0/ 6.19	648/33.13	668/34.15	448/22.90	70/ 3.58	1/ 0.05	/	1956/ 100	5.30762

NUMBER OF BAD RECORDS: 84

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	6/ 0.31	13/ 0.66	/	/	/	19.0/ 0.97	8.29605
NNE	/	/	2/ 0.10	7/ 0.36	/	/	/	9.0/0.46	8.28889
NE	/	/	/	2/ 0.10	/	/	/	2.0/ 0.10	8.76250
ENE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.25000
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.30000
ESE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	8.05000
SE	/	/	/	/	/	/	/	/	
SSE	/	/	2/ 0.10	1/ 0.05	1/ 0.05	/	/	4.0/ 0.20	9.13125
S	/	/	2/ 0.10	/	1/ 0.05	/	/	3.0/ 0.15	8.98333
SSW	/	/	1/ 0.05	14/ 0.72	2/ 0.10	/	/	17.0/ 0.87	10.10000
SW	/	/	1/ 0.05	31/ 1.58	12/ 0.61	/	/	44.0/ 2.25	10.84716
WSW	/	1/ 0.05	5/ 0.26	10/ 0.51	5/ 0.26	/	/	21.0/ 1.07	9.52619
W	/	/	1/ 0.05	8/ 0.41	1/ 0.05	/	/	10.0/ 0.51	9.14750
WNW	/	/	3/ 0.15	11/ 0.56	/	/	/	14.0/ 0.72	8.23333
NW	/	/	4/ 0.20	10/ 0.51	/	/	/	14.0/ 0.72	8.93988
NNW	/	/	6/ 0.31	11/ 0.56	2/ 0.10	/	/	19.0/ 0.97	8.63947
TOTAL	/	1/ 0.05	35/ 1.79	119/ 6.08	24/ 1.23	/	/	179.0/ 9.15	9.36257

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	6/ 0.31	4/ 0.20	/	/	/	10.0/ 0.51	7.13000
NNE	/	1/ 0.05	4/ 0.20	3/ 0.15	/	/	/	8.0/ 0.41	6.56250
NE	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	6.21250
ENE	/	/	/	/	/	/	/	/	
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.05000
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.87500
SE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	8.62500
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.20000
S	/	/	1/ 0.05	2/ 0.10	1/ 0.05	/	/	4.0/ 0.20	10.62500
SSW	/	/	2/ 0.10	4/ 0.20	2/ 0.10	/	/	8.0/ 0.41	9.63437
SW	/	/	2/ 0.10	9/ 0.46	2/ 0.10	/	/	13.0/ 0.66	10.39808
WSW	/	3/ 0.15	1/ 0.05	3/ 0.15	/	/	/	7.0/ 0.36	6.60357
W	/	/	/	1/ 0.05	1/ 0.05	/	/	2.0/ 0.10	10.22500
WNW	/	/	1/ 0.05	5/ 0.26	3/ 0.15	/	/	9.0/ 0.46	10.57778
NW	/	/	4/ 0.20	8/ 0.41	3/ 0.15	/	/	15.0/ 0.77	9.43500
NNW	/	/	4/ 0.20	8/ 0.41	1/ 0.05	/	/	13.0/ 0.66	8.08654
TOTAL	/	4/ 0.20	30/ 1.53	48/ 2.45	13/ 0.66	/	/	95.0/ 4.86	8.65526

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	5/ 0.26	4/ 0.20	/	/	/	9.0/ 0.46	7.55000
NNE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	6.55000
NE	/	/	3/ 0.15	/	/	/	/	3.0/ 0.15	4.15000
ENE	/	/	1/ 0.05	2/ 0.10	/	/	/	3.0/ 0.15	7.47500
E	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	4.72500
ESE	/	/	/	/	/	/	/	/	
SE	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	4.62500
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.12500
S	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.42500
SSW	/	1/ 0.05	2/ 0.10	5/ 0.26	2/ 0.10	/	/	10.0/ 0.51	9.91250
SW	/	/	2/ 0.10	6/ 0.31	1/ 0.05	/	/	9.0/ 0.46	9.98889
WSW	/	/	3/ 0.15	4/ 0.20	2/ 0.10	/	/	9.0/ 0.46	9.10000
W	/	/	2/ 0.10	3/ 0.15	/	/	/	5.0/ 0.26	8.15500
WNW	/	/	4/ 0.20	5/ 0.26	2/ 0.10	/	/	11.0/ 0.56	9.76136
NW	/	/	6/ 0.31	11/ 0.56	4/ 0.20	/	/	21.0/ 1.07	9.23294
NNW	/	/	5/ 0.26	8/ 0.41	/	/	/	13.0/ 0.66	7.52500
TOTAL	/	1/ 0.05	40/ 2.04	49/ 2.51	11/ 0.56	/	/	101.0/ 5.16	8.45511

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	1.0/ 0.05	6/ 0.31	36/ 1.84	31/ 1.58	1/ 0.05	/	/	75.0/ 3.83	6.91567
NNE	1.6/ 0.08	9/ 0.46	21/ 1.07	4/ 0.20	/	/	/	35.6/ 1.82	4.75211
NE	0.7/ 0.04	4/ 0.20	5/ 0.26	1/ 0.05	1/ 0.05	/	/	11.7/ 0.60	4.57906
ENE	0.3/ 0.02	2/ 0.10	6/ 0.31	/	/	/	/	8.3/ 0.42	3.81024
E	0.3/ 0.02	2/ 0.10	4/ 0.20	/	/	/	/	6.3/ 0.32	3.49603
ESE	0.3/ 0.02	2/ 0.10	15/ 0.77	/	/	/	/	17.3/ 0.88	4.73266
SE	0.3/ 0.02	2/ 0.10	8/ 0.41	2/ 0.10	1/ 0.05	/	/	13.3/ 0.68	6.17481
SSE	0.7/ 0.04	4/ 0.20	7/ 0.36	7/ 0.36	/	/	/	18.7/ 0.96	5.85628
S	0.9/ 0.05	5/ 0.26	10/ 0.51	8/ 0.41	2/ 0.10	/	/	25.9/ 1.32	6.99710
SSW	1.0/ 0.05	6/ 0.31	23/ 1.18	7/ 0.36	4/ 0.20	/	/	41.0/ 2.10	6.58841
SW	1.2/ 0.06	7/ 0.36	20/ 1.02	12/ 0.61	1/ 0.05	/	/	41.2/ 2.11	6.29308
WSW	0.3/ 0.02	2/ 0.10	17/ 0.87	11/ 0.56	/	1/ 0.05	/	31.3/ 1.60	7.15974
W	0.5/ 0.03	3/ 0.15	5/ 0.26	7/ 0.36	/	/	/	15.5/ 0.79	6.70403
WNW	0.3/ 0.02	2/ 0.10	17/ 0.87	20/ 1.02	2/ 0.10	/	/	41.3/ 2.11	7.72700
NW	/	/	14/ 0.72	35/ 1.79	7/ 0.36	/	/	56.0/ 2.86	9.34851
NNW	1.2/ 0.06	7/ 0.36	24/ 1.23	8/ 0.41	1/ 0.05	/	/	41.2/ 2.11	5.87136
TOTAL	11.0/ 0.56	63/ 3.22	232/11.86	153/ 7.82	20/ 1.02	1/ 0.05	/	480.0/24.54	6.64951

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.3/ 0.02	19/ 0.97	28/ 1.43	5/ 0.26	/	/	/	52.3/ 2.67	4.64651
NNE	0.4/ 0.02	23/ 1.18	18/ 0.92	1/ 0.05	/	/	/	42.4/ 2.17	3.49410
NE	0.4/ 0.02	23/ 1.18	2/ 0.10	/	/	/	/	25.4/ 1.30	2.62894
ENE	0.2/ 0.01	11/ 0.56	1/ 0.05	/	/	/	/	12.2/ 0.62	2.47336
E	0.1/ 0.01	7/ 0.36	3/ 0.15	/	/	/	/	10.1/ 0.52	3.34158
ESE	0.1/ 0.01	7/ 0.36	7/ 0.36	/	/	/	/	14.1/ 0.72	3.73581
SE	0.1/ 0.01	8/ 0.41	7/ 0.36	/	/	/	/	15.1/ 0.77	3.70033
SSE	0.0/ 0.00	1/ 0.05	10/ 0.51	1/ 0.05	/	/	/	12.0/ 0.61	5.44792
S	0.1/ 0.01	9/ 0.46	17/ 0.87	5/ 0.26	/	/	/	31.1/ 1.59	5.20619
SSW	0.3/ 0.02	17/ 0.87	45/ 2.30	12/ 0.61	1/ 0.05	/	/	75.3/ 3.85	5.69422
SW	0.2/ 0.01	12/ 0.61	30/ 1.53	21/ 1.07	/	/	/	63.2/ 3.23	6.28402
WSW	0.1/ 0.01	7/ 0.36	16/ 0.82	18/ 0.92	/	/	/	41.1/ 2.10	6.83323
W	0.1/ 0.01	7/ 0.36	10/ 0.51	2/ 0.10	/	/	/	19.1/ 0.98	4.87107
WNW	0.2/ 0.01	14/ 0.72	13/ 0.66	1/ 0.05	/	/	/	28.2/ 1.44	3.80585
NW	0.1/ 0.01	7/ 0.36	21/ 1.07	5/ 0.26	1/ 0.05	/	/	34.1/ 1.74	5.66276
NNW	0.2/ 0.01	13/ 0.66	15/ 0.77	5/ 0.26	/	/	/	33.2/ 1.70	4.74623
TOTAL	3.0/ 0.15	185/ 9.46	243/12.42	76/ 3.89	2/ 0.10	/	/	509.0/26.02	4.94206

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.2/ 0.01	13/ 0.66	6/ 0.31	2/ 0.10	/	/	/	21.2/ 1.08	3.45873
NNE	0.1/ 0.01	7/ 0.36	2/ 0.10	/	/	/	/	9.1/ 0.47	3.12637
NE	0.1/ 0.01	5/ 0.26	/	/	/	/	/	5.1/ 0.26	2.07108
ENE	0.0/ 0.00	4/ 0.20	2/ 0.10	/	/	/		6.0/ 0.31	3.30000
E	0.1/ 0.01	11/ 0.56	1/ 0.05	/	/	/	/	12.1/ 0.62	2.37397
ESE	0.1/ 0.01	8/ 0.41	1/ 0.05	/	/	/	/	9.1/ 0.47	2.49725
SE	0.1/ 0.01	5/ 0.26	2/ 0.10	/	/	/	/	7.1/ 0.36	2.75352
SSE	0.1/ 0.01	9/ 0.46	3/ 0.15	/	/	/	/	12.1/ 0.62	2.54545
S	0.3/ 0.02	23/ 1.18	10/ 0.51	/	/	/	/	33.3/ 1.70	2.84159
SSW	0.2/ 0.01	13/ 0.66	13/ 0.66	/	/	/	/	26.2/ 1.34	3.31202
SW	0.2/ 0.01	16/ 0.82	7/ 0.36	/	/	/	/	23.2/ 1.19	2.92780
WSW	0.1/ 0.01	7/ 0.36	8/ 0.41	/	/	/	/	15.1/ 0.77	4.26656
W	0.1/ 0.01	8/ 0.41	/	/	/	/	/	8.1/ 0.41	2.45833
WNW	0.1/ 0.01	11/ 0.56	2/ 0.10	/	/	/	/	13.1/ 0.67	2.81298
NW	0.2/ 0.01	13/ 0.66	8/ 0.41	/	/	/	/	21.2/ 1.08	3.06368
NNW	0.1/ 0.01	10/ 0.51	8/ 0.41	1/ 0.05	/	/	/	19.1/ 0.98	3.79516
TOTAL	2.0/ 0.10	163/ 8.33	73/ 3.73	3/ 0.15	/	/	/	241.0/12.32	3.07837

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	6.4/ 0.33	14/ 0.72	1/ 0.05	/	/	/	/	21.4/ 1.09	1.43964
NNE	9.5/ 0.49	21/ 1.07	/	/	/	/	/	30.5/ 1.56	1.02254
NE	5.0/ 0.26	11/ 0.56	/	/	/	/	/	16.0/ 0.82	1.16562
ENE	5.5/ 0.28	12/ 0.61	/	/	/	/	/	17.5/ 0.89	1.46452
E	8.2/ 0.42	18/ 0.92	/	/	/	/	/	26.2/ 1.34	1.13740
ESE	8.6/ 0.44	19/ 0.97	/	/	/	/	/	27.6/ 1.41	1.20592
SE	4.5/ 0.23	10/ 0.51	1/ 0.05	/	/	/	/	15.5/ 0.79	1.33548
SSE	5.5/ 0.28	12/ 0.61	2/ 0.10	/	/	/	/	19.5/ 1.00	1.73077
S	12.7/ 0.65	28/ 1.43	1/ 0.05	/	/	/	/	41.7/ 2.13	1.30216
SSW	6.4/ 0.33	14/ 0.72	/	/	/	/	/	20.4/ 1.04	1.34681
SW	7.3/ 0.37	16/ 0.82	4/ 0.20	/	/	/	/	27.3/ 1.40	1.85989
WSW	7.7/ 0.39	17/ 0.87	5/ 0.26	/	/	/	/	29.7/ 1.52	1.78507
W	3.2/ 0.16	7/ 0.36	/	/	/	/	/	10.2/ 0.52	1.40850
WNW	4.5/ 0.23	10/ 0.51	/	/	/	/	/	14.5/ 0.74	1.27845
NW	5.5/ 0.28	12/ 0.61	1/ 0.05	/	/	/	/	18.5/ 0.95	1.64324
NNW	4.5/ 0.23	10/ 0.51	/	/	/	/	/	14.5/ 0.74	1.00948
TOTAL	105.0/ 5.37	231/11.81	15/ 0.77	/	/	/	/	351.0/ 17.94	1.38837

NUMBER OF RECORDS: 47

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	6.6/ 0.30	46/ 2.12	100/ 4.61	41/ 1.89	1/ 0.05	/	/	194.6/ 8.98	5.24743
NNE	5.6/ 0.26	39/ 1.80	66/ 3.04	10/ 0.46	/	/	/	120.6/ 5.56	4.35966
NE	5.4/ 0.25	38/ 1.75	37/ 1.71	9/ 0.42	/	/	/	89.4/ 4.12	4.12108
ENE	6.7/ 0.31	47/ 2.17	40/ 1.85	20/ 0.92	1/ 0.05	/	/	114.7/ 5.29	4.56778
E	5.3/ 0.24	37/ 1.71	30/ 1.38	3/ 0.14	/	/	/	75.3/ 3.47	3.34700
ESE	5.3/ 0.24	37/ 1.71	51/ 2.35	2/ 0.09	/	/	/	95.3/ 4.40	3.95370
SE	5.0/ 0.23	35/ 1.61	57/ 2.63	14/ 0.65	/	/	/	111.0/ 5.12	4.55953
SSE	6.2/ 0.29	43/ 1.98	54/ 2.49	22/ 1.01	1/ 0.05	/	/	126.2/ 5.82	4.80111
S	9.6/ 0.44	67/ 3.09	76/ 3.51	27/ 1.25	3/ 0.14	/	/	182.6/ 8.42	4.75164
SSW	6.9/ 0.32	48/ 2.21	117/ 5.40	73/ 3.37	12/ 0.55	/	/	256.9/11.85	6.24377
SW	5.3/ 0.24	37/ 1.71	63/ 2.91	52/ 2.40	38/ 1.75	5/ 0.23	/	200.3/ 9.24	7.84791
WSW	3.9/ 0.18	27/ 1.25	53/ 2.44	23/ 1.06	8/ 0.37	3/ 0.14	/	117.9/ 5.44	6.36157
W	3.4/ 0.16	24/ 1.11	40/ 1.85	10/ 0.46	3/ 0.14	2/ 0.09	/	82.4/ 3.80	5.53580
WNW	3.0/ 0.14	21/ 0.97	36/ 1.66	34/ 1.57	18/ 0.83	4/ 0.18	/	116.0/ 5.35	7.88527
NW	3.4/ 0.16	24/ 1.11	65/ 3.00	34/ 1.57	6/ 0.28	/	/	132.4/ 6.11	6.19725
NNW	4.3/ 0.20	30/ 1.38	82/ 3.78	35/ 1.61	1/ 0.05	/	/	152.3/ 7.02	5.63280
TOTAL	86.0/ 3.97	600/27.68	967/44.60	409/18.87	92/ 4.24	14/ 0.65	/	2168/ 100	5.54565

NUMBER OF BAD RECORDS: 64

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	6/ 0.28	5/ 0.23	/	/	/	11.0/ 0.51	7.16591
NNE	/	/	6/ 0.28	4/ 0.18	/	/	/	10.0/ 0.46	7.38000
NE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.80000
ENE	/	/	/	/	/	/	/	/	
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.60000
ESE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.75000
SE	/	/	/	/	/	/	/	/	
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.60000
S	/	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	3.0/ 0.14	11.40000
SSW	/	/	6/ 0.28	21/ 0.97	1/ 0.05	/	/	28.0/ 1.29	9.17589
SW	/	1/ 0.05	7/ 0.32	14/ 0.65	15/ 0.69	/	/	37.0/ 1.71	10.53513
WSW	/	/	6/ 0.28	4/ 0.18	1/ 0.05	/	/	11.0/ 0.51	7.71591
W	/	/	1/ 0.05	/	/	1/ 0.05	/	2.0/ 0.09	13.67500
WNW	/	/	1/ 0.05	7/ 0.32	4/ 0.18	1/ 0.05	/	13.0/ 0.60	12.73974
NW	/	/	6/ 0.28	5/ 0.23	/	/	/	11.0/ 0.51	8.02045
NNW	/	/	3/ 0.14	3/ 0.14	/	/	/	6.0/ 0.28	8.20417
TOTAL	/	1/ 0.05	48/ 2.21	64/ 2.95	22/ 1.01	2/ 0.09	/	137.0/ 6.32	9.30906

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	8/ 0.37	1/ 0.05	/	/	/	9.0/ 0.42	6.75278
NNE	/	/	2/ 0.09	2/ 0.09	/	/	/	4.0/ 0.18	7.31875
NE	/	/	/	/	/	/	/	/	
ENE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	3.10000
E	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	7.67500
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	7.00000
SE	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	8.08333
SSE	/	/	/	/	/	/	/	/	
S	/	1/ 0.05	/	2/ 0.09	/	/	/	3.0/ 0.14	6.02500
SSW	/	/	1/ 0.05	4/ 0.18	/	/	/	5.0/ 0.23	9.17000
SW	/	/	5/ 0.23	1/ 0.05	7/ 0.32	2/ 0.09	/	15.0/ 0.69	11.70000
WSW	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	8.21667
W	/	/	1/ 0.05	/	/	1/ 0.05	/	2.0/ 0.09	13.92499
WNW	/	/	/	2/ 0.09	5/ 0.23	1/ 0.05	/	8.0/ 0.37	13.95937
NW	/	/	4/ 0.18	6/ 0.28	/	/	/	10.0/ 0.46	8.86750
NNW	/	/	3/ 0.14	1/ 0.05	/	/	/	4.0/ 0.18	7.22500
TOTAL	/	2/ 0.09	29/ 1.34	22/ 1.01	12/ 0.55	4/ 0.18	/	69.0/ 3.18	9.46739

NUMBER OF BAD RECORDS: 4

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	5/ 0.23	8/ 0.37	/	/	/	13.0/ 0.60	7.59038
NNE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.17500
NE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.72500
ENE	/	/	1/ 0.05	2/ 0.09	/	/	/	3.0/ 0.14	8.82500
E	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.09	5.15833
ESE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	7.50000
SE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	6.91250
SSE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	6.63750
S	/	/	2/ 0.09	1/ 0.05	1/ 0.05	/	/	4.0/ 0.18	9.18750
SSW	/	/	3/ 0.14	2/ 0.09	2/ 0.09	/	/	7.0/ 0.32	8.92500
SW	/	/	2/ 0.09	3/ 0.14	6/ 0.28	/	/	11.0/ 0.51	11.71818
WSW	/	/	3/ 0.14	4/ 0.18	/	1/ 0.05	/	8.0/ 0.37	9.93750
W	/	/	6/ 0.28	2/ 0.09	/	/	/	8.0/ 0.37	7.14687
WNW	/	1/ 0.05	1/ 0.05	/	4/ 0.18	1/ 0.05	/	7.0/ 0.32	11.54286
NW	/	/	5/ 0.23	6/ 0.28	2/ 0.09	/	/	13.0/ 0.60	8.29551
NNW	/	/	5/ 0.23	3/ 0.14	/	/	/	8.0/ 0.37	7.45312
TOTAL	/	2/ 0.09	41/ 1.89	34/ 1.57	15/ 0.69	2/ 0.09	/	94.0/ 4.34	8.64291

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.1/ 0.00	7/ 0.32	48/ 2.21	23/ 1.06	1/ 0.05	/	/	79.1/ 3.65	6.34545
NNE	0.1/ 0.00	8/ 0.37	27/ 1.25	2/ 0.09	/	/	/	37.1/ 1.71	5.05290
NE	0.1/ 0.00	7/ 0.32	5/ 0.23	5/ 0.23	/	/	/	17.1/ 0.79	5.15570
ENE	0.1/ 0.00	8/ 0.37	22/ 1.01	9/ 0.42	/	/	/	39.1/ 1.80	5.61285
E	0.1/ 0.00	5/ 0.23	11/ 0.51	2/ 0.09	/	/	/	18.1/ 0.83	4.82251
ESE	0.0/ 0.00	3/ 0.14	31/ 1.43	1/ 0.05	/	/	/	35.0/ 1.61	5.34428
SE	0.1/ 0.00	4/ 0.18	28/ 1.29	11/ 0.51	/	/	/	43.1/ 1.99	6.14704
SSE	/	/	24/ 1.11	18/ 0.83	1/ 0.05	/	/	43.0/ 1.98	7.41337
S	0.1/ 0.00	4/ 0.18	14/ 0.65	7/ 0.32	/	/	/	25.1/ 1.16	6.30827
SSW	0.1/ 0.00	4/ 0.18	24/ 1.11	30/ 1.38	7/ 0.32	/	/	65.1/ 3.00	7.99962
SW	0.1/ 0.00	5/ 0.23	23/ 1.06	28/ 1.29	9/ 0.42	3/ 0.14	/	68.1/ 3.14	8.74908
WSW	0.0/ 0.00	2/ 0.09	28/ 1.29	11/ 0.51	6/ 0.28	2/ 0.09	/	49.0/ 2.26	7.92143
W	0.1/ 0.00	5/ 0.23	11/ 0.51	4/ 0.18	3/ 0.14	/	/	23.1/ 1.07	6.69697
WNW	0.0/ 0.00	3/ 0.14	13/ 0.60	19/ 0.88	5/ 0.23	/	/	40.0/ 1.85	8.20500
NW	0.1/ 0.00	5/ 0.23	16/ 0.74	12/ 0.55	3/ 0.14	/	/	36.1/ 1.67	6.95914
NNW	0.0/ 0.00	1/ 0.05	29/ 1.34	21/ 0.97	1/ 0.05	/	/	52.0/ 2.40	7.10817
TOTAL	1.0/ 0.05	71/ 3.27	354/16.33	203/ 9.36	36/ 1.66	5/ 0.23	/	670.0/30.90	6.89834

NUMBER OF BAD RECORDS: 21

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.1/ 0.00	5/ 0.23	28/ 1.29	4/ 0.18	/	/	/	37.1/ 1.71	5.05559
NNE	0.3/ 0.01	11/ 0.51	22/ 1.01	2/ 0.09	/	/	/	35.3/ 1.63	4.45999
NE	0.1/ 0.00	3/ 0.14	27/ 1.25	4/ 0.18	/	/	/	34.1/ 1.57	5.66239
ENE	0.2/ 0.01	7/ 0.32	10/ 0.46	9/ 0.42	1/ 0.05	/	/	27.2/ 1.25	6.46507
E	0.1/ 0.00	5/ 0.23	11/ 0.51	/	/	/	/	16.1/ 0.74	3.77329
ESE	0.1/ 0.00	5/ 0.23	11/ 0.51	/	/	/	/	16.1/ 0.74	4.23913
SE	0.1/ 0.00	5/ 0.23	25/ 1.15	1/ 0.05	/	/	/	31.1/ 1.43	4.84847
SSE	0.2/ 0.01	10/ 0.46	23/ 1.06	3/ 0.14	/	/	/	36.2/ 1.67	4.71063
S	0.4/ 0.02	16/ 0.74	44/ 2.03	16/ 0.74	1/ 0.05	/	/	77.4/ 3.57	5.76712
SSW	0.3/ 0.01	13/ 0.60	65/ 3.00	16/ 0.74	2/ 0.09	/	/	96.3/ 4.44	5.91433
SW	0.3/ 0.01	13/ 0.60	18/ 0.83	5/ 0.23	1/ 0.05	/	/	37.3/ 1.72	5.18230
WSW	0.2/ 0.01	10/ 0.46	9/ 0.42	3/ 0.14	1/ 0.05	/	/	23.2/ 1.07	4.74784
W	0.2/ 0.01	8/ 0.37	12/ 0.55	4/ 0.18	/	/	/	24.2/ 1.12	4.85537
WNW	0.1/ 0.00	4/ 0.18	15/ 0.69	6/ 0.28	/	1/ 0.05	/	26.1/ 1.20	6.28640
NW	0.1/ 0.00	6/ 0.28	32/ 1.48	4/ 0.18	1/ 0.05	/	/	43.1/ 1.99	5.49362
NNW	0.2/ 0.01	8/ 0.37	32/ 1.48	7/ 0.32	/	/	/	47.2/ 2.18	5.39725
TOTAL	3.0/ 0.14	129/ 5.95	384/17.71	84/ 3.87	7/ 0.32	1/ 0.05	/	608.0/28.04	5.35645

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.4/ 0.02	7/ 0.32	4/ 0.18	/	/	/	/	11.4/ 0.53	2.91667
NNE	0.5/ 0.02	8/ 0.37	7/ 0.32	/	/	/	/	15.5/ 0.71	3.05968
NE	0.6/ 0.03	11/ 0.51	1/ 0.05	/	/	/	/	12.6/ 0.58	2.56548
ENE	0.6/ 0.03	10/ 0.46	4/ 0.18	/	/	/	/	14.6/ 0.67	2.84931
E	0.5/ 0.02	9/ 0.42	5/ 0.23	/	/	/	/	14.5/ 0.67	3.14828
ESE	0.6/ 0.03	10/ 0.46	3/ 0.14	/	/	/	/	13.6/ 0.63	2.67647
SE	0.2/ 0.01	4/ 0.18	1/ 0.05	/	/	/	/	5.2/ 0.24	2.62981
SSE	0.9/ 0.04	15/ 0.69	4/ 0.18	/	/	/	/	19.9/ 0.92	2.70352
S	1.6/ 0.07	28/ 1.29	12/ 0.55	/	/	/	/	41.6/ 1.92	2.87139
SSW	0.8/ 0.04	13/ 0.60	16/ 0.74	/	/	/	/	29.8/ 1.37	3.32382
SW	0.5/ 0.02	9/ 0.42	7/ 0.32	/	/	/	/	16.5/ 0.76	2.95454
WSW	0.3/ 0.01	5/ 0.23	4/ 0.18	/	/	/	/	9.3/ 0.43	3.62366
W	0.4/ 0.02	7/ 0.32	8/ 0.37	/	/	/	/	15.4/ 0.71	3.66071
WNW	0.3/ 0.01	6/ 0.28	4/ 0.18	/	/	/	/	10.3/ 0.48	3.48786
NW	0.2/ 0.01	4/ 0.18	/	/	/	/	/	4.2/ 0.19	2.50595
NNW	0.5/ 0.02	9/ 0.42	10/ 0.46	/	/	/	/	19.5/ 0.90	3.65641
TOTAL	9.0/ 0.42	155/ 7.15	90/ 4.15	/	/	/	/	254.0/11.72	3.06772

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	8.2/ 0.38	27/ 1.25	1/ 0.05	/	/	/	/	36.2/ 1.67	1.57320
NNE	3.6/ 0.17	12/ 0.55	/	/	/	/	/	15.6/ 0.72	1.01923
NE	5.2/ 0.24	17/ 0.78	1/ 0.05	/	/	/	/	23.2/ 1.07	1.59914
ENE	6.4/ 0.30	21/ 0.97	3/ 0.14	/	/	/	/	30.4/ 1.40	1.74507
E	5.2/ 0.24	17/ 0.78	1/ 0.05	/	/	/	/	23.2/ 1.07	1.43319
ESE	5.8/ 0.27	19/ 0.88	2/ 0.09	/	/	/	/	26.8/ 1.24	1.88433
SE	6.7/ 0.31	22/ 1.01	/	/	/	/	/	28.7/ 1.32	1.25073
SSE	5.5/ 0.25	18/ 0.83	1/ 0.05	/	/	/	/	24.5/ 1.13	1.65918
S	5.5/ 0.25	18/ 0.83	3/ 0.14	/	/	/	/	26.5/ 1.22	1.73962
SSW	5.5/ 0.25	18/ 0.83	2/ 0.09	/	/	/	/	25.5/ 1.18	1.74020
SW	2.7/ 0.12	9/ 0.42	1/ 0.05	1/ 0.05	/	/	/	13.7/ 0.63	2.48084
WSW	3.0/ 0.14	10/ 0.46	1/ 0.05	/	/	/	/	14.0/ 0.65	1.74762
W	1.2/ 0.06	4/ 0.18	1/ 0.05	/	/	/	/	6.2/ 0.29	1.67742
WNW	2.1/ 0.10	7/ 0.32	2/ 0.09	/	/	/	/	11.1/ 0.51	2.22748
NW	2.7/ 0.12	9/ 0.42	2/ 0.09	1/ 0.05	/	/	/	14.7/ 0.68	2.41922
NNW	3.6/ 0.17	12/ 0.55	/	/	/	/	/	15.6/ 0.72	1.27885
TOTAL	73.0/ 3.37	240/11.07	21/ 0.97	2/ 0.09	/	/	/	336.0/15.50	1.67519

NUMBER OF BAD RECORDS: 20

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	5.0/ 0.25	35/ 1.78	64/ 3.26	35/ 1.78	9/ 0.46	/	/	148.0/ 7.54	5.98564
NNE	6.7/ 0.34	47/ 2.40	54/ 2.75	12/ 0.61	/	/	/	119.7/ 6.10	4.25345
NE	5.8/ 0.30	41/ 2.09	32/ 1.63	12/ 0.61	1/ 0.05	/	/	91.8/ 4.68	4.00735
ENE	4.8/ 0.24	34/ 1.73	35/ 1.78	5/ 0.25	/	/	/	78.8/ 4.02	3.57297
E	7.3/ 0.37	51/ 2.60	31/ 1.58	5/ 0.25	/	/	/	94.3/ 4.81	3.29706
ESE	5.6/ 0.29	39/ 1.99	25/ 1.27	2/ 0.10	/	/	/	71.6/ 3.65	3.17842
SE	4.3/ 0.22	30/ 1.53	29/ 1.48	/	/	/	/	63.3/ 3.23	3.20524
SSE	6.6/ 0.34	46/ 2.34	38/ 1.94	1/ 0.05	/	/	/	91.6/ 4.67	3.42440
S	10.4/ 0.53	73/ 3.72	57/ 2.91	11/ 0.56	1/ 0.05	/	/	152.4/ 7.77	3.64091
SSW	9.0/ 0.46	63/ 3.21	114/ 5.81	25/ 1.27	4/ 0.20	/	/	215.0/10.96	4.87198
SW	9.0/ 0.46	63/ 3.21	80/ 4.08	53/ 2.70	9/ 0.46	/	/	214.0/10.91	5.76355
WSW	4.7/ 0.24	33/ 1.68	63/ 3.21	33/ 1.68	21/ 1.07	4/ 0.20	1/ 0.05	159.7/ 8.14	7.20157
W	2.9/ 0.15	20/ 1.02	38/ 1.94	32/ 1.63	4/ 0.20	/	/	96.9/ 4.94	6.33591
WNW	3.9/ 0.20	27/ 1.38	51/ 2.60	29/ 1.48	9/ 0.46	/	/	119.9/ 6.11	6.22790
NW	5.4/ 0.28	38/ 1.94	40/ 2.04	36/ 1.83	1/ 0.05	/	/	120.4/ 6.14	5.39867
NNW	4.7/ 0.24	33/ 1.68	53/ 2.70	26/ 1.33	8/ 0.41	/	/	124.7/ 6.36	5.82739
TOTAL	96.0/ 4.89	673/34.30	804/40.98	317/16.16	67/ 3.41	4/ 0.20	1/ 0.05	1962/ 100	5.00646

NUMBER OF BAD RECORDS: 198

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	3/ 0.15	4/ 0.20	/	/	/	7.0/ 0.36	7.70000
NNE	/	/	2/ 0.10	1/ 0.05	/	/	/	3.0/ 0.15	6.75833
NE	/	/	1/ 0.05	3/ 0.15	/	/	/	4.0/ 0.20	7.94375
ENE	/	/	3/ 0.15	/	/	/	/	3.0/ 0.15	5.07500
E	/	/	/	/	/	/	/	/	
ESE	/	/	/	/	/	/	/	/	
SE	/	/	/	/	/	/	/	/	
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.00000
S	/	/	/	/	/	/	/	/	
SSW	/	/	2/ 0.10	3/ 0.15	/	/	/	5.0/ 0.25	7.18500
SW	/	/	1/ 0.05	3/ 0.15	1/ 0.05	/	/	5.0/ 0.25	10.56500
WSW	/	/	1/ 0.05	2/ 0.10	6/ 0.31	/	/	9.0/ 0.46	12.21944
W	/	/	/	5/ 0.25	1/ 0.05	/	/	6.0/ 0.31	11.47917
WNW	/	/	4/ 0.20	5/ 0.25	5/ 0.25	/	/	14.0/ 0.71	10.62143
NW	/	/	3/ 0.15	2/ 0.10	1/ 0.05	/	/	6.0/ 0.31	8.47083
NNW	/	/	2/ 0.10	2/ 0.10	/	/	/	4.0/ 0.20	8.08125
TOTAL	/	/	23/ 1.17	30/ 1.53	14/ 0.71	/	/	67.0/ 3.41	9.33769

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	9/ 0.46	2/ 0.10	/	/	/	11.0/ 0.56	6.36364
NNE	/	/	14/ 0.71	1/ 0.05	/	/	/	15.0/ 0.76	6.51000
NE	/	/	4/ 0.20	1/ 0.05	/	/	/	5.0/ 0.25	6.04500
ENE	/	/	/	/	/	/	/	/	
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.62500
ESE	/	/	3/ 0.15	/	/	/	/	3.0/ 0.15	5.41667
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	3.60000
SSE	/	/	/	/	/	/	/	/	
S	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.35000
SSW	/	/	6/ 0.31	1/ 0.05	/	/	/	7.0/ 0.36	6.45000
SW	/	/	3/ 0.15	2/ 0.10	/	/	/	5.0/ 0.25	8.32000
WSW	/	1/ 0.05	1/ 0.05	3/ 0.15	3/ 0.15	1/ 0.05	1/ 0.05	10.0/ 0.51	12.22500
W	/	/	1/ 0.05	3/ 0.15	/	/	/	4.0/ 0.20	9.28750
WNW	/	/	5/ 0.25	5/ 0.25	1/ 0.05	/	/	11.0/ 0.56	7.92954
NW	/	1/ 0.05	2/ 0.10	7/ 0.36	/	/	/	10.0/ 0.51	8.27750
NNW	/	/	3/ 0.15	2/ 0.10	/	/	/	5.0/ 0.25	7.43500
TOTAL	/	2/ 0.10	54/ 2.75	27/ 1.38	4/ 0.20	1/ 0.05	1/ 0.05	89.0/ 4.54	7.68567

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	4/ 0.20	1/ 0.05	/	/	/	5.0/ 0.25	6.48000
NNE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.97500
NE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	7.72500
ENE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	7.33750
E	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.10	5.13750
ESE	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	4.96250
SE	/	/	4/ 0.20	/	/	/	/	4.0/ 0.20	4.63750
SSE	/	/	5/ 0.20	/	/	/	/	5.0/ 0.25	6.15500
S	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	6.00000
SSW	/	/	9/ 0.46	2/ 0.10	/	/	/	11.0/ 0.56	6.78409
SW	/	/	6/ 0.31	2/ 0.10	/	/	/	8.0/ 0.41	6.93437
WSW	/	/	2/ 0.10	5/ 0.25	3/ 0.15	/	/	10.0/ 0.51	11.64750
W	/	/	2/ 0.10	1/ 0.05	1/ 0.05	/	/	4.0/ 0.20	9.08125
WNW	/	/	4/ 0.20	1/ 0.05	1/ 0.05	/	/	6.0/ 0.31	7.40833
NW	/	/	4/ 0.20	4/ 0.20	/	/	/	8.0/ 0.41	7.23750
NNW	/	/	8/ 0.41	2/ 0.10	/	/	/	10.0/ 0.51	6.22000
TOTAL	/	1/ 0.05	55/ 2.80	20/ 1.02	5/ 0.25	/	/	81.0/ 4.13	7.28086

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	3/ 0.15	22/ 1.12	9/ 0.46	6/ 0.31	/	/	40.0/ 2.04	7.25000
NNE	/	2/ 0.10	9/ 0.46	8/ 0.41	/	/	/	19.0/ 0.97	6.69868
NE	/	2/ 0.10	7/ 0.36	1/ 0.05	1/ 0.05	/	/	11.0/ 0.56	5.65227
ENE	/	2/ 0.10	8/ 0.41	2/ 0.10	/	/	/	12.0/ 0.61	5.01667
E	/	2/ 0.10	15/ 0.76	1/ 0.05	/	/	/	18.0/ 0.92	5.26111
ESE	/	5/ 0.25	10/ 0.51	/	/	/	/	15.0/ 0.76	4.31500
SE	/	3/ 0.15	14/ 0.71	/	/	/	/	17.0/ 0.87	4.60833
SSE	/	3/ 0.15	13/ 0.66	1/ 0.05	/	/	/	17.0/ 0.87	4.80882
S	/	2/ 0.10	15/ 0.76	6/ 0.31	1/ 0.05	/	/	24.0/ 1.22	6.41979
SSW	/	3/ 0.15	37/ 1.89	13/ 0.66	3/ 0.15	/	/	56.0/ 2.85	6.95152
SW	/	6/ 0.31	21/ 1.07	33/ 1.68	8/ 0.41	/	/	68.0/ 3.47	8.24228
WSW	/	5/ 0.25	17/ 0.87	16/ 0.82	6/ 0.31	3/ 0.15	/	47.0/ 2.40	8.74397
W	/	5/ 0.25	16/ 0.82	12/ 0.61	2/ 0.10	/	/	35.0/ 1.78	7.12786
WNW	/	6/ 0.31	20/ 1.02	12/ 0.61	2/ 0.10	/	/	40.0/ 2.04	6.69500
NW	/	3/ 0.15	15/ 0.76	20/ 1.02	/	/	/	38.0/ 1.94	7.33026
NNW	/	6/ 0.31	24/ 1.22	18/ 0.92	2/ 0.10	/	/	50.0/ 2.55	6.95850
TOTAL	/	58/ 2.96	263/ 13.40	152/ 7.75	31/ 1.58	3/ 0.15	/	507.0/ 25.84	6.93719

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.2/ 0.01	9/ 0.46	17/ 0.87	19/ 0.97	3/ 0.15	/	/	48.2/ 2.46	7.13537
NNE	0.2/ 0.01	11/ 0.56	21/ 1.07	2/ 0.10	/	/	/	34.2/ 1.74	4.55044
NE	0.2/ 0.01	11/ 0.56	16/ 0.82	6/ 0.31	/	/	/	33.2/ 1.69	4.91792
ENE	0.1/ 0.01	6/ 0.31	19/ 0.97	2/ 0.10	/	/	/	27.1/ 1.38	4.78275
E	0.2/ 0.01	9/ 0.46	8/ 0.41	4/ 0.20	/	/	/	21.2/ 1.08	4.85849
ESE	0.2/ 0.01	10/ 0.51	10/ 0.51	2/ 0.10	/	/	/	22.2/ 1.13	4.13063
SE	0.2/ 0.01	10/ 0.51	5/ 0.25	/	/	/	/	15.2/ 0.77	3.37500
SSE	0.3/ 0.02	17/ 0.87	16/ 0.82	/	/	/	/	33.3/ 1.70	3.89790
S	0.4/ 0.02	20/ 1.02	30/ 1.53	5/ 0.25	/	/	/	55.4/ 2.82	4.30596
SSW	0.2/ 0.01	13/ 0.66	39/ 1.99	6/ 0.31	1/ 0.05	/	/	59.2/ 3.02	5.19510
SW	0.2/ 0.01	12/ 0.61	29/ 1.48	13/ 0.66	/	/	/	54.2/ 2.76	5.86946
WSW	0.1/ 0.01	3/ 0.15	28/ 1.43	7/ 0.36	3/ 0.15	/	/	41.1/ 2.09	6.52920
W	0.1/ 0.01	4/ 0.20	14/ 0.71	11/ 0.56	/	/	/	29.1/ 1.48	6.18943
WNW	0.2/ 0.01	9/ 0.46	14/ 0.71	6/ 0.31	/	/	/	29.2/ 1.49	5.23288
NW	0.2/ 0.01	11/ 0.56	9/ 0.46	3/ 0.15	/	/	/	23.2/ 1.18	4.45259
NNW	0.1/ 0.01	7/ 0.36	10/ 0.51	2/ 0.10	6/ 0.31	/	/	25.1/ 1.28	6.85458
TOTAL	3.0/ 0.15	162/ 8.26	285/14.53	88/ 4.49	13/ 0.66	/	/	551.0/28.08	5.27963

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.2/ 0.01	5/ 0.25	5/ 0.25	/	/	/	/	10.2/ 0.52	3.52696
NNE	0.5/ 0.03	12/ 0.61	5/ 0.25	/	/	/	/	17.5/ 0.89	3.07143
NE	0.3/ 0.02	8/ 0.41	4/ 0.20	/	/	/	/	12.3/ 0.63	2.88821
ENE	0.3/ 0.02	7/ 0.36	3/ 0.15	/	/	/	/	10.3/ 0.52	2.85680
E	0.3/ 0.02	7/ 0.36	4/ 0.20	/	/	/	/	11.3/ 0.58	3.14823
ESE	0.4/ 0.02	9/ 0.46	/	/	/	/	/	9.4/ 0.48	1.92553
SE	0.2/ 0.01	6/ 0.31	4/ 0.20	/	/	/	/	10.2/ 0.52	2.92647
SSE	0.4/ 0.02	10/ 0.51	2/ 0.10	/	/	/	/	12.4/ 0.63	2.20968
S	0.8/ 0.04	21/ 1.07	9/ 0.46	/	/	/	/	30.8/ 1.57	2.87906
SSW	0.7/ 0.04	18/ 0.92	21/ 1.07	/	/	/	/	39.7/ 2.02	3.53400
SW	0.8/ 0.04	20/ 1.02	18/ 0.92	/	/	/	/	38.8/ 1.98	3.67912
WSW	0.4/ 0.02	10/ 0.51	13/ 0.66	/	/	/	/	23.4/ 1.19	3.61752
W	0.0/ 0.00	1/ 0.05	5/ 0.25	/	/	/	/	6.0/ 0.31	3.66667
WNW	0.1/ 0.01	2/ 0.10	3/ 0.15	/	/	/	/	5.1/ 0.26	3.46569
NW	0.2/ 0.01	6/ 0.31	5/ 0.25	/	/	/	/	11.2/ 0.57	2.97768
NNW	0.4/ 0.02	9/ 0.46	4/ 0.20	/	/	/	/	13.4/ 0.68	2.77425
TOTAL	6.0/ 0.31	151/ 7.70	105/ 5.35	/	/	/	/	262.0/13.35	3.17624

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	5.2/ 0.27	18/ 0.92	4/ 0.20	/	/	/	/	27.2/ 1.39	2.07812
NNE	6.4/ 0.33	22/ 1.12	2/ 0.10	/	/	/	/	30.4/ 1.55	1.52796
NE	5.8/ 0.30	20/ 1.02	/	/	/	/	/	25.8/ 1.31	1.20543
ENE	5.5/ 0.28	19/ 0.97	1/ 0.05	/	/	/	/	25.5/ 1.30	1.23431
E	9.3/ 0.47	32/ 1.63	2/ 0.10	/	/	/	/	43.3/ 2.21	1.38453
ESE	4.4/ 0.22	15/ 0.76	/	/	/	/	/	19.4/ 0.99	1.16495
SE	3.2/ 0.16	11/ 0.56	1/ 0.05	/	/	/	/	15.2/ 0.77	1.36349
SSE	4.7/ 0.24	16/ 0.82	1/ 0.05	/	/	/	/	21.7/ 1.11	1.63940
S	8.7/ 0.44	30/ 1.53	/	/	/	/	/	38.7/ 1.97	1.13534
SSW	8.4/ 0.43	29/ 1.48	/	/	/	/	/	37.4/ 1.91	1.22126
SW	7.3/ 0.37	25/ 1.27	2/ 0.10	/	/	/	/	34.3/ 1.75	1.60204
WSW	4.1/ 0.21	14/ 0.71	1/ 0.05	/	/	/	/	19.1/ 0.97	1.59031
W	2.9/ 0.15	10/ 0.51	/	/	/	/	/	12.9/ 0.66	1.30717
WNW	2.9/ 0.15	10/ 0.51	1/ 0.05	/	/	/	/	13.9/ 0.71	1.77698
NW	4.9/ 0.25	17/ 0.87	2/ 0.10	/	/	/	/	23.9/ 1.22	1.64226
NNW	3.2/ 0.16	11/ 0.56	2/ 0.10	/	/	/	/	16.2/ 0.83	2.07562
TOTAL	87.0/ 4.43	299/15.24	19/ 0.97	/	/	/	/	405.0/20.64	1.46611

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	10.9/ 0.51	76/ 3.53	60/ 2.79	22/ 1.02	/	/	/	168.9/ 7.85	4.13901
NNE	11.0/ 0.51	77/ 3.58	50/ 2.32	12/ 0.56	/	/	/	150.0/ 6.97	3.57183
NE	12.7/ 0.59	89/ 4.14	53/ 2.46	11/ 0.51	/	/	/	165.7/ 7.70	3.51260
ENE	8.9/ 0.41	62/ 2.88	49/ 2.28	7/ 0.33	/	/	/	126.9/ 5.90	3.56925
E	8.7/ 0.40	61/ 2.83	52/ 2.42	7/ 0.33	/	/	/	128.7/ 5.98	3.55818
ESE	7.6/ 0.35	53/ 2.46	35/ 1.63	8/ 0.37	/	/	/	103.6/ 4.81	3.44401
SE	5.9/ 0.27	41/ 1.91	43/ 2.00	14/ 0.65	/	/	/	103.9/ 4.83	4.02635
SSE	9.9/ 0.46	69/ 3.21	47/ 2.18	11/ 0.51	2/ 0.09	/	/	138.9/ 6.45	3.90065
S	11.2/ 0.52	78/ 3.62	64/ 2.97	16/ 0.74	1/ 0.05	/	/	170.2/ 7.91	3.84724
SSW	12.7/ 0.59	89/ 4.14	54/ 2.51	20/ 0.93	/	/	/	175.7/ 8.16	3.69586
SW	10.5/ 0.49	73/ 3.39	50/ 2.32	43/ 2.00	/	/	/	176.5/ 8.20	4.63510
WSW	6.6/ 0.31	46/ 2.14	55/ 2.56	31/ 1.44	2/ 0.09	/	/	140.6/ 6.53	5.18279
W	5.0/ 0.23	35/ 1.63	41/ 1.91	16/ 0.74	1/ 0.05	/	/	98.0/ 4.55	4.81122
WNW	4.4/ 0.20	31/ 1.44	31/ 1.44	13/ 0.60	5/ 0.23	/	/	84.4/ 3.92	5.10940
NW	4.4/ 0.20	31/ 1.44	32/ 1.49	17/ 0.79	/	/	/	84.4/ 3.92	4.55351
NNW	5.6/ 0.26	39/ 1.81	41/ 1.91	45/ 2.09	5/ 0.23	/	/	135.6/ 6.30	5.75885
TOTAL	136.0/ 6.32	950/44.14	757/35.18	293/13.62	16/ 0.74	/	/	2152/ 100	4.16525

NUMBER OF BAD RECORDS: 80

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	3/ 0.14	1/ 0.05	/	/	/	5.0/ 0.23	6.56333
NNE	/	/	6/ 0.28	5/ 0.23	/	/	/	11.0/ 0.51	7.15000
NE	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	/	3.0/ 0.14	5.32500
ENE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.85000
E	/	/	/	/	/	/	/	/	
ESE	/	2/ 0.09	1/ 0.05	/	/	/	/	3.0/ 0.14	2.65833
SE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	8.72500
SSE	/	/	/	/	/	/	/	/	
S	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.07500
SSW	/	1/ 0.05	6/ 0.28	1/ 0.05	/	/	/	8.0/ 0.37	6.23437
SW	/	/	6/ 0.28	2/ 0.09	/	/	/	8.0/ 0.37	6.33958
WSW	/	/	8/ 0.37	5/ 0.23	/	/	/	13.0/ 0.60	6.95000
W	/	/	6/ 0.28	1/ 0.05	/	/	/	7.0/ 0.33	5.99286
WNW	/	/	7/ 0.33	5/ 0.23	/	/	/	12.0/ 0.56	7.09583
NW	/	/	8/ 0.37	5/ 0.23	/	/	/	13.0/ 0.60	6.39808
NNW	/	/	4/ 0.19	15/ 0.70	4/ 0.19	/	/	23.0/ 1.07	9.65000
TOTAL	/	5/ 0.23	58/ 2.70	42/ 1.95	4/ 0.19	/	/	109.0/ 5.07	7.14893

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	1/ 0.05	/	/	/	/	1.0/ 0/05	4.65000
NNE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	8.11250
NE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	7.07500
ENE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	7.36250
E	/	/	/	2/ 0.09	/	/	/	2.0/ 0.09	8.08750
ESE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	7.77500
SE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	9.40000
SSE	/	/	/	/	/	/	/	/	
S	/	/	/	/	/	/	/	/	
SSW	/	/	2/ 0.09	2/ 0.09	/	/	/	4.0/ 0.19	6.73125
SW	/	/	2/ 0.09	3/ 0.14	/	/	/	5.0/ 0.23	8.17000
WSW	/	1/ 0.05	2/ 0.09	4/ 0.19	/	/	/	7.0/ 0.33	7.29643
W	/	/	2/ 0.09	2/ 0.09	1/ 0.05	/	/	5.0/ 0.23	9.18000
WNW	/	1/ 0.05	1/ 0.05	1/ 0.05	2/ 0.09	/	/	5.0/ 0.23	10.26000
NW	/	1/ 0.05	2/ 0.09	1/ 0.05	/	/	/	4.0/ 0.19	6.02292
NNW	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	7.40000
TOTAL	/	3/0.14	17/ 0.79	19/ 0.88	3/ 0.14	/	/	42.0/ 1.95	7.87242

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	9.00000
NNE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.45000
NE	/	/	/	/	/	/	/	/	
ENE	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.09	4.68750
E	/	/	5/ 0.23	/	/	/	/	5.0/ 0.23	6.30000
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	3.95000
SE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	7.51250
SSE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.47500
S	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.02500
SSW	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	6.02500
SW	/	1/ 0.05	/	6/ 0.28	/	/	/	7.0/ 0.33	8.55714
WSW	/	/	/	5/ 0.23	1/ 0.05	/	/	6.0/ 0.28	10.38333
W	/	/	/	5/ 0.23	/	/	/	5.0/ 0.23	10.36500
WNW	/	/	1/ 0.05	1/ 0.05	3/ 0.14	/	/	5.0/ 0.23	11.81000
NW	/	1/ 0.05	1/ 0.05	2/ 0.09	/	/	/	4.0/ 0.19	7.20000
NNW	/	/	3/ 0.14	2/ 0.09	1/ 0.05	/	/	6.0/ 0.28	8.22083
TOTAL	/	3/ 0.14	19/ 0.88	24/ 1.12	5/ 0.23	/	/	51.0/ 2.37	8.26569

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	3/ 0.14	31/ 1.44	14/ 0.65	/	/	/	48.0/ 2.23	6.39844
NNE	/	6/ 0.28	15/ 0.70	5/ 0.23	/	/	/	26.0/ 1.21	5.31635
NE	/	5/ 0.23	28/ 1.30	7/ 0.33	/	/	/	40.0/ 1.86	5.50750
ENE	/	5/ 0.23	31/ 1.44	5/ 0.23	/	/	/	41.0/ 1.91	5.38658
E	/	10/ 0.46	24/ 1.12	2/ 0.09	/	/	/	36.0/ 1.67	4.70208
ESE	/	6/ 0.28	14/ 0.65	4/ 0.19	/	/	/	24.0/ 1.12	5.18437
SE	/	6/ 0.28	29/ 1.35	11/ 0.51	/	/	/	46.0/ 2.14	5.47717
SSE	/	10/ 0.46	26/ 1.21	9/ 0.42	/	/	/	45.0/ 2.09	5.54555
S	/	8/ 0.37	23/ 1.07	3/ 0.14	/	/	/	34.0/ 1.58	4.89706
SSW	/	6/ 0.28	19/ 0.88	13/ 0.60	/	/	/	38.0/ 1.77	5.81974
SW	/	5/ 0.23	26/ 1.21	24/ 1.12	/	/	/	55.0/ 2.56	6.80258
WSW	/	8/ 0.37	30/ 1.39	17/ 0.79	1/ 0.05	/	/	56.0/ 2.60	6.50223
W	/	7/ 0.33	20/ 0.93	8/ 0.37	/	/	/	35.0/ 1.63	5.81714
WNW	/	2/ 0.09	14/ 0.65	6/ 0.28	/	/	/	22.0/ 1.02	6.24470
NW	/	2/ 0.09	13/ 0.60	9/ 0.42	/	/	/	24.0/ 1.12	6.49687
NNW	/	1/ 0.05	13/ 0.60	25/ 1.16	/	/	/	39.0/ 1.81	7.83846
TOTAL	/	90/ 4.18	356/16.54	162/ 7.53	1/ 0.05	/	/	609.0/28.30	5.92812

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.2/ 0.01	13/ 0.60	19/ 0.88	6/ 0.28	/	/	/	38.2/ 1.78	5.41427
NNE	0.4/ 0.02	24/ 1.12	22/ 1.02	1/ 0.05	/	/	/	47.4/ 2.20	3.87500
NE	0.5/ 0.02	37/ 1.72	22/ 1.02	2/ 0.09	/	/	/	61.5/ 2.86	3.76037
ENE	0.3/ 0.01	19/ 0.88	12/ 0.56	2/ 0.09	/	/	/	33.3/ 1.55	3.57620
E	0.3/ 0.01	19/ 0.88	21/ 0.98	3/ 0.14	/	/	/	43.3/ 2.01	3.88251
ESE	0.1/ 0.00	10/ 0.46	17/ 0.79	3/ 0.14	/	/	/	30.1/ 1.40	4.51204
SE	0.2/ 0.01	12/ 0.56	10/ 0.46	/	/	/	/	22.2/ 1.03	3.37049
SSE	0.4/ 0.02	24/ 1.12	19/ 0.88	2/ 0.09	2/ 0.09	/	/	47.4/ 2.20	4.18565
S	0.4/ 0.02	26/ 1.21	34/ 1.58	13/ 0.60	1/ 0.05	/	/	74.4/ 3.46	4.91700
SSW	0.4/ 0.02	27/ 1.25	22/ 1.02	3/ 0.14	/	/	/	52.4/ 2.43	3.96565
SW	0.2/ 0.01	15/ 0.70	14/ 0.65	8/ 0.37	/	/	/	37.2/ 1.73	4.74933
WSW	0.0/ 0.00	3/ 0.14	13/ 0.60	/	/	/	/	16.0/ 0.74	5.06875
W	0.1/ 0.00	8/ 0.37	9/ 0.42	/	/	/	/	17.1/ 0.79	3.88743
WNW	0.2/ 0.01	11/ 0.51	6/ 0.28	/	/	/	/	17.2/ 0.80	3.19767
NW	0.2/ 0.01	13/ 0.60	7/ 0.33	/	/	/	/	20.2/ 0.94	3.07426
NNW	0.2/ 0.01	12/ 0.56	16/ 0.74	3/ 0.14	/	/	/	31.2/ 1.45	4.24119
TOTAL	4.0/ 0.19	273/12.69	263/12.22	46/ 2.14	3/ 0.14	/	/	589.0/27.37	4.18557

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.5/ 0.02	16/ 0.74	5/ 0.23	/	/	/	/	21.5/ 1.00	2.49535
NNE	0.5/ 0.02	16/ 0.74	3/ 0.14	/	/	/	/	19.5/ 0.91	2.20256
NE	0.5/ 0.02	16/ 0.74	1/ 0.05	/	/	/	/	17.5/ 0.81	1.97857
ENE	0.4/ 0.02	13/ 0.60	1/ 0.05	/	/	/	/	14.4/ 0.67	1.95486
E	0.3/ 0.01	9/ 0.42	2/ 0.09	/	/	/	/	11.3/ 0.53	2.50000
ESE	0.4/ 0.02	14/ 0.65	1/ 0.05	/	/	/	/	15.4/ 0.72	2.05519
SE	0.4/ 0.02	12/ 0.56	1/ 0.05	/	/	/	/	13.4/ 0.62	2.24067
SSE	0.6/ 0.03	20/ 0.93	/	/	/	/	/	20.6/ 0.96	2.13228
S	0.5/ 0.02	15/ 0.70	5/ 0.23	/	/	/	/	20.5/ 0.95	2.71341
SSW	1.0/ 0.05	33/ 1.53	3/ 0.14	/	/	/	/	37.0/ 1.72	2.19459
SW	0.7/ 0.03	22/ 1.02	1/ 0.05	/	/	/	/	23.7/ 1.10	2.13291
WSW	0.3/ 0.01	9/ 0.42	1/ 0.05	/	/	/	/	10.3/ 0.48	2.50485
W	0.3/ 0.01	8/ 0.37	4/ 0.19	/	/	/	/	12.3/ 0.57	2.94309
WNW	0.3/ 0.01	8/ 0.37	2/ 0.09	/	/	/	/	10.3/ 0.48	2.27913
NW	0.1/ 0.00	3/ 0.14	1/ 0.05	/	/	/	/	4.1/ 0.19	2.82317
NNW	0.3/ 0.01	9/ 0.42	3/ 0.14	/	/	/	/	12.3/ 0.57	2.39024
TOTAL	7.0/ 0.33	223/10.36	34/ 1.58	/	/	/	/	264.0/12.27	2.29943

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	15.2/ 0.71	43/ 2.00	1/ 0.05	/	/	/	/	59.2/ 2.75	1.43159
NNE	11.0/ 0.51	31/ 1.44	2/ 0.09	/	/	/	/	44.0/ 2.04	1.50682
NE	10.6/ 0.49	30/ 1.39	/	/	/	/	/	40.6/ 1.89	1.38793
ENE	8.5/ 0.39	24/ 1.12	1/ 0.05	/	/	/	/	33.5/ 1.56	1.51940
E	8.1/ 0.38	23/ 1.07	/	/	/	/	/	31.1/ 1.45	1.23593
ESE	7.4/ 0.34	21/ 0.98	1/ 0.05	/	/	/	/	29.4/ 1.37	1.44983
SE	3.9/ 0.18	11/ 0.51	2/ 0.09	/	/	/	/	16.9/ 0.79	1.48669
SSE	5.3/ 0.25	15/ 0.70	/	/	/	/	/	20.3/ 0.94	1.32697
S	10.3/ 0.48	29/ 1.35	/	/	/	/	/	39.3/ 1.83	1.11419
SSW	7.8/ 0.36	22/ 1.02	/	/	/	/	/	29.8/ 1.38	1.14681
SW	10.6/ 0.49	30/ 1.39	1/ 0.05	/	/	/	/	41.6/ 1.93	1.49399
WSW	8.9/ 0.41	25/ 1.16	1/ 0.05	/	/	/	/	34.9/ 1.62	1.45988
W	4.2/ 0.20	12/ 0.56	/	/	/	/	/	16.2/ 0.75	1.22994
WNW	3.2/ 0.15	9/ 0.42	/	/	/	/	/	12.2/ 0.57	1.21721
NW	3.9/ 0.18	11/ 0.51	/	/	/	/	/	14.9/ 0.69	1.03272
NNW	6.0/ 0.28	17/ 0.79	1/ 0.05	/	/	/	/	24.0/ 1.12	1.30625
TOTAL	125.0/ 5.81	353/16.40	10/ 0.46	/	/	/	/	488.0/22.68	1.36066

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	6.1/ 0.29	51/ 2.40	51/ 2.40	9/ 0.42	/	/	/	117.1/ 5.51	3.75801
NNE	7.8/ 0.37	65/ 3.06	53/ 2.49	4/ 0.19	/	/	/	129.8/ 6.11	3.58417
NE	6.7/ 0.32	56/ 2.64	60/ 2.82	10/ 0.47	2/ 0.09	/	/	134.7/ 6.34	4.45397
ENE	7.9/ 0.37	66/ 3.11	36/ 1.69	3/ 0.14	/	/	/	112.9/ 5.31	3.04218
E	6.6/ 0.31	55/ 2.59	37/ 1.74	1/ 0.05	/	/	/	99.6/ 4.69	3.06375
ESE	7.3/ 0.34	61/ 2.87	21/ 0.99	/	/	/	/	89.3/ 4.20	2.61058
SE	6.1/ 0.29	51/ 2.40	29/ 1.36	/	/	/	/	86.1/ 4.05	2.83217
SSE	11.0/ 0.52	92/ 4.33	59/ 2.78	2/ 0.09	/	/	/	164.0/ 7.72	3.15051
S	15.6/ 0.73	131/ 6.16	83/ 3.91	9/ 0.42	/	/	/	238.6/11.23	3.36966
SSW	11.3/ 0.53	95/ 4.47	101/ 4.75	19/ 0.89	/	/	/	226.3/10.65	4.13292
SW	10.9/ 0.51	91/ 4.28	72/ 3.39	35/ 1.65	1/ 0.05	/	/	209.9/ 9.88	4.40718
WSW	6.4/ 0.30	54/ 2.54	76/ 3.58	39/ 1.84	/	/	/	175.4/ 8.25	5.17717
W	2.9/ 0.14	24/ 1.13	29/ 1.36	9/ 0.42	2/ 0.09	/	/	66.9/ 3.15	4.57374
WNW	3.2/ 0.15	27/ 1.27	43/ 2.02	4/ 0.19	2/ 0.09	/	/	79.2/ 3.73	4.39930
NW	3.9/ 0.18	33/ 1.55	32/ 1.51	20/ 0.94	2/ 0.09	/	/	90.9/ 4.28	5.06436
NNW	5.4/ 0.25	45/ 2.12	40/ 1.88	14/ 0.66	/	/	/	104.4/ 4.91	4.12739
TOTAL	119.0/ 5.60	997/46.92	822/38.68	178/ 8.38	9/ 0.42	/	/	2125/ 100	3.88969

NUMBER OF BAD RECORDS: 35

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	/	/	/	/	/	/	
NNE	/	/	/	/	/	/	/	/	
NE	/	/	/	2/ 0.09	1/ 0.05	/	/	3.0/ 0.14	11.86667
ENE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	7.51250
E	/	/	/	/	/	/	/	/	
ESE	/	/	/	/	/	/	/	/	
SE	/	/	/	/	/	/	/	/	
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.85000
S	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.85000
SSW	/	/	2/ 0.09	4/ 0.19	/	/	/	6.0/ 0.28	9.02500
SW	/	1/ 0.05	1/ 0.05	4/ 0.19	/	/	/	6.0/ 0.28	7.36250
WSW	/	2/ 0.09	1/ 0.05	2/ 0.09	/	/	/	5.0/ 0.24	6.41000
W	/	/	/	1/ 0.05	1/ 0.05	/	/	2.0/ 0.09	12.15000
WNW	/	/	/	/	1/ 0.05	/	/	1.0/ 0.05	13.50000
NW	/	/	/	3/ 0.14	/	/	/	3.0/ 0.14	10.48333
NNW	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	9.97500
TOTAL	/	3/ 0.14	7/ 0.33	18/ 0.85	3/ 0.14	/	/	31.0/ 1.46	8.83629

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	5.86667
NNE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	7.50000
NE	/	/	2/ 0.09	1/ 0.05	1/ 0.05	/	/	4.0/ 0.19	7.98125
ENE	/	/	5/ 0.24	/	/	/	/	5.0/ 0.24	5.58500
E	/	/	4/ 0.19	/	/	/	/	4.0/ 0.19	5.11250
ESE	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.09	4.37500
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.17500
SSE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.05000
S	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	8.46667
SSW	/	/	3/ 0.14	3/ 0.14	/	/	/	6.0/ 0.28	7.06667
SW	/	1/ 0.05	1/ 0.05	7/ 0.33	/	/	/	9.0/ 0.42	8.60555
WSW	/	1/ 0.05	3/ 0.14	14/ 0.66	/	/	/	18.0/ 0.85	8.43611
W	/	1/ 0.05	3/ 0.14	1/ 0.05	/	/	/	5.0/ 0.24	4.83500
WNW	/	/	4/ 0.19	1/ 0.05	/	/	/	5.0/ 0.24	7.39000
NW	/	/	2/ 0.09	4/ 0.19	/	/	/	6.0/ 0.28	7.71250
NNW	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	10.15000
TOTAL	/	4/ 0.19	35/ 1.65	35/ 1.65	1/ 0.05	/	/	75.0/ 3.53	7.25433

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	4/ 0.19	3/ 0.14	/	/	/	8.0/ 0.38	5.77812
NNE	/	/	6/ 0.28	/	/	/	/	6.0/ 0.28	5.32917
NE	/	1/ 0.05	2/ 0.09	3/ 0.14	/	/	/	6.0/ 0.28	7.56667
ENE	/	1/ 0.05	3/ 0.14	/	/	/	/	4.0/ 0.19	3.80625
E	/	/	5/ 0.24	/	/	/	/	5.0/ 0.24	5.03000
ESE	/	/	/	/	/	/	/	/	
SE	/	1/ 0.05	3/ 0.14	/	/	/	/	4.0/ 0.19	3.85625
SSE	/	1/ 0.05	3/ 0.14	1/ 0.05	/	/	/	5.0/ 0.24	5.97000
S	/	/	5/ 0.24	2/ 0.09	/	/	/	7.0/ 0.33	6.71786
SSW	/	/	7/ 0.33	5/ 0.24	/	/	/	12.0/ 0.56	6.69167
SW	/	1/ 0.05	5/ 0.24	4/ 0.19	/	/	/	10.0/ 0.47	6.86000
WSW	/	/	8/ 0.38	7/ 0.33	/	/	/	15.0/ 0.71	7.53833
W	/	1/ 0.05	3/ 0.14	1/ 0.05	/	/	/	5.0/ 0.24	5.88167
WNW	/	/	5/ 0.24	/	/	/	/	5.0/ 0.24	5.35000
NW	/	1/ 0.05	4/ 0.19	1/ 0.05	2/ 0.09	/	/	8.0/ 0.38	8.18125
NNW	/	/	5/ 0.24	2/ 0.09	/	/	/	7.0/ 0.33	7.05714
TOTAL	/	8/ 0.38	68/ 3.20	29/ 1.36	2/ 0.09	/	/	107.0/ 5.04	6.44167

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	7/ 0.33	23/ 1.08	5/ 0.24	/	/	/	35.0/ 1.65	5.33714
NNE	/	18/ 0.85	31/ 1.46	3/ 0.14	/	/	/	52.0/ 2.45	4.72019
NE	/	6/ 0.28	44/ 2.07	4/ 0.19	/	/	/	54.0/ 2.54	6.05926
ENE	/	8/ 0.38	20/ 0.94	1/ 0.05	/	/	/	29.0/ 1.36	4.47586
E	/	7/ 0.33	22/ 1.04	1/ 0.05	/	/	/	30.0/ 1.41	4.48750
ESE	/	10/ 0.47	14/ 0.66	/	/	/	/	24.0/ 1.13	3.03921
SE	/	5/ 0.24	17/ 0.80	/	/	/	/	22.0/ 1.04	4.48409
SSE	/	8/ 0.38	38/ 1.79	1/ 0.05	/	/	/	47.0/ 2.21	4.78050
S	/	11/ 0.52	34/ 1.60	6/ 0.28	/	/	/	51.0/ 2.40	5.09804
SSW	/	9/ 0.42	45/ 2.12	6/ 0.28	/	/	/	60.0/ 2.82	5.46694
SW	/	14/ 0.66	37/ 1.74	17/ 0.80	1/ 0.05	/	/	69.0/ 3.25	5.77162
WSW	/	7/ 0.33	39/ 1.84	15/ 0.71	/	/	/	61.0/ 2.87	6.25943
W	/	8/ 0.38	14/ 0.66	4/ 0.19	1/ 0.05	/	/	27.0/ 1.27	5.13241
WNW	/	3/ 0.14	18/ 0.85	2/ 0.09	1/ 0.05	/	/	24.0/ 1.13	5.47604
NW	/	6/ 0.28	20/ 0.94	9/ 0.42	/	/	/	35.0/ 1.65	5.93143
NNW	/	8/ 0.38	21/ 0.99	9/ 0.42	/	/	/	38.0/ 1.79	5.79079
TOTAL	/	135/ 6.35	437/20.56	83/ 3.91	3/ 0.14	/	/	658.0/30.96	5.33019

NUMBER OF BAD RECORDS: 7

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.1/ 0.00	5/ 0.24	6/ 0.28	/	/	/	/	11.1/ 0.52	3.46847
NNE	0.1/ 0.00	6/ 0.28	14/ 0.66	/	/	/	/	20.1/ 0.95	4.31468
NE	0.1/ 0.00	13/ 0.61	8/ 0.38	/	/	/	/	21.1/ 0.99	3.55450
ENE	0.1/ 0.00	14/ 0.66	5/ 0.24	1/ 0.05	/	/	/	20.1/ 0.95	3.05937
E	0.1/ 0.00	10/ 0.47	6/ 0.28	/	/	/	/	16.1/ 0.76	3.40528
ESE	0.1/ 0.00	12/ 0.56	4/ 0.19	/	/	/	/	16.1/ 0.76	2.84161
SE	0.1/ 0.00	14/ 0.66	8/ 0.38	/	/	/	/	22.1/ 1.04	2.95588
SSE	0.3/ 0.01	33/ 1.55	15/ 0.71	/	/	/	/	48.3/ 2.27	3.00362
S	0.6/ 0.03	62/ 2.92	38/ 1.79	/	/	/	/	100.6/ 4.73	3.32256
SSW	0.4/ 0.02	39/ 1.84	42/ 1.98	1/ 0.05	/	/	/	82.4/ 3.88	3.85862
SW	0.3/ 0.01	33/ 1.55	26/ 1.22	3/ 0.14	/	/	/	62.3/ 2.93	3.89868
WSW	0.2/ 0.01	19/ 0.89	25/ 1.18	1/ 0.05	/	/	/	45.2/ 2.13	4.19358
W	0.0/ 0.00	3/ 0.14	7/ 0.33	2/ 0.09	/	/	/	12.0/ 0.56	4.87917
WNW	0.1/ 0.00	5/ 0.24	13/ 0.61	1/ 0.05	/	/	/	19.1/ 0.90	4.76767
NW	0.1/ 0.00	11/ 0.52	5/ 0.24	3/ 0.14	/	/	/	19.1/ 0.90	3.88416
NNW	0.1/ 0.00	12/ 0.56	9/ 0.42	1/ 0.05	/	/	/	22.1/ 1.04	3.46550
TOTAL	3.0/ 0.14	291/13.69	231/10.87	13/ 0.61	/	/	/	538.0/25.32	3.63854

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.4/ 0.02	10/ 0.47	15/ 0.71	/	/	/	/	25.4/ 1.20	3.60138
NNE	0.6/ 0.03	16/ 0.75	2/ 0.09	/	/	/	/	18.6/ 0.88	2.40054
NE	0.5/ 0.02	12/ 0.56	3/ 0.14	/	/	/	/	15.5/ 0.73	2.31935
ENE	0.8/ 0.04	20/ 0.94	1/ 0.05	/	/	/	/	21.8/ 1.03	2.10206
E	0.5/ 0.02	14/ 0.66	/	/	/	/	/	14.5/ 0.68	1.68190
ESE	0.9/ 0.04	23/ 1.08	2/ 0.09	/	/	/	/	25.9/ 1.22	2.03668
SE	0.5/ 0.02	13/ 0.61	/	/	/	/	/	13.5/ 0.64	2.23426
SSE	1.4/ 0.07	38/ 1.79	/	/	/	/	/	39.4/ 1.85	1.88388
S	1.5/ 0.07	40/ 1.88	3/ 0.14	/	/	/	/	44.5/ 2.09	2.01798
SSW	1.4/ 0.07	37/ 1.74	1/ 0.05	/	/	/	/	39.4/ 1.85	2.04378
SW	1.0/ 0.05	27/ 1.27	2/ 0.09	/	/	/	/	30.0/ 1.41	1.94167
WSW	0.6/ 0.03	16/ 0.75	/	/	/	/	/	16.6/ 0.78	1.30572
W	0.1/ 0.00	3/ 0.14	2/ 0.09	/	/	/	/	5.1/ 0.24	3.23039
WNW	0.3/ 0.01	7/ 0.33	3/ 0.14	/	/	/	/	10.3/ 0.48	2.70388
NW	0.3/ 0.01	9/ 0.42	1/ 0.05	/	/	/	/	10.3/ 0.48	2.03883
NNW	0.2/ 0.01	6/ 0.28	4/ 0.19	/	/	/	/	10.2/ 0.48	2.72059
TOTAL	11.0/ 0.52	291/13.69	39/ 1.84	/	/	/	/	341.0/16.05	2.17815

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	11.1/ 0.52	28/ 1.32	1/ 0.05	/	/	/	/	40.1/ 1.89	1.52868
NNE	9.9/ 0.47	25/ 1.18	/	/	/	/	/	34.9/ 1.64	1.27185
NE	9.5/ 0.45	24/ 1.13	1/ 0.05	/	/	/	/	34.5/ 1.62	1.34493
ENE	9.1/ 0.43	23/ 1.08	1/ 0.05	/	/	/	/	33.1/ 1.56	1.41994
E	9.5/ 0.45	24/ 1.13	/	/	/	/	/	33.5/ 1.58	1.25709
ESE	5.9/ 0.28	15/ 0.71	/	/	/	/	/	20.9/ 0.98	1.38935
SE	7.1/ 0.33	18/ 0.85	/	/	/	/	/	25.1/ 1.18	1.09412
SSE	4.8/ 0.23	12/ 0.56	/	/	/	/	/	16.8/ 0.79	0.95684
S	7.1/ 0.33	18/ 0.85	/	/	/	/	/	25.1/ 1.18	1.09014
SSW	4.0/ 0.19	10/ 0.47	1/ 0.05	/	/	/	/	15.0/ 0.71	1.58167
SW	5.5/ 0.26	14/ 0.66	/	/	/	/	/	19.5/ 0.92	1.14167
WSW	3.6/ 0.17	9/ 0.42	/	/	/	/	/	12.6/ 0.59	1.00397
W	3.2/ 0.15	8/ 0.38	/	/	/	/	/	11.2/ 0.53	1.02009
WNW	4.8/ 0.23	12/ 0.56	/	/	/	/	/	16.8/ 0.79	1.08482
NW	2.4/ 0.11	6/ 0.28	/	/	/	/	/	8.4/ 0.40	1.16369
NNW	7.5/ 0.35	19/ 0.89	1/ 0.05	/	/	/	/	27.5/ 1.29	1.33818
TOTAL	105.0/ 4.94	265/12.47	5/ 0.24	/	/	/	/	375.0/17.65	1.26933

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	5.6/ 0.27	52/ 2.47	61/ 2.90	9/ 0.43	/	/	/	127.6/ 6.06	3.88911
NNE	5.7/ 0.27	53/ 2.52	54/ 2.57	5/ 0.24	/	/	/	117.7/ 5.59	3.71676
NE	7.9/ 0.38	73/ 3.47	57/ 2.71	5/ 0.24	/	/	/	142.9/ 6.79	3.60995
ENE	8.3/ 0.39	77/ 3.66	34/ 1.62	/	/	/	/	119.3/ 5.67	2.92645
E	5.4/ 0.26	50/ 2.38	42/ 2.00	1/ 0.05	/	/	/	98.4/ 4.68	3.27490
ESE	4.5/ 0.21	42/ 2.00	14/ 0.67	1/ 0.05	/	/	/	61.5/ 2.92	2.67337
SE	5.0/ 0.24	46/ 2.19	25/ 1.19	1/ 0.05	/	/	/	77.0/ 3.66	2.88896
SSE	7.3/ 0.35	68/ 3.23	42/ 2.00	4/ 0.19	/	/	/	121.3/ 5.77	3.11098
S	11.3/ 0.54	105/ 4.99	69/ 3.28	5/ 0.24	/	/	/	190.3/ 9.04	3.30235
SSW	15.9/ 0.76	147/ 6.99	85/ 4.04	5/ 0.24	1/ 0.05	/	/	253.9/12.07	3.27905
SW	7.7/ 0.37	71/ 3.37	77/ 3.66	23/ 1.09	4/ 0.19	/	/	182.7/ 8.68	4.59743
WSW	7.4/ 0.35	69/ 3.28	103/ 4.90	13/ 0.62	/	/	/	192.4/ 9.14	4.14566
W	4.6/ 0.22	43/ 2.04	61/ 2.90	28/ 1.33	3/ 0.14	/	/	139.6/ 6.63	5.23149
WNW	3.3/ 0.16	31/ 1.47	61/ 2.90	12/ 0.57	1/ 0.05	/	/	108.3/ 5.15	4.60088
NW	2.3/ 0.11	21/ 1.00	39/ 1.85	6/ 0.29	/	/	/	68.3/ 3.25	4.32156
NNW	4.7/ 0.22	44/ 2.09	47/ 2.23	7/ 0.33	/	/	/	102.7/ 4.88	3.73040
TOTAL	107.0/ 5.09	992/47.15	871/41.40	125/ 5.94	9/ 0.43	/	/	2104/ 100	3.75029

NUMBER OF BAD RECORDS: 128

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	5/ 0.24	/	/	/	/	6.0/ 0.29	5.10000
NNE	/	/	6/ 0.29	2/ 0.10	/	/	/	8.0/ 0.38	6.45625
NE	/	1/ 0.05	6/ 0.29	1/ 0.05	/	/	/	8.0/ 0.38	5.87d12
ENE	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	6.34167
E	/	1/ 0.05	2/ 0.10	/	/	/	/	3.0/ 0.14	5.45000
ESE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	7.50000
SE	/	1/ 0.05	3/ 0.14	/	/	/	/	4.0/ 0.19	4.83750
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	3.95000
S	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	6.35833
SSW	/	/	4/ 0.19	1/ 0.05	1/ 0.05	/	/	6.0/ 0.29	7.71250
SW	/	1/ 0.05	7/ 0.33	9/ 0.43	3/ 0.14	/	/	20.0/ 0.95	9.05125
WSW	/	/	9/ 0.43	4/ 0.19	/	/	/	13.0/ 0.62	6.71923
W	/	1/ 0.05	10/ 0.48	1/ 0.05	/	/	/	12.0/ 0.57	5.50208
WNW	/	/	8/ 0.38	6/ 0.29	/	/	/	14.0/ 0.67	6.79107
NW	/	/	4/ 0.19	1/ 0.05	/	/	/	5.0/ 0.24	5.76000
NNW	/	/	4/ 0.19	/	/	/	/	4.0/ 0.19	5.50625
TOTAL	/	6/ 0.29	76/ 3.61	26/ 1.24	4/ 0.19	/	/	112.0/ 5.32	6.68393

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	4/ 0.19	2/ 0.10	/	/	/	6.0/ 0.29	6.68750
NNE	/	/	8/ 0.38	/	/	/	/	8.0/ 0.38	5.79062
NE	/	/	10/ 0.48	/	/	/	/	10.0/ 0.48	6.26750
ENE	/	/	4/ 0.19	/	/	/	/	4.0/ 0.19	6.51250
E	/	/	5/ 0.24	1/ 0.05	/	/	/	6.0/ 0.29	6.97917
ESE	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	6.43750
SE	/	/	/	/	/	/	/	/	
SSE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	7.65000
S	/	/	4/ 0.19	1/ 0.05	/	/	/	5.0/ 0.24	6.93500
SSW	/	/	8/ 0.38	1/ 0.05	/	/	/	9.0/ 0.43	6.14722
SW	/	/	9/ 0.43	3/ 0.14	/	/	/	12.0/ 0.57	6.78125
WSW	/	1/ 0.05	13/ 0.62	3/ 0.14	/	/	/	17.0/ 0.81	6.03529
W	/	1/ 0.05	3/ 0.14	7/ 0.33	/	/	/	11.0/ 0.52	7.72954
WNW	/	/	12/ 0.57	2/ 0.10	/	/	/	14.0/ 0.67	5.75536
NW	/	/	8/ 0.38	4/ 0.19	/	/	/	12.0/ 0.57	6.50000
NNW	/	/	5/ 0.24	3/ 0.14	/	/	/	8.0/ 0.38	6.71250
TOTAL	/	2/ 0.10	96/ 4.56	28/ 1.33	/	/	/	126.0/ 5.99	6.48016

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	6/ 0.29	3/ 0.14	/	/	/	10.0/ 0.48	5.76750
NNE	/	/	4/ 0.19	2/ 0.10	/	/	/	6.0/ 0.29	5.71667
NE	/	1/ 0.05	8/ 0.38	/	/	/	/	9.0/ 0.43	5.31944
ENE	/	/	10/ 0.48	/	/	/	/	10.0/ 0.48	5.82750
E	/	2/ 0.10	9/ 0.43	/	/	/	/	11.0/ 0.52	4.54318
ESE	/	1/ 0.05	3/ 0.14	/	/	/	/	4.0/ 0.19	4.38125
SE	/	2/ 0.10	1/ 0.05	/	/	/	/	3.0/ 0.14	2.94167
SSE	/	1/ 0.05	4/ 0.19	1/ 0.05	/	/	/	6.0/ 0.29	5.03750
S	/	/	6/ 0.29	1/ 0.05	/	/	/	7.0/ 0.33	5.98214
SSW	/	2/ 0.10	9/ 0.43	/	/	/	/	11.0/ 0.52	4.99091
SW	/	2/ 0.10	9/ 0.43	1/ 0.05	/	/	/	12.0/ 0.57	5.68958
WSW	/	/	22/ 1.05	1/ 0.05	/	/	/	23.0/ 1.09	6.11196
W	/	/	17/ 0.81	3/ 0.14	/	/	/	20.0/ 0.95	6.22250
WNW	/	3/ 0.14	9/ 0.43	1/ 0.05	/	/	/	13.0/ 0.62	5.12115
NW	/	/	11/ 0.52	1/ 0.05	/	/	/	12.0/ 0.57	6.01042
NNW	/	/	6/ 0.29	1/ 0.05	/	/	/	7.0/ 0.33	5.56786
TOTAL	/	15/ 0.71	134/ 6.37	15/ 0.71	/	/	/	164.0/ 7.79	5.56357

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	7/ 0.33	17/ 0.81	3/ 0.14	/	/	/	27.0/ 1.28	4.78148
NNE	/	5/ 0.24	27/ 1.28	/	/	/	/	32.0/ 1.52	5.07422
NE	/	4/ 0.19	23/ 1.09	4/ 0.19	/	/	/	31.0/ 1.47	5.58468
ENE	/	6/ 0.29	10/ 0.48	/	/	/	/	16.0/ 0.76	3.98281
E	/	5/ 0.24	16/ 0.76	/	/	/	/	21.0/ 1.00	4.31309
ESE	/	6/ 0.29	5/ 0.24	/	/	/	/	11.0/ 0.52	3.62500
SE	/	3/ 0.14	12/ 0.57	1/ 0.05	/	/	/	16.0/ 0.76	4.70937
SSE	/	9/ 0.43	20/ 0.95	2/ 0.10	/	/	/	31.0/ 1.47	4.59919
S	/	12/ 0.57	25/ 1.19	3/ 0.14	/	/	/	40.0/ 1.90	4.73062
SSW	/	13/ 0.62	35/ 1.66	2/ 0.10	/	/	/	50.0/ 2.38	4.66600
SW	/	7/ 0.33	29/ 1.38	9/ 0.43	/	/	/	45.0/ 2.14	5.76500
WSW	/	7/ 0.33	37/ 1.76	3/ 0.14	/	/	/	47.0/ 2.23	4.99840
W	/	7/ 0.33	20/ 0.95	15/ 0.71	3/ 0.14	/	/	45.0/ 2.14	6.79537
WNW	/	7/ 0.33	24/ 1.14	2/ 0.10	1/ 0.05	/	/	34.0/ 1.62	5.05147
NW	/	5/ 0.24	8/ 0.38	/	/	/	/	13.0/ 0.62	3.92500
NNW	/	5/ 0.24	16/ 0.76	3/ 0.14	/	/	/	24.0/ 1.14	4.81042
TOTAL	/	108/ 5.13	324/15.40	47/ 2.23	4/ 0.19	/	/	483.0/22.96	5.04677

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.1/ 0.00	8/ 0.38	12/ 0.57	1/ 0.05	/	/	/	21.1/ 1.00	4.37500
NNE	0.1/ 0.00	6/ 0.29	7/ 0.33	1/ 0.05	/	/	/	14.1/ 0.67	3.74379
NE	0.1/ 0.00	8/ 0.38	10/ 0.48	/	/	/	/	18.1/ 0.86	3.80249
ENE	0.2/ 0.01	13/ 0.62	7/ 0.33	/	/	/	/	20.2/ 0.96	3.14356
E	0.1/ 0.00	8/ 0.38	10/ 0.48	/	/	/	/	18.1/ 0.86	3.20580
ESE	0.1/ 0.00	6/ 0.29	1/ 0.05	/	/	/	/	7.1/ 0.34	2.38732
SE	0.2/ 0.01	17/ 0.81	7/ 0.33	/	/	/	/	24.2/ 1.15	2.97727
SSE	0.5/ 0.02	32/ 1.52	16/ 0.76	/	/	/	/	48.5/ 2.31	2.87680
S	0.5/ 0.02	36/ 1.71	28/ 1.33	/	/	/	/	64.5/ 3.07	3.35116
SSW	0.8/ 0.04	54/ 2.57	28/ 1.33	1/ 0.05	/	/	/	83.8/ 3.98	3.18825
SW	0.4/ 0.02	27/ 1.28	22/ 1.05	1/ 0.05	1/ 0.05	/	/	51.4/ 2.44	3.58658
WSW	0.4/ 0.02	25/ 1.19	21/ 1.00	2/ 0.10	/	/	/	48.4/ 2.30	3.53151
W	0.2/ 0.01	15/ 0.71	11/ 0.52	2/ 0.10	/	/	/	28.2/ 1.34	3.84752
WNW	0.1/ 0.00	8/ 0.38	7/ 0.33	1/ 0.05	/	/	/	16.1/ 0.77	3.61102
NW	0.1/ 0.00	8/ 0.38	7/ 0.33	/	/	/	/	15.1/ 0.72	3.16556
NNW	0.1/ 0.00	9/ 0.43	11/ 0.52	/	/	/	/	20.1/ 0.96	3.94154
TOTAL	4.0/ 0.19	280/13.31	205/ 9.74	9/ 0.43	1/ 0.05	/	/	499.0/23.72	3.39927

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.3/ 0.01	7/ 0.33	12/ 0.57	/	/	/	/	19.3/ 0.92	4.00129
NNE	0.5/ 0.02	10/ 0.48	1/ 0.05	/	/	/	/	11.5/ 0.55	2.68696
NE	0.7/ 0.03	14/ 0.67	/	/	/	/	/	14.7/ 0.70	2.40561
ENE	1.4/ 0.07	30/ 1.43	/	/	/	/	/	31.4/ 1.49	2.17595
E	0.9/ 0.04	18/ 0.86	/	/	/	/	/	18.9/ 0.90	2.02182
ESE	0.6/ 0.03	13/ 0.62	2/ 0.10	/	/	/	/	15.6/ 0.74	2.45994
SE	0.6/ 0.03	13/ 0.62	1/ 0.05	/	/	/	/	14.6/ 0.69	1.77740
SSE	1.0/ 0.05	20/ 0.95	/	/	/	/	/	21.0/ 1.00	1.51786
S	2.0/ 0.10	42/ 2.00	3/ 0.14	/	/	/	/	47.0/ 2.23	2.10798
SSW	2.8/ 0.13	59/ 2.80	1/ 0.05	/	/	/	/	62.8/ 2.98	2.07126
SW	0.7/ 0.03	15/ 0.71	1/ 0.05	/	/	/	/	16.7/ 0.79	1.89671
WSW	1.0/ 0.05	21/ 1.00	1/ 0.05	/	/	/	/	23.0/ 1.09	1.71413
W	0.4/ 0.02	9/ 0.43	/	/	/	/	/	9.4/ 0.45	1.93617
WNW	0.3/ 0.01	7/ 0.33	1/ 0.05	/	/	/	/	8.3/ 0.39	1.78313
NW	0.2/ 0.01	4/ 0.19	1/ 0.05	/	/	/	/	5.2/ 0.25	2.14423
NNW	0.5/ 0.02	11/ 0.52	4/ 0.19	/	/	/	/	15.5/ 0.74	2.34193
TOTAL	14.0/ 0.67	293/13.93	28/ 1.33	/	/	/	/	335.0/15.92	2.16992

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	8.7/ 0.41	28/ 1.33	5/ 0.24	/	/	/	/	41.7/ 1.98	1.67326
NNE	9.9/ 0.47	32/ 1.52	1/ 0.05	/	/	/	/	42.9/ 2.04	1.41317
NE	13.9/ 0.66	45/ 2.14	/	/	/	/	/	58.9/ 2.80	1.31812
ENE	8.7/ 0.41	28/ 1.33	/	/	/	/	/	36.7/ 1.74	1.30620
E	4.9/ 0.23	16/ 0.76	/	/	/	/	/	20.9/ 0.99	1.12021
ESE	4.9/ 0.23	16/ 0.76	/	/	/	/	/	20.9/ 0.99	1.01017
SE	3.1/ 0.15	10/ 0.48	1/ 0.05	/	/	/	/	14.1/ 0.67	1.29255
SSE	1.9/ 0.09	6/ 0.29	/	/	/	/	/	7.9/ 0.38	0.90665
S	4.6/ 0.22	15/ 0.71	/	/	/	/	/	19.6/ 0.93	0.98214
SSW	5.9/ 0.28	19/ 0.90	/	/	/	/	/	24.9/ 1.18	1.07881
SW	5.9/ 0.28	19/ 0.90	/	/	/	/	/	24.9/ 1.18	1.02861
WSW	4.6/ 0.22	15/ 0.71	/	/	/	/	/	19.6/ 0.93	0.83801
W	3.1/ 0.15	10/ 0.48	/	/	/	/	/	13.1/ 0.62	1.29485
WNW	1.9/ 0.09	6/ 0.29	/	/	/	/	/	7.9/ 0.38	0.94462
NW	1.2/ 0.06	4/ 0.19	/	/	/	/	/	5.2/ 0.25	0.83654
NNW	5.9/ 0.28	19/ 0.90	1/ 0.05	/	/	/	/	25.9/ 1.23	1.42954
TOTAL	89.0/ 4.23	288/13.69	8/ 0.38	/	/	/	/	385.0/18.30	1.24633

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	8.3/ 0.38	83/ 3.77	50/ 2.27	12/ 0.54	/	/	/	153.3/ 6.96	3.57917
NNE	8.5/ 0.39	85/ 3.86	51/ 2.32	14/ 0.64	/	/	/	158.5/ 7.19	3.52290
NE	9.0/ 0.41	90/ 4.09	55/ 2.50	47/ 2.13	1/ 0.05	/	/	202.0/ 9.17	4.59926
ENE	7.2/ 0.33	72/ 3.27	52/ 2.36	22/ 1.00	/	/	/	153.2/ 6.95	4.05173
E	7.4/ 0.34	74/ 3.36	30/ 1.36	2/ 0.09	/	/	/	113.4/ 5.15	2.68342
ESE	6.2/ 0.28	62/ 2.81	32/ 1.45	1/ 0.05	/	/	/	101.2/ 4.59	2.85079
SE	5.5/ 0.25	55/ 2.50	30/ 1.36	2/ 0.09	/	/	/	92.5/ 4.20	2.98869
SSE	9.7/ 0.44	97/ 4.40	41/ 1.86	3/ 0.14	/	/	/	150.7/ 6.84	2.86388
S	13.8/ 0.63	138/ 6.26	84/ 3.81	10/ 0.45	/	/	/	245.8/11.16	3.29455
SSW	16.1/ 0.73	161/ 7.31	121/ 5.49	17/ 0.77	/	/	/	315.1/14.30	3.75258
SW	5.3/ 0.24	53/ 2.41	121/ 5.49	22/ 1.00	/	/	/	201.3/ 9.14	4.85798
WSW	3.7/ 0.17	37/ 1.68	66/ 3.00	20/ 0.91	/	/	/	126.7/ 5.75	4.95748
W	1.7/ 0.08	17/ 0.77	24/ 1.09	2/ 0.09	/	/	/	44.7/ 2.03	4.09070
WNW	2.2/ 0.10	22/ 1.00	12/ 0.54	1/ 0.05	/	/	/	37.2/ 1.69	3.00874
NW	2.2/ 0.10	22/ 1.00	18/ 0.82	1/ 0.05	/	/	/	43.2/ 1.96	3.42535
NNW	4.1/ 0.19	41/ 1.86	18/ 0.82	1/ 0.05	/	/	/	64.1/ 2.91	2.80220
TOTAL	111.0/ 5.04	1109/50.34	805/36.54	177/ 8.03	1/ 0.05	/	/	2203/ 100	3.71236

NUMBER OF BAD RECORDS: 29

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	2/ 0.09	3/ 0.14	/	/	/	6.0/ 0.27	7.03750
NNE	/	/	1/ 0.05	8/ 0.36	/	/	/	9.0/ 0.41	8.67685
NE	/	/	6/ 0.27	15/ 0.68	1/ 0.05	/	/	22.0/ 1.00	8.51818
ENE	/	/	8/ 0.36	6/ 0.27	/	/	/	14.0/ 0.64	7.43571
E	/	/	5/ 0.23	1/ 0.05	/	/	/	6.0/ 0.27	6.24167
ESE	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	4.56667
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.55000
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.27500
S	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	6.49167
SSW	/	/	7/ 0.32	7/ 0.32	/	/	/	14.0/ 0.64	7.56964
SW	/	/	8/ 0.36	5/ 0.23	/	/	/	13.0/ 0.59	6.89423
WSW	/	/	3/ 0.14	2/ 0.09	/	/	/	5.0/ 0.23	6.74000
W	/	/	/	/	/	/	/	/	
WNW	/	/	/	/	/	/	/	/	
NW	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	3.92500
NNW	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.72500
TOTAL	/	1/ 0.05	50/ 2.27	48/ 2.18	1/ 0.05	/	/	100.0/ 4.54	7.38942

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	9/ 0.41	2/ 0.09	/	/	/	11.0/ 0.50	6.19545
NNE	/	/	7/ 0.32	1/ 0.05	/	/	/	8.0/ 0.36	6.02812
NE	/	4/ 0.18	4/ 0.18	6/ 0.27	/	/	/	14.0/ 0.64	5.78929
ENE	/	1/ 0.05	5/ 0.23	3/ 0.14	/	/	/	9.0/ 0.41	6.32778
E	/	1/ 0.05	3/ 0.14	/	/	/	/	4.0/ 0.18	4.21875
ESE	/	1/ 0.05	8/ 0.36	/	/	/	/	9.0/ 0.41	4.57778
SE	/	2/ 0.09	3/ 0.14	/	/	/	/	5.0/ 0.23	3.65000
SSE	/	/	/	/	/	/	/	/	
S	/	1/ 0.05	9/ 0.41	3/ 0.14	/	/	/	13.0/ 0.59	6.21731
SSW	/	/	17/ 0.77	/	/	/	/	17.0/ 0.77	6.05735
SW	/	1/ 0.05	12/ 0.54	6/ 0.27	/	/	/	19.0/ 0.86	6.73816
WSW	/	1/ 0.05	6/ 0.27	2/ 0.09	/	/	/	9.0/ 0.41	5.90555
W	/	1/ 0.05	2/ 0.09	/	/	/	/	3.0/ 0.14	4.55556
WNW	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	5.98333
NW	/	/	7/ 0.32	/	/	/	/	7.0/ 0.32	5.10714
NNW	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	3.92500
TOTAL	/	13/ 0.59	96/ 4.36	24/ 1.09	/	/	/	133.0/ 6.04	5.79618

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	7/ 0.32	1/ 0.05	/	/	/	9.0/ 0.41	5.65555
NNE	/	3/ 0.14	6/ 0.27	2/ 0.09	/	/	/	11.0/ 0.50	5.06818
NE	/	1/ 0.05	6/ 0.27	3/ 0.14	/	/	/	10.0/ 0.45	5.89250
ENE	/	1/ 0.05	5/ 0.23	3/ 0.14	/	/	/	9.0/ 0.41	5.78333
E	/	2/ 0.09	5/ 0.23	1/ 0.05	/	/	/	8.0/ 0.36	4.42812
ESE	/	2/ 0.09	6/ 0.27	/	/	/	/	8.0/ 0.36	4.19062
SE	/	/	6/ 0.27	/	/	/	/	6.0/ 0.27	4.69028
SSE	/	2/ 0.09	5/ 0.23	1/ 0.05	/	/	/	8.0/ 0.36	4.68750
S	/	1/ 0.05	12/ 0.54	4/ 0.18	/	/	/	17.0/ 0.77	5.41471
SSW	/	3/ 0.14	20/ 0.91	4/ 0.18	/	/	/	27.0/ 1.23	5.89074
SW	/	/	29/ 1.32	3/ 0.14	/	/	/	32.0/ 1.45	6.21016
WSW	/	1/ 0.05	16/ 0.73	10/ 0.45	/	/	/	27.0/ 1.23	6.90741
W	/	2/ 0.09	11/ 0.50	/	/	/	/	13.0/ 0.59	4.63077
WNW	/	1/ 0.05	5/ 0.23	/	/	/	/	6.0/ 0.27	4.55000
NW	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	6.44167
NNW	/	3/ 0.14	7/ 0.32	/	/	/	/	10.0/ 0.45	4.32583
TOTAL	/	23/ 1.04	149/ 6.76	32/ 1.45	/	/	/	204.0/ 9.26	5.58149

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.1/ 0.00	7/ 0.32	12/ 0.54	6/ 0.27	/	/	/	25.1/ 1.14	5.21265
NNE	0.1/ 0.00	9/ 0.41	15/ 0.68	2/ 0.09	/	/	/	26.1/ 1.18	4.62787
NE	0.1/ 0.00	13/ 0.59	23/ 1.04	17/ 0.77	/	/	/	53.1/ 2.41	6.01530
ENE	0.1/ 0.00	9/ 0.41	21/ 0.95	8/ 0.36	/	/	/	38.1/ 1.73	5.24639
E	0.1/ 0.00	7/ 0.32	11/ 0.50	/	/	/	/	18.1/ 0.82	3.59392
ESE	0.1/ 0.00	9/ 0.41	10/ 0.45	/	/	/	/	19.1/ 0.87	3.72382
SE	0.1/ 0.00	11/ 0.50	17/ 0.77	1/ 0.05	/	/	/	29.1/ 1.32	4.12070
SSE	0.1/ 0.00	9/ 0.41	26/ 1.18	2/ 0.09	/	/	/	37.1/ 1.68	4.56840
S	0.1/ 0.00	7/ 0.32	41/ 1.86	2/ 0.09	/	/	/	50.1/ 2.27	5.00175
SSW	0.1/ 0.00	19/ 0.86	41/ 1.86	6/ 0.27	/	/	/	66.1/ 3.00	5.14618
SW	0.1/ 0.00	8/ 0.36	52/ 2.36	7/ 0.32	/	/	/	67.1/ 3.05	5.49311
WSW	0.1/ 0.00	8/ 0.36	37/ 1.68	5/ 0.23	/	/	/	50.1/ 2.27	5.40544
W	0.0/ 0.00	4/ 0.18	6/ 0.27	1/ 0.05	/	/	/	11.0/ 0.50	4.80682
WNW	0.0/ 0.00	4/ 0.18	3/ 0.14	/	/	/	/	7.0/ 0.32	3.57500
NW	0.0/ 0.00	5/ 0.23	3/ 0.14	1/ 0.05	/	/	/	9.0/ 0.41	4.50000
NNW	0.1/ 0.00	7/ 0.32	3/ 0.14	1/ 0.05	/	/	/	11.1/ 0.50	3.93581
TOTAL	1.0/ 0.05	136/ 6.17	321/14.57	59/ 2.68	/	/	/	517.0/ 23.47	5.00723

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=08 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	1.2/ 0.05	24/ 1.09	8/ 0.36	/	/	/	/	33.2/ 1.51	2.61822
NNE	1.1/ 0.05	23/ 1.04	/	/	/	/	/	24.1/ 1.09	1.99429
NE	1.2/ 0.05	24/ 1.09	1/ 0.05	/	/	/	/	26.2/ 1.19	2.08779
ENE	0.9/ 0.04	18/ 0.82	2/ 0.09	/	/	/	/	20.9/ 0.95	2.35526
E	0.8/ 0.04	17/ 0.77	/	/	/	/	/	17.8/ 0.81	1.65028
ESE	0.8/ 0.04	17/ 0.77	/	/	/	/	/	17.8/ 0.81	1.60815
SE	0.9/ 0.04	19/ 0.86	1/ 0.05	/	/	/	/	20.9/ 0.95	1.87799
SSE	1.9/ 0.09	38/ 1.72	/	/	/	/	/	39.9/ 1.81	1.54605
S	1.9/ 0.09	38/ 1.72	1/ 0.05	/	/	/	/	40.9/ 1.86	1.66504
SSW	2.4/ 0.11	48/ 2.18	2/ 0.09	/	/	/	/	52.4/ 2.38	1.91746
SW	0.8/ 0.04	17/ 0.77	1/ 0.05	/	/	/	/	18.8/ 0.85	1.83112
WSW	0.5/ 0.02	10/ 0.45	/	/	/	/	/	10.5/ 0.48	1.42262
W	0.2/ 0.01	5/ 0.23	1/ 0.05	/	/	/	/	6.2/ 0.28	2.02016
WNW	0.4/ 0.02	8/ 0.36	/	/	/	/	/	8.4/ 0.38	1.55952
NW	0.4/ 0.02	8/ 0.36	1/ 0.05	/	/	/	/	9.4/ 0.43	2.03191
NNW	0.5/ 0.02	10/ 0.45	1/ 0.05	/	/	/	/	11.5/ 0.52	1.98478
TOTAL	16.0/ 0.73	324/14.71	19/ 0.86	/	/	/	/	359.0/16.30	1.90345

NUMBER OF BAD RECORDS: 6

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	17.7/ 0.80	36/ 1.63	/	/	/	/	/	53.7/ 2.44	1.37034
NNE	14.8/ 0.67	30/ 1.36	/	/	/	/	/	44.8/ 2.03	1.12500
NE	11.8/ 0.54	24/ 1.09	/	/	/	/	/	35.8/ 1.63	1.11662
ENE	11.8/ 0.54	24/ 1.09	/	/	/	/	/	35.8/ 1.63	0.94344
E	8.4/ 0.38	17/ 0.77	/	/	/	/	/	25.4/ 1.15	0.98228
ESE	3.9/ 0.18	8/ 0.36	/	/	/	/	/	11.9/ 0.54	1.09979
SE	2.5/ 0.11	5/ 0.23	/	/	/	/	/	7.5/ 0.34	0.96167
SSE	1.5/ 0.07	3/ 0.14	/	/	/	/	/	4.5/ 0.20	0.90278
S	3.4/ 0.15	7/ 0.32	/	/	/	/	/	10.4/ 0.47	1.10577
SSW	1.5/ 0.07	3/ 0.14	/	/	/	/	/	4.5/ 0.20	0.89167
SW	1.5/ 0.07	3/ 0.14	/	/	/	/	/	4.5/ 0.20	0.90278
WSW	0.5/ 0.02	1/ 0.05	/	/	/	/	/	1.5/ 0.07	0.69167
W	0.5/ 0.02	1/ 0.05	/	/	/	/	/	1.5/ 0.07	0.80833
WNW	2.0/ 0.09	4/ 0.18	/	/	/	/	/	6.0/ 0.27	0.89167
NW	3.9/ 0.18	8/ 0.36	/	/	/	/	/	11.9/ 0.54	0.92752
NNW	5.4/ 0.25	11/ 0.50	/	/	/	/	/	16.4/ 0.74	0.93598
TOTAL	91.0/ 4.13	185/ 8.40	/	/	/	/	/	276.0/12.53	1.08918

NUMBER OF BAD RECORDS: 4

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	20.0/ 0.94	107/ 5.03	83/ 3.90	5/ 0.23	1/ 0.05	/	/	216.0/10.15	3.32558
NNE	20.9/ 0.98	112/ 5.26	103/ 4.84	3/ 0.14	/	/	/	238.9/11.22	3.35752
NE	19.0/ 0.89	102/ 4.79	130/ 6.11	9/ 0.42	/	/	/	260.0/12.21	3.86260
ENE	16.8/ 0.79	90/ 4.23	76/ 3.57	9/ 0.42	/	/	/	191.8/ 9.01	3.46637
E	8.6/ 0.40	46/ 2.16	35/ 1.64	3/ 0.14	/	/	/	92.6/ 4.35	3.05912
ESE	8.0/ 0.38	43/ 2.02	18/ 0.85	1/ 0.05	/	/	/	70.0/ 3.29	2.55571
SE	9.3/ 0.44	50/ 2.35	17/ 0.80	1/ 0.05	/	/	/	77.3/ 3.63	2.59525
SSE	12.3/ 0.58	66/ 3.10	36/ 1.69	2/ 0.09	/	/	/	116.3/ 5.46	3.00484
S	12.1/ 0.57	65/ 3.05	36/ 1.69	1/ 0.05	/	/	/	114.1/ 5.36	3.04349
SSW	16.1/ 0.76	86/ 4.04	57/ 2.68	2/ 0.09	/	/	/	161.1/ 7.57	3.04989
SW	11.9/ 0.56	64/ 3.01	56/ 2.63	17/ 0.80	/	/	/	148.9/ 6.99	4.03585
WSW	8.0/ 0.38	43/ 2.02	32/ 1.50	19/ 0.89	/	/	/	102.0/ 4.79	4.19918
W	8.2/ 0.39	44/ 2.07	36/ 1.69	1/ 0.05	/	/	/	89.2/ 4.19	3.11257
WNW	3.9/ 0.18	21/ 0.99	24/ 1.13	3/ 0.14	/	/	/	51.9/ 2.44	3.53251
NW	4.7/ 0.22	25/ 1.17	21/ 0.99	5/ 0.23	/	/	/	55.7/ 2.62	3.83505
NNW	13.1/ 0.62	70/ 3.29	51/ 2.40	8/ 0.38	1/ 0.05	/	/	143.1/ 6.72	3.56184
TOTAL	193.0/ 9.07	1034/48.57	811/38.09	89/ 4.18	2/ 0.09	/	/	2129/ 100	3.40721

NUMBER OF BAD RECORDS: 31

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.0/ 0.00	1/ 0.05	14/ 0.66	2/ 0.09	1/ 0.05	/	/	18.0/ 0.85	5.82222
NNE	0.0/ 0.00	1/ 0.05	15/ 0.70	2/ 0.09	/	/	/	18.0/ 0.85	5.63472
NE	0.1/ 0.00	2/ 0.09	14/ 0.66	1/ 0.05	/	/	/	17.1/ 0.80	5.68202
ENE	0.0/ 0.00	1/ 0.05	12/ 0.56	1/ 0.05	/	/	/	14.0/ 0.66	5.65536
E	0.1/ 0.00	2/ 0.09	3/ 0.14	/	/	/	/	5.1/ 0.24	3.92647
ESE	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	4.67500
SE	0.1/ 0.00	3/ 0.14	7/ 0.33	/	/	/	/	10.1/ 0.47	4.60643
SSE	0.0/ 0.00	1/ 0.05	13/ 0.61	1/ 0.05	/	/	/	15.0/ 0.70	5.42667
S	0.1/ 0.00	2/ 0.09	15/ 0.70	/	/	/	/	17.1/ 0.80	5.03216
SSW	/	/	22/ 1.03	2/ 0.09	/	/	/	24.0/ 1.13	5.69687
SW	0.1/ 0.00	2/ 0.09	15/ 0.70	7/ 0.33	/	/	/	24.1/ 1.13	6.53371
WSW	0.1/ 0.00	3/ 0.14	9/ 0.42	6/ 0.28	/	/	/	18.1/ 0.85	6.26865
W	0.1/ 0.00	3/ 0.14	8/ 0.38	/	/	/	/	11.1/ 0.52	4.25676
WNW	0.1/ 0.00	3/ 0.14	5/ 0.23	2/ 0.09	/	/	/	10.1/ 0.47	4.87005
NW	/	/	6/ 0.28	3/ 0.14	/	/	/	9.0/ 0.42	6.75278
NNW	0.0/ 0.00	1/ 0.05	9/ 0.42	6/ 0.28	1/ 0.05	/	/	17.0/ 0.80	7.52794
TOTAL	1.0/ 0.05	25/ 1.17	170/ 7.98	33/ 1.55	2/ 0.09	/	/	231.0/10.85	5.72911

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	7/ 0.33	/	/	/	/	7.0/ 0.33	5.32500
NNE	/	1/ 0.05	4/ 0.19	1/ 0.05	/	/	/	6.0/ 0.28	6.12083
NE	/	1/ 0.05	20/ 0.94	3/ 0.14	/	/	/	24.0/ 1.13	5.53333
ENE	/	2/ 0.09	5/ 0.23	/	/	/	/	7.0/ 0.33	4.80357
E	/	1/ 0.05	4/ 0.19	/	/	/	/	5.0/ 0.23	4.72000
ESE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.13750
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.55000
SSE	/	1/ 0.05	3/ 0.14	1/ 0.05	/	/	/	5.0/ 0.23	5.29500
S	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.93750
SSW	/	1/ 0.05	9/ 0.42	/	/	/	/	10.0/ 0.47	5.21250
SW	/	1/ 0.05	7/ 0.33	3/ 0.14	/	/	/	11.0/ 0.52	6.94545
WSW	/	1/ 0.05	3/ 0.14	5/ 0.23	/	/	/	9.0/ 0.42	6.84722
W	/	3/ 0.14	6/ 0.28	/	/	/	/	9.0/ 0.42	4.45833
WNW	/	2/ 0.09	8/ 0.38	1/ 0.05	/	/	/	11.0/ 0.52	4.73409
NW	/	2/ 0.09	3/ 0.14	2/ 0.09	/	/	/	7.0/ 0.33	5.65000
NNW	/	1/ 0.05	8/ 0.38	/	/	/	/	9.0/ 0.42	5.64722
TOTAL	/	17/ 0.80	92/ 4.32	16/ 0.75	/	/	/	125.0/ 5.87	5.51940

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	9/ 0.42	1/ 0.05	/	/	/	11.0/ 0.52	5.15454
NNE	/	/	7/ 0.33	/	/	/	/	7.0/ 0.33	5.07857
NE	/	/	12/ 0.56	2/ 0.09	/	/	/	14.0/ 0.66	5.90178
ENE	/	1/ 0.05	16/ 0.75	/	/	/	/	17.0/ 0.80	5.02206
E	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	4.92500
ESE	/	/	6/ 0.28	/	/	/	/	6.0/ 0.28	5.02083
SE	/	1/ 0.05	2/ 0.09	1/ 0.05	/	/	/	4.0/ 0.19	5.59375
SSE	/	1/ 0.05	7/ 0.33	/	/	/	/	8.0/ 0.38	4.60937
S	/	3/ 0.14	6/ 0.28	1/ 0.05	/	/	/	10.0/ 0.47	5.07750
SSW	/	2/ 0.09	7/ 0.33	/	/	/	/	9.0/ 0.42	4.22222
SW	/	3/ 0.14	5/ 0.23	1/ 0.05	/	/	/	9.0/ 0.42	5.04722
WSW	/	1/ 0.05	6/ 0.28	3/ 0.14	/	/	/	10.0/ 0.47	6.40750
W	/	2/ 0.09	9/ 0.42	1/ 0.05	/	/	/	12.0/ 0.56	5.29305
WNW	/	1/ 0.05	6/ 0.28	/	/	/	/	7.0/ 0.33	4.22500
NW	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	4.15000
NNW	/	2/ 0.09	3/ 0.14	/	/	/	/	5.0/ 0.23	3.69000
TOTAL	/	18/ 0.85	106/ 4.98	10/ 0.47	/	/	/	134.0/ 6.29	5.09341

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	7/ 0.33	22/ 1.03	1/ 0.05	/	/	/	30.0/ 1.41	4.63500
NNE	/	14/ 0.66	61/ 2.87	/	/	/	/	75.0/ 3.52	4.77200
NE	/	11/ 0.52	65/ 3.05	3/ 0.14	/	/	/	79.0/ 3.71	5.03133
ENE	/	8/ 0.38	36/ 1.69	5/ 0.23	/	/	/	49.0/ 2.30	5.29031
E	/	5/ 0.23	19/ 0.89	2/ 0.09	/	/	/	26.0/ 1.22	4.32404
ESE	/	6/ 0.28	5/ 0.23	1/ 0.05	/	/	/	12.0/ 0.56	4.01042
SE	/	3/ 0.14	7/ 0.33	/	/	/	/	10.0/ 0.47	4.11000
SSE	/	9/ 0.42	9/ 0.42	/	/	/	/	18.0/ 0.85	4.09305
S	/	5/ 0.23	10/ 0.47	/	/	/	/	15.0/ 0.70	4.62500
SSW	/	9/ 0.42	5/ 0.23	/	/	/	/	14.0/ 0.66	3.24464
SW	/	12/ 0.56	19/ 0.89	5/ 0.23	/	/	/	36.0/ 1.69	4.92430
WSW	/	9/ 0.42	12/ 0.56	5/ 0.23	/	/	/	26.0/ 1.22	4.99487
W	/	10/ 0.47	12/ 0.56	/	/	/	/	22.0/ 1.03	3.65795
WNW	/	6/ 0.28	5/ 0.23	/	/	/	/	11.0/ 0.52	3.38636
NW	/	2/ 0.09	9/ 0.42	/	/	/	/	11.0/ 0.52	4.98182
NNW	/	6/ 0.28	19/ 0.89	1/ 0.05	/	/	/	26.0/ 1.22	5.00961
TOTAL	/	122/ 5.73	315/14.80	23/ 1.08	/	/	/	460.0/21.61	4.68194

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.3/ 0.01	18/ 0.85	20/ 0.94	1/ 0.05	/	/	/	39.3/ 1.85	3.79103
NNE	0.3/ 0.01	22/ 1.03	14/ 0.66	/	/	/	/	36.3/ 1.71	3.11295
NE	0.3/ 0.01	17/ 0.80	17/ 0.80	/	/	/	/	34.3/ 1.61	3.93149
ENE	0.2/ 0.01	15/ 0.70	6/ 0.28	3/ 0.14	/	/	/	24.2/ 1.14	3.91116
E	0.2/ 0.01	11/ 0.52	6/ 0.28	1/ 0.05	/	/	/	18.2/ 0.85	3.70192
ESE	0.2/ 0.01	14/ 0.66	2/ 0.09	/	/	/	/	16.2/ 0.76	2.45216
SE	0.3/ 0.01	20/ 0.94	/	/	/	/	/	20.3/ 0.95	2.08313
SSE	0.2/ 0.01	13/ 0.61	4/ 0.19	/	/	/	/	17.2/ 0.81	2.96948
S	0.4/ 0.02	27/ 1.27	3/ 0.14	/	/	/	/	30.4/ 1.43	2.26562
SSW	0.6/ 0.03	36/ 1.69	14/ 0.66	/	/	/	/	50.6/ 2.38	2.77619
SW	0.3/ 0.01	20/ 0.94	10/ 0.47	1/ 0.05	/	/	/	31.3/ 1.47	2.97005
WSW	0.2/ 0.01	14/ 0.66	1/ 0.05	/	/	/	/	15.2/ 0.71	1.99671
W	0.1/ 0.00	6/ 0.28	1/ 0.05	/	/	/	/	7.1/ 0.33	2.04930
WNW	0.0/ 0.00	3/ 0.14	/	/	/	/	/	3.0/ 0.14	1.75833
NW	0.1/ 0.00	4/ 0.19	1/ 0.05	/	/	/	/	5.1/ 0.24	2.79412
NNW	0.3/ 0.01	16/ 0.75	9/ 0.42	1/ 0.05	/	/	/	26.3/ 1.24	3.03850
TOTAL	4.0/ 0.19	256/12.02	108/ 5.07	7/ 0.33	/	/	/	375.0/17.61	3.03627

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	1.4/ 0.07	24/ 1.13	7/ 0.33	/	/	/	/	32.4/ 1.52	2.87963
NNE	1.2/ 0.06	20/ 0.94	2/ 0.09	/	/	/	/	23.2/ 1.09	2.34159
NE	1.5/ 0.07	26/ 1.22	1/ 0.05	/	/	/	/	28.5/ 1.34	2.31667
ENE	1.1/ 0.05	19/ 0.89	/	/	/	/	/	20.1/ 0.94	1.70460
E	0.5/ 0.02	8/ 0.38	/	/	/	/	/	8.5/ 0.40	1.53382
ESE	0.6/ 0.03	10/ 0.47	/	/	/	/	/	10.6/ 0.50	1.29009
SE	0.5/ 0.02	9/ 0.42	/	/	/	/	/	9.5/ 0.45	1.46710
SSE	1.3/ 0.06	22/ 1.03	/	/	/	/	/	23.3/ 1.09	1.80633
S	1.1/ 0.05	19/ 0.89	/	/	/	/	/	20.1/ 0.94	1.71704
SSW	1.5/ 0.07	26/ 1.22	/	/	/	/	/	27.5/ 1.29	1.71409
SW	1.2/ 0.06	20/ 0.94	/	/	/	/	/	21.2/ 1.00	1.54599
WSW	0.4/ 0.02	7/ 0.33	1/ 0.05	/	/	/	/	8.4/ 0.39	1.44940
W	0.4/ 0.02	6/ 0.28	/	/	/	/	/	6.4/ 0.30	1.33594
WNW	0.1/ 0.00	1/ 0.05	/	/	/	/	/	1.1/ 0.05	1.01136
NW	0.5/ 0.02	8/ 0.38	/	/	/	/	/	8.5/ 0.40	2.02794
NNW	0.8/ 0.04	13/ 0.61	1/ 0.05	/	/	/	/	14.8/ 0.70	2.02703
TOTAL	14.0/ 0.66	238/11.18	12/ 0.56	/	/	/	/	264.0/12.40	1.94754

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=0 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	27.2/ 1.28	56/ 2.63	4/ 0.19	/	/	/	/	87.2/ 4.10	1.49083
NNE	26.2/ 1.23	54/ 2.54	/	/	/	/	/	80.2/ 3.77	1.12999
NE	21.9/ 1.03	45/ 2.11	1/ 0.05	/	/	/	/	67.9/ 3.19	1.32327
ENE	21.4/ 1.01	44/ 2.07	1/ 0.05	/	/	/	/	66.4/ 3.12	1.14947
E	9.2/ 0.43	19/ 0.89	/	/	/	/	/	28.2/ 1.32	0.93262
ESE	6.3/ 0.30	13/ 0.61	/	/	/	/	/	19.3/ 0.91	0.86464
SE	6.8/ 0.32	14/ 0.66	/	/	/	/	/	20.8/ 0.98	1.07211
SSE	9.2/ 0.43	19/ 0.89	/	/	/	/	/	28.2/ 1.32	0.99734
S	4.4/ 0.21	9/ 0.42	/	/	/	/	/	13.4/ 0.63	1.09142
SSW	5.8/ 0.27	12/ 0.56	/	/	/	/	/	17.8/ 0.84	0.92837
SW	2.9/ 0.14	6/ 0.28	/	/	/	/	/	8.9/ 0.42	0.81320
WSW	3.9/ 0.18	8/ 0.38	/	/	/	/	/	11.9/ 0.56	0.82248
W	6.8/ 0.32	14/ 0.66	/	/	/	/	/	20.8/ 0.98	0.80769
WNW	2.4/ 0.11	5/ 0.23	/	/	/	/	/	7.4/ 0.35	0.74662
NW	4.4/ 0.21	9/ 0.42	/	/	/	/	/	13.4/ 0.63	1.14366
NNW	15.1/ 0.71	31/ 1.46	2/ 0.09	/	/	/	/	48.1/ 2.26	1.34927
TOTAL	174.0/ 8.17	358/16.82	8/ 0.38	/	/	/	/	540.0/25.36	1.16840

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	16.7/ 0.80	106/ 5.08	143/ 6.86	48/ 2.30	/	/	/	313.7/15.05	4.50049
NNE	15.9/ 0.76	101/ 4.84	107/ 5.13	42/ 2.01	/	/	/	265.9/12.75	4.26192
NE	13.7/ 0.66	87/ 4.17	60/ 2.88	19/ 0.91	/	/	/	179.7/ 8.62	3.77247
ENE	11.4/ 0.55	72/ 3.45	30/ 1.44	3/ 0.14	/	/	/	116.4/ 5.58	2.89583
E	6.8/ 0.33	43/ 2.06	22/ 1.06	2/ 0.10	/	/	/	73.8/ 3.54	2.75948
ESE	6.9/ 0.33	44/ 2.11	6/ 0.29	2/ 0.10	/	/	/	58.9/ 2.82	2.13646
SE	6.5/ 0.31	41/ 1.97	10/ 0.48	2/ 0.10	/	/	/	59.5/ 2.85	2.41912
SSE	7.4/ 0.35	47/ 2.25	39/ 1.87	2/ 0.10	/	/	/	95.4/ 4.58	3.36452
S	8.8/ 0.42	56/ 2.69	31/ 1.49	5/ 0.24	/	/	/	100.8/ 4.83	3.19990
SSW	9.8/ 0.47	62/ 2.97	39/ 1.87	14/ 0.67	1/ 0.05	/	/	125.8/ 6.03	3.88951
SW	6.0/ 0.29	38/ 1.82	39/ 1.87	12/ 0.58	/	/	/	95.0/ 4.56	4.11947
WSW	6.1/ 0.29	39/ 1.87	47/ 2.25	13/ 0.62	2/ 0.10	/	/	107.1/ 5.14	4.57260
W	3.6/ 0.17	23/ 1.10	45/ 2.16	13/ 0.62	4/ 0.19	/	/	88.6/ 4.25	5.21134
WNW	5.5/ 0.26	35/ 1.68	23/ 1.10	19/ 0.91	4/ 0.19	/	/	86.5/ 4.15	5.04827
NW	6.5/ 0.31	41/ 1.97	57/ 2.73	30/ 1.44	/	/	/	134.5/ 6.45	4.88708
NNW	8.4/ 0.40	53/ 2.54	53/ 2.54	66/ 3.17	3/ 0.14	/	/	183.4/ 8.80	5.64695
TOTAL	140.0/ 6.71	888/42.59	751/36.02	292/14.00	14/ 0.67	/	/	2085/ 100	4.14327

NUMBER OF BAD RECORDS: 147

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	24/ 1.15	18/ 0.86	/	/	/	43.0/ 2.06	7.10756
NNE	/	/	17/ 0.82	22/ 1.06	/	/	/	39.0/ 1.87	7.92564
NE	/	/	18/ 0.86	13/ 0.62	/	/	/	31.0/ 1.49	7.50806
ENE	/	/	8/ 0.38	3/ 0.14	/	/	/	11.0/ 0.53	6.15000
E	/	/	5/ 0.24	2/ 0.10	/	/	/	7.0/ 0.34	6.35000
ESE	/	1/ 0.05	5/ 0.24	/	/	/	/	6.0/ 0.29	4.29167
SE	/	1/ 0.05	2/ 0.10	/	/	/	/	3.0/ 0.14	4.29167
SSE	/	/	4/ 0.19	/	/	/	/	4.0/ 0.19	5.88750
S	/	2/ 0.10	2/ 0.10	3/ 0.14	/	/	/	7.0/ 0.34	6.00357
SSW	/	2/ 0.10	4/ 0.19	9/ 0.43	/	/	/	15.0/ 0.72	7.37500
SW	/	1/ 0.05	12/ 0.58	6/ 0.29	/	/	/	19.0/ 0.91	6.38421
WSW	/	2/ 0.10	26/ 1.25	7/ 0.34	/	/	/	35.0/ 1.68	6.49286
W	/	1/ 0.05	20/ 0.96	6/ 0.29	/	/	/	27.0/ 1.29	6.18611
WNW	/	/	10/ 0.48	11/ 0.53	4/ 0.19	/	/	25.0/ 1.20	8.65900
NW	/	1/ 0.05	14/ 0.67	11/ 0.53	/	/	/	26.0/ 1.25	6.82596
NNW	/	1/ 0.05	12/ 0.58	28/ 1.34	/	/	/	41.0/ 1.97	8.21646
TOTAL	/	13/ 0.62	183/ 8.78	139/ 6.67	4/ 0.19	/	/	339.0/16.26	7.14100

NUMBER OF BAD RECORDS: 31

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	8/ 0.38	5/ 0.24	/	/	/	14.0/ 0.67	7.02500
NNE	/	/	3/ 0.14	4/ 0.19	/	/	/	7.0/ 0.34	7.23928
NE	/	/	6/ 0.29	1/ 0.05	/	/	/	7.0/ 0.34	5.93214
ENE	/	1/ 0.05	2/ 0.10	/	/	/	/	3.0/ 0.14	4.40833
E	/	/	/	/	/	/	/	/	
ESE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	3.02500
SE	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	5.45000
SSE	/	1/ 0.05	2/ 0.10	1/ 0.05	/	/	/	4.0/ 0.19	5.18125
S	/	2/ 0.10	5/ 0.24	/	/	/	/	7.0/ 0.34	4.35357
SSW	/	2/ 0.10	4/ 0.19	/	/	/	/	6.0/ 0.29	4.25417
SW	/	/	7/ 0.34	1/ 0.05	/	/	/	8.0/ 0.38	4.87187
WSW	/	1/ 0.05	4/ 0.19	3/ 0.14	1/ 0.05	/	/	9.0/ 0.43	7.06389
W	/	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	3.0/ 0.14	9.13333
WNW	/	/	/	2/ 0.10	/	/	/	2.0/ 0.10	8.22500
NW	/	/	3/ 0.14	6/ 0.29	/	/	/	9.0/ 0.43	8.03055
NNW	/	/	3/ 0.14	6/ 0.29	2/ 0.10	/	/	11.0/ 0.53	9.06818
TOTAL	/	9/ 0.43	50/ 2.40	30/ 1.44	4/ 0.19	/	/	93.0/ 4.46	6.58978

NUMBER OF BAD RECORDS: 9

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	9/ 0.43	15/ 0.72	/	/	/	24.0/ 1.15	7.89271
NNE	/	/	7/ 0.34	5/ 0.24	/	/	/	12.0/ 0.58	7.32361
NE	/	2/ 0.10	5/ 0.24	/	/	/	/	7.0/ 0.34	4.34286
ENE	/	1/ 0.05	4/ 0.19	/	/	/	/	5.0/ 0.24	4.89000
E	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.10	2.71250
ESE	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.10	3.42500
SE	/	/	/	/	/	/	/	/	
SSE	/	1/ 0.05	4/ 0.19	/	/	/	/	5.0/ 0.24	5.69500
S	/	1/ 0.05	4/ 0.19	/	/	/	/	5.0/ 0.24	4.34500
SSW	/	2/ 0.10	5/ 0.24	2/ 0.10	/	/	/	9.0/ 0.43	5.31111
SW	/	2/ 0.10	5/ 0.24	2/ 0.10	/	/	/	9.0/ 0.43	5.21111
WSW	/	3/ 0.14	1/ 0.05	/	1/ 0.05	/	/	5.0/ 0.24	5.75000
W	/	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	3.0/ 0.14	10.22500
WNW	/	1/ 0.05	1/ 0.05	3/ 0.14	/	/	/	5.0/ 0.24	7.65000
NW	/	/	1/ 0.05	5/ 0.24	/	/	/	6.0/ 0.29	7.83333
NNW	/	/	6/ 0.29	9/ 0.43	1/ 0.05	/	/	16.0/ 0.77	8.33906
TOTAL	/	15/ 0.72	55/ 2.64	42/ 2.01	3/ 0.14	/	/	115.0/ 5.52	6.67333

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	13/ 0.62	60/ 2.88	9/ 0.43	/	/	/	82.0/ 3.93	5.25478
NNE	/	10/ 0.48	48/ 2.30	10/ 0.48	/	/	/	68.0/ 3.26	5.09853
NE	/	8/ 0.38	19/ 0.91	4/ 0.19	/	/	/	31.0/ 1.49	4.96210
ENE	/	8/ 0.38	10/ 0.48	/	/	/	/	18.0/ 0.86	4.42083
E	/	4/ 0.19	12/ 0.58	/	/	/	/	16.0/ 0.77	4.58281
ESE	/	2/ 0.10	/	/	/	/	/	2.0/ 0.10	1.67500
SE	/	4/ 0.19	4/ 0.19	1/ 0.05	/	/	/	9.0/ 0.43	4.27222
SSE	/	4/ 0.19	15/ 0.72	/	/	/	/	19.0/ 0.91	4.54737
S	/	5/ 0.24	6/ 0.29	/	/	/	/	11.0/ 0.53	4.06136
SSW	/	11/ 0.53	11/ 0.53	2/ 0.10	/	/	/	24.0/ 1.15	4.55000
SW	/	4/ 0.19	10/ 0.48	3/ 0.14	/	/	/	17.0/ 0.82	5.73088
WSW	/	3/ 0.14	4/ 0.19	3/ 0.14	/	/	/	10.0/ 0.48	5.78750
W	/	3/ 0.14	7/ 0.34	3/ 0.14	1/ 0.05	/	/	14.0/ 0.67	6.08393
WNW	/	6/ 0.29	5/ 0.24	3/ 0.14	/	/	/	14.0/ 0.67	5.13571
NW	/	6/ 0.29	17/ 0.82	5/ 0.24	/	/	/	28.0/ 1.34	5.64196
NNW	/	6/ 0.29	17/ 0.82	15/ 0.72	/	/	/	38.0/ 1.82	6.17763
TOTAL	/	97/ 4.65	245/11.75	58/ 2.78	1/ 0.05	/	/	401.0/19.23	5.16581

NUMBER OF BAD RECORDS: 29

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.5/ 0.02	21/ 1.01	38/ 1.82	1/ 0.05	/	/	/	60.5/ 2.90	3.93864
NNE	0.7/ 0.03	30/ 1.44	26/ 1.25	1/ 0.05	/	/	/	57.7/ 2.77	3.59554
NE	0.4/ 0.02	16/ 0.77	10/ 0.48	1/ 0.05	/	/	/	27.4/ 1.31	3.57025
ENE	0.3/ 0.01	11/ 0.53	6/ 0.29	/	/	/	/	17.3/ 0.83	3.55058
E	0.1/ 0.00	3/ 0.14	3/ 0.14	/	/	/	/	6.1/ 0.29	3.38115
ESE	0.2/ 0.01	7/ 0.34	/	2/ 0.10	/	/	/	9.2/ 0.44	3.65489
SE	0.2/ 0.01	10/ 0.48	2/ 0.10	1/ 0.05	/	/	/	13.2/ 0.63	2.73485
SSE	0.4/ 0.02	18/ 0.86	14/ 0.67	1/ 0.05	/	/	/	33.4/ 1.60	3.47829
S	0.5/ 0.02	20/ 0.96	13/ 0.62	2/ 0.10	/	/	/	35.5/ 1.70	3.60035
SSW	0.5/ 0.02	19/ 0.91	15/ 0.72	1/ 0.05	1/ 0.05	/	/	36.5/ 1.75	3.93356
SW	0.2/ 0.01	9/ 0.43	4/ 0.19	/	/	/	/	13.2/ 0.63	3.03788
WSW	0.1/ 0.00	5/ 0.24	8/ 0.38	/	/	/	/	13.1/ 0.63	3.68702
W	0.1/ 0.00	5/ 0.24	14/ 0.67	2/ 0.10	1/ 0.05	/	/	22.1/ 1.06	5.29638
WNW	0.2/ 0.01	10/ 0.48	4/ 0.19	/	/	/	/	14.2/ 0.68	3.00000
NW	0.2/ 0.01	9/ 0.43	17/ 0.82	3/ 0.14	/	/	/	29.2/ 1.40	4.45976
NNW	0.2/ 0.01	7/ 0.34	13/ 0.62	8/ 0.38	/	/	/	28.2/ 1.35	5.39362
TOTAL	5.0/ 0.24	200/ 9.59	187/ 8.97	23/ 1.10	2/ 0.10	/	/	417.0/20.00	3.86879

NUMBER OF BAD RECORDS: 45

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.3/ 0.01	18/ 0.86	3/ 0.14	/	/	/	/	21.3/ 1.02	2.29225
NNE	0.2/ 0.01	15/ 0.72	4/ 0.19	/	/	/	/	19.2/ 0.92	2.41667
NE	0.2/ 0.01	14/ 0.67	1/ 0.05	/	/	/	/	15.2/ 0.73	2.24013
ENE	0.1/ 0.00	8/ 0.38	/	/	/	/	/	8.1/ 0.39	2.24846
E	0.1/ 0.00	7/ 0.34	1/ 0.05	/	/	/	/	8.1/ 0.39	1.58951
ESE	0.1/ 0.00	6/ 0.29	/	/	/	/	/	6.1/ 0.29	1.57172
SE	0.1/ 0.00	8/ 0.38	/	/	/	/	/	8.1/ 0.39	1.83179
SSE	0.1/ 0.00	9/ 0.43	/	/	/	/	/	9.1/ 0.44	2.14972
S	0.1/ 0.00	9/ 0.43	1/ 0.05	/	/	/	/	10.1/ 0.48	2.25742
SSW	0.1/ 0.00	6/ 0.29	/	/	/	/	/	6.1/ 0.29	1.94877
SW	0.1/ 0.00	9/ 0.43	/	/	/	/	/	9.1/ 0.44	2.20742
WSW	0.1/ 0.00	5/ 0.24	4/ 0.19	/	/	/	/	9.1/ 0.44	3.26923
W	0.1/ 0.00	5/ 0.24	2/ 0.10	/	/	/	/	7.1/ 0.34	2.52113
WNW	0.0/ 0.00	3/ 0.14	3/ 0.14	/	/	/	/	6.0/ 0.29	3.13333
NW	0.1/ 0.00	7/ 0.34	5/ 0.24	/	/	/	/	12.1/ 0.58	3.32851
NNW	0.2/ 0.01	12/ 0.58	2/ 0.10	/	/	/	/	14.2/ 0.68	2.15669
TOTAL	2.0/ 0.10	141/ 6.76	26/ 1.25	/	/	/	/	169.0/ 8.11	2.34601

NUMBER OF BAD RECORDS: 14

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	16.7/ 0.80	52/ 2.49	1/ 0.05	/	/	/	/	69.7/ 3.34	1.36191
NNE	14.8/ 0.71	46/ 2.21	2/ 0.10	/	/	/	/	62.8/ 3.01	1.26075
NE	15.1/ 0.72	47/ 2.25	1/ 0.05	/	/	/	/	63.1/ 3.03	1.33201
ENE	13.8/ 0.66	43/ 2.06	/	/	/	/	/	56.8/ 2.72	1.14701
E	9.0/ 0.43	28/ 1.34	/	/	/	/	/	37.0/ 1.77	1.15743
ESE	8.4/ 0.40	26/ 1.25	/	/	/	/	/	34.4/ 1.65	1.14026
SE	5.8/ 0.28	18/ 0.86	/	/	/	/	/	23.8/ 1.14	1.08823
SSE	4.5/ 0.22	14/ 0.67	/	/	/	/	/	18.5/ 0.89	1.07770
S	5.5/ 0.26	17/ 0.82	/	/	/	/	/	22.5/ 1.08	1.14833
SSW	6.4/ 0.31	20/ 0.96	/	/	/	/	/	26.4/ 1.27	1.10511
SW	4.2/ 0.20	13/ 0.62	1/ 0.05	/	/	/	/	18.2/ 0.87	1.27198
WSW	6.4/ 0.31	20/ 0.96	/	/	/	/	/	26.4/ 1.27	1.04640
W	2.9/ 0.14	9/ 0.43	/	/	/	/	/	11.9/ 0.57	1.04307
WNW	4.8/ 0.23	15/ 0.72	/	/	/	/	/	19.8/ 0.95	1.30808
NW	5.8/ 0.28	18/ 0.86	/	/	/	/	/	23.8/ 1.14	1.14496
NNW	8.7/ 0.42	27/ 1.29	/	/	/	/	/	35.7/ 1.71	1.09699
TOTAL	133.0/ 6.38	413/19.81	5/ 0.24	/	/	/	/	551.0/26.43	1.20091

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	7.9/ 0.37	91/ 4.27	104/ 4.88	39/ 1.83	/	/	/	241.9/11.36	4.39391
NNE	9.2/ 0.43	105/ 4.93	152/ 7.14	19/ 0.89	/	/	/	285.2/13.40	4.06565
NE	6.7/ 0.31	77/ 3.62	59/ 2.77	3/ 0.14	/	/	/	145.7/ 6.84	3.26192
ENE	4.7/ 0.22	54/ 2.54	55/ 2.58	6/ 0.28	/	/	/	119.7/ 5.62	3.84116
E	5.3/ 0.25	61/ 2.87	25/ 1.17	8/ 0.38	2/ 0.09	/	/	101.3/ 4.76	3.61130
ESE	3.7/ 0.17	42/ 1.97	24/ 1.13	/	/	/	/	69.7/ 3.27	2.99641
SE	2.5/ 0.12	29/ 1.36	17/ 0.80	3/ 0.14	/	/	/	51.5/ 2.42	3.20364
SSE	4.0/ 0.19	46/ 2.16	24/ 1.13	2/ 0.09	/	/	/	76.0/ 3.57	3.07204
S	4.9/ 0.23	56/ 2.63	32/ 1.50	9/ 0.42	/	/	/	101.9/ 4.79	3.58035
SSW	6.4/ 0.30	73/ 3.43	79/ 3.71	15/ 0.70	2/ 0.09	/	/	175.4/ 8.24	4.25285
SW	5.8/ 0.27	67/ 3.15	85/ 3.99	24/ 1.13	3/ 0.14	/	/	184.8/ 8.68	4.71293
WSW	4.4/ 0.21	50/ 2.35	40/ 1.88	25/ 1.17	6/ 0.28	/	/	125.4/ 5.89	5.27020
W	3.1/ 0.15	35/ 1.64	30/ 1.41	11/ 0.52	1/ 0.05	/	/	80.1/ 3.76	4.31180
WNW	2.9/ 0.14	33/ 1.55	21/ 0.99	33/ 1.55	5/ 0.23	/	/	94.9/ 4.46	6.03872
NW	3.3/ 0.16	38/ 1.78	39/ 1.83	26/ 1.22	11/ 0.52	/	/	117.3/ 5.51	5.76236
NNW	6.2/ 0.29	71/ 3.33	56/ 2.63	23/ 1.08	2/ 0.09	/	/	158.2/ 7.43	4.34529
TOTAL	81.0/ 3.80	928/43.59	842/39.55	246/11.55	32/ 1.50	/	/	2129/ 100	4.25318

NUMBER OF BAD RECORDS: 31

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	9/ 0.42	9/ 0.42	/	/	/	18.0/ 0.85	7.30694
NNE	/	/	3/ 0.14	4/ 0.19	/	/	/	7.0/ 0.33	7.68571
NE	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	5.43333
ENE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	7.03750
E	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	5.89167
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.30000
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	/	/	/	/	/	
S	/	/	/	/	/	/	/	/	
SSW	/	/	3/ 0.14	3/ 0.14	/	/	/	6.0/ 0.28	7.95417
SW	/	/	8/ 0.38	3/ 0.14	/	/	/	11.0/ 0.52	6.91818
WSW	/	/	2/ 0.09	10/ 0.47	/	/	/	12.0/ 0.56	9.23750
W	/	/	/	4/ 0.19	/	/	/	4.0/ 0.19	10.02500
WNW	/	/	/	7/ 0.33	/	/	/	7.0/ 0.33	11.30357
NW	/	/	3/ 0.14	7/ 0.33	8/ 0.38	/	/	18.0/ 0.85	10.94167
NNW	/	/	5/ 0.23	3/ 0.14	/	/	/	8.0/ 0.38	6.50937
TOTAL	/	/	41/ 1.93	51/ 2.40	8/ 0.38	/	/	100.0/ 4.70	8.41600

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	7.28333
NNE	/	/	2/ 0.09	2/ 0.09	/	/	/	4.0/ 0.19	6.97500
NE	/	/	5/ 0.23	/	/	/	/	5.0/ 0.23	5.81833
ENE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.81833
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.25000
ESE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.16250
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	/	/	/	/	/	
S	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	2.45000
SSW	/	1/ 0.05	4/ 0.19	2/ 0.09	/	/	/	7.0/ 0.33	6.40357
SW	/	1/ 0.05	5/ 0.23	3/ 0.14	/	/	/	9.0/ 0.42	6.36944
WSW	/	/	7/ 0.33	6/ 0.28	/	/	/	13.0/ 0.61	7.62500
W	/	/	/	5/ 0.23	/	/	/	5.0/ 0.23	8.93500
WNW	/	/	1/ 0.05	9/ 0.42	1/ 0.05	/	/	11.0/ 0.52	9.87727
NW	/	/	1/ 0.05	4/ 0.19	2/ 0.09	/	/	7.0/ 0.33	10.43928
NNW	/	/	8/ 0.38	2/ 0.09	/	/	/	10.0/ 0.47	6.17250
TOTAL	/	3/ 0.14	40/ 1.88	34/ 1.60	3/ 0.14	/	/	80.0/ 3.76	7.49333

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	1/ 0.05	7/ 0.33	7/ 0.33	/	/	/	15.0/ 0.70	6.65000
NNE	/	/	9/ 0.42	1/ 0.05	/	/	/	10.0/ 0.47	6.09500
NE	/	1/ 0.05	2/ 0.09	/	/	/	/	3.0/ 0.14	4.58333
ENE	/	/	4/ 0.19	/	/	/	/	4.0/ 0.19	4.31250
E	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.60000
ESE	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	5.66667
SE	/	1/ 0.05	3/ 0.14	/	/	/	/	4.0/ 0.19	4.56250
SSE	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	5.03333
S	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	4.71250
SSW	/	/	8/ 0.38	2/ 0.09	/	/	/	10.0/ 0.47	6.44000
SW	/	3/ 0.14	9/ 0.42	6/ 0.28	/	/	/	18.0/ 0.85	6.83750
WSW	/	2/ 0.09	7/ 0.33	2/ 0.09	1/ 0.05	/	/	12.0/ 0.56	6.77292
W	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	5.75833
WNW	/	/	/	2/ 0.09	2/ 0.09	/	/	4.0/ 0.19	12.89375
NW	/	2/ 0.09	2/ 0.09	6/ 0.28	/	/	/	10.0/ 0.47	7.25750
NNW	/	1/ 0.05	6/ 0.28	2/ 0.09	1/ 0.05	/	/	10.0/ 0.47	6.83750
TOTAL	/	11/ 0.52	70/ 3.29	28/ 1.32	4/ 0.19	/	/	113.0/ 5.31	6.55951

NUMBER OF BAD RECORDS: 4

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.1/ 0.00	13/ 0.61	57/ 2.68	22/ 1.03	/	/	/	92.1/ 4.33	5.70670
NNE	0.3/ 0.01	30/ 1.41	94/ 4.42	11/ 0.52	/	/	/	135.3/ 6.36	4.78095
NE	0.2/ 0.01	21/ 0.99	27/ 1.27	1/ 0.05	/	/	/	49.2/ 2.31	3.98391
ENE	0.1/ 0.00	9/ 0.42	26/ 1.22	2/ 0.09	/	/	/	37.1/ 1.74	4.87972
E	0.0/ 0.00	4/ 0.19	9/ 0.42	6/ 0.28	/	/	/	19.0/ 0.89	5.69210
ESE	0.0/ 0.00	2/ 0.09	12/ 0.56	/	/	/	/	14.0/ 0.66	5.18571
SE	0.0/ 0.00	1/ 0.05	4/ 0.19	3/ 0.14	/	/	/	8.0/ 0.38	6.15625
SSE	0.0/ 0.00	2/ 0.09	9/ 0.42	2/ 0.09	/	/	/	13.0/ 0.61	5.48077
S	0.1/ 0.00	8/ 0.38	16/ 0.75	5/ 0.23	/	/	/	29.1/ 1.37	5.03050
SSW	0.2/ 0.01	22/ 1.03	29/ 1.36	7/ 0.33	2/ 0.09	/	/	60.2/ 2.83	4.93522
SW	0.2/ 0.01	17/ 0.80	32/ 1.50	9/ 0.42	2/ 0.09	/	/	60.2/ 2.83	5.37749
WSW	0.2/ 0.01	18/ 0.85	18/ 0.85	7/ 0.33	4/ 0.19	/	/	47.2/ 2.22	5.61405
W	0.1/ 0.00	9/ 0.42	13/ 0.61	2/ 0.09	/	/	/	24.1/ 1.13	4.36463
WNW	0.1/ 0.00	5/ 0.23	7/ 0.33	12/ 0.56	2/ 0.09	/	/	26.1/ 1.23	7.24521
NW	0.1/ 0.00	5/ 0.23	9/ 0.42	8/ 0.38	1/ 0.05	/	/	23.1/ 1.09	6.44480
NNW	0.2/ 0.01	19/ 0.89	20/ 0.94	14/ 0.66	/	/	/	53.2/ 2.50	5.59210
TOTAL	2.0/ 0.09	185/ 8.69	382/17.94	111/ 5.21	11/ 0.52	/	/	691.0/32.46	5.24400

NUMBER OF BAD RECORDS: 13

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	12/ 0.56	17/ 0.80	/	/	/	/	29.0/ 1.36	3.75690
NNE	/	17/ 0.80	38/ 1.78	1/ 0.05	/	/	/	56.0/ 2.63	4.17991
NE	/	10/ 0.47	22/ 1.03	1/ 0.05	/	/	/	33.0/ 1.55	4.20076
ENE	/	10/ 0.47	18/ 0.85	4/ 0.19	/	/	/	32.0/ 1.50	4.85781
E	/	18/ 0.85	8/ 0.38	2/ 0.09	2/ 0.09	/	/	30.0/ 1.41	4.74583
ESE	/	9/ 0.42	6/ 0.28	/	/	/	/	15.0/ 0.70	3.61833
SE	/	6/ 0.28	10/ 0.47	/	/	/	/	16.0/ 0.75	3.85469
SSE	/	14/ 0.66	11/ 0.52	/	/	/	/	25.0/ 1.17	3.41900
S	/	18/ 0.85	10/ 0.47	4/ 0.19	/	/	/	32.0/ 1.50	4.25234
SSW	/	16/ 0.75	28/ 1.32	1/ 0.05	/	/	/	45.0/ 2.11	4.26667
SW	/	8/ 0.38	22/ 1.03	3/ 0.14	1/ 0.05	/	/	34.0/ 1.60	5.12426
WSW	/	4/ 0.19	5/ 0.23	/	1/ 0.05	/	/	10.0/ 0.47	4.41750
W	/	6/ 0.28	11/ 0.52	/	1/ 0.05	/	/	18.0/ 0.85	4.65694
WNW	/	3/ 0.14	7/ 0.33	3/ 0.14	/	/	/	13.0/ 0.61	5.39038
NW	/	7/ 0.33	23/ 1.08	1/ 0.05	/	/	/	31.0/ 1.46	4.43629
NNW	/	7/ 0.33	10/ 0.47	2/ 0.09	1/ 0.05	/	/	20.0/ 0.94	4.55875
TOTAL	/	165/ 7.75	246/11.55	22/ 1.03	6/ 0.28	/	/	439.0/20.62	4.35074

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.2/ 0.01	20/ 0.94	11/ 0.52	/	/	/	/	31.2/ 1.47	3.03926
NNE	0.2/ 0.01	20/ 0.94	6/ 0.28	/	/	/	/	26.2/ 1.23	2.55248
NE	0.1/ 0.00	13/ 0.61	/	/	/	/	/	13.1/ 0.62	1.90172
ENE	0.1/ 0.00	12/ 0.56	/	/	/	/	/	12.1/ 0.57	2.06921
E	0.1/ 0.00	10/ 0.47	2/ 0.09	/	/	/	/	12.1/ 0.57	2.58471
ESE	0.1/ 0.00	11/ 0.52	/	/	/	/	/	11.1/ 0.52	1.88176
SE	0.1/ 0.00	6/ 0.28	/	/	/	/	/	6.1/ 0.29	1.95697
SSE	0.2/ 0.01	16/ 0.75	1/ 0.05	/	/	/	/	17.2/ 0.81	2.31395
S	0.2/ 0.01	17/ 0.80	4/ 0.19	/	/	/	/	21.2/ 1.00	2.42689
SSW	0.1/ 0.00	15/ 0.70	7/ 0.33	/	/	/	/	22.1/ 1.04	2.75226
SW	0.2/ 0.01	16/ 0.75	8/ 0.38	/	/	/	/	24.2/ 1.14	2.92975
WSW	0.1/ 0.00	11/ 0.52	1/ 0.05	/	/	/	/	12.1/ 0.57	2.50620
W	0.1/ 0.00	6/ 0.28	3/ 0.14	/	/	/	/	9.1/ 0.43	2.76648
WNW	0.1/ 0.00	10/ 0.47	5/ 0.23	/	/	/	/	15.1/ 0.71	2.74007
NW	0.1/ 0.00	9/ 0.42	1/ 0.05	/	/	/	/	10.1/ 0.47	2.11386
NNW	0.1/ 0.00	13/ 0.61	6/ 0.28	/	/	/	/	19.1/ 0.90	2.82330
TOTAL	2.0/ 0.09	205/ 9.63	55/ 2.58	/	/	/	/	262.0/12.31	2.56059

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	9.7/ 0.46	45/ 2.11	1/ 0.05	/	/	/	/	55.7/ 2.62	1.41921
NNE	8.2/ 0.39	38/ 1.78	/	/	/	/	/	46.2/ 2.17	1.34794
NE	6.9/ 0.32	32/ 1.50	1/ 0.05	/	/	/	/	39.9/ 1.87	1.36529
ENE	4.9/ 0.23	23/ 1.08	3/ 0.14	/	/	/	/	30.9/ 1.45	1.69175
E	6.2/ 0.29	29/ 1.36	/	/	/	/	/	35.2/ 1.65	1.25994
ESE	4.3/ 0.20	20/ 0.94	/	/	/	/	/	24.3/ 1.14	1.12500
SE	3.2/ 0.15	15/ 0.70	/	/	/	/	/	18.2/ 0.85	1.22527
SSE	3.0/ 0.14	14/ 0.66	/	/	/	/	/	17.0/ 0.80	1.09559
S	2.6/ 0.12	12/ 0.56	/	/	/	/	/	14.6/ 0.69	1.01199
SSW	4.1/ 0.19	19/ 0.89	/	/	/	/	/	23.1/ 1.09	1.35877
SW	4.7/ 0.22	22/ 1.03	1/ 0.05	/	/	/	/	27.7/ 1.30	1.47563
WSW	3.2/ 0.15	15/ 0.70	/	/	/	/	/	18.2/ 0.85	1.38049
W	3.0/ 0.14	14/ 0.66	/	/	/	/	/	17.0/ 0.80	1.51765
WNW	3.2/ 0.15	15/ 0.70	1/ 0.05	/	/	/	/	19.2/ 0.90	1.63802
NW	3.2/ 0.15	15/ 0.70	/	/	/	/	/	18.2/ 0.85	1.21154
NNW	6.6/ 0.31	31/ 1.46	1/ 0.05	/	/	/	/	38.6/ 1.81	1.51813
TOTAL	77.0/ 3.62	359/16.86	8/ 0.38	/	/	/	/	444.0/ 20.85	1.37556

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 SUMMARY OVER ALL STABILITY

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	4.1/ 0.20	67/ 3.34	103/ 5.14	12/ 0.60	1/ 0.05	/	/	187.1/ 9.34	4.34794
NNE	5.4/ 0.27	89/ 4.44	83/ 4.14	4/ 0.20	/	/	/	181.4/ 9.05	3.56500
NE	4.2/ 0.21	70/ 3.49	38/ 1.90	6/ 0.30	/	/	/	118.2/ 5.90	3.23033
ENE	4.0/ 0.20	66/ 3.29	25/ 1.25	1/ 0.05	/	/	/	96.0/ 4.79	2.77474
E	2.9/ 0.14	48/ 2.40	15/ 0.75	/	/	/	/	65.9/ 3.29	2.44879
ESE	2.5/ 0.12	42/ 2.10	23/ 1.15	1/ 0.05	/	/	/	68.5/ 3.42	2.94179
SE	2.5/ 0.12	42/ 2.10	11/ 0.55	2/ 0.10	/	/	/	57.5/ 2.87	2.60587
SSE	1.7/ 0.08	28/ 1.40	10/ 0.50	3/ 0.15	/	/	/	42.7/ 2.13	3.01083
S	2.8/ 0.14	46/ 2.30	41/ 2.05	9/ 0.45	1/ 0.05	/	/	99.8/ 4.98	3.97428
SSW	4.6/ 0.23	76/ 3.79	67/ 3.34	49/ 2.45	14/ 0.70	/	/	210.6/10.51	5.65254
SW	3.5/ 0.17	57/ 2.84	62/ 3.09	46/ 2.30	3/ 0.15	/	/	171.5/ 8.56	5.40632
WSW	1.9/ 0.09	32/ 1.60	60/ 2.99	44/ 2.20	2/ 0.10	/	/	139.9/ 6.98	6.17387
W	2.1/ 0.10	35/ 1.75	48/ 2.40	14/ 0.70	4/ 0.20	/	/	103.1/ 5.14	4.87949
WNW	1.9/ 0.09	31/ 1.55	49/ 2.45	42/ 2.10	17/ 0.85	/	/	140.9/ 7.03	6.99272
NW	1.9/ 0.09	31/ 1.55	65/ 3.24	39/ 1.95	13/ 0.65	/	/	149.9/ 7.48	6.48760
NNW	3.9/ 0.19	64/ 3.19	69/ 3.44	27/ 1.35	7/ 0.35	/	/	170.9/ 8.53	4.84986
TOTAL	50.0/ 2.50	824/41.12	769/38.37	299/14.92	62/ 3.09	/	/	2004/ 100	4.69927

NUMBER OF BAD RECORDS: 228

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=A

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	14/ 0.70	5/ 0.25	1/ 0.05	/	/	20.0/ 1.00	7.17000
NNE	/	/	7/ 0.35	/	/	/	/	7.0/ 0.35	5.79524
NE	/	/	5/ 0.25	3/ 0.15	/	/	/	8.0/ 0.40	6.96250
ENE	/	/	3/ 0.15	1/ 0.05	/	/	/	4.0/ 0.20	5.98125
E	/	/	/	/	/	/	/	/	
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.82500
SE	/	1/ 0.05	2/ 0.10	/	/	/	/	3.0/ 0.15	4.82500
SSE	/	1/ 0.05	2/ 0.10	/	/	/	/	3.0/ 0.15	4.00833
S	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	4.81250
SSW	/	1/ 0.05	2/ 0.10	4/ 0.20	/	/	/	7.0/ 0.35	7.07143
SW	/	/	1/ 0.05	8/ 0.40	/	/	/	9.0/ 0.45	9.12778
WSW	/	/	3/ 0.15	1/ 0.05	/	/	/	4.0/ 0.20	5.93125
W	/	/	2/ 0.10	/	1/ 0.05	/	/	3.0/ 0.15	8.24167
WNW	/	/	2/ 0.10	8/ 0.40	3/ 0.15	/	/	13.0/ 0.65	10.66154
NW	/	/	5/ 0.25	10/ 0.50	2/ 0.10	/	/	17.0/ 0.85	8.72059
NNW	/	/	5/ 0.25	3/ 0.15	2/ 0.10	/	/	10.0/ 0.50	8.34750
TOTAL	/	3/ 0.15	56/ 2.79	43/ 2.15	9/ 0.45	/	/	111.0/ 5.54	7.72042

NUMBER OF BAD RECORDS: 21

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=B

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	3/ 0.15	2/ 0.10	/	/	/	5.0/ 0.25	7.07000
NNE	/	/	3/ 0.15	1/ 0.05	/	/	/	4.0/ 0.20	5.83750
NE	/	/	/	2/ 0.10	/	/	/	2.0/ 0.10	7.88750
ENE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.35000
E	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	4.81250
ESE	/	/	5/ 0.25	/	/	/	/	5.0/ 0.25	5.56000
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	7.92500
S	/	1/ 0.05	3/ 0.15	2/ 0.10	/	/	/	6.0/ 0.30	6.60417
SSW	/	/	2/ 0.10	1/ 0.05	/	/	/	3.0/ 0.15	6.62500
SW	/	/	4/ 0.20	1/ 0.05	/	/	/	5.0/ 0.25	5.38500
WSW	/	/	3/ 0.15	2/ 0.10	/	/	/	5.0/ 0.25	8.14000
W	/	/	1/ 0.05	/	1/ 0.05	/	/	2.0/ 0.10	8.57500
WNW	/	/	2/ 0.10	4/ 0.20	1/ 0.05	/	/	7.0/ 0.35	9.85000
NW	/	1/ 0.05	1/ 0.05	3/ 0.15	5/ 0.25	/	/	10.0/ 0.50	11.58500
NNW	/	/	2/ 0.10	3/ 0.15	/	/	/	5.0/ 0.25	8.00500
TOTAL	/	2/ 0.10	32/ 1.60	22/ 1.10	7/ 0.35	/	/	63.0/ 3.14	7.86151

NUMBER OF BAD RECORDS: 4

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=C

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	/	/	7/ 0.35	/	/	/	/	7.0/ 0.35	5.87857
NNE	/	1/ 0.05	3/ 0.15	/	/	/	/	4.0/ 0.20	5.06875
NE	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	6.50000
ENE	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	3.77500
E	/	/	/	/	/	/	/	/	
ESE	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	4.68750
SE	/	/	/	/	/	/	/	/	
SSE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	7.42500
S	/	1/ 0.05	4/ 0.20	/	/	/	/	5.0/ 0.25	5.34000
SSW	/	/	3/ 0.15	4/ 0.20	/	/	/	7.0/ 0.35	7.63929
SW	/	1/ 0.05	2/ 0.10	2/ 0.10	/	/	/	5.0/ 0.25	6.60667
WSW	/	/	3/ 0.15	6/ 0.30	/	/	/	9.0/ 0.45	8.87778
W	/	/	1/ 0.05	3/ 0.15	1/ 0.05	/	/	5.0/ 0.25	8.68000
WNW	/	1/ 0.05	2/ 0.10	2/ 0.10	4/ 0.20	/	/	9.0/ 0.45	10.63333
NW	/	1/ 0.05	5/ 0.25	3/ 0.15	/	/	/	9.0/ 0.45	6.97500
NNW	/	/	4/ 0.20	3/ 0.15	2/ 0.10	/	/	9.0/ 0.45	8.14722
TOTAL	/	5/ 0.25	41/ 2.05	24/ 1.20	7/ 0.35	/	/	77.0/ 3.84	7.46115

NUMBER OF BAD RECORDS: 8

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=D

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.2/ 0.01	15/ 0.75	47/ 2.35	4/ 0.20	/	/	/	66.2/ 3.30	4.90634
NNE	0.2/ 0.01	10/ 0.50	43/ 2.15	3/ 0.15	/	/	/	56.2/ 2.80	4.84920
NE	0.1/ 0.00	7/ 0.35	15/ 0.75	1/ 0.05	/	/	/	23.1/ 1.15	4.34578
ENE	0.2/ 0.01	11/ 0.55	11/ 0.55	/	/	/	/	22.2/ 1.11	3.75788
E	0.2/ 0.01	10/ 0.50	10/ 0.50	/	/	/	/	20.2/ 1.01	3.55198
ESE	0.1/ 0.00	5/ 0.25	11/ 0.55	/	/	/	/	16.1/ 0.80	4.15062
SE	0.0/ 0.00	2/ 0.10	5/ 0.25	1/ 0.05	/	/	/	8.0/ 0.40	5.13125
SSE	0.0/ 0.00	2/ 0.10	2/ 0.10	1/ 0.05	/	/	/	5.0/ 0.25	4.57000
S	0.1/ 0.00	6/ 0.30	9/ 0.45	3/ 0.15	1/ 0.05	/	/	19.1/ 0.95	5.23080
SSW	0.2/ 0.01	11/ 0.55	24/ 1.20	30/ 1.50	14/ 0.70	/	/	79.2/ 3.95	8.25663
SW	0.2/ 0.01	15/ 0.75	19/ 0.95	20/ 1.00	2/ 0.10	/	/	56.2/ 2.80	6.11610
WSW	0.1/ 0.00	5/ 0.25	31/ 1.55	26/ 1.30	2/ 0.10	/	/	64.1/ 3.20	7.30538
W	0.1/ 0.00	5/ 0.25	16/ 0.80	9/ 0.45	1/ 0.05	/	/	31.1/ 1.55	6.53617
WNW	0.1/ 0.00	7/ 0.35	11/ 0.55	19/ 0.95	9/ 0.45	/	/	46.1/ 2.30	8.35683
NW	0.1/ 0.00	7/ 0.35	25/ 1.25	18/ 0.90	6/ 0.30	/	/	56.1/ 2.80	7.26812
NNW	0.2/ 0.01	13/ 0.65	33/ 1.65	15/ 0.75	3/ 0.15	/	/	64.2/ 3.20	6.17796
TOTAL	2.0/ 0.10	131/ 6.54	312/15.57	150/ 7.49	38/ 1.90	/	/	633.0/31.59	6.22802

NUMBER OF BAD RECORDS: 33

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=E

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.1/ 0.00	10/ 0.50	16/ 0.80	1/ 0.05	/	/	/	27.1/ 1.35	4.09271
NNE	0.3/ 0.01	25/ 1.25	24/ 1.20	/	/	/	/	49.3/ 2.46	3.55933
NE	0.2/ 0.01	19/ 0.95	13/ 0.65	/	/	/	/	32.2/ 1.61	3.37189
ENE	0.1/ 0.00	12/ 0.60	7/ 0.35	/	/	/	/	19.1/ 0.95	3.16099
E	0.1/ 0.00	10/ 0.50	3/ 0.15	/	/	/	/	13.1/ 0.65	2.70420
ESE	0.1/ 0.00	6/ 0.30	4/ 0.20	1/ 0.05	/	/	/	11.1/ 0.55	3.85698
SE	0.1/ 0.00	9/ 0.45	4/ 0.20	1/ 0.05	/	/	/	14.1/ 0.70	3.04344
SSE	0.1/ 0.00	7/ 0.35	5/ 0.25	/	/	/	/	12.1/ 0.60	3.30372
S	0.1/ 0.00	9/ 0.45	20/ 1.00	4/ 0.20	/	/	/	33.1/ 1.65	4.57666
SSW	0.2/ 0.01	16/ 0.80	26/ 1.30	10/ 0.50	/	/	/	52.2/ 2.60	5.04981
SW	0.2/ 0.01	17/ 0.85	27/ 1.35	15/ 0.75	1/ 0.05	/	/	60.2/ 3.00	5.75955
WSW	0.1/ 0.00	6/ 0.30	16/ 0.80	9/ 0.45	/	/	/	31.1/ 1.55	6.02050
W	0.1/ 0.00	9/ 0.45	17/ 0.85	2/ 0.10	/	/	/	28.1/ 1.40	4.39012
WNW	0.1/ 0.00	6/ 0.30	23/ 1.15	9/ 0.45	/	/	/	38.1/ 1.90	5.80413
NW	0.1/ 0.00	10/ 0.50	26/ 1.30	5/ 0.25	/	/	/	41.1/ 2.05	4.87865
NNW	0.1/ 0.00	12/ 0.60	18/ 0.90	3/ 0.15	/	/	/	33.1/ 1.65	4.08950
TOTAL	2.0/ 0.10	183/ 9.13	249/ 12.43	60/ 2.99	1/ 0.05	/	/	495.0/ 24.70	4.53715

NUMBER OF BAD RECORDS: 41

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=F

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	0.2/ 0.01	14/ 0.70	13/ 0.65	/	/	/	/	27.2/ 1.36	3.30974
NNE	0.2/ 0.01	18/ 0.90	3/ 0.15	/	/	/	/	21.2/ 1.06	2.81604
NE	0.2/ 0.01	15/ 0.75	2/ 0.10	/	/	/	/	17.2/ 0.86	2.26308
ENE	0.2/ 0.01	13/ 0.65	1/ 0.05	/	/	/	/	14.2/ 0.71	2.59859
E	0.0/ 0.00	4/ 0.20	/	/	/	/	/	4.0/ 0.20	2.13125
ESE	0.1/ 0.00	8/ 0.40	/	/	/	/	/	8.1/ 0.40	1.73611
SE	0.1/ 0.00	7/ 0.35	/	/	/	/	/	7.1/ 0.35	1.82218
SSE	0.1/ 0.00	5/ 0.25	/	/	/	/	/	5.1/ 0.25	2.37500
S	0.1/ 0.00	10/ 0.50	2/ 0.10	/	/	/	/	12.1/ 0.60	2.87810
SSW	0.2/ 0.01	19/ 0.95	9/ 0.45	/	/	/	/	28.2/ 1.41	3.05940
SW	0.2/ 0.01	16/ 0.80	8/ 0.40	/	/	/	/	24.2/ 1.21	2.93078
WSW	0.1/ 0.00	10/ 0.50	4/ 0.20	/	/	/	/	14.1/ 0.70	3.05142
W	0.1/ 0.00	7/ 0.35	10/ 0.50	/	/	/	/	17.1/ 0.85	3.81140
WNW	0.0/ 0.00	3/ 0.15	7/ 0.35	/	/	/	/	10.0/ 0.50	4.04250
NW	0.0/ 0.00	1/ 0.05	3/ 0.15	/	/	/	/	4.0/ 0.20	3.71875
NNW	0.2/ 0.01	16/ 0.80	7/ 0.35	/	/	/	/	23.2/ 1.16	2.76042
TOTAL	2.0/ 0.10	166/ 8.28	69/ 3.44	/	/	/	/	237.0/11.83	2.92301

NUMBER OF BAD RECORDS: 20

TABLE 2.3.3-16 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=1 STABILITY=G

LOWNDSPD

LOWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE LOWNDWSPD
N	3.7/ 0.18	28/ 1.40	3/ 0.15	/	/	/	/	34.7/ 1.73	1.87248
NNE	4.6/ 0.23	35/ 1.75	/	/	/	/	/	39.5/ 1.98	1.28914
NE	3.8/ 0.19	29/ 1.45	1/ 0.05	/	/	/	/	33.8/ 1.69	1.42012
ENE	4.0/ 0.20	30/ 1.50	/	/	/	/	/	34.0/ 1.70	1.33015
E	3.2/ 0.16	24/ 1.20	/	/	/	/	/	27.2/ 1.36	1.29779
ESE	3.0/ 0.15	23/ 1.15	/	/	/	/	/	26.0/ 1.30	1.24423
SE	3.0/ 0.15	23/ 1.15	/	/	/	/	/	26.0/ 1.30	1.42019
SSE	1.7/ 0.08	13/ 0.65	/	/	/	/	/	14.7/ 0.73	1.20153
S	2.5/ 0.12	19/ 0.95	1/ 0.05	/	/	/	/	22.5/ 1.12	1.44333
SSW	3.8/ 0.19	29/ 1.45	1/ 0.05	/	/	/	/	33.8/ 1.69	1.78698
SW	1.1/ 0.05	8/ 0.40	1/ 0.05	/	/	/	/	10.1/ 0.50	1.98267
WSW	1.4/ 0.07	11/ 0.55	/	/	/	/	/	12.4/ 0.62	1.52218
W	1.8/ 0.09	14/ 0.70	1/ 0.05	/	/	/	/	16.8/ 0.84	1.45982
WNW	1.8/ 0.09	14/ 0.70	2/ 0.10	/	/	/	/	17.8/ 0.89	1.90871
NW	1.4/ 0.07	11/ 0.55	/	/	/	/	/	12.4/ 0.62	1.64718
NNW	3.0/ 0.15	23/ 1.15	/	/	/	/	/	26.0/ 1.30	1.22404
TOTAL	44.0/ 2.20	334/16.67	10/ 0.50	/	/	/	/	388.0/19.36	1.47941

NUMBER OF BAD RECORDS: 48

TABLE 2.3.3-17

JOINT OCCURRENCE FREQUENCIES FOR
60 M WIND DIRECTION AND 60M WIND SPEED
RANGES INCLUDE LOWER END POINT, EXCLUDE UPPER END POINT
SITE=SHNP YEAR=76-78 MONTH=01 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.0/ 0.00	3/ 0.16	58/ 3.15	50/ 2.72	7/ 0.38	/	/	118.0/ 6.41	7.83630
NNE	0.1/ 0.01	14/ 0.76	36/ 1.96	43/ 2.34	5/ 0.27	1/ 0.05	/	99.1/ 5.39	7.62677
NE	0.1/ 0.01	9/ 0.49	23/ 1.25	15/ 0.82	2/ 0.11	/	/	49.1/ 2.67	6.28683
ENE	0.1/ 0.01	17/ 0.92	18/ 0.98	12/ 0.65	/	/	/	47.1/ 2.56	5.22744
E	0.1/ 0.01	9/ 0.49	18/ 0.98	18/ 0.98	/	/	/	45.1/ 2.45	6.40272
ESE	0.0/ 0.00	5/ 0.27	25/ 1.36	13/ 0.71	1/ 0.05	/	/	44.0/ 2.39	6.33352
SE	0.1/ 0.01	8/ 0.43	23/ 1.25	11/ 0.60	1/ 0.05	/	/	43.1/ 2.34	6.24652
SSE	0.1/ 0.01	7/ 0.38	30/ 1.63	28/ 1.52	12/ 0.65	3/ 0.16	7/ 0.38	87.1/ 4.73	10.29420
S	0.1/ 0.01	7/ 0.38	36/ 1.96	34/ 1.85	8/ 0.43	3/ 0.16	1/ 0.05	89.1/ 4.84	8.62869
SSW	0.1/ 0.01	7/ 0.38	45/ 2.45	45/ 2.45	22/ 1.20	11/ 0.60	/	130.1/ 7.07	9.93899
SW	0.1/ 0.01	8/ 0.43	35/ 1.90	51/ 2.77	29/ 1.58	13/ 0.71	8/ 0.43	144.1/ 7.83	11.80933
WSW	0.1/ 0.01	8/ 0.43	53/ 2.88	69/ 3.75	24/ 1.30	6/ 0.33	7/ 0.38	167.1/ 9.08	10.16532
W	0.1/ 0.01	8/ 0.43	41/ 2.23	113/ 6.14	31/ 1.68	8/ 0.43	/	201.1/10.93	9.80165
WNW	0.1/ 0.01	12/ 0.65	45/ 2.45	123/ 6.68	73/ 3.97	21/ 1.14	6/ 0.33	280.1/15.22	11.49580
NW	0.0/ 0.00	5/ 0.27	40/ 2.17	96/ 5.22	34/ 1.85	7/ 0.38	/	182.0/ 9.89	9.92486
NNW	0.1/ 0.01	8/ 0.43	29/ 1.58	69/ 3.75	8/ 0.43	/	/	114.1/ 6.20	8.51161
TOTAL	1.0/ 0.05	135/ 7.34	555/30.16	790/42.93	257/13.97	73/ 3.97	29/ 1.58	1840/ 100	9.45625

NUMBER OF BAD RECORDS: 65

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	2/ 0.11	6/ 0.33	/	/	/	9.0/ 0.49	8.26944
NNE	/	1/ 0.05	/	1/ 0.05	/	/	/	2.0/ 0.11	6.96250
NE	/	/	/	/	/	/	/	/	
ENE	/	/	/	/	/	/	/	/	
E	/	/	/	/	/	/	/	/	
ESE	/	/	/	/	/	/	/	/	
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	/	/	/	/	/	
S	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	/	3.0/ 0.16	6.55000
SSW	/	/	1/ 0.05	4/ 0.22	2/ 0.11	/	/	7.0/ 0.38	10.80000
SW	/	2/ 0.11	/	3/ 0.16	4/ 0.22	1/ 0.05	2/ 0.11	12.0/ 0.65	15.14791
WSW	/	/	3/ 0.16	4/ 0.22	5/ 0.27	4/ 0.22	4/ 0.22	20.0/ 1.09	17.29874
W	/	/	1/ 0.05	2/ 0.11	7/ 0.38	2/ 0.11	/	12.0/ 0.65	14.68541
WNW	/	1/ 0.05	1/ 0.05	10/ 0.54	18/ 0.98	5/ 0.27	2/ 0.11	37.0/ 2.01	14.88716
NW	/	/	3/ 0.16	10/ 0.54	5/ 0.27	/	/	18.0/ 0.98	10.60000
NNW	/	1/ 0.05	/	5/ 0.27	1/ 0.05	/	/	7.0/ 0.38	9.28571
TOTAL	/	7/ 0.38	12/ 0.65	46/ 2.50	42/ 2.28	12/ 0.65	8/ 0.43	127.0/ 6.90	13.34015

NUMBER OF BAD RECORDS: 7

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=B

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	7/ 0.38	/	/	/	/	7.0/ 0.38	6.41428
NNE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.11	8.41250
NE	/	/	/	/	/	/	/	/	
ENE	/	1/ 0.05	2/ 0.11	/	/	/	/	3.0/ 0.16	4.63333
E	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	3.45000
ESE	/	/	/	/	/	/	/	/	
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	/	/	/	/	/	
S	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.11	9.42500
SSW	/	/	2/ 0.11	2/ 0.11	/	/	/	4.0/ 0.22	7.38750
SW	/	/	3/ 0.16	2/ 0.11	2/ 0.11	1/ 0.05	/	8.0/ 0.43	10.80000
WSW	/	/	2/ 0.11	1/ 0.05	5/ 0.27	/	1/ 0.05	9.0/ 0.49	13.54444
W	/	/	/	4/ 0.22	2/ 0.11	2/ 0.11	/	8.0/ 0.43	13.87500
WNW	/	1/ 0.05	4/ 0.22	6/ 0.33	13/ 0.71	1/ 0.05	1/ 0.05	26.0/ 1.41	12.78461
NW	/	/	5/ 0.27	6/ 0.33	5/ 0.27	/	/	16.0/ 0.87	10.66094
NNW	/	1/ 0.05	/	3/ 0.16	1/ 0.05	/	/	5.0/ 0.27	9.58000
TOTAL	/	4/ 0.22	27/ 1.47	26/ 1.41	28/ 1.52	4/ 0.22	2/ 0.11	91.0/ 4.95	10.96318

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	4/ 0.22	1/ 0.05	/	/	/	5.0/ 0.27	7.08000
NNE	/	/	4/ 0.22	1/ 0.05	/	/	/	5.0/ 0.27	6.65000
NE	/	/	2/ 0.11	/	/	/	/	2.0/ 0.11	4.68750
ENE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.11	7.55000
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.82500
ESE	/	/	2/ 0.11	/	/	/	/	2.0/ 0.11	4.60000
SE	/	/	2/ 0.11	/	/	/	/	2.0/ 0.11	4.66250
SSE	/	/	2/ 0.11	/	/	/	/	2.0/ 0.11	5.25000
S	/	/	4/ 0.22	1/ 0.05	1/ 0.05	/	/	6.0/ 0.33	7.60833
SSW	/	2/ 0.11	6/ 0.33	/	1/ 0.05	/	/	9.0/ 0.49	5.60833
SW	/	/	/	2/ 0.11	2/ 0.11	/	2/ 0.11	6.0/ 0.33	19.86666
WSW	/	/	2/ 0.11	2/ 0.11	2/ 0.11	1/ 0.05	/	7.0/ 0.38	11.67143
W	/	/	1/ 0.05	5/ 0.27	6/ 0.33	/	/	12.0/ 0.65	11.76458
WNW	/	/	1/ 0.05	6/ 0.33	8/ 0.43	5/ 0.27	/	20.0/ 1.09	14.34125
NW	/	/	1/ 0.05	4/ 0.22	7/ 0.38	/	/	12.0/ 0.65	11.58750
NNW	/	/	5/ 0.27	4/ 0.22	/	/	/	9.0/ 0.49	7.80000
TOTAL	/	2/ 0.11	38/ 2.07	27/ 1.47	27/ 1.47	6/ 0.33	2/ 0.11	102.0/ 5.54	10.40441

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.0/ 0.00	1/ 0.05	27/ 1.47	18/ 0.98	1/ 0.05	/	/	47.0/ 2.55	7.43333
NNE	0.1/ 0.01	8/ 0.43	16/ 0.87	27/ 1.47	4/ 0.22	1/ 0.05	/	56.1/ 3.05	8.03795
NE	0.1/ 0.01	7/ 0.38	10/ 0.54	8/ 0.43	2/ 0.11	/	/	27.1/ 1.47	6.15867
ENE	0.1/ 0.01	8/ 0.43	9/ 0.49	5/ 0.27	/	/	/	22.1/ 1.20	4.76301
E	0.1/ 0.01	6/ 0.33	9/ 0.49	7/ 0.38	/	/	/	22.1/ 1.20	5.79581
ESE	0.1/ 0.01	5 /0.27	10/ 0.54	11/ 0.60	/	/	/	26.1/ 1.42	6.40469
SE	0.1/ 0.01	6/ 0.33	17/ 0.92	3/ 0.16	1/ 0.05	/	/	27.1/ 1.47	5.58118
SSE	0.1/ 0.01	4/ 0.22	7/ 0.38	5/ 0.27	11/ 0.60	3/ 0.16	5/ 0.27	35.1/ 1.91	13.11324
S	0.0/ 0.00	2/ 0.11	9/ 0.49	7/ 0.38	1/ 0.05	1/ 0.05	1/ 0.05	21.0/ 1.14	9.06071
SSW	0.0/ 0.00	2/ 0.11	13/ 0.71	8/ 0.43	2/ 0.11	6/ 0.33	/	31.0/ 1.68	10.55242
SW	0.0/ 0.00	1/ 0.05	14/ 0.76	13/ 0.71	8/ 0.43	9/ 0.49	4/ 0.22	49.0/ 2.66	13.54591
WSW	0.1/ 0.01	4/ 0.22	11/ 0.60	20/ 1.09	5/ 0.27	1/ 0.05	2/ 0.11	43.1/ 2.34	10.03538
W	0.1/ 0.01	4/ 0.22	11/ 0.60	22/ 1.20	8/ 0.43	4/ 0.22	/	49.1/ 2.67	9.51298
WNW	0.1/ 0.01	5/ 0.27	14/ 0.76	38/ 2.07	23/ 1.25	9/ 0.49	3/ 0.16	92.1/ 5.01	11.71960
NW	0.0/ 0.00	2/ 0.11	10/ 0.54	25/ 1.36	10/ 0.54	7/ 0.38	/	54.0/ 2.93	11.24027
NNW	0.0/ 0.00	2/ 0.11	7/ 0.38	12/ 0.65	5/ 0.27	/	/	26.0/ 1.41	9.07596
TOTAL	1.0/ 0.05	67/ 3.64	194/10.54	229/12.45	81/4.40	41/ 2.23	15/ 0.82	626.0/34.13	9.52599

NUMBER OF BAD RECORDS: 29

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	8/ 0.43	17/ 0.92	6/ 0.33	/	/	32.0/ 1.74	9.13099
NNE	/	2/ 0.11	2/ 0.11	8/ 0.43	/	/	/	12.0/ 0.65	8.03958
NE	/	1/ 0.05	3/ 0.16	5/ 0.27	/	/	/	9.0/ 0.49	7.55926
ENE	/	3/ 0.16	3/ 0.16	5/ 0.27	/	/	/	11.0/ 0.60	6.44545
E	/	/	4/ 0.22	8/ 0.43	/	/	/	12.0/ 0.65	7.89375
ESE	/	/	8/ 0.43	2/ 0.11	/	/	/	10.0/ 0.54	6.26250
SE	/	/	3/ 0.16	7/ 0.38	/	/	/	10.0/ 0.54	8.91750
SSE	/	/	7/ 0.38	14/ 0.76	1/ 0.05	/	2/ 0.11	24.0/ 1.30	10.47187
S	/	1/ 0.05	3/ 0.16	11/ 0.60	3/ 0.16	2/ 0.11	/	20.0/ 1.09	10.65000
SSW	/	2/ 0.11	5/ 0.27	8/ 0.43	16/ 0.87	5/ 0.27	/	36.0/ 1.96	12.64861
SW	/	2/ 0.11	5/ 0.27	10/ 0.54	13/ 0.71	2/ 0.11	/	32.0/ 1.74	11.68515
WSW	/	/	3/ 0.16	7/ 0.38	2/ 0.11	/	/	12.0/ 0.65	9.86458
W	/	/	5/ 0.27	27/ 1.47	4/ 0.22	/	/	36.0/ 1.96	10.26458
WNW	/	1/ 0.05	10/ 0.54	40/ 2.17	9/ 0.49	1/ 0.05	/	61.0/ 3.32	10.13401
NW	/	/	8/ 0.43	41/ 2.23	7/ 0.38	/	/	56.0/ 3.04	9.39955
NNW	/	/	7/ 0.38	32/ 1.74	1/ 0.05	/	/	40.0/ 2.17	9.11250
TOTAL	/	13/ 0.71	84/ 4.57	242/13.15	62/ 3.37	10/ 0.54	2/ 0.11	413.0/22.45	9.84183

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	4/ 0.22	4/ 0.22	/	/	/	8.0/ 0.43	7.50000
NNE	/	/	2/ 0.11	3/ 0.16	1/ 0.05	/	/	6.0/ 0.33	9.42083
NE	/	/	/	/	/	/	/	/	
ENE	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	/	3.0/ 0.16	6.18333
E	/	1/ 0.05	4/ 0.22	3/ 0.16	/	/	/	8.0/ 0.43	7.07500
ESE	/	/	2/ 0.11	/	1/ 0.05	/	/	3.0/ 0.16	8.70000
SE	/	1/ 0.05	/	1/ 0.05	/	/	/	2.0/ 0.11	6.18750
SSE	/	/	7/ 0.38	1/ 0.05	/	/	/	8.0/ 0.43	5.97187
S	/	1/ 0.05	10/ 0.54	7/ 0.38	1/ 0.05	/	/	19.0/ 1.03	7.56579
SSW	/	/	4/ 0.22	7/ 0.38	1/ 0.05	/	/	12.0/ 0.65	8.63750
SW	/	2/ 0.11	1/ 0.05	9/ 0.49	/	/	/	12.0/ 0.65	8.07292
WSW	/	3/ 0.16	2/ 0.11	12/ 0.65	2/ 0.11	/	/	19.0/ 1.03	8.80131
W	/	/	9/ 0.49	35/ 1.90	4/ 0.22	/	/	48.0/ 2.61	9.74792
WNW	/	1/ 0.05	5/ 0.27	12/ 0.65	1/ 0.05	/	/	19.0/ 1.03	9.06316
NW	/	/	3/ 0.16	5/ 0.27	/	/	/	8.0/ 0.43	7.43750
NNW	/	/	4/ 0.22	6/ 0.33	/	/	/	10.0/ 0.54	7.94500
TOTAL	/	10/ 0.54	58/ 3.15	106/ 5.76	11/ 0.60	/	/	185.0/10.05	8.47824

NUMBER OF BAD RECORDS: 6

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=01 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	6/ 0.33	4/ 0.22	/	/	/	10.0/ 0.54	6.84000
NNE	/	3/ 0.16	11/ 0.60	2/ 0.11	/	/	/	16.0/ 0.87	5.49271
NE	/	1/ 0.05	8/ 0.43	2/ 0.11	/	/	/	11.0/ 0.60	5.85227
ENE	/	4/ 0.22	2/ 0.11	/	/	/	/	6.0/ 0.33	3.75000
E	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	1.07500
ESE	/	/	3/ 0.16	/	/	/	/	3.0/ 0.16	4.56667
SE	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.11	3.55000
SSE	/	3/ 0.16	7/ 0.38	8/ 0.43	/	/	/	18.0/ 0.98	7.04167
S	/	2/ 0.11	8/ 0.43	6/ 0.33	2/ 0.11	/	/	18.0/ 0.98	7.63426
SSW	/	1/ 0.05	14/ 0.76	16/ 0.87	/	/	/	31.0/ 1.68	8.10081
SW	/	1/ 0.05	12/ 0.65	12/ 0.65	/	/	/	25.0/ 1.36	7.18300
WSW	/	1/ 0.05	30/ 1.63	23/ 1.25	3/ 0.16	/	/	57.0/ 3.10	7.56009
W	/	4/ 0.22	14/ 0.76	18/ 0.98	/	/	/	36.0/ 1.96	6.61667
WNW	/	3/ 0.16	10/ 0.54	11/ 0.60	1/ 0.05	/	/	25.0/ 1.36	7.20700
NW	/	3/ 0.16	10/ 0.54	5/ 0.27	/	/	/	18.0/ 0.98	6.28055
NNW	/	4/ 0.22	6/ 0.33	7/ 0.38	/	/	/	17.0/ 0.92	6.35294
TOTAL	/	32/ 1.74	142/ 7.72	114/ 6.20	6/ 0.33	/	/	294.0/ 15.98	6.90561

NUMBER OF BAD RECORDS: 11

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.0/ 0.00	2/ 0.11	43/ 2.27	96/ 5.07	38/ 2.01	1/ 0.05	/	180.0/ 9.50	9.92528
NNE	0.0/ 0.00	4/ 0.21	46/ 2.43	60/ 3.17	10/ 0.53	1/ 0.05	/	121.0/ 6.39	8.57651
NE	0.0/ 0.00	1/ 0.05	24/ 1.27	24/ 1.27	/	/	/	49.0/ 2.59	7.32653
ENE	0.0/ 0.00	2/ 0.11	15/ 0.79	13/ 0.69	1/ 0.05	/	/	31.0/ 1.64	7.44193
E	0.1/ 0.01	5/ 0.26	21/ 1.11	19/ 1.00	1/ 0.05	/	/	46.1/ 2.43	7.14100
ESE	0.1/ 0.01	10/ 0.53	25/ 1.32	19/ 1.00	/	/	/	54.1/ 2.85	6.48683
SE	0.0/ 0.00	3/ 0.16	15/ 0.79	30/ 1.58	2/ 0.11	1/ 0.05	/	51.0/ 2.69	8.32402
SSE	0.1/ 0.01	5/ 0.26	20/ 1.06	16/ 0.84	15/ 0.79	/	/	56.1/ 2.96	8.48128
S	0.1/ 0.01	7/ 0.37	14/ 0.74	45/ 2.37	33/ 1.74	/	1/ 0.05	100.1/ 5.28	10.63973
SSW	0.1/ 0.01	7/ 0.37	24/ 1.27	111/ 5.86	89/ 4.70	17/ 0.90	1/ 0.05	249.1/13.15	11.96186
SW	0.1/ 0.01	8/ 0.42	48/ 2.53	96/ 5.07	75/ 3.96	13/ 0.69	1/ 0.05	241.1/12.72	11.29210
WSW	0.1/ 0.01	13/ 0.69	28/ 1.48	99/ 5.22	50/ 2.64	10/ 0.53	1/ 0.05	201.1/10.61	10.76570
W	0.1/ 0.01	5/ 0.26	25/ 1.32	38/ 2.01	9/ 0.47	/	/	77.1/ 4.07	8.40823
WNW	0.1/ 0.01	5/ 0.26	34/ 1.79	56/ 2.96	30/ 1.58	9/ 0.47	/	134.1/ 7.08	10.34790
NW	0.1/ 0.01	6/ 0.32	41/ 2.16	61/ 3.22	34/ 1.79	8/ 0.42	/	150.1/ 7.92	10.01385
NNW	0.1/ 0.01	5/ 0.26	48/ 2.53	74/ 3.91	26/ 1.37	1/ 0.05	/	154.1/ 6.13	9.24554
TOTAL	1.0/ 0.05	88/ 4.64	471/24.85	857/45.22	413/21.79	61/ 3.22	4/ 0.21	1895/ 100	9.96858

NUMBER OF BAD RECORDS: 145

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	4/ 0.21	10/ 0.53	1/ 0.05	/	/	15.0/ 0.79	9.46333
NNE	/	/	3/ 0.16	6/ 0.32	2/ 0.11	/	/	11.0/ 0.58	9.60000
NE	/	/	/	2/ 0.11	/	/	/	2.0/ 0.11	12.12500
ENE	/	/	/	/	/	/	/	/	
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.40000
ESE	/	/	/	/	/	/	/	/	
SE	/	/	/	/	/	/	/	/	
SSE	/	/	2/ 0.11	/	1/ 0.05	/	/	3.0/ 0.16	10.63333
S	/	1/ 0.05	1/ 0.05	1/ 0.05	1/ 0.05	/	/	4.0/ 0.21	8.57500
SSW	/	/	/	6/ 0.32	11/ 0.58	4/ 0.21	/	21.0/ 1.11	15.08452
SW	/	/	5/ 0.26	12/ 0.63	15/ 0.79	6/ 0.32	1/ 0.05	39.0/ 2.06	13.76538
WSW	/	/	1/ 0.05	16/ 0.84	7/ 0.37	3/ 0.16	/	27.0/ 1.42	12.70833
W	/	/	/	4/ 0.21	4/ 0.21	/	/	8.0/ 0.42	12.11562
WNW	/	/	1/ 0.05	5/ 0.26	5/ 0.26	/	/	11.0/ 0.58	11.30833
NW	/	1/ 0.05	3/ 0.16	5/ 0.26	3/ 0.16	1/ 0.05	/	13.0/ 0.69	10.43269
NNW	/	/	1/ 0.05	9/ 0.47	9/ 0.47	/	/	19.0/ 1.00	12.00263
TOTAL	/	2/ 0.11	22/ 1.16	76/ 4.01	59/ 3.11	14/ 0.74	1/ 0.05	174.0/ 9.18	12.21920

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	4/ 0.21	4/ 0.21	/	/	/	9.0/ 0.47	7.73055
NNE	/	/	2/ 0.11	4/ 0.21	1/ 0.05	/	/	7.0/ 0.37	9.85000
NE	/	/	/	2/ 0.11	/	/	/	2.0/ 0.11	7.95000
ENE	/	/	/	/	/	/	/	/	
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.17500
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.07500
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.52500
SSE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.11	7.56250
S	/	/	1/ 0.05	/	3/ 0.16	/	/	4.0/ 0.21	14.57499
SSW	/	/	/	2/ 0.11	4/ 0.21	2/ 0.11	/	8.0/ 0.42	14.73749
SW	/	/	/	5/ 0.26	7/ 0.37	1/ 0.05	/	13.0/ 0.69	14.31730
WSW	/	1/ 0.05	1/ 0.05	4/ 0.21	1/ 0.05	/	/	7.0/ 0.37	8.40000
W	/	/	2/ 0.11	/	2/ 0.11	/	/	4.0/ 0.21	10.43749
WNW	/	/	/	6/ 0.32	2/ 0.11	3/ 0.16	/	11.0/ 0.58	14.29318
NW	/	/	3/ 0.16	5/ 0.26	1/ 0.05	1/ 0.05	/	10.0/ 0.53	10.38500
NNW	/	/	3/ 0.16	7/ 0.37	1/ 0.05	/	/	11.0/ 0.58	10.07045
TOTAL	/	2/ 0.11	20/ 1.06	40/ 2.11	22/ 1.16	7/ 0.37	/	91.0/ 4.80	11.19835

NUMBER OF BAD RECORDS: 4

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	5/ 0.26	2/ 0.11	/	/	7.0/ 0.37	10.97857
NNE	/	/	1/ 0.05	2/ 0.11	1/ 0.05	/	/	4.0/ 0.21	9.26250
NE	/	/	3/ 0.16	1/ 0.05	/	/	/	4.0/ 0.21	5.76875
ENE	/	/	/	2/ 0.11	/	/	/	2.0/ 0.11	9.58750
E	/	/	/	/	/	/	/	/	
ESE	/	/	2/ 0.11	/	/	/	/	2.0/ 0.11	4.53750
SE	/	/	3/ 0.16	/	/	/	/	3.0/ 0.16	4.36667
SSE	/	/	/	/	/	/	/	/	
S	/	1/ 0.05	/	/	1/ 0.05	/	/	2.0/ 0.11	9.94999
SSW	/	/	/	3/ 0.16	5/ 0.26	/	1/ 0.05	9.0/ 0.47	14.54722
SW	/	/	2/ 0.11	2/ 0.11	4/ 0.21	2/ 0.11	/	10.0/ 0.53	13.26500
WSW	/	1/ 0.05	/	3/ 0.16	5/ 0.26	2/ 0.11	/	11.0/ 0.58	13.74091
W	/	/	/	/	/	/	/	/	
WNW	/	/	2/ 0.11	8/ 0.42	3/ 0.16	3/ 0.16	/	16.0/ 0.84	12.63437
NW	/	/	2/ 0.11	8/ 0.42	2/ 0.11	2/ 0.11	/	14.0/ 0.74	11.49583
NNW	/	1/ 0.05	4/ 0.21	4/ 0.21	3/ 0.16	/	/	12.0/ 0.63	8.82500
TOTAL	/	3/ 0.16	19/ 1.00	38/ 2.01	26/ 1.37	9/ 0.47	1/ 0.05	96.0/ 5.07	11.27022

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	15/ 0.79	26/ 1.37	20/ 1.06	1/ 0.05	/	62.0/ 3.27	10.45322
NNE	/	1/ 0.05	17/ 0.90	15/ 0.79	3/ 0.16	1/ 0.05	/	37.0/ 1.95	8.46441
NE	/	1/ 0.05	7/ 0.37	2/ 0.11	/	/	/	10.0/ 0.53	5.60250
ENE	/	1/ 0.05	3/ 0.16	1/ 0.05	/	/	/	5.0/ 0.26	5.88500
E	/	2/ 0.11	7/ 0.37	1/ 0.05	/	/	/	10.0/ 0.53	5.95500
ESE	/	3/ 0.16	9/ 0.47	3/ 0.16	/	/	/	15.0/ 0.79	5.66000
SE	/	2/ 0.11	3/ 0.16	2/ 0.11	2/ 0.11	1/ 0.05	/	10.0/ 0.53	10.30500
SSE	/	/	5/ 0.26	4/ 0.21	6/ 0.32	/	/	15.0/ 0.79	10.03000
S	/	2/ 0.11	2/ 0.11	8/ 0.42	7/ 0.37	/	1/ 0.05	20.0/ 1.06	11.51875
SSW	/	3/ 0.16	5/ 0.26	13/ 0.69	7/ 0.37	7/ 0.37	/	35.0/ 1.85	12.09428
SW	/	3/ 0.16	20/ 1.06	13/ 0.69	8/ 0.42	/	/	44.0/ 2.32	8.63295
WSW	/	3/ 0.16	4/ 0.21	12/ 0.63	8/ 0.42	2/ 0.11	1/ 0.05	30.0/ 1.58	11.91250
W	/	/	4/ 0.21	5/ 0.26	2/ 0.11	/	/	11.0/ 0.58	9.16363
WNW	/	/	5/ 0.26	11/ 0.58	14/ 0.74	3/ 0.16	/	33.0/ 1.74	12.10076
NW	/	/	7/ 0.37	8/ 0.42	18/ 0.95	4/ 0.21	/	37.0/ 1.95	13.08918
NNW	/	3/ 0.16	18/ 0.95	16/ 0.84	4/ 0.21	/	/	41.0/ 2.16	7.99634
TOTAL	/	24/ 1.27	131/ 6.91	140/ 7.39	99/ 5.22	19/ 1.00	2/ 0.11	415.0/21.90	9.99484

NUMBER OF BAD RECORDS: 66

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	4/ 0.21	28/ 1.48	9/ 0.47	/	/	41.0/ 2.16	10.81158
NNE	/	1/ 0.05	15/ 0.79	27/ 1.42	/	/	/	43.0/ 2.27	8.42674
NE	/	/	11/ 0.58	16/ 0.84	/	/	/	27.0/ 1.42	7.93981
ENE	/	1/ 0.05	6/ 0.32	6/ 0.32	/	/	/	13.0/ 0.69	6.82115
E	/	1/ 0.05	9/ 0.47	3/ 0.16	1/ 0.05	/	/	14.0/ 0.74	6.91964
ESE	/	1/ 0.05	9/ 0.47	6/ 0.32	/	/	/	16.0/ 0.84	7.11719
SE	/	/	6/ 0.32	9/ 0.47	/	/	/	15.0/ 0.79	7.68500
SSE	/	/	3/ 0.16	4/ 0.21	7/ 0.37	/	/	14.0/ 0.74	10.55178
S	/	2/ 0.11	1/ 0.05	7/ 0.37	13/ 0.69	/	/	23.0/ 1.21	11.63696
SSW	/	3/ 0.16	4/ 0.21	26/ 1.37	39/ 2.06	4/ 0.21	/	76.0/ 4.01	12.21612
SW	/	/	8/ 0.42	19/ 1.00	35/ 1.85	4/ 0.21	/	66.0/ 3.48	13.01060
WSW	/	2/ 0.11	5/ 0.26	16/ 0.84	14/ 0.74	3/ 0.16	/	40.0/ 2.11	11.65021
W	/	1/ 0.05	5/ 0.26	8/ 0.42	/	/	/	14.0/ 0.74	7.86250
WNW	/	1/ 0.05	6/ 0.32	9/ 0.47	2/ 0.11	/	/	18.0/ 0.95	8.66944
NW	/	1/ 0.05	9/ 0.47	15/ 0.79	7/ 0.37	/	/	32.0/ 1.69	9.11719
NNW	/	1/ 0.05	8/ 0.42	18/ 0.95	6/ 0.32	1/ 0.05	/	34.0/ 1.79	9.95441
TOTAL	/	15/ 0.79	109/ 5.75	217/11.45	133/ 7.02	12/ 0.63	/	486.0/25.65	10.28710

NUMBER OF BAD RECORDS: 25

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	7/ 0.37	11/ 0.58	6/ 0.32	/	/	24.0/ 1.27	10.03958
NNE	/	1/ 0.05	2/ 0.11	5/ 0.26	3/ 0.16	/	/	11.0/ 0.58	9.60909
NE	/	/	/	/	/	/	/	/	
ENE	/	/	/	4/ 0.21	1/ 0.05	/	/	5.0/ 0.26	11.32000
E	/	/	2/ 0.11	7/ 0.37	/	/	/	9.0/ 0.47	8.36944
ESE	/	1/ 0.05	1/ 0.05	4/ 0.21	/	/	/	6.0/ 0.32	8.13750
SE	/	/	1/ 0.05	13/ 0.69	/	/	/	14.0/ 0.74	8.78214
SSE	/	2/ 0.11	1/ 0.05	4/ 0.21	1/ 0.05	/	/	8.0/ 0.42	7.83125
S	/	/	/	4/ 0.21	2/ 0.11	/	/	6.0/ 0.32	11.14167
SSW	/	/	/	30/ 1.58	14/ 0.74	/	/	44.0/ 2.32	11.53182
SW	/	/	3/ 0.16	16/ 0.84	4/ 0.21	/	/	23.0/ 1.21	10.06087
WSW	/	/	2/ 0.11	12/ 0.63	7/ 0.37	/	/	21.0/ 1.11	11.31309
W	/	/	2/ 0.11	6/ 0.32	/	/	/	8.0/ 0.42	8.66250
WNW	/	/	8/ 0.42	12/ 0.63	1/ 0.05	/	/	21.0/ 1.11	8.75357
NW	/	1/ 0.05	5/ 0.26	9/ 0.47	3/ 0.16	/	/	18.0/ 0.95	8.83889
NNW	/	/	3/ 0.16	13/ 0.69	3/ 0.16	/	/	19.0/ 1.00	9.86184
TOTAL	/	5/ 0.26	37/ 1.95	150/ 7.92	45/ 2.37	/	/	237.0/12.51	9.94019

NUMBER OF BAD RECORDS: 9

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=02 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.0/ 0.00	1/ 0.05	9/ 0.47	12/ 0.63	/	/	/	22.0/ 1.16	7.53864
NNE	0.0/ 0.00	1/ 0.05	6/ 0.32	1/ 0.05	/	/	/	8.0/ 0.42	5.61562
NE	/	/	3/ 0.16	1/ 0.05	/	/	/	4.0/ 0.21	6.34375
ENE	/	/	6/ 0.32	/	/	/	/	6.0/ 0.32	6.13750
E	0.1/ 0.01	2/ 0.11	1/ 0.05	8/ 0.42	/	/	/	11.1/ 0.59	7.82320
ESE	0.1/ 0.01	5/ 0.26	3/ 0.16	6/ 0.32	/	/	/	14.1/ 0.74	6.32535
SE	0.0/ 0.00	1/ 0.05	1/ 0.05	6/ 0.32	/	/	/	8.0/ 0.42	8.07812
SSE	0.1/ 0.01	3/ 0.16	8/ 0.42	3/ 0.16	/	/	/	14.1/ 0.74	4.81649
S	0.0/ 0.00	1/ 0.05	9/ 0.47	25/ 1.32	6/ 0.32	/	/	41.0/ 2.16	9.45061
SSW	0.0/ 0.00	1/ 0.05	15/ 0.79	31/ 1.64	9/ 0.47	/	/	56.0/ 2.96	9.90625
SW	0.1/ 0.01	5/ 0.26	10/ 0.53	29/ 1.53	2/ 0.11	/	/	46.1/ 2.43	8.60900
WSW	0.2/ 0.01	6/ 0.32	15/ 0.79	36/ 1.90	8/ 0.42	/	/	65.2/ 3.44	8.45130
W	0.1/ 0.01	4/ 0.21	12/ 0.63	15/ 0.79	1/ 0.05	/	/	32.1/ 1.69	7.14720
WNW	0.1/ 0.01	4/ 0.21	12/ 0.63	5/ 0.26	3/ 0.16	/	/	24.1/ 1.27	6.83195
NW	0.1/ 0.01	3/ 0.16	12/ 0.63	11/ 0.58	/	/	/	26.1/ 1.38	6.41523
NNW	/	/	11/ 0.58	7/ 0.37	/	/	/	18.0/ 0.95	7.00833
TOTAL	1.0/ 0.05	37/ 1.95	133/ 7.02	196/10.34	29/ 1.53	/	/	396.0/20.90	7.97809

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.0/ 0.00	2/ 0.09	42/ 1.94	90/ 4.15	28/ 1.29	2/ 0.09	/	164.0/ 7.56	9.51946
NNE	0.1/ 0.00	8/ 0.37	32/ 1.48	54/ 2.49	12/ 0.55	/	/	106.1/ 4.89	8.35792
NE	0.1/ 0.00	7/ 0.32	22/ 1.01	39/ 1.80	15/ 0.69	/	/	83.1/ 3.83	8.86181
ENE	0.1/ 0.00	6/ 0.28	29/ 1.34	39/ 1.80	17/ 0.78	3/ 0.14	/	94.1/ 4.34	9.25075
E	0.1/ 0.00	5/ 0.23	30/ 1.38	40/ 1.84	7/ 0.32	/	/	82.1/ 3.79	8.03806
ESE	0.1/ 0.00	7/ 0.32	31/ 1.43	47/ 2.17	5/ 0.23	/	/	90.1/ 4.15	7.77553
SE	0.1/ 0.00	7/ 0.32	22/ 1.01	49/ 2.26	9/ 0.41	1/ 0.05	/	88.1/ 4.06	9.10286
SSE	0.0/ 0.00	4/ 0.18	26/ 1.20	54/ 2.49	30/ 1.38	/	/	114.0/ 5.26	10.23289
S	0.1/ 0.00	6/ 0.28	31/ 1.43	74/ 3.41	45/ 2.07	7/ 0.32	1/ 0.05	164.1/ 7.57	10.68830
SSW	0.1/ 0.00	7/ 0.32	35/ 1.61	144/ 6.64	94/ 4.33	25/ 1.15	/	305.1/14.07	11.79101
SW	0.1/ 0.00	7/ 0.32	53/ 2.44	88/ 4.06	60/ 2.77	26/ 1.20	5/ 0.23	239.1/11.02	11.71525
WSW	0.1/ 0.00	6/ 0.28	44/ 2.03	59/ 2.72	20/ 0.92	7/ 0.32	7/ 0.32	143.1/ 6.60	10.41998
W	0.1/ 0.00	6/ 0.28	25/ 1.15	24/ 1.11	18/ 0.83	3/ 0.14	3/ 0.14	79.1/ 3.65	10.36694
WNW	0.1/ 0.00	5/ 0.23	24/ 1.11	51/ 2.35	33/ 1.52	10/ 0.46	3/ 0.14	126.1/ 5.81	11.54269
NW	0.0/ 0.00	2/ 0.09	32/ 1.48	72/ 3.32	30/ 1.38	2/ 0.09	2/ 0.09	140.0/ 6.45	10.31125
NNW	0.1/ 0.00	11/ 0.51	27/ 1.24	86/ 3.96	27/ 1.24	/	/	151.1/ 6.97	9.53938
TOTAL	1.0/ 0.05	96/ 4.43	505/23.28	1010/46.57	450/20.75	86/ 3.96	21/ 0.97	2169/ 100	10.22961

NUMBER OF BAD RECORDS: 63

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	4/ 0.18	8/ 0.37	2/ 0.09	/	/	14.0/ 0.65	9.37678
NNE	/	/	4/ 0.18	2/ 0.09	/	/	/	6.0/ 0.28	7.38333
NE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	7.16250
ENE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.83750
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.92500
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.00000
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.57500
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.42500
S	/	/	/	1/ 0.05	/	/	1/ 0.05	2.0/ 0.09	16.86250
SSW	/	1/ 0.05	2/ 0.09	10/ 0.46	7/ 0.32	2/ 0.09	/	22.0/ 1.01	11.77159
SW	/	/	9/ 0.41	8/ 0.37	19/ 0.88	8/ 0.37	1/ 0.05	45.0/ 2.07	14.00759
WSW	/	1/ 0.05	3/ 0.14	4/ 0.18	1/ 0.05	/	1/ 0.05	10.0/ 0.46	10.27000
W	/	/	1/ 0.05	/	/	/	1/ 0.05	2.0/ 0.09	18.21250
WNW	/	/	1/ 0.05	1/ 0.05	10/ 0.46	1/ 0.05	1/ 0.05	14.0/ 0.65	15.83035
NW	/	/	5/ 0.23	/	4/ 0.18	/	/	9.0/ 0.41	9.88611
NNW	/	/	3/ 0.14	1/ 0.05	2/ 0.09	/	/	6.0/ 0.28	8.89167
TOTAL	/	2/ 0.09	39/ 1.80	36/ 1.66	45/ 2.07	11/ 0.51	5/ 0.23	138.0/ 6.36	11.94649

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	1/ 0.05	7/ 0.32	/	/	/	8.0/ 0.37	8.61250
NNE	/	/	/	2/ 0.09	1/ 0.05	/	/	3.0/ 0.14	11.11667
NE	/	/	/	/	/	/	/	/	
ENE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.97500
E	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	6.10833
ESE	/	1/ 0.05	/	1/ 0.05	/	/	/	2.0/ 0.09	5.17500
SE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	7.97500
SSE	/	/	1/ 0.05	/	1/ 0.05	/	/	2.0/ 0.09	9.38750
S	/	/	/	2/ 0.09	/	/	/	2.0/ 0.09	9.63750
SSW	/	/	1/ 0.05	1/ 0.05	2/ 0.09	4/ 0.18	/	8.0/ 0.37	16.85937
SW	/	/	2/ 0.09	3/ 0.14	2/ 0.09	1/ 0.05	1/ 0.05	9.0/ 0.41	12.90555
WSW	/	/	1/ 0.05	1/ 0.05	1/ 0.05	/	2/ 0.09	5.0/ 0.23	18.43499
W	/	/	1/ 0.05	/	/	/	1/ 0.05	2.0/ 0.09	18.81249
WNW	/	1/ 0.05	2/ 0.09	3/ 0.14	3/ 0.14	3/ 0.14	1/ 0.05	13.0/ 0.60	14.23910
NW	/	/	3/ 0.14	2/ 0.09	5/ 0.23	/	/	10.0/ 0.46	11.17500
NNW	/	/	/	2/ 0.09	1/ 0.05	/	/	3.0/ 0.14	10.40000
TOTAL	/	2/ 0.09	16/ 0.74	26/ 1.20	16/ 0.74	8/ 0.37	5/ 0.23	73.0/ 3.37	12.31210

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	7/ 0.32	8/ 0.37	2/ 0.09	/	/	17.0/ 0.78	8.34363
NNE	/	/	4/ 0.18	/	/	/	/	4.0/ 0.18	6.30000
NE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.17500
ENE	/	/	/	1/ 0.05	2/ 0.09	/	/	3.0/ 0.14	12.23333
E	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	6.81250
ESE	/	/	/	2/ 0.09	/	/	/	2.0/ 0.09	9.27500
SE	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	7.29167
SSE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.67500
S	/	1/ 0.05	/	1/ 0.05	/	2/ 0.09	/	4.0/ 0.18	12.89374
SSW	/	/	2/ 0.09	1/ 0.05	2/ 0.09	5/ 0.23	/	10.0/ 0.46	15.70499
SW	/	/	1/ 0.05	2/ 0.09	3/ 0.14	4/ 0.18	/	10.0/ 0.46	15.47249
WSW	/	/	3/ 0.14	1/ 0.05	2/ 0.09	/	1/ 0.05	7.0/ 0.32	11.46071
W	/	/	2/ 0.09	3/ 0.14	2/ 0.09	/	/	7.0/ 0.32	10.58214
WNW	/	/	/	/	3/ 0.14	1/ 0.05	1/ 0.05	5.0/ 0.23	19.26499
NW	/	/	4/ 0.18	2/ 0.09	6/ 0.28	/	/	12.0/ 0.55	10.41250
NNW	/	/	/	3/ 0.14	2/ 0.09	/	/	5.0/ 0.23	11.70000
TOTAL	/	1/ 0.05	29/ 1.34	26/ 1.20	24/ 1.11	12/ 0.55	2/ 0.09	94.0/ 4.33	11.40150

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	2/ 0.09	15/ 0.69	32/ 1.48	12/ 0.55	2/ 0.09	/	63.0/ 2.90	9.83333
NNE	/	5/ 0.23	10/ 0.46	17/ 0.78	3/ 0.14	/	/	35.0/ 1.61	7.64643
NE	/	5/ 0.23	9/ 0.41	3/ 0.14	7/ 0.32	/	/	24.0/ 1.11	7.97292
ENE	/	4/ 0.18	11/ 0.51	15/ 0.69	5/ 0.23	/	/	35.0/ 1.61	8.31238
E	/	1/ 0.05	14/ 0.65	7/ 0.32	/	/	/	22.0/ 1.01	7.07841
ESE	/	4/ 0.18	17/ 0.78	15/ 0.69	/	/	/	36.0/ 1.66	6.70903
SE	/	4/ 0.18	11/ 0.51	23/ 1.06	6/ 0.28	1/ 0.05	/	45.0/ 2.07	9.29555
SSE	/	2/ 0.09	14/ 0.65	17/ 0.78	16/ 0.74	/	/	49.0/ 2.26	10.18265
S	/	1/ 0.05	7/ 0.32	8/ 0.37	6/ 0.28	/	/	22.0/ 1.01	9.85795
SSW	/	3/ 0.14	9/ 0.41	12/ 0.55	21/ 0.97	11/ 0.51	/	56.0/ 2.58	13.04955
SW	/	3/ 0.14	20/ 0.92	26/ 1.20	22/ 1.01	10/ 0.46	3/ 0.14	84.0/ 3.87	12.30029
WSW	/	1/ 0.05	12/ 0.55	18/ 0.83	5/ 0.23	5/ 0.23	3/ 0.14	44.0/ 2.03	11.59147
W	/	3/ 0.14	5/ 0.23	3/ 0.14	4/ 0.18	3/ 0.14	1/ 0.05	19.0/ 0.88	11.54736
WNW	/	1/ 0.05	8/ 0.37	13/ 0.60	9/ 0.41	5/ 0.23	/	36.0/ 1.66	11.69583
NW	/	/	6/ 0.28	16/ 0.74	7/ 0.32	1/ 0.05	1/ 0.05	31.0/ 1.43	11.44839
NNW	/	3/ 0.14	9/ 0.41	31/ 1.43	9/ 0.41	/	/	52.0/ 2.40	9.57884
TOTAL	/	42/ 1.94	177/ 8.16	256/11.80	132/ 6.09	38/ 1.75	8/ 0.37	653.0/30.11	10.21173

NUMBER OF BAD RECORDS: 38

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	6/ 0.28	19/ 0.88	5/ 0.23	/	/	30.0/ 1.38	9.75167
NNE	/	/	6/ 0.28	17/ 0.78	6/ 0.28	/	/	29.0/ 1.34	10.04483
NE	/	/	3/ 0.14	27/ 1.24	8/ 0.37	/	/	38.0/ 1.75	10.43662
ENE	/	/	3/ 0.14	10/ 0.46	7/ 0.32	3/ 0.14	/	23.0/ 1.06	12.46739
E	/	1/ 0.05	4/ 0.18	12/ 0.55	1/ 0.05	/	/	18.0/ 0.83	8.12361
ESE	/	1/ 0.05	5/ 0.23	17/ 0.78	1/ 0.05	/	/	24.0/ 1.11	8.85521
SE	/	/	4/ 0.18	14/ 0.65	1/ 0.05	/	/	19.0/ 0.88	9.60000
SSE	/	/	2/ 0.09	16/ 0.74	8/ 0.37	/	/	26.0/ 1.20	11.37019
S	/	1/ 0.05	11/ 0.51	38/ 1.75	26/ 1.20	5/ 0.23	/	81.0/ 3.73	11.45710
SSW	/	1/ 0.05	9/ 0.41	47/ 2.17	50/ 2.31	3/ 0.14	/	110.0/ 5.07	12.19932
SW	/	1/ 0.05	10/ 0.46	19/ 0.88	6/ 0.28	3/ 0.14	/	39.0/ 1.80	10.13654
WSW	/	/	6/ 0.28	8/ 0.37	7/ 0.32	2/ 0.09	/	23.0/ 1.06	10.67717
W	/	1/ 0.05	9/ 0.41	5/ 0.23	5/ 0.23	/	/	20.0/ 0.92	8.88250
WNW	/	1/ 0.05	2/ 0.09	17/ 0.78	5/ 0.23	/	/	25.0/ 1.15	10.25300
NW	/	/	4/ 0.18	36/ 1.66	7/ 0.32	1/ 0.05	1/ 0.05	49.0/ 2.26	11.13928
NNW	/	1/ 0.05	9/ 0.41	25/ 1.15	10/ 0.46	/	/	45.0/ 2.07	10.34611
TOTAL	/	8/ 0.37	93/ 4.29	327/15.08	153/ 7.05	17/ 0.78	1/ 0.05	599.0/27.62	10.78496

NUMBER OF BAD RECORDS: 12

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	2/ 0.09	7/ 0.32	5/ 0.23	/	/	14.0/ 0.65	10.84107
NNE	/	/	3/ 0.14	5/ 0.23	1/ 0.05	/	/	9.0/ 0.41	8.66944
NE	/	1/ 0.05	2/ 0.09	3/ 0.14	/	/	/	6.0/ 0.28	7.23750
ENE	/	/	2/ 0.09	6/ 0.28	2/ 0.09	/	/	10.0/ 0.46	9.19000
E	/	3/ 0.14	4/ 0.18	12/ 0.55	4/ 0.18	/	/	23.0/ 1.06	8.83587
ESE	/	/	4/ 0.18	9/ 0.41	3/ 0.14	/	/	16.0/ 0.74	9.13281
SE	/	1/ 0.05	/	4/ 0.18	/	/	/	5.0/ 0.23	8.60000
SSE	/	/	1/ 0.05	5/ 0.23	3/ 0.14	/	/	9.0/ 0.41	11.52222
S	/	2/ 0.09	4/ 0.18	15/ 0.69	10/ 0.46	/	/	31.0/ 1.43	10.15000
SSW	/	/	6/ 0.28	39/ 1.80	7/ 0.32	/	/	52.0/ 2.40	10.29615
SW	/	/	1/ 0.05	13/ 0.60	7/ 0.32	/	/	21.0/ 0.97	10.83452
WSW	/	1/ 0.05	2/ 0.09	7/ 0.32	2/ 0.09	/	/	12.0/ 0.55	9.92083
W	/	/	/	3/ 0.14	7/ 0.32	/	/	10.0/ 0.46	12.65500
WNW	/	1/ 0.05	/	9/ 0.41	/	/	/	10.0/ 0.46	8.80250
NW	/	/	2/ 0.09	7/ 0.32	1/ 0.05	/	/	10.0/ 0.46	9.68250
NNW	/	/	2/ 0.09	13/ 0.60	3/ 0.14	/	/	18.0/ 0.83	10.28889
TOTAL	/	9/ 0.41	35/ 1.61	157/ 7.24	55/ 2.54	/	/	256.0/11.80	9.97812

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=03 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	7/ 0.32	9/ 0.41	2/ 0.09	/	/	18.0/ 0.83	8.63055
NNE	0.1/ 0.00	3/ 0.14	5/ 0.23	11/ 0.51	1/ 0.05	/	/	20.1/ 0.93	7.31219
NE	0.0/ 0.00	1/ 0.05	6/ 0.28	5/ 0.23	/	/	/	12.0/ 0.55	7.11667
ENE	0.1/ 0.00	2/ 0.09	10/ 0.46	7/ 0.32	1/ 0.05	/	/	20.1/ 0.93	7.33955
E	/	/	4/ 0.18	7/ 0.32	2/ 0.09	/	/	13.0/ 0.60	9.05577
ESE	0.0/ 0.00	1/ 0.05	4/ 0.18	3/ 0.14	1/ 0.05	/	/	9.0/ 0.41	7.37222
SE	0.1/ 0.00	2/ 0.09	3/ 0.14	6/ 0.28	2/ 0.09	/	/	13.1/ 0.60	8.68893
SSE	0.1/ 0.00	2/ 0.09	5/ 0.23	16/ 0.74	2/ 0.09	/	/	25.1/ 1.16	9.31773
S	0.0/ 0.00	1/ 0.05	9/ 0.41	9/ 0.41	3/ 0.14	/	/	22.0/ 1.01	8.61818
SSW	0.1/ 0.00	2/ 0.09	6/ 0.28	34/ 1.57	5/ 0.23	/	/	47.1/ 2.17	9.30785
SW	0.1/ 0.00	3/ 0.14	10/ 0.46	17/ 0.78	1/ 0.05	/	/	31.1/ 1.43	7.83762
WSW	0.1/ 0.00	3/ 0.14	17/ 0.78	20/ 0.92	2/ 0.09	/	/	42.1/ 1.94	8.10629
W	0.1/ 0.00	2/ 0.09	7/ 0.32	10/ 0.46	/	/	/	19.1/ 0.88	7.75851
WNW	0.0/ 0.00	1/ 0.05	11/ 0.51	8/ 0.37	3/ 0.14	/	/	23.0/ 1.06	8.12391
NW	0.1/ 0.00	2/ 0.09	8/ 0.37	9/ 0.41	/	/	/	19.1/ 0.88	6.30694
NNW	0.2/ 0.01	7/ 0.32	4/ 0.18	11/ 0.51	/	/	/	22.2/ 1.02	6.73649
TOTAL	1.0/ 0.05	32/ 1.48	116/ 5.35	182/ 8.39	25/ 1.15	/	/	356.0/ 16.41	8.10421

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.9/ 0.05	8/ 0.41	36/ 1.85	52/ 2.67	19/ 0.98	13/ 0.67	/	128.9/ 6.63	9.91773
NNE	0.5/ 0.03	4/ 0.21	33/ 1.70	40/ 2.06	20/ 1.03	2/ 0.10	/	99.5/ 5.12	9.28731
NE	0.6/ 0.03	5/ 0.26	35/ 1.80	46/ 2.37	9/ 0.46	/	/	95.6/ 4.92	8.45659
ENE	0.9/ 0.05	8/ 0.41	24/ 1.23	21/ 1.08	4/ 0.21	1/ 0.05	/	58.9/ 3.03	7.59953
E	0.8/ 0.04	7/ 0.36	26/ 1.34	25/ 1.29	7/ 0.36	/	/	65.8/ 3.38	7.87120
ESE	0.6/ 0.03	5/ 0.26	37/ 1.90	27/ 1.39	4/ 0.21	/	/	73.6/ 3.78	7.36866
SE	0.6/ 0.03	5/ 0.26	38/ 1.95	21/ 1.08	2/ 0.10	/	/	66.6/ 3.42	6.77628
SSE	0.7/ 0.04	6/ 0.31	29/ 1.49	48/ 2.47	4/ 0.21	/	/	87.7/ 4.51	7.92275
S	0.5/ 0.03	4/ 0.21	38/ 1.95	86/ 4.42	32/ 1.65	3/ 0.15	/	163.5/ 8.41	9.76246
SSW	0.9/ 0.05	8/ 0.41	41/ 2.11	115/ 5.91	75/ 3.86	9/ 0.46	/	248.9/12.80	10.68486
SW	0.9/ 0.05	8/ 0.41	31/ 1.59	82/ 4.22	61/ 3.14	11/ 0.57	8/ 0.41	201.9/10.38	11.86837
WSW	0.7/ 0.04	6/ 0.31	45/ 2.31	70/ 3.60	42/ 2.16	10/ 0.51	8/ 0.41	181.7/ 9.34	11.39983
W	0.2/ 0.01	2/ 0.10	31/ 1.59	43/ 2.21	26/ 1.34	5/ 0.26	/	107.2/ 5.51	10.13386
WNW	0.4/ 0.02	3/ 0.15	36/ 1.85	67/ 3.44	19/ 0.98	4/ 0.21	/	129.4/ 6.65	9.56543
NW	0.7/ 0.04	6/ 0.31	39/ 2.01	41/ 2.11	32/ 1.65	2/ 0.10	/	120.7/ 6.21	9.30437
NNW	0.9/ 0.05	8/ 0.41	36/ 1.85	45/ 2.31	16/ 0.82	9/ 0.46	/	114.9/ 5.91	9.46138
TOTAL	11.0/ 0.57	93/ 4.78	555/28.53	829/42.62	372/19.13	69/ 3.55	16/ 0.82	1945/ 100	9.72874

NUMBER OF BAD RECORDS: 215

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	2/ 0.10	5/ 0.26	/	/	/	7.0/ 0.36	8.27857
NNE	/	/	2/ 0.10	2/ 0.10	1/ 0.05	/	/	5.0/ 0.26	9.30500
NE	/	/	2/ 0.10	3/ 0.15	1/ 0.05	/	/	6.0/ 0.31	8.53750
ENE	/	/	/	/	/	/	/	/	
E	/	/	/	/	/	/	/	/	
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.45000
SE	/	/	/	/	/	/	/	/	
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.20000
S	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.62500
SSW	/	/	1/ 0.05	1/ 0.05	2/ 0.10	/	/	4.0/ 0.21	10.97500
SW	/	1/ 0.05	/	1/ 0.05	3/ 0.15	1/ 0.05	/	6.0/ 0.31	12.75833
WSW	/	/	/	/	5/ 0.26	4/ 0.21	/	9.0/ 0.46	17.83610
W	/	/	3/ 0.15	1/ 0.05	5/ 0.26	/	/	9.0/ 0.46	11.50555
WNW	/	/	1/ 0.05	2/ 0.10	6/ 0.31	2/ 0.10	/	11.0/ 0.57	14.18333
NW	/	/	/	/	4/ 0.21	1/ 0.05	/	5.0/ 0.26	15.88000
NNW	/	/	1/ 0.05	/	1/ 0.05	/	/	2.0/ 0.10	9.40000
TOTAL	/	1/ 0.05	15/ 0.77	15/ 0.77	28/ 1.44	8/ 0.41	/	67.0/ 3.44	12.07039

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	3/ 0.15	5/ 0.26	1/ 0.05	/	/	9.0/ 0.46	8.65000
NNE	/	/	6/ 0.31	11/ 0.57	/	/	/	17.0/ 0.87	8.07206
NE	/	/	6/ 0.31	2/ 0.10	/	/	/	8.0/ 0.41	6.44687
ENE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	7.45000
E	/	/	/	/	/	/	/	/	
ESE	/	/	3/ 0.15	/	/	/	/	3.0/ 0.15	6.45000
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.02500
SSE	/	/	/	/	/	/	/	/	
S	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	7.92500
SSW	/	/	2/ 0.10	6/ 0.31	1/ 0.05	/	/	9.0/ 0.46	9.41944
SW	/	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	3.0/ 0.15	10.45833
WSW	/	1/ 0.05	/	3/ 0.15	3/ 0.15	2/ 0.10	2/ 0.10	11.0/ 0.57	17.40227
W	/	/	/	/	2/ 0.10	/	/	2.0/ 0.10	13.67500
WNW	/	/	3/ 0.15	7/ 0.36	1/ 0.05	/	/	11.0/ 0.57	9.32273
NW	/	/	1/ 0.05	4/ 0.21	5/ 0.26	1/ 0.05	/	11.0/ 0.57	12.31818
NNW	/	/	/	2/ 0.10	/	/	/	2.0/ 0.10	9.96250
TOTAL	/	1/ 0.05	27/ 1.39	42/ 2.16	14/ 0.72	3/ 0.15	2/ 0.10	89.0/ 4.58	10.09326

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	6/ 0.31	/	/	/	/	6.0/ 0.31	6.40833
NNE	/	/	3/ 0.15	1/ 0.05	/	/	/	4.0/ 0.21	5.68750
NE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	11.62500
ENE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	7.07500
E	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	9.02500
ESE	/	/	3/ 0.15	/	/	/	/	3.0/ 0.15	5.65000
SE	/	/	4/ 0.21	/	/	/	/	4.0/ 0.21	6.08750
SSE	/	1/ 0.05	2/ 0.10	2/ 0.10	/	/	/	5.0/ 0.26	7.00000
S	/	/	1/ 0.05	3/ 0.15	/	/	/	4.0/ 0.21	7.63125
SSW	/	/	1/ 0.05	9/ 0.46	2/ 0.10	/	/	12.0/ 0.62	10.57916
SW	/	/	1/ 0.05	2/ 0.10	/	1/ 0.05	1/ 0.05	5.0/ 0.26	15.23500
WSW	/	/	2/ 0.10	3/ 0.15	3/ 0.15	1/ 0.05	1/ 0.05	10.0/ 0.51	13.30000
W	/	/	1/ 0.05	/	1/ 0.05	1/ 0.05	/	3.0/ 0.15	13.01666
WNW	/	/	4/ 0.21	3/ 0.15	1/ 0.05	/	/	8.0/ 0.41	8.99687
NW	/	/	3/ 0.15	2/ 0.10	2/ 0.10	/	/	7.0/ 0.36	8.97500
NNW	/	/	3/ 0.15	2/ 0.10	1/ 0.05	/	/	6.0/ 0.31	8.58333
TOTAL	/	1/ 0.05	35/ 1.80	29/ 1.49	10/ 0.51	3/ 0.15	2/ 0.10	80.0/ 4.11	9.46531

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=C

UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	6/ 0.31	/	/	/	/	6.0/ 0.31	6.40833
NNE	/	/	3/ 0.15	1/ 0.05	/	/	/	4.0/ 0.21	5.68750
NE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	11.62500
ENE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	7.07500
E	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	9.02500
ESE	/	/	3/ 0.15	/	/	/	/	3.0/ 0.15	5.65000
SE	/	/	4/ 0.21	/	/	/	/	4.0/ 0.21	6.08750
SSE	/	1/ 0.05	2/ 0.10	2/ 0.10	/	/	/	5.0/ 0.26	7.00000
S	/	/	1/ 0.05	3/ 0.15	/	/	/	4.0/ 0.21	7.63125
SSW	/	/	1/ 0.05	9/ 0.46	2/ 0.10	/	/	12.0/ 0.62	10.57916
SW	/	/	1/ 0.05	2/ 0.10	/	1/ 0.05	1/ 0.05	5.0/ 0.26	15.23500
WSW	/	/	2/ 0.10	3/ 0.15	3/ 0.15	1/ 0.05	1/ 0.05	10.0/ 0.51	13.30000
W	/	/	1/ 0.05	/	1/ 0.05	1/ 0.05	/	3.0/ 0.15	13.01666
WNW	/	/	4/ 0.21	3/ 0.15	1/ 0.05	/	/	8.0/ 0.41	8.99687
NW	/	/	3/ 0.15	2/ 0.10	2/ 0.10	/	/	7.0/ 0.36	8.97500
NNW	/	/	3/ 0.15	2/ 0.10	1/ 0.05	/	/	6.0/ 0.31	8.58333
TOTAL	/	1/ 0.05	35/ 1.80	29/ 1.49	10/ 0.51	3/ 0.15	2/ 0.10	80.0/ 4.11	9.46531

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.01	3/ 0.15	13/ 0.67	12/ 0.62	3/ 0.15	4/ 0.21	/	35.1/ 1.80	9.23611
NNE	0.1/ 0.01	2/ 0.10	9/ 0.46	6/ 0.31	5/ 0.26	2/ 0.10	/	24.1/ 1.24	9.94657
NE	0.0/ 0.00	1/ 0.05	7/ 0.36	4/ 0.21	/	/	/	12.0/ 0.62	6.63958
ENE	0.1/ 0.01	4/ 0.21	8/ 0.41	3/ 0.15	/	/	/	15.1/ 0.78	5.47434
E	0.0/ 0.00	1/ 0.05	13/ 0.67	8/ 0.41	/	/	/	22.0/ 1.13	6.87386
ESE	0.1/ 0.01	2/ 0.10	7/ 0.36	3/ 0.15	/	/	/	12.1/ 0.62	5.67459
SE	/	/	13/ 0.67	4/ 0.21	/	/	/	17.0/ 0.87	6.19706
SSE	0.1/ 0.01	2/ 0.10	7/ 0.36	10/ 0.51	1/ 0.05	/	/	20.1/ 1.03	7.54478
S	0.0/ 0.00	1/ 0.05	6/ 0.31	18/ 0.93	4/ 0.21	3/ 0.15	/	32.0/ 1.65	10.73672
SSW	0.1/ 0.01	2/ 0.10	9/ 0.46	24/ 1.23	20/ 1.03	8/ 0.41	/	63.1/ 3.24	12.00257
SW	0.2/ 0.01	5/ 0.26	12/ 0.62	12/ 0.62	17/ 0.87	8/ 0.41	5/ 0.26	59.2/ 3.04	12.79772
WSW	0.1/ 0.01	4/ 0.21	13/ 0.67	8/ 0.41	10/ 0.51	1/ 0.05	5/ 0.26	41.1/ 2.11	11.89902
W	/	/	12/ 0.62	8/ 0.41	8/ 0.41	3/ 0.15	/	31.0/ 1.59	10.64435
WNW	0.1/ 0.01	2/ 0.10	9/ 0.46	19/ 0.98	4/ 0.21	2/ 0.10	/	36.1/ 1.86	9.46226
NW	/	/	10/ 0.51	13/ 0.67	17/ 0.87	/	/	40.0/ 2.06	10.62500
NNW	0.1/ 0.01	4/ 0.21	15/ 0.77	15/ 0.77	7/ 0.36	3/ 0.15	/	44.1/ 2.27	9.43849
TOTAL	1.0/ 0.05	33/ 1.70	163/ 8.38	167/ 8.59	96/ 4.94	34/ 1.75	10/ 0.51	504.0/25.91	10.04670

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	2/ 0.10	7/ 0.36	12/ 0.62	12/ 0.62	9/ 0.46	/	42.0/ 2.16	12.59246
NNE	/	/	9/ 0.46	9/ 0.46	7/ 0.36	/	/	25.0/ 1.29	9.89300
NE	/	3/ 0.15	15/ 0.77	19/ 0.98	6/ 0.31	/	/	43.0/ 2.21	8.95058
ENE	/	/	6/ 0.31	11/ 0.57	4/ 0.21	1/ 0.05	/	22.0/ 1.13	9.70682
E	/	3/ 0.15	8/ 0.41	6/ 0.31	5/ 0.26	/	/	22.0/ 1.13	8.64659
ESE	/	1/ 0.05	13/ 0.67	8/ 0.41	1/ 0.05	/	/	23.0/ 1.18	7.91848
SE	/	2/ 0.10	11/ 0.57	8/ 0.41	/	/	/	21.0/ 1.08	6.78929
SSE	/	1/ 0.05	11/ 0.57	21/ 1.08	2/ 0.10	/	/	35.0/ 1.80	8.55000
S	/	2/ 0.10	13/ 0.67	31/ 1.59	17/ 0.87	/	/	63.0/ 3.24	10.03809
SSW	/	/	9/ 0.46	29/ 1.49	26/ 1.34	1/ 0.05	/	65.0/ 3.34	11.19115
SW	/	1/ 0.05	6/ 0.31	18/ 0.93	17/ 0.87	1/ 0.05	2/ 0.10	45.0/ 2.31	12.40000
WSW	/	1/ 0.05	13/ 0.67	14/ 0.72	8/ 0.41	2/ 0.10	/	38.0/ 1.95	10.04473
W	/	/	6/ 0.31	16/ 0.82	8/ 0.41	1/ 0.05	/	31.0/ 1.59	10.09726
WNW	/	/	6/ 0.31	20/ 1.03	2/ 0.10	/	/	28.0/ 1.44	9.27857
NW	/	1/ 0.05	5/ 0.26	10/ 0.51	1/ 0.05	/	/	17.0/ 0.87	8.69118
NNW	/	2/ 0.10	6/ 0.31	9/ 0.46	2/ 0.10	6/ 0.31	/	25.0/ 1.29	11.02000
TOTAL	/	19/ 0.98	144/ 7.40	241/12.39	118/ 6.07	21/ 1.08	2/ 0.10	545.0/28.02	10.10919

NUMBER OF BAD RECORDS: 11

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	3/ 0.15	3/ 0.15	3/ 0.15	/	/	9.0/ 0.46	9.53889
NNE	/	/	/	6/ 0.31	3/ 0.15	/	/	9.0/ 0.46	11.56389
NE	/	/	2/ 0.10	9/ 0.46	2/ 0.10	/	/	13.0/ 0.67	10.10385
ENE	/	/	3/ 0.15	4/ 0.21	/	/	/	7.0/ 0.36	9.18214
E	0.3/ 0.02	1/ 0.05	3/ 0.15	9/ 0.46	1/ 0.05	/	/	14.3/ 0.74	9.07343
ESE	/	/	5/ 0.26	6/ 0.31	1/ 0.05	/	/	12.0/ 0.62	7.84375
SE	/	/	2/ 0.10	4/ 0.21	1/ 0.05	/	/	7.0/ 0.36	9.06071
SSE	0.3/ 0.02	1/ 0.05	2/ 0.10	9/ 0.46	/	/	/	12.3/ 0.63	7.78557
S	/	/	4/ 0.21	21/ 1.08	8/ 0.41	/	/	33.0/ 1.70	10.21818
SSW	/	/	7/ 0.36	21/ 1.08	18/ 0.93	/	/	46.0/ 2.37	10.84674
SW	/	/	2/ 0.10	15/ 0.77	18/ 0.93	/	/	35.0/ 1.80	12.35143
WSW	/	/	3/ 0.15	11/ 0.57	10/ 0.51	/	/	24.0/ 1.23	11.37396
W	/	/	1/ 0.05	7/ 0.36	1/ 0.05	/	/	9.0/ 0.46	9.24722
WNW	/	/	2/ 0.10	3/ 0.15	2/ 0.10	/	/	7.0/ 0.36	10.30714
NW	/	/	4/ 0.21	7/ 0.36	2/ 0.10	/	/	13.0/ 0.67	9.02308
NNW	0.3/ 0.02	1/ 0.05	1/ 0.05	8/ 0.41	1/ 0.05	/	/	11.3/ 0.58	9.23230
TOTAL	1.0/ 0.05	3/ 0.15	44/ 2.26	143/ 7.35	71/ 3.65	/	/	262.0/13.47	10.25587

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=04 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.8/ 0.04	3/ 0.15	2/ 0.10	15/ 0.77	/	/	/	20.8/ 1.07	7.91466
NNE	0.5/ 0.03	2/ 0.10	4/ 0.21	5/ 0.26	4/ 0.21	/	/	15.5/ 0.80	8.15968
NE	0.3/ 0.02	1/ 0.05	3/ 0.15	8/ 0.41	/	/	/	12.3/ 0.63	7.93191
ENE	1.0/ 0.05	4/ 0.21	5/ 0.26	3/ 0.15	/	/	/	13.0/ 0.67	5.55000
E	0.5/ 0.03	2/ 0.10	2/ 0.10	1/ 0.05	1/ 0.05	/	/	6.5/ 0.33	5.80000
ESE	0.5/ 0.03	2/ 0.10	5/ 0.26	10/ 0.51	2/ 0.10	/	/	19.5/ 1.00	7.98120
SE	0.8/ 0.04	3/ 0.15	7/ 0.36	5/ 0.26	1/ 0.05	/	/	16.8/ 0.86	6.65923
SSE	0.3/ 0.02	1/ 0.05	6/ 0.31	6/ 0.31	1/ 0.05	/	/	14.3/ 0.74	7.61189
S	0.3/ 0.02	1/ 0.05	13/ 0.67	12/ 0.62	3/ 0.15	/	/	29.3/ 1.51	8.17150
SSW	1.5/ 0.08	6/ 0.31	12/ 0.62	25/ 1.29	6/ 0.31	/	/	50.5/ 2.60	8.33366
SW	0.3/ 0.02	1/ 0.05	9/ 0.46	33/ 1.70	5/ 0.26	/	/	48.3/ 2.48	9.58747
WSW	/	/	14/ 0.72	31/ 1.59	3/ 0.15	/	/	48.0/ 2.47	9.19427
W	0.5/ 0.03	2/ 0.10	8/ 0.41	11/ 0.57	1/ 0.05	/	/	22.5/ 1.16	7.35333
WNW	0.3/ 0.02	1/ 0.05	11/ 0.57	13/ 0.67	3/ 0.15	/	/	28.3/ 1.46	8.25353
NW	1.3/ 0.07	5/ 0.26	16/ 0.82	5/ 0.26	1/ 0.05	/	/	28.3/ 1.46	5.50883
NNW	0.3/ 0.02	1/ 0.05	10/ 0.51	9/ 0.46	4/ 0.21	/	/	24.3/ 1.25	8.23971
TOTAL	9.0/ 0.46	35/ 1.80	127/ 6.53	192/ 9.87	35/ 1.80	/	/	398.0/20.46	8.02687

NUMBER OF BAD RECORDS: 8

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	11/ 0.52	40/ 1.88	51/ 2.39	22/ 1.03	1/ 0.05	/	125.1/ 5.86	8.78474
NNE	0.0/ 0.00	4/ 0.19	37/ 1.73	55/ 2.58	17/ 0.80	/	/	113.0/ 5.30	8.75774
NE	0.0/ 0.00	8/ 0.38	42/ 1.97	61/ 2.86	23/ 1.08	/	/	134.0/ 6.28	8.90951
ENE	0.1/ 0.00	13/ 0.61	41/ 1.92	65/ 3.05	10/ 0.47	/	/	129.1/ 6.05	7.92854
E	0.1/ 0.00	11/ 0.52	40/ 1.88	70/ 3.28	8/ 0.38	/	/	129.1/ 6.05	8.29318
ESE	0.1/ 0.00	15/ 0.70	41/ 1.92	44/ 2.06	15/ 0.70	/	/	115.1/ 5.40	7.82493
SE	0.1/ 0.00	11/ 0.52	26/ 1.22	44/ 2.06	10/ 0.47	/	/	91.1/ 4.27	8.17343
SSE	0.1/ 0.00	18/ 0.84	49/ 2.30	86/ 4.03	19/ 0.89	3/ 0.14	2/ 0.09	177.1/ 8.30	8.95313
S	0.0/ 0.00	7/ 0.33	51/ 2.39	72/ 3.38	34/ 1.59	/	/	164.0/ 7.69	9.24817
SSW	0.0/ 0.00	8/ 0.38	47/ 2.20	92/ 4.31	37/ 1.73	2/ 0.09	/	186.0/ 8.72	9.52796
SW	0.1/ 0.00	12/ 0.56	51/ 2.39	85/ 3.98	37/ 1.73	/	/	185.1/ 8.68	9.17900
WSW	0.1/ 0.00	9/ 0.42	44/ 2.06	81/ 3.80	29/ 1.36	2/ 0.09	/	165.1/ 7.74	9.18466
W	0.0/ 0.00	8/ 0.38	37/ 1.73	54/ 2.53	10/ 0.47	5/ 0.23	1/ 0.05	115.0/ 5.39	9.06282
WNW	0.1/ 0.00	11/ 0.52	36/ 1.69	39/ 1.83	4/ 0.19	1/ 0.05	/	91.1/ 4.27	7.46208
NW	0.0/ 0.00	6/ 0.28	37/ 1.73	33/ 1.55	13/ 0.61	/	/	89.0/ 4.17	8.19326
NNW	0.0/ 0.00	8/ 0.38	29/ 1.36	60/ 2.81	27/ 1.27	/	/	124.0/ 5.81	9.52990
TOTAL	1.0/ 0.05	160/ 7.50	648/30.38	992/46.51	315/14.77	14/ 0.66	3/ 0.14	2133/ 100	8.78800

NUMBER OF BAD RECORDS: 99

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	/	2/ 0.09	1/ 0.05	/	/	4.0/ 0.19	8.62083
NNE	/	/	1/ 0.05	8/ 0.38	3/ 0.14	/	/	12.0/ 0.56	10.22292
NE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	7.00000
ENE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	2.75000
E	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	9.92500
ESE	/	2/ 0.09	1/ 0.05	/	1/ 0.05	/	/	4.0/ 0.19	5.59375
SE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	2.67500
SSE	/	/	/	/	/	/	/	/	
S	/	1/ 0.05	/	3/ 0.14	/	/	/	4.0/ 0.19	7.33750
SSW	/	/	/	7/ 0.33	2/ 0.09	/	/	9.0/ 0.42	9.82500
SW	/	/	4/ 0.19	5/ 0.23	/	/	/	9.0/ 0.42	7.38241
WSW	/	/	/	7/ 0.33	1/ 0.05	/	/	8.0/ 0.38	10.98125
W	/	/	5/ 0.23	3/ 0.14	1/ 0.05	/	/	9.0/ 0.42	7.96944
WNW	/	/	2/ 0.09	10/ 0.47	1/ 0.05	/	/	13.0/ 0.61	9.55000
NW	/	/	5/ 0.23	6/ 0.28	/	/	/	11.0/ 0.52	7.72954
NNW	/	/	/	12/ 0.56	10/ 0.47	/	/	22.0/ 1.03	12.85113
TOTAL	/	6/ 0.28	19/ 0.89	64/ 3.00	20/ 0.94	/	/	109.0/ 5.11	9.51904

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.82500
NNE	/	/	2/ 0.09	/	2/ 0.09	/	/	4.0/ 0.19	10.87500
NE	/	/	/	/	/	/	/	/	
ENE	/	/	/	2/ 0.09	/	/	/	2.0/ 0.09	10.21250
E	/	/	/	3/ 0.14	/	/	/	3.0/ 0.14	11.45000
ESE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	11.30000
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	/	/	/	/	/	
S	/	/	/	/	/	/	/	/	
SSW	/	/	1/ 0.05	3/ 0.14	2/ 0.09	/	/	6.0/ 0.28	11.15833
SW	/	/	2/ 0.09	1/ 0.05	1/ 0.05	/	/	4.0/ 0.19	9.40000
WSW	/	1/ 0.05	1/ 0.05	1/ 0.05	5/ 0.23	/	/	8.0/ 0.38	11.11875
W	/	/	2/ 0.09	/	1/ 0.05	2/ 0.09	1/ 0.05	6.0/ 0.28	15.94583
WNW	/	/	/	/	/	/	/	/	
NW	/	1/ 0.05	3/ 0.14	2/ 0.09	/	/	/	6.0/ 0.28	6.54167
NNW	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	10.30000
TOTAL	/	2/ 0.09	12/ 0.56	14/ 0.66	11/ 0.52	2/ 0.09	1/ 0.05	42.0/ 1.97	10.83631

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	/	/	/	/	/	
NNE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	11.62500
NE	/	/	/	/	/	/	/	/	
ENE	/	1/ 0.05	1/ 0.05	2/ 0.09	/	/	/	4.0/ 0.19	8.00625
E	/	/	1/ 0.05	2/ 0.09	/	/	/	3.0/ 0.14	8.71667
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.25000
SE	/	/	/	3/ 0.14	/	/	/	3.0/ 0.14	9.60833
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.72500
S	/	/	/	3/ 0.14	/	/	/	3.0/ 0.14	7.95833
SSW	/	/	/	1/ 0.05	2/ 0.09	/	/	3.0/ 0.14	12.55000
SW	/	1/ 0.05	/	1/ 0.05	7/ 0.33	/	/	9.0/ 0.42	12.69166
WSW	/	/	/	/	3/ 0.14	2/ 0.09	/	5.0/ 0.23	16.87999
W	/	/	/	1/ 0.05	1/ 0.05	3/ 0.14	/	5.0/ 0.23	18.10999
WNW	/	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	3.0/ 0.14	11.24166
NW	/	1/ 0.05	2/ 0.09	/	1/ 0.05	/	/	4.0/ 0.19	7.10625
NNW	/	/	/	4/ 0.19	2/ 0.09	/	/	6.0/ 0.28	11.96250
TOTAL	/	3/ 0.14	7/ 0.33	19/ 0.89	17/ 0.80	5/ 0.23	/	51.0/ 2.39	11.67108

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	5/ 0.23	12/ 0.56	20/ 0.94	8/ 0.38	/	/	45.0/ 2.11	8.82222
NNE	/	/	12/ 0.56	13/ 0.61	5/ 0.23	/	/	30.0/ 1.41	8.70083
NE	/	2/ 0.09	15/ 0.70	15/ 0.70	7/ 0.33	/	/	39.0/ 1.83	9.00000
ENE	/	3/ 0.14	18/ 0.84	19/ 0.89	4/ 0.19	/	/	44.0/ 2.06	7.70966
E	/	4/ 0.19	14/ 0.66	11/ 0.52	2/ 0.09	/	/	31.0/ 1.45	7.15242
ESE	/	4/ 0.19	17/ 0.80	10/ 0.47	8/ 0.38	/	/	39.0/ 1.83	7.94744
SE	/	2/ 0.09	16/ 0.75	6/ 0.28	6/ 0.28	/	/	30.0/ 1.41	8.04917
SSE	/	8/ 0.38	15/ 0.70	25/ 1.17	7/ 0.33	/	/	55.0/ 2.58	8.49545
S	/	2/ 0.09	20/ 0.94	7/ 0.33	5/ 0.23	/	/	34.0/ 1.59	7.73970
SSW	/	3/ 0.14	17/ 0.80	7/ 0.33	17/ 0.80	/	/	44.0/ 2.06	9.38579
SW	/	4/ 0.19	17/ 0.80	15/ 0.70	19/ 0.89	/	/	55.0/ 2.58	9.66757
WSW	/	1/ 0.05	18/ 0.84	22/ 1.03	8/ 0.38	/	/	49.0/ 2.30	8.86888
W	/	4/ 0.19	9/ 0.42	15/ 0.70	4/ 0.19	/	/	32.0/ 1.50	8.42422
WNW	/	1/ 0.05	6/ 0.28	9/ 0.42	/	/	/	16.0/ 0.75	7.33125
NW	/	/	3/ 0.14	13/ 0.61	9/ 0.42	/	/	25.0/ 1.17	11.14200
NNW	/	1/ 0.05	9/ 0.42	20/ 0.94	7/ 0.33	/	/	37.0/ 1.73	9.53198
TOTAL	/	44/ 2.06	218/10.22	227/10.64	116/ 5.44	/	/	605.0/28.36	8.67632

NUMBER OF BAD RECORDS: 4

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	2/ 0.09	13/ 0.61	14/ 0.66	8/ 0.38	1/ 0.05	/	38.0/ 1.78	9.35329
NNE	/	2/ 0.09	10/ 0.47	19/ 0.89	5/ 0.23	/	/	36.0/ 1.69	8.77708
NE	/	4/ 0.19	15/ 0.70	30/ 1.41	10/ 0.47	/	/	59.0/ 2.77	8.95339
ENE	/	1/ 0.05	16/ 0.75	23/ 1.08	2/ 0.09	/	/	42.0/ 1.97	8.32024
E	/	2/ 0.09	15/ 0.70	22/ 1.03	4/ 0.19	/	/	43.0/ 2.02	8.71686
ESE	/	/	14/ 0.66	19/ 0.89	3/ 0.14	/	/	36.0/ 1.69	8.64514
SE	/	4/ 0.19	4/ 0.19	12/ 0.56	1/ 0.05	/	/	21.0/ 0.98	7.61786
SSE	/	3/ 0.14	17/ 0.80	33/ 1.55	11/ 0.52	3/ 0.14	2/ 0.09	69.0/ 3.23	10.17753
S	/	1/ 0.05	6/ 0.28	24/ 1.13	24/ 1.13	/	/	55.0/ 2.58	11.49773
SSW	/	3/ 0.14	10/ 0.47	40/ 1.88	7/ 0.33	2/ 0.09	/	62.0/ 2.91	9.96532
SW	/	1/ 0.05	10/ 0.47	9/ 0.42	6/ 0.28	/	/	26.0/ 1.22	8.91058
WSW	/	4/ 0.19	8/ 0.38	12/ 0.56	1/ 0.05	/	/	25.0/ 1.17	7.18800
W	/	2/ 0.09	5/ 0.23	7/ 0.33	1/ 0.05	/	/	15.0/ 0.70	6.60500
WNW	/	4/ 0.19	7/ 0.33	5/ 0.23	/	1/ 0.05	/	17.0/ 0.80	6.89167
NW	/	3/ 0.14	5/ 0.23	6/ 0.28	1/ 0.05	/	/	15.0/ 0.70	7.08000
NNW	/	5/ 0.23	10/ 0.47	11/ 0.52	3/ 0.14	/	/	29.0/ 1.36	7.46207
TOTAL	/	41/ 1.92	165/ 7.74	286/13.41	87/ 4.08	7/ 0.33	2/ 0.09	588.0/27.57	9.00988

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	7/ 0.33	10/ 0.47	2/ 0.09	/	/	19.0/ 0.89	8.96974
NNE	/	2/ 0.09	6/ 0.28	5/ 0.23	1/ 0.05	/	/	14.0/ 0.66	7.02679
NE	/	2/ 0.09	5/ 0.23	8/ 0.38	2/ 0.09	/	/	17.0/ 0.80	7.73971
ENE	/	3/ 0.14	2/ 0.09	6/ 0.28	/	/	/	11.0/ 0.52	6.89545
E	/	2/ 0.09	3/ 0.14	13/ 0.61	1/ 0.05	/	/	19.0/ 0.89	8.54342
ESE	/	2/ 0.09	2/ 0.09	5/ 0.23	1/ 0.05	/	/	10.0/ 0.47	7.73750
SE	/	1/ 0.05	1/ 0.05	11/ 0.52	/	/	/	13.0/ 0.61	8.42308
SSE	/	2/ 0.09	6/ 0.28	17/ 0.80	/	/	/	25.0/ 1.17	8.52900
S	/	/	8/ 0.38	16/ 0.75	5/ 0.23	/	/	29.0/ 1.36	9.42672
SSW	/	/	3/ 0.14	19/ 0.89	4/ 0.19	/	/	26.0/ 1.22	10.27115
SW	/	1/ 0.05	5/ 0.23	22/ 1.03	/	/	/	28.0/ 1.31	8.77232
WSW	/	/	5/ 0.23	7/ 0.33	/	/	/	16.0/ 0.75	9.00156
W	/	1/ 0.05	3/ 0.14	7/ 0.33	1/ 0.19	/	/	12.0/ 0.56	8.47292
WNW	/	2/ 0.09	3/ 0.14	6/ 0.28	1/ 0.05	/	/	12.0/ 0.56	7.68333
NW	/	/	1/ 0.05	3/ 0.14	2/ 0.09	/	/	6.0/ 0.28	9.95000
NNW	/	1/ 0.05	1/ 0.05	2/ 0.09	2/ 0.09	/	/	6.0/ 0.28	9.42917
TOTAL	/	19/ 0.89	61/ 2.86	157/ 7.36	26/ 1.22	/	/	263.0/ 12.33	8.66492

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=05 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	3/ 0.14	7/ 0.33	5/ 0.23	3/ 0.14	/	/	18.1/ 0.85	7.44613
NNE	/	/	6/ 0.28	9/ 0.42	1/ 0.05	/	/	16.0/ 0.75	8.52812
NE	/	/	6/ 0.28	8/ 0.38	4/ 0.19	/	/	18.0/ 0.84	9.78055
ENE	0.1/ 0.00	4/ 0.19	4/ 0.19	13/ 0.61	4/ 0.19	/	/	25.1/ 1.18	8.12151
E	0.1/ 0.00	3/ 0.14	7/ 0.33	18/ 0.84	1/ 0.05	/	/	29.1/ 1.36	8.29381
ESE	0.2/ 0.01	7/ 0.33	6/ 0.28	9/ 0.42	2/ 0.09	/	/	24.2/ 1.13	6.70764
SE	0.1/ 0.00	3/ 0.14	5/ 0.23	12/ 0.56	3/ 0.14	/	/	23.1/ 1.08	8.75108
SSE	0.1/ 0.00	5/ 0.23	10/ 0.47	11/ 0.52	1/ 0.05	/	/	27.1/ 1.27	7.27214
S	0.1/ 0.00	3/ 0.14	17/ 0.80	19/ 0.89	/	/	/	39.1/ 1.83	7.53676
SSW	0.0/ 0.00	2/ 0.09	16/ 0.75	15/ 0.70	3/ 0.14	/	/	36.0/ 1.69	7.81389
SW	0.1/ 0.00	5/ 0.23	13/ 0.61	32/ 1.50	4/ 0.19	/	/	54.1/ 2.54	8.71996
WSW	0.1/ 0.00	3/ 0.14	12/ 0.56	32/ 1.50	7/ 0.33	/	/	54.1/ 2.54	9.18392
W	0.0/ 0.00	1/ 0.05	13/ 0.61	21/ 0.98	1/ 0.05	/	/	36.0/ 1.69	8.72083
WNW	0.1/ 0.00	4/ 0.19	17/ 0.80	8/ 0.38	1/ 0.05	/	/	30.1/ 1.41	6.48588
NW	0.0/ 0.00	1/ 0.05	18/ 0.84	3/ 0.14	/	/	/	22.0/ 1.03	6.00227
NNW	0.0/ 0.00	1/ 0.05	9/ 0.42	10/ 0.47	3/ 0.14	/	/	23.0/ 1.08	8.31522
TOTAL	1.0/ 0.05	45/ 2.11	166/ 7.78	225/10.55	38/ 1.78	/	/	475.0/22.27	8.06545

NUMBER OF BAD RECORDS: 13

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	6/ 0.30	30/ 1.50	30/ 1.50	12/ 0.60	/	/	78.0/ 3.90	8.28045
NNE	/	3/ 0.15	41/ 2.05	47/ 2.35	16/ 0.80	/	/	107.0/ 5.35	8.51075
NE	/	8/ 0.40	30/ 1.50	57/ 2.85	17/ 0.85	6/ 0.30	/	118.0/ 5.90	9.80127
ENE	/	8/ 0.40	44/ 2.20	28/ 1.40	5/ 0.25	/	/	85.0/ 4.25	7.06323
E	/	6/ 0.30	37/ 1.85	31/ 1.55	2/ 0.10	/	/	76.0/ 3.80	7.36743
ESE	/	6/ 0.30	34/ 1.70	36/ 1.80	2/ 0.10	/	/	78.0/ 3.90	7.50833
SE	/	9/ 0.45	52/ 2.60	28/ 1.40	/	/	/	89.0/ 4.45	6.52472
SSE	/	7/ 0.35	50/ 2.50	61/ 3.05	7/ 0.35	/	/	125.0/ 6.25	7.94587
S	/	12/ 0.60	70/ 3.50	122/ 6.10	15/ 0.75	3/ 0.15	/	222.0/11.11	8.46250
SSW	/	11/ 0.55	77/ 3.85	128/ 6.40	16/ 0.80	1/ 0.05	/	233.0/11.66	8.43147
SW	/	14/ 0.70	75/ 3.75	112/ 5.60	34/ 1.70	2/ 0.10	/	237.0/11.86	8.86413
WSW	/	12/ 0.60	55/ 2.75	126/ 6.30	18/ 0.90	1/ 0.05	/	212.0/10.61	8.66568
W	/	4/ 0.20	46/ 2.30	39/ 1.95	3/ 0.15	1/ 0.05	/	93.0/ 4.65	7.36935
WNW	/	7/ 0.35	26/ 1.30	22/ 1.10	11/ 0.55	2/ 0.10	/	68.0/ 3.40	8.28934
NW	/	15/ 0.75	31/ 1.55	19/ 0.95	14/ 0.70	2/ 0.10	/	81.0/ 4.05	7.88333
NNW	/	12/ 0.60	42/ 2.10	32/ 1.60	11/ 0.55	/	/	97.0/ 4.85	7.48737
TOTAL	/	140/ 7.00	740/37.02	918/45.92	183/ 9.15	18/ 0.90	/	1999/ 100	8.21808

NUMBER OF BAD RECORDS: 161

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	/	/	/	/	/	
NNE	/	/	/	/	/	/	/	/	
NE	/	/	/	/	1/ 0.05	3/ 0.15	/	4.0/ 0.20	19.13123
ENE	/	/	/	/	/	/	/	/	
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	3.65000
ESE	/	/	/	/	/	/	/	/	
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	8.95000
S	/	/	1/ 0.05	1/ 0.05	/	1/ 0.05	/	3.0/ 0.15	11.80000
SSW	/	/	1/ 0.05	1/ 0.05	2/ 0.10	/	/	4.0/ 0.20	12.07499
SW	/	1/ 0.05	1/ 0.05	1/ 0.05	5/ 0.25	/	/	8.0/ 0.40	12.11250
WSW	/	1/ 0.05	/	1/ 0.05	1/ 0.05	/	/	3.0/ 0.15	8.95833
W	/	/	/	/	1/ 0.05	1/ 0.05	/	2.0/ 0.10	17.78750
WNW	/	/	/	/	/	1/ 0.05	/	1.0/ 0.05	21.34999
NW	/	/	/	/	3/ 0.15	/	/	3.0/ 0.15	13.90833
NNW	/	/	/	/	1/ 0.05	/	/	1.0/ 0.05	13.25000
TOTAL	/	2/ 0.10	4/ 0.20	5/ 0.25	14/ 0.70	6/ 0.30	/	31.0/ 1.55	13.17741

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	1/ 0.05	2/ 0.10	/	/	/	3.0/ 0.15	8.83333
NNE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.65000
NE	/	/	1/ 0.05	3/ 0.15	/	1/ 0.05	/	5.0/ 0.25	11.34500
ENE	/	/	5/ 0.25	1/ 0.05	/	/	/	6.0/ 0.30	6.27917
E	/	1/ 0.05	3/ 0.15	/	/	/	/	4.0/ 0.20	5.75625
ESE	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.10	4.27500
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.32500
SSE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.62500
S	/	/	1/ 0.05	2/ 0.10	1/ 0.05	/	/	4.0/ 0.20	11.76875
SSW	/	/	/	4/ 0.20	1/ 0.05	/	/	5.0/ 0.25	11.05000
SW	/	/	3/ 0.15	1/ 0.05	7/ 0.35	1/ 0.05	/	12.0/ 0.60	12.31458
WSW	/	/	1/ 0.05	9/ 0.45	6/ 0.30	/	/	16.0/ 0.80	12.35469
W	/	/	/	/	/	/	/	/	
WNW	/	/	2/ 0.10	4/ 0.20	1/ 0.05	/	/	7.0/ 0.35	8.96071
NW	/	/	/	2/ 0.10	2/ 0.10	/	/	4.0/ 0.20	12.00000
NNW	/	/	2/ 0.10	/	1/ 0.05	/	/	3.0/ 0.15	6.85000
TOTAL	/	2/ 0.10	23/ 1.15	28/ 1.40	19/ 0.95	2/ 0.10	/	74.0/ 3.70	10.13682

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	2/ 0.10	4/ 0.20	/	/	/	7.0/ 0.35	7.50357
NNE	/	/	3/ 0.15	3/ 0.15	/	/	/	6.0/ 0.30	7.34583
NE	/	/	2/ 0.10	2/ 0.10	1/ 0.05	1/ 0.05	/	6.0/ 0.30	11.57916
ENE	/	1/ 0.05	2/ 0.10	1/ 0.05	/	/	/	4.0/ 0.20	5.85626
E	/	1/ 0.05	3/ 0.15	1/ 0.05	/	/	/	5.0/ 0.25	5.90000
ESE	/	1/ 0.05	2/ 0.10	/	/	/	/	3.0/ 0.15	4.39167
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.15000
SSE	/	/	2/ 0.10	2/ 0.10	1/ 0.05	/	/	5.0/ 0.25	9.14500
S	/	/	6/ 0.30	2/ 0.10	1/ 0.05	1/ 0.05	/	10.0/ 0.50	8.82500
SSW	/	1/ 0.05	2/ 0.10	5/ 0.25	3/ 0.15	/	/	11.0/ 0.55	9.45682
SW	/	/	2/ 0.10	5/ 0.25	2/ 0.10	/	/	9.0/ 0.45	10.13333
WSW	/	/	2/ 0.10	12/ 0.60	3/ 0.15	1/ 0.05	/	18.0/ 0.90	10.67083
W	/	/	2/ 0.10	1/ 0.05	/	/	/	3.0/ 0.15	6.16667
WNW	/	/	1/ 0.05	4/ 0.20	/	/	/	5.0/ 0.25	7.86500
NW	/	/	1/ 0.05	2/ 0.10	/	2/ 0.10	/	5.0/ 0.25	12.57499
NNW	/	/	2/ 0.10	5/ 0.25	2/ 0.10	/	/	9.0/ 0.45	9.24722
TOTAL	/	5/ 0.25	35/ 1.75	49/ 2.45	13/ 0.65	5/ 0.25	/	107.0/ 5.35	8.98621

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	3/ 0.15	15/ 0.75	8/ 0.40	/	/	/	26.0/ 1.30	6.50769
NNE	/	3/ 0.15	23/ 1.15	25/ 1.25	5/ 0.25	/	/	56.0/ 2.80	7.85446
NE	/	5/ 0.25	12/ 0.60	30/ 1.50	7/ 0.35	1/ 0.05	/	55.0/ 2.75	9.46091
ENE	/	3/ 0.15	23/ 1.15	10/ 0.50	1/ 0.05	/	/	37.0/ 1.85	6.44054
E	/	4/ 0.20	9/ 0.45	9/ 0.45	/	/	/	22.0/ 1.10	6.61477
ESE	/	3/ 0.15	14/ 0.70	3/ 0.15	/	/	/	20.0/ 1.00	5.50750
SE	/	3/ 0.15	20/ 1.00	6/ 0.30	/	/	/	29.0/ 1.45	6.05690
SSE	/	5/ 0.25	24/ 1.20	17/ 0.85	3/ 0.15	/	/	49.0/ 2.45	7.39150
S	/	8/ 0.40	12/ 0.60	20/ 1.00	4/ 0.20	1/ 0.05	/	45.0/ 2.25	7.89555
SSW	/	3/ 0.15	24/ 1.20	13/ 0.65	3/ 0.15	1/ 0.05	/	44.0/ 2.20	7.49337
SW	/	5/ 0.25	28/ 1.40	19/ 0.95	12/ 0.60	/	/	64.0/ 3.20	8.55742
WSW	/	2/ 0.10	19 /0.95	33/ 1.65	5/ 0.25	/	/	59.0/ 2.95	8.50254
W	/	/	14/ 0.70	16/ 0.80	1/ 0.05	/	/	31.0/ 1.55	7.53790
WNW	/	2/ 0.10	9/ 0.45	9/ 0.45	1/ 0.05	1/ 0.05	/	22.0/ 1.10	8.08182
NW	/	7/ 0.35	15/ 0.75	9/ 0.45	5/ 0.25	/	/	36.0/ 1.80	7.68819
NNW	/	4/ 0.20	15/ 0.75	9/ 0.45	3/ 0.15	/	/	31.0/ 1.55	7.34274
TOTAL	/	60/ 3.00	276/13.81	236/11.81	50/ 2.50	4/ 0.20	/	626.0/31.32	7.68597

NUMBER OF BAD RECORDS: 39

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	4/ 0.20	6/ 0.30	/	/	/	11.0/ 0.55	7.57273
NNE	/	/	5/ 0.25	7/ 0.35	7/ 0.35	/	/	19.0/ 0.95	10.05263
NE	/	2/ 0.10	6/ 0.30	6/ 0.30	3/ 0.15	/	/	17.0/ 0.85	8.47647
ENE	/	3/ 0.15	5/ 0.25	7/ 0.35	1/ 0.05	/	/	16.0/ 0.80	7.79375
E	/	/	13/ 0.65	8/ 0.40	1/ 0.05	/	/	22.0/ 1.10	6.98182
ESE	/	1/ 0.05	4/ 0.20	3/ 0.15	/	/	/	8.0/ 0.40	6.98437
SE	/	4/ 0.20	13/ 0.65	9/ 0.45	/	/	/	26.0/ 1.30	6.52211
SSE	/	2/ 0.10	11/ 0.55	20/ 1.00	1/ 0.05	/	/	34.0/ 1.70	7.69485
S	/	3/ 0.15	21/ 1.05	72/ 3.60	7/ 0.35	/	/	103.0/ 5.15	8.96893
SSW	/	2/ 0.10	20/ 1.00	56/ 2.80	7/ 0.35	/	/	85.0/ 4.25	8.96382
SW	/	/	21/ 1.05	33/ 1.65	4/ 0.20	1/ 0.05	/	59.0/ 2.95	8.75763
WSW	/	5/ 0.25	9/ 0.45	28/ 1.40	3/ 0.15	/	/	45.0/ 2.25	8.55333
W	/	3/ 0.15	6/ 0.30	10/ 0.50	/	/	/	19.0/ 0.95	7.18553
WNW	/	3/ 0.15	4/ 0.20	2/ 0.10	8/ 0.40	/	/	17.0/ 0.85	9.25441
NW	/	3/ 0.15	4/ 0.20	2/ 0.10	3/ 0.15	/	/	12.0/ 0.60	7.69792
NNW	/	5/ 0.25	7/ 0.35	11/ 0.55	/	/	/	23.0/ 1.15	7.06196
TOTAL	/	37/ 1.85	153/ 7.65	280/14.01	45/ 2.25	1/ 0.05	/	516.0/25.81	8.37161

NUMBER OF BAD RECORDS: 23

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	1/ 0.05	3/ 0.15	9/ 0.45	/	/	14.0/ 0.70	12.18571
NNE	/	/	4/ 0.20	1/ 0.05	4/ 0.20	/	/	9.0/ 0.45	9.63333
NE	/	/	4/ 0.20	9/ 0.45	4/ 0.20	/	/	17.0/ 0.85	10.13382
ENE	/	/	3/ 0.15	5/ 0.25	1/ 0.05	/	/	9.0/ 0.45	9.38889
E	/	/	4/ 0.20	3/ 0.15	/	/	/	7.0/ 0.35	8.27857
ESE	/	/	9/ 0.45	19/ 0.95	2/ 0.10	/	/	30.0/ 1.50	8.77750
SE	/	1/ 0.05	6/ 0.30	9/ 0.45	/	/	/	16.0/ 0.80	7.72344
SSE	/	/	7/ 0.35	9/ 0.45	1/ 0.05	/	/	17.0/ 0.85	8.61471
S	/	/	17/ 0.85	13/ 0.65	2/ 0.10	/	/	32.0/ 1.60	7.84609
SSW	/	4/ 0.20	11/ 0.55	23/ 1.15	/	/	/	38.0/ 1.90	7.95921
SW	/	4/ 0.20	10/ 0.50	38/ 1.90	4/ 0.20	/	/	56.0/ 2.80	8.71875
WSW	/	2/ 0.10	12/ 0.60	33/ 1.65	/	/	/	47.0/ 2.35	8.01383
W	/	/	8/ 0.40	4/ 0.20	1/ 0.05	/	/	13.0/ 0.65	7.64423
WNW	/	/	3/ 0.15	2/ 0.10	1/ 0.05	/	/	6.0/ 0.30	8.52500
NW	/	1/ 0.05	1/ 0.05	2/ 0.10	1/ 0.05	/	/	5.0/ 0.25	7.80500
NNW	/	1/ 0.05	4/ 0.20	2/ 0.10	2/ 0.10	/	/	9.0/ 0.45	7.54722
TOTAL	/	14/ 0.70	104/ 5.20	175/ 8.75	32/ 1.60	/	/	325.0/16.26	8.55777

NUMBER OF BAD RECORDS: 16

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=06 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	7/ 0.35	7/ 0.35	3/ 0.15	/	/	17.0/ 0.85	8.45588
NNE	/	/	5/ 0.25	11/ 0.55	/	/	/	16.0/ 0.80	8.96094
NE	/	1/ 0.05	5/ 0.25	7/ 0.35	1/ 0.05	/	/	14.0/ 0.70	8.36428
ENE	/	1/ 0.05	6/ 0.30	4/ 0.20	2/ 0.10	/	/	13.0/ 0.65	7.05961
E	/	/	4/ 0.20	10/ 0.50	1/ 0.05	/	/	15.0/ 0.75	9.77833
ESE	/	/	4/ 0.20	11/ 0.55	/	/	/	15.0/ 0.75	8.97167
SE	/	1/ 0.05	11/ 0.55	4/ 0.20	/	/	/	16.0/ 0.80	6.33906
SSE	/	/	5/ 0.25	12/ 0.60	1/ 0.05	/	/	18.0/ 0.90	8.98194
S	/	1/ 0.05	12/ 0.60	12/ 0.60	/	/	/	25.0/ 1.25	7.11100
SSW	/	1/ 0.05	19/ 0.95	26/ 1.30	/	/	/	46.0/ 2.30	7.88859
SW	/	4/ 0.20	10/ 0.50	15/ 0.75	/	/	/	29.0/ 1.45	7.32069
WSW	/	2/ 0.10	12/ 0.60	10/ 0.50	/	/	/	24.0/ 1.20	6.55417
W	/	1/ 0.05	16/ 0.80	8/ 0.40	/	/	/	25.0/ 1.25	6.46800
WNW	/	2/ 0.10	7/ 0.35	1/ 0.05	/	/	/	10.0/ 0.50	5.40000
NW	/	4/ 0.20	10/ 0.50	2/ 0.10	/	/	/	16.0/ 0.80	4.86094
NNW	/	2/ 0.10	12/ 0.60	5/ 0.25	2/ 0.10	/	/	21.0/ 1.05	7.20357
TOTAL	/	20/ 1.00	145/ 7.25	145/ 7.25	10/ 0.50	/	/	320.0/16.01	7.48547

NUMBER OF BAD RECORDS: 55

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=07 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	5/ 0.24	32/ 1.54	30/ 1.45	7/ 0.34	1/ 0.05	/	75.1/ 3.62	8.07917
NNE	0.1/ 0.00	9/ 0.43	18/ 0.87	55/ 2.65	7/ 0.34	/	/	89.1/ 4.29	8.55920
NE	0.1/ 0.00	11/ 0.53	28/ 1.35	50/ 2.41	4/ 0.19	/	/	93.1/ 4.48	7.65449
ENE	0.1/ 0.00	11/ 0.53	27/ 1.30	41/ 1.97	8/ 0.39	/	/	87.1/ 4.20	7.98077
E	0.1/ 0.00	10/ 0.48	26/ 1.25	49/ 2.36	5/ 0.24	/	/	90.1/ 4.34	7.71365
ESE	0.1/ 0.00	9/ 0.43	26/ 1.25	32/ 1.54	1/ 0.05	/	/	68.1/ 3.28	6.99339
SE	0.1/ 0.00	10/ 0.48	33/ 1.59	37/ 1.78	/	/	/	80.1/ 3.86	6.90808
SSE	0.1/ 0.00	8/ 0.39	47/ 2.26	57/ 2.75	4/ 0.19	/	/	116.1/ 5.59	7.45392
S	0.1/ 0.00	11/ 0.53	65/ 3.13	98/ 4.72	8/ 0.39	/	/	182.1/ 8.77	8.01991
SSW	0.2/ 0.01	15/ 0.72	64/ 3.08	130/ 6.26	14/ 0.67	1/ 0.05	/	224.2/10.80	8.53557
SW	0.1/ 0.00	9/ 0.43	77/ 3.71	160/ 7.71	14/ 0.67	6/ 0.29	/	266.1/12.82	8.62378
WSW	0.1/ 0.00	11/ 0.53	107/ 5.15	143/ 6.89	14/ 0.67	1/ 0.05	/	276.1/13.30	8.10309
W	0.3/ 0.01	24/ 1.16	70/ 3.37	53/ 2.55	15/ 0.72	2/ 0.10	/	164.3/ 7.91	7.42328
WNW	0.1/ 0.00	8/ 0.39	56/ 2.70	33/ 1.59	3/ 0.14	/	/	100.1/ 4.82	6.92383
NW	0.1/ 0.00	11/ 0.53	39/ 1.88	28/ 1.35	2/ 0.10	/	/	80.1/ 3.86	6.42603
NNW	0.1/ 0.00	10/ 0.48	43/ 2.07	24/ 1.16	7/ 0.34	/	/	84.1/ 4.05	7.04697
TOTAL	2.0/ 0.10	172/ 8.29	758/36.51	1020/49.13	113/ 5.44	11/ 0.53	/	2076/ 100	7.84844

NUMBER OF BAD RECORDS: 156

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=07 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	2/ 0.10	2/ 0.10	/	/	/	4.0/ 0.19	7.24583
NNE	/	1/ 0.05	2/ 0.10	7/ 0.34	/	/	/	10.0/ 0.48	7.89750
NE	/	/	2/ 0.10	5/ 0.24	/	/	/	7.0/ 0.34	8.35357
ENE	/	1/ 0.05	2/ 0.10	3/ 0.14	/	/	/	6.0/ 0.29	6.84167
E	/	/	/	4/ 0.19	/	/	/	4.0/ 0.19	9.35625
ESE	/	/	1/ 0.05	2/ 0.10	/	/	/	3.0/ 0.14	7.00000
SE	/	/	/	/	/	/	/	/	
SSE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	6.21250
S	/	1/ 0.05	1/ 0.05	3/ 0.14	/	/	/	5.0/ 0.24	6.60500
SSW	/	/	1/ 0.05	3/ 0.14	1/ 0.05	1/ 0.05	/	6.0/ 0.29	12.22083
SW	/	/	2/ 0.10	8/ 0.39	5/ 0.24	5/ 0.24	/	20.0/ 0.96	12.98125
WSW	/	/	2/ 0.10	10/ 0.48	1/ 0.05	/	/	13.0/ 0.63	9.11731
W	/	1/ 0.05	6/ 0.29	3/ 0.14	1/ 0.05	/	/	11.0/ 0.53	7.00909
WNW	/	/	4/ 0.19	8/ 0.39	/	/	/	12.0/ 0.58	8.26458
NW	/	/	4/ 0.19	3/ 0.14	/	/	/	7.0/ 0.34	6.75714
NNW	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	7.03750
TOTAL	/	4/ 0.19	31/ 1.49	63/ 3.03	8/ 0.39	6/ 0.29	/	112.0/ 5.39	8.93289

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=07 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	4/ 0.19	3/ 0.14	1/ 0.05	/	/	8.0/ 0.39	8.86562
NNE	/	/	/	2/ 0.10	/	/	/	2.0/ 0.10	9.91250
NE	/	/	3/ 0.14	11/ 0.53	/	/	/	14.0/ 0.67	8.62678
ENE	/	/	1/ 0.05	2/ 0.10	/	/	/	3.0/ 0.14	8.38333
E	/	/	1/ 0.05	4/ 0.19	/	/	/	5.0/ 0.24	8.36500
ESE	/	/	1/ 0.05	3/ 0.14	/	/	/	4.0/ 0.19	8.18750
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	11.17500
S	/	/	2/ 0.10	4/ 0.19	/	/	/	6.0/ 0.29	9.08750
SSW	/	/	4/ 0.19	4/ 0.19	1/ 0.05	/	/	9.0/ 0.43	8.88055
SW	/	/	5/ 0.24	8/ 0.39	1/ 0.05	/	/	14.0/ 0.67	8.79821
WSW	/	1/ 0.05	6/ 0.29	9/ 0.43	3/ 0.14	/	/	19.0/ 0.92	8.98684
W	/	/	4/ 0.19	4/ 0.19	2/ 0.10	/	/	10.0/ 0.48	8.33500
WNW	/	/	8/ 0.39	8/ 0.39	/	/	/	16.0/ 0.77	7.30937
NW	/	/	1/ 0.05	8/ 0.39	/	/	/	9.0/ 0.43	9.03889
NNW	/	/	3/ 0.14	3/ 0.14	/	/	/	6.0/ 0.29	8.17083
TOTAL	/	1/ 0.05	43/ 2.07	74/ 3.56	8/ 0.39	/	/	126.0/ 6.07	8.58313

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=07 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	4/ 0.19	4/ 0.19	/	/	/	9.0/ 0.43	6.58333
NNE	/	/	1/ 0.05	3/ 0.14	1/ 0.05	/	/	5.0/ 0.24	9.94500
NE	/	/	5/ 0.24	4/ 0.19	/	/	/	9.0/ 0.43	7.58056
ENE	/	1/ 0.05	5/ 0.24	5/ 0.24	/	/	/	11.0/ 0.53	7.13409
E	/	1/ 0.05	4/ 0.19	5/ 0.24	/	/	/	10.0/ 0.48	6.91000
ESE	/	1/ 0.05	2/ 0.10	/	/	/	/	3.0/ 0.14	3.85000
SE	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	5.61250
SSE	/	2/ 0.10	3/ 0.14	3/ 0.14	/	/	/	8.0/ 0.39	6.08437
S	/	/	2/ 0.10	6/ 0.29	/	/	/	8.0/ 0.39	8.93750
SSW	/	1/ 0.05	6/ 0.29	4/ 0.19	/	/	/	11.0/ 0.53	7.01364
SW	/	1/ 0.05	3/ 0.14	6/ 0.29	1/ 0.05	/	/	11.0/ 0.53	7.89545
WSW	/	/	12/ 0.58	13/ 0.63	/	/	/	25.0/ 1.20	7.51300
W	/	1/ 0.05	12/ 0.58	7/ 0.34	2/ 0.10	/	/	22.0/ 1.06	7.90568
WNW	/	1/ 0.05	5/ 0.24	2/ 0.10	/	/	/	8.0/ 0.39	6.93125
NW	/	1/ 0.05	3/ 0.14	7/ 0.34	/	/	/	11.0/ 0.53	7.46364
NNW	/	/	7/ 0.34	4/ 0.19	/	/	/	11.0/ 0.53	7.20682
TOTAL	/	11/ 0.53	76/ 3.66	73/ 3.52	4/ 0.19	/	/	164.0/ 7.90	7.37988

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=07 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	3/ 0.14	10/ 0.48	5/ 0.24	1/ 0.05	/	/	19.0/ 0.92	6.00526
NNE	/	5/ 0.24	8/ 0.39	15/ 0.72	2/ 0.10	/	/	30.0/ 1.45	7.72417
NE	/	3/ 0.14	8/ 0.39	17/ 0.82	1/ 0.05	/	/	29.0/ 1.40	7.88391
ENE	/	6/ 0.29	11/ 0.53	5/ 0.24	/	/	/	22.0/ 1.06	5.75000
E	/	3/ 0.14	12/ 0.58	2/ 0.10	/	/	/	17.0/ 0.82	5.50000
ESE	/	4/ 0.19	6/ 0.29	2/ 0.10	/	/	/	12.0/ 0.58	4.86042
SE	/	4/ 0.19	10/ 0.48	4/ 0.19	/	/	/	18.0/ 0.87	5.37222
SSE	/	1/ 0.05	15/ 0.72	11/ 0.53	3/ 0.14	/	/	30.0/ 1.45	7.93500
S	/	2/ 0.10	21/ 1.01	17/ 0.82	3/ 0.14	/	/	43.0/ 2.07	7.78546
SSW	/	4/ 0.19	10/ 0.48	13/ 0.63	7/ 0.34	/	/	34.0/ 1.64	8.72573
SW	/	2/ 0.10	21/ 1.01	26/ 1.25	3/ 0.14	/	/	52.0/ 2.50	7.99567
WSW	/	4/ 0.19	23/ 1.11	22/ 1.06	4/ 0.19	1/ 0.05	/	54.0/ 2.60	8.24305
W	/	8/ 0.39	21/ 1.01	12/ 0.58	8/ 0.39	2/ 0.10	/	51.0/ 2.46	8.34281
WNW	/	3/ 0.14	11/ 0.53	6/ 0.29	1/ 0.05	/	/	21.0/ 1.01	6.78452
NW	/	2/ 0.10	13/ 0.63	2/ 0.10	1/ 0.05	/	/	18.0/ 0.87	5.87222
NNW	/	5/ 0.24	11/ 0.53	5/ 0.24	1/ 0.05	/	/	22.0/ 1.06	5.82386
TOTAL	/	59/ 2.84	211/10.16	164/ 7.90	35/ 1.69	3/ 0.14	/	472.0/22.74	7.37641

NUMBER OF BAD RECORDS: 11

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=07 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	4/ 0.19	8/ 0.39	1/ 0.05	1/ 0.05	/	14.0/ 0.67	10.25357
NNE	/	1/ 0.05	1/ 0.05	10/ 0.48	/	/	/	12.0/ 0.58	8.90000
NE	/	2/ 0.10	5/ 0.24	6/ 0.29	1/ 0.05	/	/	14.0/ 0.67	7.17143
ENE	/	/	6/ 0.29	9/ 0.43	1/ 0.05	/	/	16.0/ 0.77	8.65937
E	/	2/ 0.10	6/ 0.29	12/ 0.58	1/ 0.05	/	/	21.0/ 1.01	7.67024
ESE	/	/	7/ 0.34	4/ 0.19	/	/	/	11.0/ 0.53	6.99091
SE	/	1/ 0.05	4/ 0.19	8/ 0.39	/	/	/	13.0/ 0.63	7.55769
SSE	/	1/ 0.05	14/ 0.67	25/ 1.20	1/ 0.05	/	/	41.0/ 1.97	7.84756
S	/	2/ 0.10	16/ 0.77	47/ 2.26	3/ 0.14	/	/	68.0/ 3.28	8.78309
SSW	/	1/ 0.05	21/ 1.01	51/ 2.46	1/ 0.05	/	/	74.0/ 3.56	8.89932
SW	/	2/ 0.10	22/ 1.06	32/ 1.54	4/ 0.19	1/ 0.05	/	61.0/ 2.94	8.50164
WSW	/	3/ 0.14	32/ 1.54	29/ 1.40	6/ 0.29	/	/	70.0/ 3.37	8.10678
W	/	4/ 0.19	7/ 0.34	16/ 0.77	2/ 0.10	/	/	29.0/ 1.40	7.76552
WNW	/	/	9/ 0.43	5/ 0.24	1/ 0.05	/	/	15.0/ 0.72	7.69333
NW	/	4/ 0.19	7/ 0.34	5/ 0.24	/	/	/	16.0/ 0.77	5.83125
NNW	/	2/ 0.10	9/ 0.43	5/ 0.24	1/ 0.05	/	/	17.0/ 0.82	7.09118
TOTAL	/	25/ 1.20	70/ 8.19	272/13.10	23/ 1.11	2/ 0.10	/	492.0/23.70	8.21870

NUMBER OF BAD RECORDS: 7

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=07 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	2/ 0.10	8/ 0.39	4/ 0.19	/	/	15.0/ 0.72	10.58500
NNE	/	/	4/ 0.19	7/ 0.34	1/ 0.05	/	/	12.0/ 0.58	9.15625
NE	/	1/ 0.05	1/ 0.05	4/ 0.19	/	/	/	6.0/ 0.29	8.12917
ENE	/	1/ 0.05	/	11/ 0.53	1/ 0.05	/	/	13.0/ 0.63	9.57308
E	/	2/ 0.10	2/ 0.10	12/ 0.58	4/ 0.19	/	/	20.0/ 0.96	9.42875
ESE	/	/	6/ 0.29	12/ 0.58	/	/	/	18.0/ 0.87	8.13333
SE	/	2/ 0.10	9/ 0.43	9/ 0.43	/	/	/	20.0/ 0.96	7.15375
SSE	/	1/ 0.05	5/ 0.24	10/ 0.48	/	/	/	16.0/ 0.77	7.70469
S	/	1/ 0.05	11/ 0.53	16/ 0.77	2/ 0.10	/	/	30.0/ 1.45	8.37750
SSW	/	1/ 0.05	6/ 0.29	32/ 1.54	3/ 0.14	/	/	42.0/ 2.02	9.43988
SW	/	1/ 0.05	12/ 0.58	51/ 2.46	/	/	/	64.0/ 3.08	8.55000
WSW	/	1/ 0.05	13/ 0.63	21/ 1.01	/	/	/	35.0/ 1.69	7.85643
W	/	2/ 0.10	7/ 0.34	5/ 0.24	/	/	/	14.0/ 0.67	6.56786
WNW	/	/	2/ 0.10	2/ 0.10	/	/	/	4.0/ 0.19	6.58125
NW	/	2/ 0.10	3/ 0.14	2/ 0.10	1/ 0.05	/	/	8.0/ 0.39	5.99687
NNW	/	/	7/ 0.34	1/ 0.05	3/ 0.14	/	/	11.0/ 0.53	8.93409
TOTAL	/	16/ 0.77	90/ 4.34	203/ 9.78	19/ 0.92	/	/	328.0/ 15.80	8.46860

NUMBER OF BAD RECORDS: 7

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=07 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	6/ 0.29	/	/	/	/	6.0/ 0.29	5.16250
NNE	0.1/ 0.00	2/ 0.10	2/ 0.10	11/ 0.53	3/ 0.14	/	/	18.1/ 0.87	9.15470
NE	0.2/ 0.01	5/ 0.24	4/ 0.19	3/ 0.14	2/ 0.10	/	/	14.2/ 0.68	6.16197
ENE	0.1/ 0.00	2/ 0.10	2/ 0.10	6/ 0.29	6/ 0.29	/	/	16.1/ 0.78	9.99689
E	0.1/ 0.00	2/ 0.10	1/ 0.05	10/ 0.48	/	/	/	13.1/ 0.63	7.89790
ESE	0.1/ 0.00	4/ 0.19	3/ 0.14	9/ 0.43	1/ 0.05	/	/	17.1/ 0.82	7.56286
SE	0.1/ 0.00	3/ 0.14	8/ 0.39	16/ 0.77	/	/	/	27.1/ 1.31	7.53090
SSE	0.1/ 0.00	3/ 0.14	9/ 0.43	6/ 0.29	/	/	/	18.1/ 0.87	6.07804
S	0.2/ 0.01	5/ 0.24	12/ 0.58	5/ 0.24	/	/	/	22.2/ 1.07	5.31982
SSW	0.3/ 0.01	8/ 0.39	16/ 0.77	23/ 1.11	1/ 0.05	/	/	48.3/ 2.33	6.86646
SW	0.1/ 0.00	3/ 0.14	12/ 0.58	29/ 1.40	/	/	/	44.1/ 2.12	7.78883
WSW	0.1/ 0.00	2/ 0.10	19/ 0.92	39/ 1.88	/	/	/	60.1/ 2.89	7.86210
W	0.3/ 0.01	8/ 0.39	13/ 0.63	6/ 0.29	/	/	/	27.3/ 1.32	5.21657
WNW	0.1/ 0.00	4/ 0.19	17/ 0.82	2/ 0.10	1/ 0.05	/	/	24.1/ 1.16	5.69709
NW	0.1/ 0.00	2/ 0.10	8/ 0.39	1/ 0.05	/	/	/	11.1/ 0.53	5.13176
NNW	0.1/ 0.00	3/ 0.14	5/ 0.24	5/ 0.24	2/ 0.10	/	/	15.1/ 0.73	6.84271
TOTAL	2.0/ 0.10	56/ 2.70	137/ 6.60	171/ 8.24	16/ 0.77	/	/	382.0/18.40	7.06220

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=08 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	11/ 0.50	37/ 1.69	36/ 1.65	7/ 0.32	/	/	91.1/ 4.17	7.97530
NNE	0.1/ 0.00	16/ 0.73	43/ 1.97	68/ 3.11	10/ 0.46	1/ 0.05	/	138.1/ 6.31	8.21153
NE	0.1/ 0.00	10/ 0.46	44/ 2.01	88/ 4.02	52/ 2.38	1/ 0.05	/	195.1/ 8.92	9.79940
ENE	0.0/ 0.00	8/ 0.37	55/ 2.51	50/ 2.29	20/ 0.91	/	/	133.0/ 6.08	8.14699
E	0.1/ 0.00	11/ 0.50	59/ 2.70	35/ 1.60	/	/	/	105.1/ 4.81	6.71087
ESE	0.1/ 0.00	11/ 0.50	56/ 2.56	30/ 1.37	1/ 0.05	/	/	98.1/ 4.49	6.59225
SE	0.1/ 0.00	10/ 0.46	42/ 1.92	23/ 1.05	1/ 0.05	/	/	76.1/ 3.48	6.61399
SSE	0.1/ 0.00	13/ 0.59	61/ 2.79	63/ 2.88	4/ 0.18	/	/	141.1/ 6.45	7.47289
S	0.0/ 0.00	9/ 0.41	85/ 3.89	150/ 6.86	12/ 0.55	1/ 0.05	/	257.0/11.75	8.43268
SSW	0.1/ 0.00	16/ 0.73	114/ 5.21	197/ 9.01	19/ 0.87	1/ 0.05	/	347.1/15.87	8.36067
SW	0.1/ 0.00	17/ 0.78	103/ 4.71	158/ 7.22	9/ 0.41	1/ 0.05	/	288.1/13.17	7.88420
WSW	0.1/ 0.00	14/ 0.64	63/ 2.88	68/ 3.11	8/ 0.37	/	/	153.1/ 7.00	7.37938
W	0.0/ 0.00	9/ 0.41	23/ 1.05	14/ 0.64	2/ 0.09	/	/	48.0/ 2.19	6.52969
WNW	0.0/ 0.00	6/ 0.27	17/ 0.78	9/ 0.41	/	/	/	32.0/ 1.46	5.71250
NW	0.1/ 0.00	11/ 0.50	21/ 0.96	8/ 0.37	/	/	/	40.1/ 1.83	5.48784
NNW	0.0/ 0.00	8/ 0.37	24/ 1.10	12/ 0.55	/	/	/	44.0/ 2.01	5.92462
TOTAL	1.0/ 0.05	180/ 8.23	847/38.73	1009/46.14	145/ 6.63	5/ 0.23	/	2187/ 100	7.87065

NUMBER OF BAD RECORDS: 45

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=08 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	/	4/ 0.18	1/ 0.05	/	/	5.0/ 0.23	11.58000
NNE	/	/	1/ 0.05	4/ 0.18	5/ 0.23	/	/	10.0/ 0.46	12.14750
NE	/	/	2/ 0.09	10/ 0.46	10/ 0.46	1/ 0.05	/	23.0/ 10.5	12.35543
ENE	/	/	4/ 0.18	6/ 0.27	3/ 0.14	/	/	13.0/ 0.59	9.70961
E	/	/	3/ 0.14	2/ 0.09	/	/	/	5.0/ 0.23	7.18500
ESE	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	5.44167
SE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.52500
SSE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.09	7.63750
S	/	/	/	2/ 0.09	/	/	/	2.0/ 0.09	10.70000
SSW	/	/	3/ 0.14	9/ 0.41	2/ 0.09	/	/	14.0/ 0.64	9.63393
SW	/	/	5/ 0.23	5/ 0.23	1/ 0.05	/	/	11.0/ 0.50	8.50909
WSW	/	/	2/ 0.09	1/ 0.05	1/ 0.05	/	/	4.0/ 0.18	8.84375
W	/	/	/	/	/	/	/	/	
WNW	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	4.32500
NW	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.57500
NNW	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	6.02500
TOTAL	/	/	29/ 1.33	44/ 2.01	23/ 1.05	1/ 0.05	/	97.0/ 4.44	9.98273

NUMBER OF BAD RECORDS: 4

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=08 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	3/ 0.14	3/ 0.14	1/ 0.05	/	/	7.0/ 0.32	9.16071
NNE	/	1/ 0.05	6/ 0.27	5/ 0.23	/	/	/	12.0/ 0.55	7.25000
NE	/	1/ 0.05	3/ 0.14	2/ 0.09	6/ 0.27	/	/	12.0/ 0.55	10.33542
ENE	/	/	3/ 0.14	2/ 0.09	2/ 0.09	/	/	7.0/ 0.32	8.61071
E	/	/	5/ 0.23	/	/	/	/	5.0/ 0.23	5.71000
ESE	/	1/ 0.05	6/ 0.27	/	/	/	/	7.0/ 0.32	5.18571
SE	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.09	3.63750
SSE	/	1/ 0.05	6/ 0.27	2/ 0.09	/	/	/	9.0/ 0.41	5.82500
S	/	/	2/ 0.09	6/ 0.27	2/ 0.09	/	/	10.0/ 0.46	9.57750
SSW	/	/	7/ 0.32	8/ 0.37	/	/	/	15.0/ 0.69	7.61833
SW	/	1/ 0.05	5/ 0.23	11/ 0.50	1/ 0.05	/	/	18.0/ 0.82	8.41111
WSW	/	2/ 0.09	4/ 0.18	3/ 0.14	1/ 0.05	/	/	10.0/ 0.46	6.88083
W	/	/	5/ 0.23	/	/	/	/	5.0/ 0.23	5.72500
WNW	/	/	3/ 0.14	2/ 0.09	/	/	/	5.0/ 0.23	6.60500
NW	/	/	5/ 0.23	/	/	/	/	5.0/ 0.23	5.86000
NNW	/	/	4/ 0.18	1/ 0.05	/	/	/	5.0/ 0.23	5.06000
TOTAL	/	8/ 0.37	68/ 3.11	45/ 2.06	13/ 0.59	/	/	134.0/ 6.13	7.51107

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=08 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	3/ 0.14	4/ 0.18	/	/	/	/	7.0/ 0.32	4.78929
NNE	/	2/ 0.09	7/ 0.32	4/ 0.18	1/ 0.05	/	/	14.0/ 0.64	6.67500
NE	/	/	4/ 0.18	3/ 0.14	3/ 0.14	/	/	10.0/ 0.46	9.31500
ENE	/	/	5/ 0.23	2/ 0.09	1/ 0.05	/	/	8.0/ 0.37	8.05000
E	/	1/ 0.05	7/ 0.32	2/ 0.09	/	/	/	10.0/ 0.46	6.35500
ESE	/	1/ 0.05	5/ 0.23	/	/	/	/	6.0/ 0.27	4.55833
SE	/	1/ 0.05	5/ 0.23	/	/	/	/	6.0/ 0.27	5.40833
SSE	/	1/ 0.05	4/ 0.18	3/ 0.14	/	/	/	8.0/ 0.37	7.26875
S	/	/	11/ 0.50	7/ 0.32	2/ 0.09	/	/	20.0/ 0.91	7.76750
SSW	/	3/ 0.14	4/ 0.18	12/ 0.55	2/ 0.09	/	/	21.0/ 0.96	8.37619
SW	/	/	15/ 0.69	15/ 0.69	3/ 0.14	/	/	33.0/ 1.51	8.52121
WSW	/	3/ 0.14	10/ 0.46	19/ 0.87	3/ 0.14	/	/	35.0/ 1.60	8.12643
W	/	/	7/ 0.32	/	/	/	/	7.0/ 0.32	5.84286
WNW	/	/	4/ 0.18	/	/	/	/	4.0/ 0.18	6.15625
NW	/	3/ 0.14	3/ 0.14	2/ 0.09	/	/	/	8.0/ 0.37	5.50937
NNW	/	1/ 0.05	3/ 0.14	3/ 0.14	/	/	/	7.0/ 0.32	7.66548
TOTAL	/	19/ 0.87	98/ 4.48	72/ 3.29	15/ 0.69	/	/	204.0/ 9.33	7.48117

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=08 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	3/ 0.14	11/ 0.50	6/ 0.27	4/ 0.18	/	/	24.0/ 1.10	7.81354
NNE	/	3/ 0.14	7/ 0.32	11/ 0.50	1/ 0.05	1/ 0.05	/	23.0/ 1.05	8.43478
NE	/	3/ 0.14	12/ 0.55	17/ 0.78	24/ 1.10	/	/	56.0/ 2.56	10.65446
ENE	/	3/ 0.14	16/ 0.73	9/ 0.41	5/ 0.23	/	/	33.0/ 1.51	7.32576
E	/	2/ 0.09	18/ 0.82	2/ 0.09	/	/	/	22.0/ 1.01	5.26932
ESE	/	3/ 0.14	15/ 0.69	3/ 0.14	/	/	/	21.0/ 0.96	5.42738
SE	/	3/ 0.14	11/ 0.50	6/ 0.27	/	/	/	20.0/ 0.91	6.29250
SSE	/	3/ 0.14	17/ 0.78	15/ 0.69	2/ 0.09	/	/	37.0/ 1.69	7.67230
S	/	3/ 0.14	23/ 1.05	30/ 1.37	6/ 0.27	1/ 0.05	/	63.0/ 2.88	8.37659
SSW	/	7/ 0.32	19/ 0.87	29/ 1.33	7/ 0.32	1/ 0.05	/	63.0/ 2.88	8.59921
SW	/	7/ 0.32	28/ 1.28	34/ 1.55	3/ 0.14	1/ 0.05	/	73.0/ 3.34	7.77294
WSW	/	3/ 0.14	22/ 1.01	23/ 1.05	1/ 0.05	/	/	49.0/ 2.24	7.31888
W	/	5/ 0.23	3/ 0.14	3/ 0.14	/	/	/	11.0/ 0.50	5.84773
WNW	/	5/ 0.23	1/ 0.05	1/ 0.05	/	/	/	7.0/ 0.32	3.36429
NW	/	3/ 0.14	6/ 0.27	1/ 0.05	/	/	/	10.0/ 0.46	4.59750
NNW	/	/	5/ 0.23	/	/	/	/	5.0/ 0.23	5.15000
TOTAL	/	56/ 2.56	214/ 9.79	190/ 8.69	53/ 2.42	4/ 0.18	/	517.0/23.64	7.76533

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=08 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	7/ 0.32	11/ 0.50	1/ 0.05	/	/	20.0/ 0.91	8.66375
NNE	/	4/ 0.18	4/ 0.18	21/ 0.96	2/ 0.09	/	/	31.0/ 1.42	9.22661
NE	/	4/ 0.18	12/ 0.55	28/ 1.28	9 /0.41	/	/	53.0/ 2.42	9.07689
ENE	/	3/ 0.14	16/ 0.73	9/ 0.41	3/ 0.14	/	/	31.0/ 1.42	7.49435
E	/	4/ 0.18	12/ 0.55	12/ 0.55	/	/	/	28.0/ 1.28	7.28214
ESE	/	2/ 0.09	16/ 0.73	9/ 0.41	1/ 0.05	/	/	28.0/ 1.28	7.01339
SE	/	1/ 0.05	15/ 0.69	6/ 0.27	1/ 0.05	/	/	23.0/ 1.05	7.05543
SSE	/	2/ 0.09	15/ 0.69	20/ 0.91	1/ 0.05	/	/	38.0/ 1.74	7.74276
S	/	1/ 0.05	31/ 1.42	61/ 2.79	1/ 0.05	/	/	94.0/ 4.30	8.39016
SSW	/	2/ 0.09	35/ 1.60	90/ 4.12	6/ 0.27	/	/	133.0/ 6.08	8.65996
SW	/	3/ 0.14	20/ 0.91	41/ 1.87	1/ 0.05	/	/	65.0/ 2.97	7.95038
WSW	/	2/ 0.09	9/ 0.41	10/ 0.46	2/ 0.09	/	/	23.0/ 1.05	7.26522
W	/	3/ 0.14	/	3/ 0.14	2/ 0.09	/	/	8.0/ 0.37	7.66562
WNW	/	1/ 0.05	5/ 0.23	3/ 0.14	/	/	/	9.0/ 0.41	5.81944
NW	/	/	1/ 0.05	2/ 0.09	/	/	/	3.0/ 0.14	8.15833
NNW	/	2/ 0.09	5/ 0.23	1/ 0.05	/	/	/	8.0/ 0.37	4.89062
TOTAL	/	35/ 1.60	203/ 9.28	327/14.95	30/ 1.37	/	/	595.0/27.21	8.11950

NUMBER OF BAD RECORDS: 22

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=08 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	2/ 0.09	7/ 0.32	6/ 0.27	/	/	/	15.1/ 0.69	7.88824
NNE	0.2/ 0.01	4/ 0.18	3/ 0.14	13/ 0.59	/	/	/	20.2/ 0.92	7.91213
NE	0.0/ 0.00	1/ 0.05	5/ 0.23	15/ 0.69	/	/	/	21.0/ 0.96	8.38214
ENE	0.0/ 0.00	1/ 0.05	7/ 0.32	12/ 0.55	4/ 0.18	/	/	24.0/ 1.10	8.71458
E	0.0/ 0.00	1/ 0.05	7/ 0.32	5/ 0.23	/	/	/	13.0/ 0.59	6.75192
ESE	0.1/ 0.00	2/ 0.09	7/ 0.32	6/ 0.27	/	/	/	15.1/ 0.69	6.75248
SE	0.0/ 0.00	1/ 0.05	4/ 0.18	1/ 0.05	/	/	/	6.0/ 0.27	6.60000
SSE	0.1/ 0.00	2/ 0.09	6/ 0.27	14/ 0.64	1/ 0.05	/	/	23.1/ 1.06	8.26299
S	0.0/ 0.00	1/ 0.05	9/ 0.41	34/ 1.55	1/ 0.05	/	/	45.0/ 2.06	9.38889
SSW	/	/	31/ 1.42	36/ 1.65	2/ 0.09	/	/	69.0/ 3.16	8.12065
SW	0.1/ 0.00	2/ 0.09	14/ 0.64	41/ 1.87	/	/	/	57.1/ 2.61	8.12894
WSW	0.1/ 0.00	2/ 0.09	9/ 0.41	7/ 0.32	/	/	/	18.1/ 0.83	6.78660
W	/	/	6/ 0.27	8/ 0.37	/	/	/	14.0/ 0.64	7.62857
WNW	/	/	2/ 0.09	3/ 0.14	/	/	/	5.0/ 0.23	8.11500
NW	0.1/ 0.00	3/ 0.14	/	3/ 0.14	/	/	/	6.1/ 0.28	5.59426
NNW	0.2/ 0.01	4/ 0.18	4/ 0.18	2/ 0.09	/	/	/	10.2/ 0.47	5.31618
TOTAL	1.0/ 0.05	26/ 1.19	121/ 5.53	206/ 9.42	8/ 0.37	/	/	362.0/16.55	7.98308

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=08 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	2/ 0.09	5/ 0.23	6/ 0.27	/	/	/	13.0/ 0.59	7.00385
NNE	/	2/ 0.09	15/ 0.69	10/ 0.46	1/ 0.05	/	/	28.0/ 1.28	6.86696
NE	/	1/ 0.05	6/ 0.27	13/ 0.59	/	/	/	20.0/ 0.91	7.82875
ENE	/	1/ 0.05	4/ 0.18	10/ 0.46	2/ 0.09	/	/	17.0/ 0.78	8.78971
E	/	3/ 0.14	7/ 0.32	12/ 0.55	/	/	/	22.0/ 1.01	7.70795
ESE	/	2/ 0.09	4/ 0.18	12/ 0.55	/	/	/	18.0/ 0.82	8.57639
SE	/	3/ 0.14	5/ 0.23	10/ 0.46	/	/	/	18.0/ 0.82	7.28889
SSE	/	4/ 0.18	12/ 0.55	8/ 0.37	/	/	/	24.0/ 1.10	6.65000
S	/	4/ 0.18	9/ 0.41	10/ 0.46	/	/	/	23.0/ 1.05	6.77283
SSW	/	4/ 0.18	15/ 0.69	13/ 0.59	/	/	/	32.0/ 1.46	6.96562
SW	/	4/ 0.18	16/ 0.73	11/ 0.50	/	/	/	31.0/ 1.42	6.34839
WSW	/	2/ 0.09	7/ 0.32	5/ 0.23	/	/	/	14.0/ 0.64	6.61250
W	/	1/ 0.05	2/ 0.09	/	/	/	/	3.0/ 0.14	3.81667
WNW	/	/	/	/	/	/	/	/	
NW	/	2/ 0.09	5/ 0.23	/	/	/	/	7.0/ 0.32	5.07143
NNW	/	1/ 0.05	2/ 0.09	5/ 0.23	/	/	/	8.0/ 0.37	7.10312
TOTAL	/	36/ 1.65	114/ 5.21	125/ 5.72	3/ 0.14	/	/	278.0/12.71	7.10755

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=09 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	13 0.62	51/ 2.42	50/ 2.38	10/ 0.48	/	/	124.1/ 5.90	7.48932
NNE	0.1/ 0.00	6/ 0.29	77/ 3.66	87/ 4.13	10/ 0.48	/	/	180.1/ 8.56	7.95669
NE	0.2/ 0.01	19/ 0.90	62/ 2.95	140/ 6.65	24/ 1.14	/	/	245.2/11.65	8.46717
ENE	0.1/ 0.00	13/ 0.62	67/ 3.18	56/ 2.66	18/ 0.86	1/ 0.05	/	155.1/ 7.37	8.06557
E	0.1/ 0.00	10/ 0.48	48/ 2.28	42/ 2.00	5/ 0.24	/	/	105.1/ 4.99	7.30558
ESE	0.0/ 0.00	5/ 0.24	43/ 2.04	21/ 1.00	1/ 0.05	/	/	70.0/ 3.33	6.51321
SE	0.1/ 0.00	8/ 0.38	28/ 1.33	14/ 0.67	1/ 0.05	/	/	51.1/ 2.43	6.54110
SSE	0.1/ 0.00	10/ 0.48	60/ 2.85	54/ 2.57	4/ 0.19	/	/	128.1/ 6.09	7.18755
S	0.1/ 0.00	10/ 0.48	69/ 3.28	62/ 2.95	3/ 0.14	/	/	144.1/ 6.85	7.39105
SSW	0.2/ 0.01	20/ 0.95	62/ 2.95	94/ 4.47	4/ 0.19	/	/	180.2/ 8.56	7.38818
SW	0.1/ 0.00	11/ 0.52	81/ 3.85	98/ 4.66	16/ 0.76	/	/	206.1/ 9.79	7.93741
WSW	0.2/ 0.01	18/ 0.86	81/ 3.85	51/ 2.42	9/ 0.43	/	/	159.2/ 7.56	6.83940
W	0.2/ 0.01	16/ 0.76	49/ 2.33	19/ 0.90	/	/	/	84.2/ 4.00	5.61283
WNW	0.2/ 0.01	16/ 0.76	45/ 2.14	20/ 0.95	/	/	/	81.2/ 3.86	5.63362
NW	0.1/ 0.00	14/ 0.67	48/ 2.28	11/ 0.52	/	/	/	73.1/ 3.47	5.28904
NNW	0.2/ 0.01	20/ 0.95	41/ 1.95	46/ 2.19	10/ 0.48	1/ 0.05	/	118.2/ 5.62	7.43020
TOTAL	2.0/ 0.10	209/ 9.93	912/43.33	865/41.09	115/ 5.46	2/ 0.10	/	2105/ 100	7.35597

NUMBER OF BAD RECORDS: 55

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=09 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	9/ 0.43	2/ 0.10	/	/	/	12.0/ 0.57	6.12708
NNE	/	1/ 0.05	10/ 0.48	6/ 0.29	/	/	/	17.0/ 0.81	6.45000
NE	/	1/ 0.05	8/ 0.38	10/ 0.48	/	/	/	19.0/ 0.90	7.59210
ENE	/	1/ 0.05	7/ 0.33	3/ 0.14	/	/	/	11.0/ 0.52	6.83182
E	/	1/ 0.05	6/ 0.29	/	/	/	/	7.0/ 0.33	4.83571
ESE	/	/	12/ 0.57	1/ 0.05	/	/	/	13.0/ 0.62	5.70385
SE	/	/	6/ 0.29	/	/	/	/	6.0/ 0.29	5.67083
SSE	/	1/ 0.05	7/ 0.33	2/ 0.10	/	/	/	10.0/ 0.48	6.45500
S	/	/	16/ 0.76	7/ 0.33	/	/	/	23.0/ 1.09	7.00761
SSW	/	/	10/ 0.48	12/ 0.57	1/ 0.05	/	/	23.0/ 1.09	7.41956
SW	/	1/ 0.05	6/ 0.29	11/ 0.52	4/ 0.19	/	/	22.0/ 1.05	9.02500
WSW	/	3/ 0.14	5/ 0.24	4/ 0.19	3/ 0.14	/	/	15.0/ 0.71	7.89667
W	/	1/ 0.05	3/ 0.14	1/ 0.05	/	/	/	5.0/ 0.24	5.12500
WNW	/	3/ 0.14	7/ 0.33	4/ 0.19	/	/	/	14.0/ 0.67	5.87857
NW	/	/	2/ 0.10	4/ 0.19	/	/	/	6.0/ 0.29	9.29583
NNW	/	1/0.05	6/ 0.29	11/ 0.52	3/ 0.14	1/ 0.05	/	22.0/ 1.05	9.56250
TOTAL	/	15/ 0.71	120/ 5.70	78/ 3.71	11/ 0.52	1/ 0.05	/	225.0/10.69	7.25355

NUMBER OF BAD RECORDS: 6

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=09 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	7/ 0.33	1/ 0.05	/	/	/	9.0/ 0.43	6.01944
NNE	/	/	4/ 0.19	3/ 0.14	1/ 0.05	/	/	8.0/ 0.38	7.93750
NE	/	1/ 0.05	8/ 0.38	8/ 0.38	2/ 0.10	/	/	19.0/ 0.90	7.91316
ENE	/	1/ 0.05	6/ 0.29	2/ 0.10	/	/	/	9.0/ 0.43	6.36111
E	/	/	6/ 0.29	1/ 0.05	/	/	/	7.0/ 0.33	5.83571
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.57500
SE	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.10	3.48750
SSE	/	1/ 0.05	2/ 0.10	3/ 0.14	/	/	/	6.0/ 0.29	6.83333
S	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	7.67500
SSW	/	/	6/ 0.29	4/ 0.19	/	/	/	10.0/ 0.48	7.25500
SW	/	1/ 0.05	2/ 0.10	7/ 0.33	3/ 0.14	/	/	13.0/ 0.62	9.49038
WSW	/	1/ 0.05	2/ 0.10	3/ 0.14	1/ 0.05	/	/	7.0/ 0.33	8.15357
W	/	1/ 0.05	5/ 0.24	3/ 0.14	/	/	/	9.0/ 0.43	6.21667
WNW	/	2/ 0.10	4/ 0.19	6/ 0.29	/	/	/	12.0/ 0.57	6.83958
NW	/	/	4/ 0.19	2/ 0.10	/	/	/	6.0/ 0.29	6.36250
NNW	/	1/ 0.05	2/ 0.10	2/ 0.10	/	/	/	5.0/ 0.24	6.34500
TOTAL	/	11/ 0.52	60/ 2.85	46/ 2.19	7/ 0.33	/	/	124.0/ 5.89	7.16351

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=09 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	7/ 0.33	1/ 0.05	/	/	/	8.0/ 0.38	5.82812
NNE	/	/	6/ 0.29	2/ 0.10	/	/	/	8.0/ 0.38	6.59687
NE	/	/	8/ 0.38	10/ 0.48	2/ 0.10	/	/	20.0/ 0.95	8.13125
ENE	/	/	7/ 0.33	3/ 0.14	/	/	/	10.0/ 0.48	6.94500
E	/	/	6/ 0.29	/	/	/	/	6.0/ 0.29	5.91389
ESE	/	/	4/ 0.19	/	/	/	/	4.0/ 0.19	5.86250
SE	/	1/ 0.05	2/ 0.10	1/ 0.05	/	/	/	4.0/ 0.19	5.28125
SSE	/	1/ 0.05	2/ 0.10	3/ 0.14	1/ 0.05	/	/	7.0/ 0.33	7.65357
S	/	1/ 0.05	7/ 0.33	4/ 0.19	/	/	/	12.0/ 0.57	6.64375
SSW	/	1/ 0.05	3/ 0.14	3/ 0.14	/	/	/	7.0/ 0.33	6.42500
SW	/	2/ 0.10	4/ 0.19	4/ 0.19	1/ 0.05	/	/	11.0/ 0.52	7.31364
WSW	/	3/ 0.14	5/ 0.24	2/ 0.10	2/ 0.10	/	/	12.0/ 0.57	7.15972
W	/	1/ 0.05	4/ 0.19	5/ 0.24	/	/	/	10.0/ 0.48	6.87750
WNW	/	/	3/ 0.14	1/ 0.05	/	/	/	4.0/ 0.19	5.64375
NW	/	1/ 0.05	5/ 0.24	/	/	/	/	6.0/ 0.29	4.73750
NNW	/	1/ 0.05	2/ 0.10	2/ 0.10	/	/	/	5.0/ 0.24	6.62000
TOTAL	/	12/ 0.57	75/ 3.56	41/ 1.95	6/ 0.29	/	/	134.0/ 6.37	6.78395

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=09 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	4/ 0.19	13/ 0.62	8/ 0.38	1/ 0.05	/	/	26.0/ 1.24	6.34615
NNE	/	3/ 0.14	28/ 1.33	38/ 1.81	5/ 0.24	/	/	74.0/ 3.52	8.34628
NE	/	4/ 0.19	14/ 0.67	56/ 2.66	11/ 0.52	/	/	85.0/ 4.04	9.13353
ENE	/	3/ 0.14	17/ 0.81	22/ 1.05	9/ 0.43	1/ 0.05	/	52.0/ 2.47	8.86602
E	/	4/ 0.19	8/ 0.38	12/ 0.57	1/ 0.05	/	/	25.0/ 1.19	7.40100
ESE	/	/	7/ 0.33	1/ 0.05	1/ 0.05	/	/	9.0/ 0.43	7.40833
SE	/	2/ 0.10	6/ 0.29	2/ 0.10	/	/	/	10.0/ 0.48	5.70000
SSE	/	2/ 0.10	6/ 0.29	6/ 0.29	/	/	/	14.0/ 0.67	6.52143
S	/	4/ 0.19	6/ 0.29	10/ 0.48	1/ 0.05	/	/	21.0/ 1.00	7.04524
SSW	/	6/ 0.29	7/ 0.33	3/ 0.14	/	/	/	16.0/ 0.76	4.90000
SW	/	2/ 0.10	10/ 0.48	13/ 0.62	7/ 0.33	/	/	32.0/ 1.52	8.97969
WSW	/	2/ 0.10	14/ 0.67	9/ 0.43	3/ 0.14	/	/	28.0/ 1.33	7.15863
W	/	3/ 0.14	11/ 0.52	3/ 0.14	/	/	/	17.0/ 0.81	5.68088
WNW	/	3/ 0.14	8/ 0.38	1/ 0.05	/	/	/	12.0/ 0.57	5.61458
NW	/	1/ 0.05	8/ 0.38	2/ 0.10	/	/	/	11.0/ 0.52	5.93864
NNW	/	2/ 0.10	7/ 0.33	15/ 0.71	1/ 0.05	/	/	25.0/ 1.19	7.70500
TOTAL	/	45/ 2.14	170/ 8.08	201/ 9.55	40/ 1.90	1/ 0.05	/	457.0/21.71	7.78129

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=09 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	2/ 0.10	4/ 0.19	24/ 1.14	2/ 0.10	/	/	32.0/ 1.52	9.20937
NNE	/	/	11/ 0.52	14/ 0.67	2/ 0.10	/	/	27.0/ 1.28	8.68148
NE	/	2/ 0.10	5/ 0.24	24/ 1.14	6/ 0.29	/	/	37.0/ 1.76	9.67635
ENE	/	1/ 0.05	10/ 0.48	8/ 0.38	7/ 0.33	/	/	26.0/ 1.24	9.09423
E	/	1/ 0.05	4/ 0.19	12/ 0.57	3/ 0.14	/	/	20.0/ 0.95	8.84250
ESE	/	2/ 0.10	6/ 0.29	9/ 0.43	/	/	/	17.0/ 0.81	6.88235
SE	/	2/ 0.10	3/ 0.14	2/ 0.10	/	/	/	7.0/ 0.33	5.56786
SSE	/	1/ 0.05	8/ 0.38	13/ 0.62	1/ 0.05	/	/	23.0/ 1.09	7.87065
S	/	1/ 0.05	17/ 0.81	14/ 0.67	/	/	/	32.0/ 1.52	7.63359
SSW	/	1/ 0.05	15/ 0.71	30/ 1.43	3/ 0.14	/	/	49.0/ 2.33	8.61173
SW	/	2/ 0.10	17/ 0.81	15/ 0.71	1/ 0.05	/	/	35.0/ 1.66	7.64214
WSW	/	5/ 0.24	15/ 0.71	4/ 0.19	/	/	/	24.0/ 1.14	5.37812
W	/	3/ 0.14	3/ 0.14	3/ 0.14	/	/	/	9.0/ 0.43	5.84444
WNW	/	1/ 0.05	3/ 0.14	2/ 0.10	/	/	/	6.0/ 0.29	6.08750
NW	/	4/ 0.19	2/ 0.10	1/ 0.05	/	/	/	7.0/ 0.33	3.55000
NNW	/	3/ 0.14	10/ 0.48	6/ 0.29	4/ 0.19	/	/	23.0/ 1.09	7.66304
TOTAL	/	31/ 1.47	133/ 6.32	181/ 8.60	29/ 1.38	/	/	374.0/17.77	7.99585

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=09 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	2/ 0.10	1/ 0.05	9/ 0.43	7/ 0.33	/	/	19.1/ 0.91	10.05236
NNE	/	/	7/ 0.33	16/ 0.76	2/ 0.10	/	/	25.0/ 1.19	8.93000
NE	0.2/ 0.01	4/ 0.19	6/ 0.29	11/ 0.52	/	/	/	21.2/ 1.01	7.11439
ENE	/	/	4/ 0.19	8/ 0.38	2/ 0.10	/	/	14.0/ 0.67	9.85357
E	0.1/ 0.00	2/ 0.10	4/ 0.19	6/ 0.29	1/ 0.05	/	/	13.1/ 0.62	7.77481
ESE	/	/	5/ 0.24	1/ 0.05	/	/	/	6.0/ 0.29	6.10000
SE	/	/	5/ 0.24	2/ 0.10	/	/	/	7.0/ 0.33	6.83571
SSE	/	/	10/ 0.48	6/ 0.29	2/ 0.10	/	/	18.0/ 0.86	7.81528
S	0.0/ 0.00	1/ 0.05	4/ 0.19	11/ 0.52	1/ 0.05	/	/	17.0/ 0.81	8.79265
SSW	0.0/ 0.00	1/ 0.05	4/ 0.19	19/ 0.90	/	/	/	24.0/ 1.14	8.75625
SW	0.0/ 0.00	1/ 0.05	7/ 0.33	34/ 1.62	/	/	/	42.0/ 2.00	8.21309
WSW	/	/	9/ 0.43	10/ 0.48	/	/	/	19.0/ 0.90	7.64210
W	0.0/ 0.00	1/ 0.05	4/ 0.19	2/ 0.10	/	/	/	7.0/ 0.33	5.62143
WNW	0.1/ 0.00	2/ 0.10	5/ 0.24	4/ 0.19	/	/	/	11.1/ 0.53	5.51464
NW	0.0/ 0.00	1/ 0.05	4/ 0.19	1/ 0.05	/	/	/	6.0/ 0.29	5.32500
NNW	0.2/ 0.01	5/ 0.24	2/ 0.10	4/ 0.19	/	/	/	11.2/ 0.53	5.37723
TOTAL	1.0/ 0.05	20/ 0.95	81/ 3.85	144/ 6.84	15/ 0.71	/	/	261.0/12.40	7.94449

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=09 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.0/ 0.00	3/ 0.14	10/ 0.48	5/ 0.24	/	/	/	18.0/ 0.86	5.74444
NNE	0.0/ 0.00	2/ 0.10	11/ 0.52	8/ 0.38	/	/	/	21.0/ 1.00	6.26905
NE	0.1/ 0.00	7/ 0.33	13/ 0.62	21/ 1.00	3/ 0.14	/	/	44.1/ 2.10	7.56916
ENE	0.1/ 0.00	7/ 0.33	16/ 0.76	10/ 0.48	/	/	/	33.1/ 1.57	6.45355
E	0.0/ 0.00	2/ 0.10	14/ 0.67	11/ 0.52	/	/	/	27.0/ 1.28	7.18179
ESE	0.0/ 0.00	3/ 0.14	8/ 0.38	9/ 0.43	/	/	/	20.0/ 0.95	6.62375
SE	0.0/ 0.00	2/ 0.10	5/ 0.24	7/ 0.33	1/ 0.05	/	/	15.0/ 0.71	8.54333
SSE	0.1/ 0.00	4/ 0.19	25/ 1.19	21/ 1.00	/	/	/	50.1/ 2.38	6.95733
S	0.0/ 0.00	3/ 0.14	19/ 0.90	15/ 0.71	1/ 0.05	/	/	38.0/ 1.81	7.22697
SSW	0.1/ 0.00	11/ 0.52	17/ 0.81	23/ 1.09	/	/	/	51.1/ 2.43	6.50611
SW	0.0/ 0.00	2/ 0.10	35/ 1.66	14/ 0.67	/	/	/	51.0/ 2.42	6.54118
WSW	0.1/ 0.00	4/ 0.19	31/ 1.47	19/ 0.90	/	/	/	54.1/ 2.57	6.51548
W	0.1/ 0.00	6/ 0.29	19/ 0.90	2/ 0.10	/	/	/	27.1/ 1.29	4.93035
WNW	0.1/ 0.00	5/ 0.24	15/ 0.71	2/ 0.10	/	/	/	22.1/ 1.05	4.76866
NW	0.1/ 0.00	7/ 0.33	23/ 1.09	1/ 0.05	/	/	/	31.1/ 1.48	4.57007
NNW	0.1/ 0.00	7/ 0.33	12/ 0.57	6/ 0.29	2/ 0.10	/	/	27.1/ 1.29	6.41882
TOTAL	1.0/ 0.05	75/ 3.56	273/12.97	174/ 8.27	7/ 0.33	/	/	530.0/25.18	6.47889

NUMBER OF BAD RECORDS: 10

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=10 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	7/ 0.33	58/ 2.75	143/ 6.77	59/ 2.79	2/ 0.09	/	269.1/12.75	10.01825
NNE	0.1/ 0.00	5/ 0.24	58/ 2.75	156/ 7.39	68/ 3.22	2/ 0.09	/	289.1/13.69	10.22557
NE	0.1/ 0.00	5/ 0.24	41/ 1.94	89/ 4.22	33/ 1.56	/	/	168.1/ 7.96	9.45166
ENE	0.1/ 0.00	8/ 0.38	26/ 1.23	44/ 2.08	14/ 0.66	/	1/ 0.05	93.1/ 4.41	9.08687
E	0.1/ 0.00	7/ 0.33	24/ 1.14	33/ 1.56	8/ 0.38	/	/	72.1/ 3.42	8.40950
ESE	0.2/ 0.01	9/ 0.43	21/ 0.99	20/ 0.95	2/ 0.09	/	/	52.2/ 2.47	7.10967
SE	0.2/ 0.01	10/ 0.47	27/ 1.28	18/ 0.85	1/ 0.05	/	/	56.2/ 2.66	6.54849
SSE	0.1/ 0.00	5/ 0.24	40/ 1.89	59/ 2.79	3/ 0.14	1/ 0.05	/	108.1/ 5.12	8.33568
S	0.2/ 0.01	14/ 0.66	33/ 1.56	57/ 2.70	17/ 0.81	2/0.09	/	123.2/ 5.84	8.60247
SSW	0.2/ 0.01	10/ 0.47	43/ 2.04	69/ 3.27	26/ 1.23	/	1/ 0.05	149.2/ 7.07	9.15706
SW	0.2/ 0.01	9/ 0.43	51/ 2.42	58/ 2.75	11/ 0.52	2/ 0.09	/	131.2/ 6.22	7.99238
WSW	0.1/ 0.00	7/ 0.33	43/ 2.04	77/ 3.65	16/ 0.76	1/ 0.05	1/ 0.05	145.1/ 6.87	8.86992
W	0.1/ 0.00	5/ 0.24	31/ 1.47	26/ 1.23	13/ 0.62	3/ 0.14	/	78.1/ 3.70	8.68614
WNW	0.1/ 0.00	4/ 0.19	26/ 1.23	47/ 2.23	17/ 0.81	/	/	94.1/ 4.46	9.27338
NW	0.1/ 0.00	7/ 0.33	43/ 2.04	69/ 3.27	18/ 0.85	/	/	137.1/ 6.49	8.75456
NNW	0.1/ 0.00	5/ 0.24	37/ 1.75	71/ 3.36	30/ 1.42	2/ 0.09	/	145.1/ 6.87	9.81292
TOTAL	2.0/ 0.09	117/ 5.54	602/28.52	1036/49.08	336/15.92	15/ 0.71	3/ 0.14	2111/ 100	9.12852

NUMBER OF BAD RECORDS: 121

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=10 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	7/ 0.33	25/ 1.18	10/ 0.47	/	/	42.0/ 1.99	10.64762
NNE	/	/	8/ 0.38	29/ 1.37	18/ 0.85	/	/	55.0/ 2.61	10.72818
NE	/	/	7/ 0.33	19/ 0.90	8/ 0.38	/	/	34.0/ 1.61	10.17647
ENE	/	/	/	7/ 0.33	/	/	/	7.0/ 0.33	9.36786
E	0.6/ 0.03	2/ 0.09	5/ 0.24	1/ 0.05	1/ 0.05	/	/	9.6/ 0.45	5.69271
ESE	0.3/ 0.01	1/ 0.05	6/ 0.28	/	/	/	/	7.3/ 0.35	5.03767
SE	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	5.46667
SSE	0.3/ 0.01	1/ 0.05	3/ 0.14	2/ 0.09	/	/	/	6.3/ 0.30	6.10516
S	/	/	1/ 0.05	2/ 0.09	2/ 0.09	/	/	5.0/ 0.24	10.45500
SSW	/	/	3/ 0.14	8/ 0.38	10/ 0.47	/	/	21.0/ 0.99	12.04405
SW	0.3/ 0.01	1/ 0.05	12/ 0.57	12/ 0.57	1/ 0.05	/	/	26.3/ 1.25	7.30893
WSW	/	/	12/ 0.57	15/ 0.71	3/ 0.14	/	/	30.0/ 1.42	8.17667
W	0.3/ 0.01	1/ 0.05	10/ 0.47	7/ 0.33	/	/	/	18.3/ 0.87	6.90505
WNW	/	/	4/ 0.19	12/ 0.57	8/ 0.38	/	/	24.0/ 1.14	10.98437
NW	0.3/ 0.01	1/ 0.05	4/ 0.19	14/ 0.66	1/ 0.05	/	/	20.3/ 0.96	8.92734
NNW	/	/	7/ 0.33	21/ 0.99	2/ 0.09	/	/	30.0/ 1.42	9.44917
TOTAL	2.0/ 0.09	7/ 0.33	91/ 4.31	175/ 8.29	64/ 3.03	/	/	339.0/ 16.06	9.41748

NUMBER OF BAD RECORDS: 31

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=10 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	2/ 0.09	5/ 0.24	7/ 0.33	1/ 0.05	/	16.0/ 0.76	11.72343
NNE	/	/	4/ 0.19	4/ 0.19	4/ 0.19	/	/	12.0/ 0.57	10.14583
NE	/	/	2/ 0.09	3/ 0.14	2/ 0.09	/	/	7.0/ 0.33	9.91071
ENE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	6.70000
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.80000
ESE	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.09	4.67500
SE	/	/	1/ 0.05	3/ 0.14	/	/	/	4.0/ 0.19	8.61875
SSE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.63750
S	/	/	4/ 0.19	4/ 0.19	1/ 0.05	/	/	9.0/ 0.43	8.34722
SSW	/	2/ 0.09	1/ 0.05	2/ 0.09	1/ 0.05	/	/	6.0/ 0.28	7.12500
SW	/	2/ 0.09	7/ 0.33	/	1/ 0.05	1/ 0.05	/	11.0/ 0.52	6.59318
WSW	/	/	1/ 0.05	4/ 0.19	1/ 0.05	/	/	6.0/ 0.28	9.37500
W	/	/	/	/	/	1/ 0.05	/	1.0/ 0.05	19.87498
WNW	/	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	3.0/ 0.14	10.00833
NW	/	/	1/ 0.05	5/ 0.24	4/ 0.19	/	/	10.0/ 0.47	11.62500
NNW	/	/	2/ 0.09	4/ 0.19	2/ 0.09	1/ 0.05	/	9.0/ 0.43	11.29444
TOTAL	/	6/ 0.28	32/ 1.52	35/ 1.66	24/ 1.14	4/ 0.19	/	101.0/ 4.78	9.57871

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=10 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	3/ 0.14	8/ 0.38	13/ 0.62	/	/	25.0/ 1.18	12.01100
NNE	/	/	2/ 0.09	6/ 0.28	6/ 0.28	2/ 0.09	/	16.0/ 0.76	12.33593
NE	/	/	1/ 0.05	2/ 0.09	/	/	/	3.0/ 0.14	7.73333
ENE	/	/	2/ 0.09	2/ 0.09	/	/	1/ 0.05	5.0/ 0.24	13.91000
E	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	5.19167
ESE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	3.35000
SE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	3.07500
SSE	/	1/ 0.05	/	5/ 0.24	/	/	/	6.0/ 0.28	7.84583
S	/	1/ 0.05	1/ 0.05	2/ 0.09	/	/	/	4.0/ 0.19	6.70000
SSW	/	2/ 0.09	2/ 0.09	2/ 0.09	4/ 0.19	/	/	10.0/ 0.47	9.63000
SW	/	1/ 0.05	6/ 0.28	/	1/ 0.05	/	/	8.0/ 0.38	6.26562
WSW	/	2/ 0.09	2/ 0.09	1/ 0.05	/	1/ 0.05	1/ 0.05	7.0/ 0.33	10.14642
W	/	1/ 0.05	/	1/ 0.05	1/ 0.05	/	/	3.0/ 0.14	7.68333
WNW	/	/	/	/	1/ 0.05	/	/	1.0/ 0.05	12.65000
NW	/	/	3/ 0.14	4/ 0.19	1/ 0.05	/	/	8.0/ 0.38	9.24375
NNW	/	/	1/ 0.05	4/ 0.19	7/ 0.33	1/ 0.05	/	13.0/ 0.62	12.99615
TOTAL	/	11/ 0.52	25/ 1.18	38/ 1.80	34/ 1.61	4/ 0.19	2/ 0.09	114.0/ 5.40	10.37127

NUMBER OF BAD RECORDS: 4

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=10 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	2/ 0.09	23/ 1.09	40/ 1.89	14/ 0.66	1/ 0.05	/	80.0/ 3.79	9.59500
NNE	/	/	23/ 1.09	38/ 1.80	18/ 0.85	/	/	79.0/ 3.74	9.81392
NE	/	/	14/ 0.66	11/ 0.52	7/ 0.33	/	/	32.0/ 1.52	9.10391
ENE	/	1/ 0.05	10/ 0.47	6/ 0.28	3/ 0.14	/	/	20.0/ 0.95	8.40125
E	/	3/ 0.14	2/ 0.09	12/ 0.57	1/ 0.05	/	/	18.0/ 0.85	8.77222
ESE	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	/	3.0/ 0.14	6.17500
SE	/	2/ 0.09	3/ 0.14	4/ 0.19	1/ 0.05	/	/	10.0/ 0.47	7.36500
SSE	/	/	11/ 0.52	7/ 0.33	1/ 0.05	/	/	19.0/ 0.90	7.75131
S	/	5/ 0.24	6/ 0.28	3/ 0.14	1/ 0.05	/	/	15.0/ 0.71	6.06167
SSW	/	1/ 0.05	12/ 0.57	5/ 0.24	7/ 0.33	/	/	25.0/ 1.18	8.83500
SW	/	1/ 0.05	2/ 0.09	6/ 0.28	2/ 0.09	1/ 0.05	/	12.0/ 0.57	9.77083
WSW	/	/	7/ 0.33	4/ 0.19	3/ 0.14	/	/	14.0/ 0.66	8.89464
W	/	/	3/ 0.14	5/ 0.24	2/ 0.09	1/ 0.05	/	11.0/ 0.52	10.36136
WNW	/	2/ 0.09	5/ 0.24	2/ 0.09	2/ 0.09	/	/	11.0/ 0.52	7.58409
NW	/	1/ 0.05	10/ 0.47	11/ 0.52	5/ 0.24	/	/	27.0/ 1.28	8.78148
NNW	/	/	13/ 0.62	12/ 0.57	8/ 0.38	/	/	33.0/ 1.56	9.34672
TOTAL	/	19/ 0.90	145/ 6.87	167/ 7.91	75/ 3.55	3/ 0.14	/	409.0/19.37	9.03696

NUMBER OF BAD RECORDS: 21

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=10 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	9/ 0.43	40/ 1.89	8/ 0.38	/	/	58.0/ 2.75	9.89827
NNE	/	2/ 0.09	7/ 0.33	46/ 2.18	16/ 0.76	/	/	71.0/ 3.36	10.65211
NE	/	/	6/ 0.28	29/ 1.37	8/ 0.38	/	/	43.0/ 2.04	10.01337
ENE	/	/	3/ 0.14	11/ 0.52	/	/	/	14.0/ 0.66	9.08393
E	/	/	3/ 0.14	5/ 0.24	4/ 0.19	/	/	12.0/ 0.57	11.04375
ESE	/	/	1/ 0.05	1/ 0.05	1/ 0.05	/	/	3.0/ 0.14	12.08333
SE	/	1/ 0.05	6/ 0.28	2/ 0.09	/	/	/	9.0/ 0.43	6.88333
SSE	/	/	13/ 0.62	26/ 1.23	1/ 0.05	1/ 0.05	/	41.0/ 1.94	9.23719
S	/	2/ 0.09	9/ 0.43	22/ 1.04	5/ 0.24	2/ 0.09	/	40.0/ 1.89	9.53937
SSW	/	1/ 0.05	4/ 0.19	28/ 1.33	3/ 0.14	/	1/ 0.05	37.0/ 1.75	10.16509
SW	/	/	3/ 0.14	7/ 0.33	1/ 0.05	/	/	11.0/ 0.52	9.01364
WSW	/	1/ 0.05	5/ 0.24	7/ 0.33	1/ 0.05	/	/	14.0/ 0.66	7.89464
W	/	1/ 0.05	5/ 0.24	3/ 0.14	7/ 0.33	1/ 0.05	/	17.0/ 0.81	10.76470
WNW	/	/	8/ 0.38	5/ 0.24	1/ 0.05	/	/	14.0/ 0.66	8.35893
NW	/	1/ 0.05	6/ 0.28	14/ 0.66	7/ 0.33	/	/	28.0/ 1.33	9.79107
NNW	/	/	5/ 0.24	12/ 0.57	11/ 0.52	/	/	28.0/ 1.33	11.24643
TOTAL	/	10/ 0.47	93/ 4.41	258/12.22	74/ 3.51	4/ 0.19	1/ 0.05	440.0/20.84	9.89553

NUMBER OF BAD RECORDS: 22

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=10 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	6/ 0.28	6/ 0.28	5/ 0.24	/	/	18.0/ 0.85	9.22500
NNE	/	/	4/ 0.19	15/ 0.71	4/ 0.19	/	/	23.0/ 1.09	10.32609
NE	/	/	3/ 0.14	11/ 0.52	6/ 0.28	/	/	20.0/ 0.95	10.81750
ENE	/	/	2/ 0.09	4/ 0.19	2/ 0.09	/	/	8.0/ 0.38	10.51875
E	/	/	2/ 0.09	2/ 0.09	/	/	/	4.0/ 0.19	7.95000
ESE	/	/	1/ 0.05	3/ 0.14	/	/	/	4.0/ 0.19	8.91875
SE	/	/	4/ 0.19	2/ 0.09	/	/	/	6.0/ 0.28	6.75833
SSE	/	2/ 0.09	2/ 0.09	5/ 0.24	1/ 0.05	/	/	10.0/ 0.47	8.23000
S	/	1/ 0.05	5/ 0.24	8/ 0.38	4/ 0.19	/	/	18.0/ 0.85	8.99028
SSW	/	2/ 0.09	2/ 0.09	4/ 0.19	/	/	/	8.0/ 0.38	7.20937
SW	/	/	2/ 0.09	3/ 0.14	3/ 0.14	/	/	8.0/ 0.38	10.59687
WSW	/	1/ 0.05	1/ 0.05	8/ 0.38	2/ 0.09	/	/	12.0/ 0.57	9.67083
W	/	/	2/ 0.09	2/ 0.09	2/ 0.09	/	/	6.0/ 0.28	9.86250
WNW	/	/	3/ 0.14	5/ 0.24	3/ 0.14	/	/	11.0/ 0.52	10.35000
NW	/	/	6/ 0.28	8/ 0.38	/	/	/	14.0/ 0.66	8.39107
NNW	/	1/ 0.05	2/ 0.09	7/ 0.33	/	/	/	10.0/ 0.47	7.74750
TOTAL	/	8/ 0.38	47/ 2.23	93/ 4.41	32/ 1.52	/	/	180.0/ 8.53	9.34819

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=10 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	8/ 0.38	19/ 0.90	2/ 0.09	/	/	30.0/ 1.42	8.43083
NNE	/	3/ 0.14	10/ 0.47	18/ 0.85	2/ 0.09	/	/	33.0/ 1.56	8.41667
NE	/	5/ 0.24	8/ 0.38	14/ 0.66	2/ 0.09	/	/	29.0/ 1.37	7.30517
ENE	/	7/ 0.33	7/ 0.33	14/ 0.66	9/ 0.43	/	/	37.0/ 1.75	8.59257
E	/	2/ 0.09	9/ 0.43	12/ 0.57	2/ 0.09	/	/	25.0/ 1.18	8.35300
ESE	/	5/ 0.24	11/ 0.52	15/ 0.71	1/ 0.05	/	/	32.0/ 1.52	7.22266
SE	/	6/ 0.28	11/ 0.52	6/ 0.28	/	/	/	23.0/ 1.09	5.98369
SSE	/	1/ 0.05	9/ 0.43	14/ 0.66	/	/	/	24.0/ 1.14	8.17187
S	/	5/ 0.24	7/ 0.33	16/ 0.76	4/ 0.19	/	/	32.0/ 1.52	8.46641
SSW	/	2/ 0.09	19/ 0.90	20/ 0.95	1/ 0.05	/	/	42.0/ 1.99	7.60059
SW	/	4/ 0.19	19/ 0.90	30/ 1.42	2/ 0.09	/	/	55.0/ 2.61	7.86591
WSW	/	3/ 0.14	15/ 0.71	38/ 1.80	6/ 0.28	/	/	62.0/ 2.94	9.08266
W	/	2/ 0.09	11/ 0.52	8/ 0.38	1/ 0.05	/	/	22.0/ 1.04	6.95909
WNW	/	2/ 0.09	5/ 0.24	22/ 1.04	1/ 0.05	/	/	30.0/ 1.42	8.39583
NW	/	4/ 0.19	13/ 0.62	13/ 0.62	/	/	/	30.0/ 1.42	6.68000
NNW	/	4/ 0.19	7/ 0.33	11/ 0.52	/	/	/	22.0/ 1.04	7.67159
TOTAL	/	56/ 2.65	169/ 8.01	270/12.79	33/ 1.56	/	/	528.0/25.01	7.94356

NUMBER OF BAD RECORDS: 26

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=11 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	12/ 0.60	63/ 3.13	100/ 4.97	34/ 1.69	/	/	209.1/10.38	8.92165
NNE	0.1/ 0.00	7/ 0.35	69/ 3.43	162/ 8.04	48/ 2.38	/	/	286.1/14.21	9.35136
NE	0.1/ 0.00	12/ 0.60	67/ 3.33	61/ 3.03	4/ 0.20	/	/	144.1/ 7.15	7.21877
ENE	0.0/ 0.00	6/ 0.30	34/ 1.69	39/ 1.94	2/ 0.10	/	/	81.0/ 4.02	7.26142
E	0.0/ 0.00	6/ 0.30	26/ 1.29	31/ 1.54	3/ 0.15	/	/	66.0/ 3.28	7.69432
ESE	0.0/ 0.00	4/ 0.20	24/ 1.19	18/ 0.89	/	/	/	46.0/ 2.28	7.20978
SE	0.0/ 0.00	5/ 0.25	9/ 0.45	6/ 0.30	1/ 0.05	/	/	21.0/ 1.04	6.10952
SSE	0.0/ 0.00	6/ 0.30	15/ 0.74	22/ 1.09	3/ 0.15	/	/	46.0/ 2.28	7.63804
S	0.1/ 0.00	9/ 0.45	48/ 2.38	44/ 2.18	29/ 1.44	/	/	130.1/ 6.46	8.67890
SSW	0.1/ 0.00	10/ 0.50	56/ 2.78	101/ 5.01	38/ 1.89	1/ 0.05	1/ 0.05	207.1/10.28	9.43534
SW	0.1/ 0.00	14/ 0.70	53/ 2.63	83/ 4.12	34/ 1.69	4/ 0.20	2/ 0.10	190.1/ 9.44	9.34298
WSW	0.1/ 0.00	11/ 0.55	50/ 2.48	74/ 3.67	22/ 1.09	5/ 0.25	/	162.1/ 8.05	8.93160
W	0.0/ 0.00	4/ 0.20	26/ 1.29	40/ 1.99	8/ 0.40	/	/	78.0/ 3.87	8.59744
WNW	0.0/ 0.00	5/ 0.25	18/ 0.89	31/ 1.54	27/ 1.34	2/ 0.10	/	83.0/ 4.12	10.69006
NW	0.0/ 0.00	6/ 0.30	45/ 2.23	56/ 2.78	19/ 0.94	4/ 0.20	/	130.0/ 6.45	9.23788
NNW	0.0/ 0.00	4/ 0.20	46/ 2.28	66/ 3.28	17/ 0.84	1/ 0.05	/	134.0/ 6.65	8.69701
TOTAL	1.0/ 0.05	121/ 6.01	649/32.22	934/46.38	289/14.35	17/ 0.84	3/ 0.15	2014/ 100	8.79829

NUMBER OF BAD RECORDS: 146

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=11 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	4/ 0.20	7/ 0.35	5/ 0.25	/	/	16.0/ 0.79	10.27448
NNE	/	/	3/ 0.15	8/ 0.40	3/ 0.15	/	/	14.0/ 0.70	10.11071
NE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	10.92500
ENE	/	/	/	2/ 0.10	/	/	/	2.0/ 0.10	9.62500
E	/	/	2/ 0.10	1/ 0.05	/	/	/	3.0/ 0.15	7.22500
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.97500
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	/	/	/	/	/	
S	/	/	/	/	/	/	/	/	
SSW	/	/	1/ 0.05	2/ 0.10	3/ 0.15	/	/	6.0/ 0.30	11.08333
SW	/	/	1/ 0.05	11/ 0.55	/	/	/	12.0/ 0.60	9.19583
WSW	/	/	1/ 0.05	5/ 0.25	4/ 0.20	1/ 0.05	/	11.0/ 0.55	12.91136
W	/	/	/	3/ 0.15	2/ 0.10	/	/	5.0/ 0.25	12.96500
WNW	/	/	/	1/ 0.05	7/ 0.35	1/ 0.05	/	9.0/ 0.45	16.11666
NW	/	/	1/ 0.05	3/ 0.15	6/ 0.30	4/ 0.20	/	14.0/ 0.70	14.57857
NNW	/	/	5/ 0.25	3/ 0.15	/	/	/	8.0/ 0.40	7.31875
TOTAL	/	/	19/ 0.94	47/ 2.33	30/ 1.49	6/ 0.30	/	102.0/ 5.06	11.32516

NUMBER OF BAD RECORDS: 1

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=11 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	7.87500
NNE	/	/	2/ 0.10	1/ 0.05	3/ 0.15	/	/	6.0/ 0.30	10.65417
NE	/	/	2/ 0.10	4/ 0.20	/	/	/	6.0/ 0.30	7.95000
ENE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	6.58750
E	/	/	2/ 0.10	/	/	/	/	2.0/ 0.10	6.08750
ESE	/	/	3/ 0.15	/	/	/	/	3.0/ 0.15	5.61667
SE	/	/	/	/	/	/	/	/	
SSE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	3.12500
S	/	/	/	/	/	/	/	/	
SSW	/	1/ 0.05	1/ 0.05	6/ 0.30	/	/	/	8.0/ 0.40	9.22500
SW	/	1/ 0.05	3/ 0.15	8/ 0.40	/	/	/	12.0/ 0.60	8.74792
WSW	/	/	2/ 0.10	5/ 0.25	2/ 0.10	/	/	9.0/ 0.45	9.33889
W	/	/	/	4/ 0.20	2/ 0.10	/	/	6.0/ 0.30	11.29167
WNW	/	/	/	2/ 0.10	8/ 0.40	/	/	10.0/ 0.50	14.73749
NW	/	/	1/ 0.05	/	3/ 0.15	/	/	4.0/ 0.20	11.72500
NNW	/	/	3/ 0.15	5/ 0.25	/	/	/	8.0/ 0.40	7.96250
TOTAL	/	3/ 0.15	21/ 1.04	37/ 1.84	18/ 0.89	/	/	79.0/ 3.92	9.63607

NUMBER OF BAD RECORDS: 3

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=11 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	4/ 0.20	6/ 0.30	2/ 0.10	/	/	13.0/ 0.65	9.01731
NNE	/	/	5/ 0.25	5/ 0.25	2/ 0.10	/	/	12.0/ 0.60	9.07083
NE	/	/	5/ 0.25	1/ 0.05	/	/	/	6.0/ 0.30	5.89167
ENE	/	/	4/ 0.20	/	/	/	/	4.0/ 0.20	5.12500
E	/	/	2/ 0.10	1/ 0.05	/	/	/	3.0/ 0.15	6.98333
ESE	/	1/ 0.05	3/ 0.15	/	/	/	/	4.0/ 0.20	5.71875
SE	/	/	/	/	/	/	/	/	
SSE	/	1/ 0.05	/	/	/	/	/	1.0/ 0.05	3.47500
S	/	/	3/ 0.15	1/ 0.05	/	/	/	4.0/ 0.20	8.00625
SSW	/	/	4/ 0.20	5/ 0.25	2/ 0.10	/	/	11.0/ 0.55	9.72954
SW	/	2/ 0.10	6/ 0.30	6/ 0.30	4/ 0.20	1/ 0.05	/	19.0/ 0.94	9.30526
WSW	/	/	4/ 0.20	2/ 0.10	1/ 0.05	/	/	7.0/ 0.35	8.00714
W	/	1/ 0.05	2/ 0.10	1/ 0.05	/	/	/	4.0/ 0.20	6.15625
WNW	/	2/ 0.10	/	/	4/ 0.20	/	/	6.0/ 0.30	11.19166
NW	/	/	2/ 0.10	3/ 0.15	1/ 0.05	/	/	6.0/ 0.30	10.68750
NNW	/	/	5/ 0.25	3/ 0.15	1/ 0.05	1/ 0.05	/	10.0/ 0.50	8.29750
TOTAL	/	8/ 0.40	49/ 2.43	34/ 1.69	17/ 0.84	2/ 0.10	/	110.0/ 5.46	8.54545

NUMBER OF BAD RECORDS: 7

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=11 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	4/ 0.20	29/ 1.44	40/ 1.99	17/ 0.84	/	/	90.0/ 4.47	9.10028
NNE	/	6/ 0.30	47/ 2.33	67/ 3.33	23/ 1.14	/	/	143.0/ 7.10	8.71958
NE	/	5/ 0.25	37/ 1.84	17/ 0.84	1/ 0.05	/	/	60.0/ 2.98	6.65250
ENE	/	5/ 0.25	13/ 0.65	17/ 0.84	1/ 0.05	/	/	36.0/ 1.79	7.14375
E	/	5/ 0.25	6/ 0.30	4/ 0.20	/	/	/	15.0/ 0.74	5.63500
ESE	/	/	3/ 0.15	6/ 0.30	/	/	/	9.0/ 0.45	8.32778
SE	/	1/ 0.05	1/ 0.05	3/ 0.15	/	/	/	5.0/ 0.25	6.73000
SSE	/	2/ 0.10	1/ 0.05	1/ 0.05	3/ 0.15	/	/	7.0/ 0.35	8.30000
S	/	4/ 0.20	10/ 0.50	13/ 0.65	5/ 0.25	/	/	32.0/ 1.59	7.81406
SSW	/	4/ 0.20	21/ 1.04	20/ 0.99	9/ 0.45	1/ 0.05	1/ 0.05	56.0/ 2.78	9.07961
SW	/	8/ 0.40	23/ 1.14	19/ 0.94	7/ 0.35	3/ 0.15	1/ 0.05	61.0/ 3.03	8.63770
WSW	/	7/ 0.35	13/ 0.65	11/ 0.55	5/ 0.25	2/ 0.10	/	38.0/ 1.89	8.08618
W	/	1/ 0.05	12/ 0.60	8/ 0.40	1/ 0.05	/	/	22.0/ 1.09	7.03523
WNW	/	1/ 0.05	3/ 0.15	6/ 0.30	5/ 0.25	1/ 0.05	/	16.0/ 0.79	11.27812
NW	/	4/ 0.20	10/ 0.50	8/ 0.40	6/ 0.30	/	/	28.0/ 1.39	8.56875
NNW	/	3/ 0.15	11/ 0.55	18/ 0.89	9/ 0.45	/	/	41.0/ 2.04	9.27439
TOTAL	/	60/ 2.98	240/11.92	258/12.81	92/ 4.57	7/ 0.35	2/ 0.10	659.0/32.72	8.37869

NUMBER OF BAD RECORDS: 45

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=11 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	2/ 0.10	4/ 0.20	18/ 0.89	4/ 0.20	/	/	28.0/ 1.39	9.24553
NNE	/	/	6/ 0.30	48/ 2.38	10/ 0.50	/	/	64.0/ 3.18	10.33047
NE	/	/	8/ 0.40	19/ 0.94	2/ 0.10	/	/	29.0/ 1.44	8.95000
ENE	/	/	3/ 0.15	4/ 0.20	1/ 0.05	/	/	8.0/ 0.40	7.81875
E	/	/	7/ 0.35	9/ 0.45	1/ 0.05	/	/	17.0/ 0.84	8.19265
ESE	/	1/ 0.05	8/ 0.40	7/ 0.35	/	/	/	16.0/ 0.79	7.35000
SE	/	/	3/ 0.15	1/ 0.05	1/ 0.05	/	/	5.0/ 0.25	8.11000
SSE	/	1/ 0.05	5/ 0.25	10/ 0.50	/	/	/	16.0/ 0.79	7.97656
S	/	1/ 0.05	13/ 0.65	8/ 0.40	15/ 0.74	/	/	37.0/ 1.84	10.22905
SSW	/	/	10/ 0.50	22/ 1.09	13/ 0.65	/	/	45.0/ 2.23	10.26055
SW	/	1/ 0.05	7/ 0.35	7/ 0.35	15/ 0.74	/	1/ 0.05	31.0/ 1.54	11.40000
WSW	/	3/ 0.15	5/ 0.25	6/ 0.30	3/ 0.15	2/ 0.10	/	19.0/ 0.94	9.58158
W	/	/	2/ 0.10	2/ 0.10	1/ 0.05	/	/	5.0/ 0.25	9.61000
WNW	/	/	/	4/ 0.20	2/ 0.10	/	/	6.0/ 0.30	11.62083
NW	/	/	2/ 0.10	28/ 1.39	2/ 0.10	/	/	32.0/ 1.59	10.39766
NNW	/	1/ 0.05	3/ 0.15	8/ 0.40	2/ 0.10	/	/	14.0/ 0.70	8.97857
TOTAL	/	10/ 0.50	86/ 4.27	201/ 9.98	72/ 3.57	2/ 0.10	1/ 0.05	372.0/18.47	9.72856

NUMBER OF BAD RECORDS: 70

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=11 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	8/ 0.40	18/ 0.89	3/ 0.15	/	/	29.0/ 1.44	9.12414
NNE	/	/	2/ 0.10	14/ 0.70	4/ 0.20	/	/	20.0/ 0.99	10.41750
NE	/	2/ 0.10	5/ 0.25	4/ 0.20	1/ 0.05	/	/	12.0/ 0.60	7.10417
ENE	/	/	1/ 0.05	1/ 0.05	/	/	/	2.0/ 0.10	7.57500
E	/	/	3/ 0.15	5/ 0.25	/	/	/	8.0/ 0.40	7.88125
ESE	/	1/ 0.05	6/ 0.30	4/ 0.20	/	/	/	11.0/ 0.55	7.38409
SE	/	1/ 0.05	1/ 0.05	/	/	/	/	2.0/ 0.10	3.48750
SSE	/	1/ 0.05	3/ 0.15	7/ 0.35	/	/	/	11.0/ 0.55	7.72727
S	/	3/ 0.15	13/ 0.65	13/ 0.65	7/ 0.35	/	/	36.0/ 1.79	8.58680
SSW	/	2/ 0.10	8/ 0.40	17/ 0.84	9/ 0.45	/	/	36.0/ 1.79	9.67014
SW	/	/	3/ 0.15	8/ 0.40	6/ 0.30	/	/	17.0/ 0.84	10.85735
WSW	/	/	5/ 0.25	11/ 0.55	1/ 0.05	/	/	17.0/ 0.84	8.30000
W	/	1/ 0.05	/	7/ 0.35	1/ 0.05	/	/	9.0/ 0.45	9.86389
WNW	/	1/ 0.05	2/ 0.10	7/ 0.35	1/ 0.05	/	/	11.0/ 0.55	9.35682
NW	/	/	6/ 0.30	6/ 0.30	1/ 0.05	/	/	13.0/ 0.65	8.06154
NNW	/	/	4/ 0.20	9/ 0.45	4/ 0.20	/	/	17.0/ 0.84	9.64118
TOTAL	/	12/ 0.60	70/ 3.48	131/ 6.50	38/ 1.89	/	/	251.0/12.46	8.97580

NUMBER OF BAD RECORDS: 14

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=11 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.2/ 0.01	5/ 0.25	13/ 0.65	10/ 0.50	3/ 0.15	/	/	31.2/ 1.55	7.23718
NNE	0.0/ 0.00	1/ 0.05	4/ 0.20	19/ 0.94	3/ 0.15	/	/	27.0/ 1.34	9.05741
NE	0.2/ 0.01	5/ 0.25	10/ 0.50	15/ 0.74	/	/	/	30.2/ 1.50	6.70116
ENE	0.0/ 0.00	1/ 0.05	12/ 0.60	14/ 0.70	/	/	/	27.0/ 1.34	7.42130
E	0.0/ 0.00	1/ 0.05	4/ 0.20	11/ 0.55	2/ 0.10	/	/	18.0/ 0.89	9.23194
ESE	0.0/ 0.00	1/ 0.05	/	1/ 0.05	/	/	/	2.0/ 0.10	6.08750
SE	0.1/ 0.00	3/ 0.15	4/ 0.20	2/ 0.10	/	/	/	9.1/ 0.45	5.19093
SSE	/	/	6/ 0.30	4/ 0.20	/	/	/	10.0/ 0.50	7.40250
S	0.0/ 0.00	1/ 0.05	9/ 0.45	9/ 0.45	2/ 0.10	/	/	21.0/ 1.04	7.58571
SSW	0.1/ 0.00	3/ 0.15	11/ 0.55	29/ 1.44	2/ 0.10	/	/	45.1/ 2.24	8.61086
SW	0.1/ 0.00	2/ 0.10	10/ 0.50	24/ 1.19	2/ 0.10	/	/	38.1/ 1.89	8.37336
WSW	0.0/ 0.00	1/ 0.05	20/ 0.99	34/ 1.69	6/ 0.30	/	/	61.0/ 3.03	8.77172
W	0.0/ 0.00	1/ 0.05	10/ 0.50	15/ 0.74	1/ 0.05	/	/	27.0/ 1.34	8.21481
WNW	0.0/ 0.00	1/ 0.05	13/ 0.65	11/ 0.55	/	/	/	25.0/ 1.24	6.98400
NW	0.1/ 0.00	2/ 0.10	23/ 1.14	8/ 0.40	/	/	/	33.1/ 1.64	6.29796
NNW	/	/	15/ 0.74	20/ 0.99	1/ 0.05	/	/	36.0/ 1.79	8.06458
TOTAL	1.0/ 0.05	28/ 1.39	164/ 8.14	226/11.22	22/ 1.09	/	/	441.0/21.90	7.86769

NUMBER OF BAD RECORDS: 6

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=12 SUMMARY OVER ALL STABILITY
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	10/ 0.46	35/ 1.61	84/ 3.87	18/ 0.83	1/ 0.05	/	148.1/ 6.82	8.92817
NNE	0.0/ 0.00	6/ 0.28	41/ 1.89	92/ 4.24	13/ 0.60	/	/	152.0/ 7.00	8.70005
NE	0.1/ 0.00	11/ 0.51	30/ 1.38	58/ 2.67	8/ 0.37	/	/	107.1/ 4.93	8.15686
ENE	0.0/ 0.00	4/ 0.18	30/ 1.38	50/ 2.30	2/ 0.09	/	/	86.0/ 3.96	7.90698
E	0.1/ 0.00	9/ 0.41	41/ 1.89	25/ 1.15	/	/	/	75.1/ 3.46	6.24118
ESE	0.0/ 0.00	6/ 0.28	31/ 1.43	16/ 0.74	3/ 0.14	/	/	56.0/ 2.58	6.96518
SE	0.1/ 0.00	9/ 0.41	15/ 0.69	11/ 0.51	3/ 0.14	/	/	38.1/ 1.75	6.56234
SSE	0.1/ 0.00	10/ 0.46	41/ 1.89	32/ 1.47	6/ 0.28	1/ 0.05	/	90.1/ 4.15	7.51734
S	0.0/ 0.00	7/ 0.32	26/ 1.20	60/ 2.76	22/ 1.01	3/ 0.14	/	118.0/ 5.43	9.72662
SSW	0.1/ 0.00	16/ 0.74	37/ 1.70	125/ 5.76	75/ 3.45	22/ 1.01	7/ 0.32	282.1/12.99	11.69044
SW	0.1/ 0.00	14/ 0.64	54/ 2.49	77/ 3.55	42/ 1.93	16/ 0.74	1/ 0.05	204.1/ 9.40	10.18467
WSW	0.1/ 0.00	11/ 0.51	62/ 2.85	81/ 3.73	43/ 1.98	4/ 0.18	/	201.1/ 9.26	9.50785
W	0.1/ 0.00	9/ 0.41	23/ 1.06	49/ 2.26	33/ 1.52	7/ 0.32	1/ 0.05	122.1/ 5.62	10.62592
WNW	0.1/ 0.00	10/ 0.46	26/ 1.20	55/ 2.53	59/ 2.72	19/ 0.87	1/ 0.05	170.1/ 7.83	11.98486
NW	0.1/ 0.00	12/ 0.55	28/ 1.29	78/ 3.59	43/ 1.98	10/ 0.46	/	171.1/ 7.88	10.53192
NNW	0.1/ 0.00	8/ 0.37	49/ 2.26	67/ 3.08	22/ 1.01	5/ 0.23	/	151.1/ 6.96	9.04939
TOTAL	1.0/ 0.05	152/ 7.00	569/26.20	960/44.20	392/18.05	88/ 4.05	10/ 0.46	2172/ 100	9.63450

NUMBER OF BAD RECORDS: 60

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=12 STABILITY=A
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	3/ 0.14	10/ 0.46	2/ 0.09	1/ 0.05	/	16.0/ 0.74	9.76250
NNE	/	/	9/ 0.41	1/ 0.05	/	/	/	10.0/ 0.46	6.39083
NE	/	1/ 0.05	1/ 0.05	8/ 0.37	/	/	/	10.0/ 0.46	9.22500
ENE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	8.35000
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	4.50000
ESE	/	1/ 0.05	2/ 0.09	1/ 0.05	/	/	/	4.0/ 0.18	6.48750
SE	/	1/ 0.05	2/ 0.09	/	/	/	/	3.0/ 0.14	4.64167
SSE	/	/	2/ 0.09	/	/	/	/	2.0/ 0.09	5.72500
S	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	7.37500
SSW	/	/	3/ 0.14	3/ 0.14	3/ 0.14	/	/	9.0/ 0.41	10.18611
SW	/	/	2/ 0.09	5/ 0.23	5/ 0.23	1/ 0.05	/	13.0/ 0.60	12.25769
WSW	/	/	5/ 0.23	/	1/ 0.05	/	/	6.0/ 0.28	6.33750
W	/	/	/	3/ 0.14	/	1/ 0.05	/	4.0/ 0.18	13.65000
WNW	/	/	2/ 0.09	11/ 0.51	7/ 0.32	2/ 0.09	1/ 0.05	23.0/ 1.06	13.01956
NW	/	/	/	11/ 0.51	6/ 0.28	1/ 0.05	/	18.0/ 0.83	12.75833
NNW	/	/	2/ 0.09	6/ 0.28	2/ 0.09	1/ 0.05	/	11.0/ 0.51	11.20454
TOTAL	/	3/ 0.14	35/ 1.61	60/ 2.76	26/ 1.20	7/ 0.32	1/ 0.05	132.0/6.08	10.45385

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=12 STABILITY=B
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	2/ 0.09	3/ 0.14	/	/	/	5.0/ 0.23	7.64500
NNE	/	/	1/ 0.05	3/ 0.14	/	/	/	4.0/ 0.18	8.37500
NE	/	/	/	2/ 0.09	/	/	/	2.0/ 0.09	11.16250
ENE	/	/	/	1/ 0.05	/	/	/	1.0/ 0.05	7.60000
E	/	/	3/ 0.14	/	/	/	/	3.0/ 0.14	5.70833
ESE	/	/	3/ 0.14	1/ 0.05	/	/	/	4.0/ 0.18	6.73125
SE	/	/	/	/	/	/	/	/	
SSE	/	1/ 0.05	1/ 0.05	2/ 0.09	/	/	/	4.0/ 0.18	7.29375
S	/	/	2/ 0.09	2/ 0.09	1/ 0.05	/	/	5.0/ 0.23	8.50500
SSW	/	/	/	2/ 0.09	3/ 0.14	/	/	5.0/ 0.23	12.53500
SW	/	/	2/ 0.09	1/ 0.05	1/ 0.05	1/ 0.05	/	5.0/ 0.23	10.91499
WSW	/	/	5/ 0.23	/	/	/	/	5.0/ 0.23	6.04000
W	/	/	/	/	1/ 0.05	/	/	1.0/ 0.05	18.27499
WNW	/	/	1/ 0.05	2/ 0.09	4/ 0.18	3/ 0.14	/	10.0/ 0.46	15.76249
NW	/	1/ 0.05	1/ 0.05	1/ 0.05	3/ 0.14	3/ 0.14	/	9.0/ 0.41	14.10000
NNW	/	/	1/ 0.05	1/ 0.05	2/ 0.09	/	/	4.0/ 0.18	11.86250
TOTAL	/	2/ 0.09	22/ 1.01	21/ 0.97	15/ 0.69	7/ 0.32	/	67.0/ 3.08	10.67313

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=12 STABILITY=C
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	1/ 0.05	3/ 0.14	/	/	/	5.0/ 0.23	7.17500
NNE	/	/	2/ 0.09	3/ 0.14	/	/	/	5.0/ 0.23	7.75000
NE	/	1/ 0.05	1/ 0.05	2/ 0.09	/	/	/	4.0/ 0.18	6.52500
ENE	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	5.86667
E	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	5.37500
ESE	/	/	1/ 0.05	/	/	/	/	1.0/ 0.05	7.27500
SE	/	/	/	/	/	/	/	/	
SSE	/	/	/	4/ 0.18	/	/	/	4.0/ 0.18	9.05000
S	/	/	2/ 0.09	1/ 0.05	/	/	/	3.0/ 0.14	7.40833
SSW	/	1/ 0.05	2/ 0.09	5/ 0.23	3/ 0.14	1/ 0.05	/	12.0/ 0.55	10.42917
SW	/	1/ 0.05	1/ 0.05	2/ 0.09	3/ 0.14	1/ 0.05	/	8.0/ 0.37	11.53333
WSW	/	/	1/ 0.05	2/ 0.09	2/ 0.09	/	/	5.0/ 0.23	12.26499
W	/	/	1/ 0.05	3/ 0.14	1/ 0.05	2/ 0.09	/	7.0/ 0.32	12.86428
WNW	/	/	/	3/ 0.14	4/ 0.18	3/ 0.14	/	10.0/ 0.46	15.03749
NW	/	1/ 0.05	1/ 0.05	4/ 0.18	3/ 0.14	/	/	9.0/ 0.41	9.87778
NNW	/	/	3/ 0.14	2/ 0.09	2/ 0.09	1/ 0.05	/	8.0/ 0.37	10.59062
TOTAL	/	5/ 0.23	19/ 0.87	35/ 1.61	18/ 0.83	8/ 0.37	/	85.0/ 3.91	10.37872

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=12 STABILITY=D
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	6/ 0.28	17/ 0.78	31/ 1.43	2/ 0.09	/	/	56.0/ 2.58	7.91428
NNE	/	3/ 0.14	11/ 0.51	38/ 1.75	6/ 0.28	/	/	58.0/ 2.67	9.22500
NE	/	5/ 0.23	11/ 0.51	14/ 0.64	1/ 0.05	/	/	31.0/ 1.43	7.12177
ENE	/	1/ 0.05	10/ 0.46	12/ 0.55	/	/	/	23.0/ 1.06	7.21848
E	/	5/ 0.23	14/ 0.64	8/ 0.37	/	/	/	27.0/ 1.24	5.56296
ESE	/	3/ 0.14	8/ 0.37	6/ 0.28	1/ 0.05	/	/	18.0/ 0.83	6.71805
SE	/	3/ 0.14	1/ 0.05	2/ 0.09	/	/	/	6.0/ 0.28	5.43333
SSE	/	1/ 0.05	8/ 0.37	/	1/ 0.05	1/ 0.05	/	11.0/ 0.51	7.04773
S	/	4/ 0.18	7/ 0.32	6/ 0.28	3/ 0.14	2/ 0.09	/	22.0/ 1.01	9.05757
SSW	/	7/ 0.32	8/ 0.37	17/ 0.78	25/ 1.15	17/ 0.78	7/ 0.32	81.0/ 3.73	14.39969
SW	/	4/ 0.18	23/ 1.06	17/ 0.78	13/ 0.60	4/ 0.18	1/ 0.05	62.0/ 2.85	9.80967
WSW	/	2/ 0.09	15/ 0.69	20/ 0.92	20/ 0.92	2/ 0.09	/	59.0/ 2.72	11.03206
W	/	3/ 0.14	3/ 0.14	9/ 0.41	13/ 0.60	4/ 0.18	1/ 0.05	33.0/ 1.52	12.37803
WNW	/	2/ 0.09	5/ 0.23	8/ 0.37	17/ 0.78	11/ 0.51	/	43.0/ 1.98	14.13546
NW	/	7/ 0.32	10/ 0.46	21/ 0.97	22/ 1.01	5/ 0.23	/	65.0/ 2.99	10.93961
NNW	/	4/ 0.18	21/ 0.97	28/ 1.29	10/ 0.46	3/ 0.14	/	66.0/ 3.04	9.27613
TOTAL	/	60/ 2.76	172/ 7.92	237/10.91	134/ 6.17	49/ 2.26	9/ 0.41	661.0/30.43	10.15224

NUMBER OF BAD RECORDS: 5

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=12 STABILITY=E
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	0.1/ 0.00	2/ 0.09	5/ 0.23	15/ 0.69	6/ 0.28	/	/	28.1/ 1.29	9.42971
NNE	0.0/ 0.00	1/ 0.05	5/ 0.23	22/ 1.01	6/ 0.28	/	/	34.0/ 1.57	9.67500
NE	0.1/ 0.00	2/ 0.09	6/ 0.28	21/ 0.97	5/ 0.23	/	/	34.1/ 1.57	9.06891
ENE	0.1/ 0.00	2/ 0.09	7/ 0.32	19/ 0.87	/	/	/	28.1/ 1.29	8.60098
E	0.1/ 0.00	2/ 0.09	16/ 0.74	5/ 0.23	/	/	/	23.1/ 1.06	6.09037
ESE	0.1/ 0.00	2/ 0.09	8/ 0.37	4/ 0.18	2/ 0.09	/	/	16.1/ 0.74	7.55279
SE	0.1/ 0.00	2/ 0.09	4/ 0.18	2/ 0.09	1/ 0.05	/	/	9.1/ 0.42	6.92857
SSE	0.1/ 0.00	3/ 0.14	8/ 0.37	5/ 0.23	4/ 0.18	/	/	20.1/ 0.93	7.79229
S	/	/	2/ 0.09	19/ 0.87	14/ 0.64	1/ 0.05	/	36.0/ 1.66	12.16250
SSW	0.1/ 0.00	2/ 0.09	2/ 0.09	47/ 2.16	28/ 1.29	3/ 0.14	/	82.1/ 3.78	11.98158
SW	0.1/ 0.00	3/ 0.14	8/ 0.37	12/ 0.55	17/ 0.78	9/ 0.41	/	49.1/ 2.26	12.68579
WSW	0.1/ 0.00	3/ 0.14	4/ 0.18	16/ 0.74	10/ 0.46	2/ 0.09	/	35.1/ 1.62	10.84580
W	0.0/ 0.00	1/ 0.05	4/ 0.18	16/ 0.74	9/ 0.41	/	/	30.0/ 1.38	10.48500
WNW	/	/	5/ 0.23	19/ 0.87	19/ 0.87	/	/	43.0/ 1.98	11.63895
NW	/	/	6/ 0.28	31/ 1.43	6/ 0.28	1/ 0.05	/	44.0/ 2.03	10.18579
NNW	0.1/ 0.00	2/ 0.09	5/ 0.23	12/ 0.55	3/ 0.14	/	/	22.1/ 1.02	8.67421
TOTAL	1.0/ 0.05	27/ 1.24	95/ 4.37	265/ 12.20	130/ 5.99	16/ 0.74	/	534.0/24.59	10.30321

NUMBER OF BAD RECORDS: 2

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-78 MONTH=12 STABILITY=F
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	1/ 0.05	3/ 0.14	14/ 0.64	6/ 0.28	/	/	24.0/ 1.10	10.48958
NNE	/	2/ 0.09	6/ 0.28	11/ 0.51	/	/	/	19.0/ 0.87	7.76579
NE	/	1/ 0.05	6/ 0.28	6/ 0.28	2/ 0.09	/	/	15.0/ 0.69	8.17833
ENE	/	/	3/ 0.14	6/ 0.28	2/ 0.09	/	/	11.0/ 0.51	9.09318
E	/	1/ 0.05	2/ 0.09	10/ 0.46	/	/	/	13.0/ 0.60	8.41538
ESE	/	/	5/ 0.23	3/ 0.14	/	/	/	8.0/ 0.37	7.26562
SE	/	/	3/ 0.14	2/ 0.09	1/ 0.05	/	/	6.0/ 0.28	8.01667
SSE	/	1/ 0.05	3/ 0.14	8/ 0.37	1/ 0.05	/	/	13.0/ 0.60	8.85192
S	/	/	/	8/ 0.37	2/ 0.09	/	/	10.0/ 0.46	10.11250
SSW	/	2/ 0.09	3/ 0.14	21/ 0.97	8/ 0.37	1/ 0.05	/	35.0/ 1.61	10.73000
SW	/	2/ 0.09	1/ 0.05	13/ 0.60	2/ 0.09	/	/	18.0/ 0.83	9.24444
WSW	/	/	1/ 0.05	11/ 0.51	5/ 0.23	/	/	17.0/ 0.78	11.34706
W	/	1/ 0.05	5/ 0.23	11/ 0.51	8/ 0.37	/	/	25.0/ 1.15	10.62800
WNW	/	1/ 0.05	3/ 0.14	4/ 0.18	7/ 0.32	/	/	15.0/ 0.69	10.18833
NW	/	/	3/ 0.14	7/ 0.32	2/ 0.09	/	/	12.0/ 0.55	9.11458
NNW	/	/	6/ 0.28	8/ 0.37	2/ 0.09	/	/	16.0/ 0.74	9.06719
TOTAL	/	12/ 0.55	53/ 2.44	143/ 6.58	48/ 2.21	1/ 0.05	/	257.0/11.83	9.57840

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-17 (Cont'd)

SITE=SHNP YEAR=76-7 MONTH=12 STABILITY=G
UPWNDSPD

UPWNDDEG	CALM	.75-3.5	3.5-7.5	7.5-12.5	12.5-18.5	18.5-25	>= 25	TOTAL	AVERAGE UPWNDSPD
N	/	/	4/ 0.18	8/ 0.37	2/ 0.09	/	/	14.0/ 0.64	9.42857
NNE	/	/	7/ 0.32	14/ 0.64	1/ 0.05	/	/	22.0/ 1.01	7.94091
NE	/	1/ 0.05	5/ 0.23	5/ 0.23	/	/	/	11.0/ 0.51	7.29318
ENE	/	1/ 0.05	8/ 0.37	10/ 0.46	/	/	/	19.0/ 0.87	7.30658
E	/	1/ 0.05	4/ 0.18	2/ 0.09	/	/	/	7.0/ 0.32	5.91786
ESE	/	/	4/ 0.18	1/ 0.05	/	/	/	5.0/ 0.23	5.88000
SE	/	3/ 0.14	5/ 0.23	5/ 0.23	1/ 0.05	/	/	14.0/ 0.64	6.59643
SSE	/	4/ 0.18	19/ 0.87	13/ 0.60	/	/	/	36.0/ 1.66	6.97847
S	/	3/ 0.14	12/ 0.55	24/ 1.10	2/ 0.09	/	/	41.0/ 1.89	8.22866
SSW	/	4/ 0.18	19/ 0.87	30/ 1.38	5/ 0.23	/	/	58.0/ 2.67	8.49526
SW	/	4/ 0.18	17/ 0.78	27/ 1.24	1/ 0.05	/	/	49.0/ 2.26	7.75306
WSW	/	6/ 0.28	31/ 1.43	32/ 1.47	5/ 0.23	/	/	74.0/ 3.41	7.54054
W	/	4/ 0.18	10/ 0.46	7/ 0.32	1/ 0.05	/	/	22.0/ 1.01	6.61591
WNW	/	7/ 0.32	10/ 0.46	8/ 0.37	1/ 0.05	/	/	26.0/ 1.20	6.53173
NW	/	3/ 0.14	7/ 0.32	3/ 0.14	1/ 0.05	/	/	14.0/ 0.64	6.26786
NNW	/	2/ 0.09	11/ 0.51	10/ 0.46	1/ 0.05	/	/	24.0/ 1.10	6.78750
TOTAL	/	43/ 1.98	173/ 7.97	199/ 9.16	21/ 0.97	/	/	436.0/20.07	7.50957

NUMBER OF BAD RECORDS: 0

TABLE 2.3.3-18

WIND INSTRUMENT HEIGHT

STATION	WIND INSTRUMENT HEIGHT (feet above ground level)
Raleigh-Durham	20[a]
Greensboro	20
Charlotte	20[b]

[a] - 26 ft. until 6/18/63

[b] - 59 ft. until 11/17/60

TABLE 2.3.4-1

WIND VELOCITY GROUPINGS FROM THE JOINT WIND FREQUENCY DISTRIBUTION AND THE CORRESPONDING COMPUTATIONAL WIND VELOCITY

Wind Velocity Grouping	χ/Q Computational Velocity
0.00 - 0.75 MPH	0.75 MPH = 0.335 m/s
0.76 - 3.50 MPH	3.50 MPH = 1.563 m/s
3.51 - 7.50 MPH	7.50 MPH = 3.350 m/s
7.51 - 12.50 MPH	12.50 MPH = 5.583 m/s
12.51 - 18.50 MPH	18.50 MPH = 8.263 m/s
18.51 - 25.00 MPH	25.00 MPH = 11.167 m/s
25.01 - 99.00 MPH	26.00 MPH = 11.613 m/s

TABLE 2.3.4-2

SHNPP SITE BOUNDARY DISTANCES
(METERS)

Site Boundary From Plant	Distance
N	2133
NNE	2133
NE	2133
ENE	2133
E	2621
ESE	2255
SE	2133
SSE	3536
S	4998
SSW	3109
SW	2438
WSW	2133
W	2255
WNW	2133
NW	2133
NNW	2012

TABLE 2.3.4-3

SHNPP EXCLUSION AREA BOUNDARY DISTANCES (METERS)

Receptor Location From Plant Center	Distance From SHNPP Plant Center to Receptor Location	Distance Used for Accident Computations [(a)]
N	2133	2024
NNE	2133	2133
NE	2133	2133
ENE	2133	2133
E	2133	2133
ESE	2133	2133
SE	2133	2133
SSE	2133	2133
S	2194	2194
SSW	2133	2133
SW	2133	2133
WSW	2133	2133
W	2133	2133
WNW	2133	2103
NW	2103	2024
NNW	2042	2024

[(a)] As defined by Draft NRC Regulatory Guide 1.145, September 30, 1977

TABLE 2.3.4-4

SUMMARY OF 0-2 HOUR SHNPP 5 AND 50 PERCENTILE VALUES BY DIRECTION OF SHORT TERM DIFFUSION ESTIMATES AT THE MIN EXCLUSION BOUNDARY AREA

(January 1976 through December 1978)

Receptor Location From Plant Center	Dilution Factors (Sec/m^3) 5 Percentile	50 Percentile
N	4.18×10^{-4}	6.03×10^{-5}
NNE	4.03×10^{-4}	5.85×10^{-5}
NE	3.72×10^{-4}	4.02×10^{-5}
ENE	3.22×10^{-4}	2.36×10^{-5}
E	2.61×10^{-4}	1.08×10^{-5}
ESE	2.07×10^{-4}	1.03×10^{-5}
SE	2.63×10^{-4}	1.42×10^{-5}
SSE	4.10×10^{-4}	3.00×10^{-5}
S	6.17×10^{-4}	6.30×10^{-5}
SSW	5.73×10^{-4}	6.73×10^{-5}
SW	5.57×10^{-4}	4.97×10^{-5}
NSW	5.07×10^{-4}	3.83×10^{-5}
W	4.65×10^{-4}	2.56×10^{-5}
WNW	3.90×10^{-4}	1.48×10^{-5}
NW	3.45×10^{-4}	1.10×10^{-5}
NNW	3.44×10^{-4}	3.28×10^{-5}
Over All Site	4.03×10^{-4}	3.65×10^{-4}

TABLE 2.3.4-5

SHNPP WORST, 5 AND 50 PERCENTILE OF CUMULATIVE FREQUENCY DISTRIBUTION OF X/Q VALUES (SEC/M^3) BASED ON ΔT (59.85M - 11.03M) STABILITY DATA AND 12.46M WIND VELOCITY DATA

(January 1976 through December 1978)

Period	Worst	5 Percentile	50% Percentile
0-2 Hour Minimum Exclusion Boundary	6.17×10^{-4} (S)	6.17×10^{-4} (S)	6.73×10^{-5} (SSW)
0-2 Hour Actual Site Boundary	7.6×10^{-4} (W)	5.1×10^{-4} (W)	5.8×10^{-5} (N)
0-8 Hour Low Population Zone	- - -	1.4×10^{-4} (W)	1.6×10^{-5} (SSW)
8-24 Hour Low Population Zone	- - -	1.0×10^{-4} (W)	1.3×10^{-5} (SSW)
1-3 Day Low Population Zone	- - -	5.9×10^{-5} (W)	7.4×10^{-6} (SSW)
4-30 Day Low Population Zone	- - -	2.4×10^{-5} (W)	3.4×10^{-6} (SSW)

TABLE 2.3.5-1

ANNUAL AVERAGE DILUTION FACTORS AT THE SHNPP MINIMUM EXCLUSION BOUNDARY

(January 1976 through December 1978)

		(Sec/m^3)			(m^{-2})
Receptor Location From Plant Center	Distance (Meters)	χ/Q No Decay Undepleted	χ/Q 2.26 Day Decay Undepleted	χ/Q 8.00 Day Decay Depleted	D/Q
N	2133	$5.6X10^{-6}$	$5.5X10^{-6}$	$4.8X10^{-6}$	$7.6X10^{-9}$
NNE	2133	$5.4X10^{-6}$	$5.3X10^{-6}$	$4.6X10^{-6}$	$8.9X10^{-9}$
NE	2133	$4.3X10^{-6}$	$4.2X10^{-6}$	$3.7X10^{-6}$	$8.0X10^{-9}$
ENE	2133	$3.3X10^{-6}$	$3.3X10^{-6}$	$2.9X10^{-6}$	$6.4X10^{-9}$
E	2133	$2.4X10^{-6}$	$2.4X10^{-6}$	$2.1X10^{-6}$	$4.1X10^{-9}$
ESE	2133	$2.1X10^{-6}$	$2.1X10^{-6}$	$1.8X10^{-6}$	$4.8X10^{-9}$
SE	2133	$2.5X10^{-6}$	$2.4X10^{-6}$	$2.1X10^{-6}$	$5.2X10^{-9}$
SSE	2133	$4.0X10^{-6}$	$3.9X10^{-6}$	$3.4X10^{-6}$	$6.1X10^{-9}$
S	2194	$6.0X10^{-6}$	$5.9X10^{-6}$	$5.1X10^{-6}$	$7.9X10^{-9}$
SSW	2133	$6.0X10^{-6}$	$5.9X10^{-6}$	$5.1X10^{-6}$	$7.5X10^{-9}$
SW	2133	$5.5X10^{-6}$	$5.3X10^{-6}$	$4.7X10^{-6}$	$6.2X10^{-9}$
WSW	2133	$4.8X10^{-6}$	$4.7X10^{-6}$	$4.1X10^{-6}$	$5.0X10^{-9}$
W	2133	$4.1X10^{-6}$	$4.0X10^{-6}$	$3.5X10^{-6}$	$4.0X10^{-9}$
WNW	2103	$3.5X10^{-6}$	$3.4X10^{-6}$	$3.0X10^{-6}$	$3.5X10^{-9}$
NW	2024	$3.3X10^{-6}$	$3.2X10^{-6}$	$2.8X10^{-6}$	$3.8X10^{-9}$
NNW	2024	$4.0X10^{-6}$	$3.9X10^{-6}$	$3.4X10^{-6}$	$5.3X10^{-9}$

TABLE 2.3.5-2

ANNUAL AVERAGE DILUTION FACTORS AT THE SHNPP ACTUAL SITE BOUNDARIES

(January 1976 through December 1978)

Receptor Location From Plant Center	Distance (Meters)	χ/Q No Decay Undepleted (Sec/m^3)	χ/Q 2.26 Day Decay Undepleted (Sec/m^3)	χ/Q 8.00 Day Decay Depleted (Sec/m^3)	D/Q (m^{-2})
N	2133	$5.0X10^{-6}$	$4.9X10^{-6}$	$4.2X10^{-6}$	$6.7X10^{-9}$
NNE	2133	$5.4X10^{-6}$	$5.3X10^{-6}$	$4.6X10^{-6}$	$8.9X10^{-9}$
NE	2133	$4.3X10^{-6}$	$4.2X10^{-6}$	$3.7X10^{-6}$	$8.0X10^{-9}$
ENE	2133	$3.3X10^{-6}$	$3.3X10^{-6}$	$2.9X10^{-6}$	$6.4X10^{-9}$
E	2621	$1.6X10^{-6}$	$1.5X10^{-6}$	$1.3X10^{-6}$	$2.5X10^{-9}$
ESE	2255	$1.9X10^{-6}$	$1.8X10^{-6}$	$1.6X10^{-6}$	$4.2X10^{-9}$
SE	2133	$2.5X10^{-6}$	$2.4X10^{-6}$	$2.1X10^{-6}$	$5.2X10^{-9}$
SSE	3536	$4.0X10^{-6}$	$3.9X10^{-6}$	$3.4X10^{-6}$	$6.1X10^{-9}$
S	4998	$1.2X10^{-6}$	$1.2X10^{-6}$	$9.7X10^{-6}$	$1.1X10^{-9}$
SSW	3109	$2.8X10^{-6}$	$2.7X10^{-6}$	$2.3X10^{-6}$	$3.0X10^{-9}$
SW	2438	$4.5X10^{-6}$	$4.4X10^{-6}$	$3.8X10^{-6}$	$5.0X10^{-9}$
WSW	2133	$4.8X10^{-6}$	$4.7X10^{-6}$	$4.1X10^{-6}$	$5.0X10^{-9}$
W	2255	$3.7X10^{-6}$	$3.6X10^{-6}$	$3.1X10^{-6}$	$3.5X10^{-9}$
WNW	2133	$3.4X10^{-6}$	$3.3X10^{-6}$	$2.9X10^{-6}$	$3.4X10^{-9}$
NW	2133	$2.9X10^{-6}$	$2.9X10^{-6}$	$2.5X10^{-6}$	$3.3X10^{-9}$
NNW	2012	$3.5X10^{-6}$	$3.5X10^{-6}$	$3.0X10^{-6}$	$4.6X10^{-9}$

TABLE 2.3.5-3

ANNUAL AVERAGE DILUTION FACTORS AT THE SHNPP LOW POPULATION ZONE BOUNDARIES

(January 1976 through December 1978)

		(Sec/m^3)			(m^{-2})
Receptor Location From Plant Center	Distance (Meters)	χ/Q No Decay Undepleted	χ/Q 2.26 Day Decay Undepleted	χ/Q 8.00 Day Decay Depleted	D/Q
N	4828	$1.0X10^{-6}$	$9.6X10^{-7}$	$7.9X10^{-7}$	$9.5X10^{-9}$
NNE	4828	$1.1X10^{-6}$	$1.0X10^{-6}$	$8.6X10^{-7}$	$1.3X10^{-10}$
NE	4828	$8.8X10^{-7}$	$8.4X10^{-7}$	$6.9X10^{-7}$	$1.1X10^{-9}$
ENE	4828	$6.8X10^{-7}$	$6.5X10^{-7}$	$5.3X10^{-7}$	$9.1X10^{-10}$
E	4828	$4.9X10^{-7}$	$4.7X10^{-7}$	$3.8X10^{-7}$	$5.9X10^{-10}$
ESE	4828	$4.2X10^{-7}$	$4.0X10^{-7}$	$3.3X10^{-7}$	$6.8X10^{-10}$
SE	4828	$5.0X10^{-7}$	$4.8X10^{-7}$	$3.9X10^{-7}$	$7.3X10^{-10}$
SSE	4828	$8.2X10^{-7}$	$7.7X10^{-7}$	$6.4X10^{-7}$	$8.6X10^{-10}$
S	4828	$1.3X10^{-6}$	$1.2X10^{-6}$	$1.0X10^{-6}$	$1.2X10^{-9}$
SSW	4828	$1.3X10^{-6}$	$1.2X10^{-6}$	$9.8X10^{-7}$	$1.1X10^{-9}$
SW	4828	$1.1X10^{-6}$	$1.1X10^{-6}$	$8.9X10^{-7}$	$8.8X10^{-10}$
WSW	4828	$1.0X10^{-6}$	$9.5X10^{-7}$	$7.9X10^{-7}$	$7.1X10^{-10}$
W	4828	$8.6X10^{-7}$	$8.1X10^{-7}$	$6.7X10^{-7}$	$5.7X10^{-10}$
WNW	4828	$7.2X10^{-7}$	$6.7X10^{-7}$	$5.6X10^{-7}$	$4.8X10^{-10}$
NW	4828	$6.1X10^{-7}$	$6.7X10^{-7}$	$4.7X10^{-7}$	$4.6X10^{-10}$
NNW	4828	$7.2X10^{-7}$	$6.9X10^{-7}$	$5.7X10^{-7}$	$6.6X10^{-10}$

TABLE 2.3.5-4

ANNUAL AVERAGE DILUTION FACTORS FOR INCREMENTAL DISTANCES AT SHNPP

(January 1976 through December 1978)

NO DECAY, UNDEPLETED

CORRECTED FOR OPEN TERRAIN RECIRCULATION

ANNUAL AVERAGE CHI/Q (SEC/METER CUBED)							DISTANCE IN MILES				
SECTOR	0.250	0.500	0.750	1.000	1.500	2.000	2.500	3.000	3.500	4.000	4.500
S	1.733E-04	5.134E-05	2.522E-05	1.237E-05	4.882E-06	2.790E-06	1.841E-06	1.326E-06	1.013E-09	8.075E-07	6.643E-07
SSW	1.630E-04	4.852E-05	2.395E-05	1.177E-05	4.650E-06	2.652E-06	1.747E-06	1.256E-06	9.591E-07	7.639E-07	6.280E-07
SW	1.486E-04	4.405E-05	2.169E-05	1.065E-05	4.208E-06	2.404E-06	1.586E-06	1.142E-06	8.722E-07	6.951E-07	5.718E-07
WSW	1.316E-04	3.692E-05	1.911E-05	9.393E-06	3.713E-06	2.124E-06	1.402E-06	1.010E-06	7.724E-07	6.158E-07	5.067E-07
W	1.127E-04	3.326E-05	1.632E-05	8.005E-06	3.161E-06	1.811E-06	1.196E-06	8.626E-07	6.600E-07	5.265E-07	4.335E-07
WNW	9.280E-05	2.742E-05	1.347E-05	6.636E-06	2.630E-06	1.504E-06	9.927E-07	7.151E-07	5.465E-07	4.357E-07	3.585E-07
NW	7.812E-05	2.324E-05	1.151E-05	5.682E-06	2.253E-06	1.284E-06	8.452E-07	6.077E-07	4.637E-07	3.691E-07	3.033E-07
NNW	9.051E-05	2.721E-05	1.367E-05	6.829E-06	2.731E-06	1.545E-06	1.011E-06	7.235E-07	5.499E-07	4.364E-07	3.577E-07
N	1.263E-04	3.797E-05	1.914E-05	9.572E-06	3.830E-06	2.165E-06	1.416E-06	1.013E-06	7.699E-07	6.108E-07	5.004E-07
NNE	1.362E-04	4.122E-05	2.086E-05	1.045E-05	4.179E-06	2.353E-06	1.535E-06	1.096E-06	8.313E-07	6.585E-07	5.389E-07
NE	1.114E-04	3.363E-05	1.688E-05	8.398E-06	3.341E-06	1.885E-06	1.231E-06	8.798E-07	6.681E-07	5.297E-07	4.338E-07
ENE	8.748E-05	2.632E-05	1.312E-05	6.498E-06	2.575E-06	1.455E-06	9.520E-07	6.812E-07	5.178E-07	4.109E-07	3.368E-07
E	6.368E-05	1.914E-05	9.521E-06	4.702E-06	1.8605-06	1.053E-06	6.903E-07	4.946E-07	3.768E-07	2.989E-07	2.452E-07
ESE	5.435E-05	1.642E-05	8.223E-06	4.066E-06	1.608E-06	9.063E-07	5.919E-07	4.230E-07	3.212E-07	2.547E-07	2.086E-07
SE	6.494E-05	1.951E-05	9.734E-06	4.798E-06	1.894E-06	1.072E-06	7.017E-07	5.025E-07	3.823E-07	3.036E-07	2.439E-07
SSE	1.060E-04	3.163E-05	1.565E-05	7.712E-06	3.049E-06	1.734E-06	1.140E-06	8.188E-07	6.242E-07	4.966E-07	4.079E-07

TABLE 2.3.5-4 (Continued)

ANNUAL AVERAGE CHI/Q (SEC/METER CUBED)						DISTANCE IN MILES					
BEARING	5.000	7.5000	10.000	15.000	20.000	25.000	30.000	35.000	40.000	45.000	50.000
S	5.601E-07	3.082E-07	2.092E-07	1.278E-07	9.028E-08	6.905E-08	5.551E-08	4.618E-08	3.939E-08	3.425E-08	3.023E-08
SSW	5.291E-07	2.905E-07	1.969E-07	1.200E-07	8.467E-08	6.468E-08	5.196E-08	4.320E-08	3.683E-08	3.200E-08	2.823E-08
SW	4.320E-07	2.651E-07	1.799E-07	1.098E-07	7.756E-08	5.930E-08	4.766E-08	3.964E-08	3.381E-08	2.939E-08	2.593E-08
WSW	4.273E-07	2.3538-07	1.598E-07	9.761E-08	6.897E-08	5.275E-09	4.241E-08	3.528E-08	3.010E-08	2.617E-08	2.310E-08
W	3.657E-07	2.016E-07	1.371E-07	8.3855-08	5.930E-08	4.538E-08	3.650E-08	3.038E-08	2.593E-08	2.255E-08	1.991E-08
WNW	3.022E-07	1.663E-07	1.129E-07	6.894E-08	4.869E-08	3.723E-08	2.992E-08	2.489E-08	2.122E-8	1.845E-08	1.628E-08
NW	2.555E-07	1.401E-07	9.485E-08	5.773E-08	4.069E-08	3.106E-08	2.493E-08	2.072E-08	1.766E-08	1.534E-08	1.358E-08
NNW	3.005E-07	1.633E-07	1.099E-07	6.633E-08	4.649E-08	3.534E-08	2.827E-08	2.343E-08	1.992E-08	1.726E-08	1.520E-08
N	4.204E-07	2.282E-07	1.535E-07	9.262E-08	6.489E-08	4.931E-08	3.943E-08	3.267E-08	2.776E-08	2.406E-08	2.119E-08
NNE	4.522E-07	2.446E-07	1.641E-07	9.866E-08	6.899E-08	5.235E-08	4.182E-08	3.461E-08	2.939E-08	2.545E-08	2.239E-08
NE	3.643E-07	1.976E-07	1.329E-07	8.014E-08	5.616E-08	4.269E-08	3.415E-08	2.830E-08	2.406E-08	2.066E-08	1.8365-08
ENE	2.831E-07	1.540E-07	1.038E-07	6.276E-08	4.407E-08	3.355E-08	2.687E-08	2.229E-08	1.897E-08	1.646E-08	1.450E-08
E	2.062E-07	1.124E-07	7.584E-08	4.596E-08	3.231E-08	2.462E-08	1.974E-08	1.638E-08	1.395E-08	1.211E-08	1.067E-08
ESE	1.752E-07	9.513E-08	6.402E-08	3.866E-08	2.713E-08	2.064E-08	1.652E-08	1.370E-08	1.165E-08	1.011E-08	8.904E-09
SE	2.093E-07	1.141E-07	7.699E-08	4.666E-08	3.281E-08	2.500E-08	2.004E-08	1.664E-08	1.416E-08	1.230E-08	1.084E-08
SSE	3.434E-07	1.830E-07	1.272E-07	7.738E-08	5.452E-08	4.162E-08	3.340E-08	2.776E-08	2.365E-08	2.054E-08	1.812E-08

TABLE 2.3.5-5

ANNUAL AVERAGE DILUTION FACTORS FOR INCREMENTAL DISTANCES AT SHNPP

(January 1976 through December 1978)

2.260 DAY DECAY, UNDEPLETED

CORRECTED FOR OPEN TERRAIN RECIRCULATION

ANNUAL AVERAGE CHI/Q (SEC/METER CUBED)							DISTANCE IN MILES				
SECTOR	0.250	0.500	0.750	1.000	1.500	2.000	2.500	3.000	3.500	4.000	4.500
S	1.724E-04	5.083E-05	2.485E-05	1.214E-05	4.744E-06	2.684E-06	1.753E-06	1.250E-06	9.454E-07	7.460E-07	6.076E-07
SSW	1.521E-04	4.804E-05	2.361E-05	1.155E-05	4.520E-06	2.553E-06	1.665E-06	1.186E-06	8.960E-07	7.065E-07	5.751E-07
SW	1.478E-04	4.360E-05	2.137E-05	1.045E-05	4.088E-06	2.312E-06	1.509E-06	1.076E-06	8.136E-07	6.419E-07	5.227E-07
WSW	1.309E-04	3.852E-05	1.883E-05	9.209E-06	3.606E-06	2.042E-06	1.334E-06	9.516E-07	7.200E-07	5.682E-07	5.227E-07
W	1.121E-05	3.292E-05	1.607E-05	7.846E-06	3.068E-06	1.740E-06	1.138E-06	8.119E-07	6.146E-07	4.853E-07	4.628E-07
WNW	9.233E-05	2.714E-05	1.327E-05	6.508E-06	2.555E-06	1.477E-06	9.452E-07	6.741E-07	5.100E-07	4.025E-07	3.954E-07
NW	7.774E-05	2.301E-05	1.135E-05	5.577E-06	2.191E-06	1.237E-06	8.061E-07	5.739E-07	4.336E-07	3.418E-07	3.279E-07
NNW	9.011E-05	2.698E-05	1.350E-05	6.718E-06	2.665E-06	1.495E-06	9.697E-07	6.880E-07	5.184E-07	4.078E-07	2.782E-07
N	1.257E-04	3.765E-05	1.890E-05	9.418E-06	3.739E-06	2.096E-06	1.359E-06	9.639E-07	7.262E-07	5.712E-07	3.313E-07
NNE	1.357E-04	4.089E-05	2.061E-05	1.029E-05	4.085E-06	2.281E-06	1.476E-06	1.045E-06	7.862E-07	6.176E-07	4.640E-07
NE	1.109E-04	3.335E-05	1.667E-05	8.264E-06	3.262E-06	1.824E-06	1.181E-06	8.371E-07	6.301E-07	4.953E-07	5.013E-07
ENE	8.708E-05	2.609E-05	1.295E-05	6.389E-06	2.511E-06	1.406E-06	9.118E-07	6.466E-07	4.871E-07	3.830E-07	4.021E-07
E	6.339E-05	1.897E-05	9.399E-06	4.624E-06	1.814E-06	1.018E-06	6.613E-07	4.696E-07	3.541E-07	2.787E-07	3.111E-07
ESE	5.413E-05	1.630E-05	8.132E-06	4.008E-06	1.573E-06	8.801E-07	5.703E-07	4.043E-07	3.046E-07	2.396E-07	2.266E-07
SE	6.465E-05	1.934E-05	9.614E-06	4.721E-06	1.849E-06	1.037E-06	6.730E-07	4.778E-07	3.602E-07	2.835E-07	1.947E-07
SSE	1.055E-04	3.133E-05	1.544E-05	7.572E-06	2.967E-06	1.672E-06	1.088E-06	7.741E-07	5.845E-07	4.605E-07	2.304E-07

TABLE 2.3.5-5 (Continued)

ANNUAL AVERAGE CHI/Q (SEC/METER CUBED)						DISTANCE IN MILES					
BEARING	5.000	7.5000	10.000	15.000	20.000	25.000	30.000	35.000	40.000	45.000	50.000
S	5.072E-07	2.655E-07	1.717E-07	9.536E-08	6.155E-08	4.320E-08	3.201E-08	2.465E-08	1.955E-08	1.587E-08	1.313E-08
SSW	4.798E-07	2.507E-07	1.619E-07	8.981E-08	5.792E-08	4.062E-08	3.008E-08	2.316E-08	1.836E-08	1.489E-08	1.231E-08
SW	4.362E-07	2.232E-07	1.474E-07	8.178E-08	5.271E-08	3.695E-08	2.734E-08	2.103E-08	1.666E-08	1.350E-08	1.115E-08
WSW	3.863E-07	2.023E-07	1.307E-07	7.255E-08	4.577E-08	3.278E-08	2.426E-08	1.866E-08	1.478E-08	1.198E-08	9.893E-09
W	3.302E-07	1.730E-07	1.118E-07	6.207E-08	4.000E-09	2.802E-03	2.071E-08	1.592E-08	1.260E-08	1.020E-08	8.416E-09
WNW	2.737E-07	1.433E-07	9.263E-08	5.144E-08	3.319E-08	2.329E-08	1.724E-08	1.327E-08	1.051E-08	8.523E-09	7.042E-09
NW	2.320E-07	1.212E-07	7.824E-08	4.341E-08	2.801E-08	1.965E-08	1.456E-08	1.122E-08	8.897E-09	7.221E-09	5.972E-09
NNW	2.760E-07	1.436E-07	9.259E-08	5.145E-08	3.332E-08	2.350E-08	1.750E-08	1.355E-08	1.080E-08	8.811E-09	7.321E-09
N	3.864E-07	2.010E-07	1.297E-07	7.211E-08	4.674E-08	3.298E-08	2.459E-08	1.906E-08	1.521E-08	1.242E-08	1.033E-08
NNE	4.172E-07	2.165E-07	1.395E-07	7.754E-08	5.029E-08	3.553E-08	2.652E-08	2.059E-08	1.645E-08	1.345E-08	1.120E-08
NE	3.349E-07	1.739E-07	1.121E-07	6.226E-08	4.033E-08	2.845E-08	2.121E-08	1.643E-08	1.311E-08	1.071E-08	8.907E-09
ENE	2.591E-07	1.347E-07	8.685E-08	4.820E-08	3.119E-08	2.196E-08	1.634E-08	1.264E-08	1.007E-08	8.211E-09	6.820E-09
E	1.886E-07	9.841E-08	6.353E-08	3.534E-08	2.289E-08	1.614E-08	1.202E-08	9.305E-09	7.417E-09	6.049E-09	5.026E-09
ESE	1.662E-07	8.465E-08	5.479E-08	3.067E-08	2.002E-08	1.423E08	1.067E-08	8.323E-09	6.680E-09	5.484E-09	4.584E-09
SE	1.920E-07	1.002E-07	6.470E-08	3.604E-08	2.338E-08	1.650E-08	1.231E-08	9.538E-09	7.612E-09	6.216E-09	5.171E-09
SSE	3.124E-07	1.631E-07	1.053E-07	5.843E-08	3.774E-08	2.652E-08	1.968E-08	1.518E-08	1.206E-08	9.805E-09	8.123E-09

TABLE 2.3.5-6

ANNUAL AVERAGE DILUTION FACTORS FOR INCREMENTAL DISTANCES AT SHNPP

(January 1976 through December 1978)

8.000 DAY DECAY, DEPLETED

CORRECTED FOR OPEN TERRAIN RECIRCULATION

ANNUAL AVERAGE CHI/Q (SEC/METER CUBED)						DISTANCE IN MILES					
SECTOR	0.250	0.500	0.750	1.000	1.500	2.000	2.500	3.000	3.500	4.000	4.500
S	1.638E-04	4.675E-05	2.238E-05	1.077E-05	4.113E-06	2.285E-06	1.470-06	1.035E-06	7.750E-07	6.059E-07	4.896E-07
SSW	1.540E-04	4.418E-05	2.126E-05	1.025E-05	3.917E-06	2.172E-06	1.396E-06	9.816E-07	7.340E-07	5.734E-07	4.630E-07
SW	1.404E-04	4.011E-05	1.926E-05	9.275E-06	3.545E-06	1.969E-06	1.266E-06	8.916E-07	6.672E-07	5.216E-07	4.213E-07
WSW	1.244E-04	3.544E-05	1.696E-05	8.176E-06	3.127E-06	1.739E-06	1.120E-06	7.890E-07	5.907E-07	4.620E-07	3.733E-07
W	1.065E-04	3.029E-05	1.448E-05	6.967E-06	2.662E-06	1.482E-06	9.553E-07	6.736E-07	5.046E-07	3.948E-07	3.192E-07
WNW	8.770E-05	2.497E-05	1.195E-05	5.777E-06	2.215E-06	1.232E-06	7.929E-07	5.585E-07	4.181E-07	3.270E-07	2.642E-07
NW	7.383E-05	2.116E-05	1.021E-05	4.948E-06	1.898E-06	1.052E-06	6.755E-07	4.749E-07	3.549E-07	2.772E-07	2.237E-07
NNW	8.555E-05	2.479E-05	1.213E-05	5.950E-06	2.303E-06	1.267E-06	8.092E-07	5.665E-07	4.219E-07	3.286E-07	2.645E-07
N	1.194E-04	3.459E-05	1.699E-05	8.341E-06	3.231E-06	1.776E-06	1.134E-06	7.934E-07	5.907E-07	4.599E-07	3.702E-07
NNE	1.288E-04	3.756E-05	1.852E-05	9.104E-06	3.526E-06	1.931E-06	1.230E-06	8.587E-07	6.383E-07	4.963E-07	3.990E-07
NE	1.053E-04	3.064E-05	1.499E-05	7.317E-06	2.818E-06	1.546E-06	9.855E-07	6.890E-07	5.126E-07	3.989E-07	3.209E-07
ENE	8.269E-05	2.397E-05	1.165E-05	5.660E-06	2.171E-06	1.193E-07	7.617E-07	5.331E-07	3.970E-07	3.092E-07	2.489E-07
E	6.019E-05	1.743E-05	8.454E-06	4.097E-06	1.569E-06	8.640E-07	5.524E-07	3.871E-07	2.886E-07	2.249E-07	1.812E-07
ESE	5.138E-05	1.496E-05	7.305E-06	3.545E-06	1.357E-06	7.442E-07	4.744E-07	3.317E-07	2.468E-07	1.921E-07	1.546E-07
SE	6.138E-05	1.777E-05	8.644E-06	4.181E-06	1.597E-06	8.790E-07	5.617E-07	3.934E-07	2.932E-07	2.285E-07	1.841E-07
SSE	1.002E-04	2.881E-05	1.389E-05	6.716E-06	2.570E-06	1.421E-06	9.112E-07	6.400E-07	4.780E-07	3.731E-07	3.010E-07

TABLE 2.3.5-6 (Continued)

ANNUAL AVERAGE CHI/Q (SEC/METER CUBED)						DISTANCE IN MILES					
BEARING	5.000	7.5000	10.000	15.000	20.000	25.000	30.000	35.000	40.000	45.000	50.000
S	4.057E-07	2.081E-07	1.326E-07	7.281E-08	4.693E-08	3.304E-08	2.460E-08	1.904E-08	1.517E-08	1.236E-08	1.024E-08
SSW	3.835E-07	1.962E-07	1.249E-07	6.844E-08	4.405E-08	3.098E-08	2.306E-08	1.784E-08	1.421E-08	1.157E-08	9.584E-09
SW	3.491E-07	1.789E-07	1.140E-07	6.254E-08	4.028E-08	2.834E-08	2.110E-08	1.633E-08	1.300E-08	1.058E-08	8.770E-09
WSW	3.094E-07	1.587E-07	1.012E-07	5.556E-08	3.580E-08	2.520E-08	1.876E-08	1.452E-08	1.156E-08	9.416E-09	7.802E-09
W	2.647E-07	1.460E-07	8.673E-06	4.767E-08	3.074E-08	2.164E-08	1.611E-08	1.247E-08	9.932E-09	8.086E-09	6.699E-09
WNW	2.190E-07	1.123E-07	7.158E-08	3.929E-08	2.531E-08	1.731E-08	1.326E-08	1.027E-08	8.177E-09	6.659E-09	5.518E-09
NW	1.852E-07	9.468E-08	6.023E-08	3.297E-08	2.121E-08	1.491E-08	1.110E-08	8.585E-09	6.836E-09	5.566E-09	4.611E-09
NNW	2.186E-07	1.109E-07	7.019E-08	3.822E-08	2.452E-08	1.721E-08	1.280E-08	9.901E-09	7.887E-09	6.424E-09	5.327E-09
N	3.058E-07	1.551E-07	9.815E-08	5.342E-08	3.427E-08	2.406E-08	1.789E-08	1.384E-08	1.103E-08	8.983E-09	7.450E-09
NNE	3.298E-07	1.664E-07	1.051E-07	5.707E-08	3.656E-08	2.565E-08	1.906E-08	1.475E-08	1.175E-08	9.570E-09	7.938E-09
NE	2.650E-07	1.342E-07	8.491E-08	4.619E-08	2.963E-08	2.030E-08	1.547E-08	1.197E-08	9.533E-09	7.765E-09	6.440E-09
ENE	2.057E-07	1.044E-07	6.615E-08	3.605E-08	2.315E-08	1.626E-08	1.209E-08	9.357E-09	7.454E-09	6.071E-09	5.034E-09
E	1.498E-07	7.623E-08	4.837E-08	2.641E-08	1.698E-08	1.194E-08	8.887E-09	6.880E-09	5.484E-09	4.469E-09	3.707E-09
ESE	1.277E-07	6.482E-08	4.108E-08	2.242E-08	1.443E-08	1.016E-08	7.574E-09	5.875E-09	4.692E-09	3.832E-09	3.185E-09
SE	1.522E-07	7.744E-08	4.914E-08	2.684E-08	1.727E-08	1.215E-08	9.044E-09	7.004E-09	5.585E-09	4.553E-09	3.778E-09
SSE	2.491E-07	1.272E-07	8.085E-08	4.424E-08	2.846E-08	2.002E-08	1.490E-08	1.153E-08	9.183E-09	7.479E-09	6.199E-09

TABLE 2.3.5-7

ANNUAL AVERAGE DILUTION FACTORS FOR INCREMENTAL DISTANCES AT SHNPP

(January 1976 through December 1978)

CORRECTED FOR OPEN TERRAIN RECIRCULATION

RELATIVE DEPOSITION PER UNIT AREA (M-2) AT FIXED POINTS BY DOWNWIND SECTORS

DIRECTION FROM	DISTANCE IN MILES										
SITE	0.25	0.50	0.75	1.00	1.50	2.00	2.50	3.00	3.50	4.00	4.50
S	2.095E-07	7.083E-08	3.637E-08	1.729E-08	6.211E-09	3.080E-09	1.814E-09	1.188E-09	8.356E-10	6.193E-10	4.772E-10
SSW	1.869E-07	6.321E-08	3.245E-08	1.543E-08	5.542E-09	2.748E-09	1.618E-09	1.060E-09	7.456E-10	5.526E-10	4.258E-10
SW	1.561E-07	5.279E-08	2.710E-08	1.289E-08	4.628E-09	2.295E-09	1.352E-09	8.850E-10	6.227E-10	4.615E-10	3.556E-10
WSW	1.257E-07	4.249E-08	2.182E-08	1.037E-08	3.726E-09	1.848E-09	1.088E-09	7.124E-10	5.013E-10	3.715E-10	2.863E-10
W	9.986E-08	3.377E-08	1.734E-08	8.243E-09	2.961E-09	1.468E-09	8.646E-10	5.661E-10	3.984E-10	2.952E-10	2.275E-10
WNW	8.492E-08	2.872E-08	1.474E-08	7.010E-09	2.518E-09	1.249E-09	7.392E-10	4.814E-10	3.388E-10	2.511E-10	1.935E-10
NW	8.186E-08	2.768E-08	1.421E-08	6.757E-09	2.427E-09	1.204E-09	7.087E-10	4.641E-10	3.265E-10	2.420E-10	1.865E-10
NNW	1.157E-07	3.914E-08	2.009E-08	9.553E-09	3.431E-09	1.702E-09	1.002E-09	6.561E-10	4.617E-10	3.421E-10	2.637E-10
N	1.668E-07	5.640E-08	2.896E-08	1.377E-06	4.945E-09	2.452E-09	1.444E-09	9.455E-10	6.653E-10	4.930E-10	3.800E-10
NNE	2.229E-07	7.538E-08	3.871E-08	1.840E-08	6.610E-09	3.278E-09	1.930E-09	1.264E-09	8.893E-10	6.590E-10	5.079E-10
NE	2.007E-07	6.783E-08	3.484E-08	1.656E-08	5.949E-09	2.950E-09	1.737E-09	1.138E-09	8.004E-10	5.932E-10	4.571E-10
ENE	1.602E-07	5.417E-08	2.781E-08	1.322E-08	4.750E-09	2.355E-09	1.387E-09	9.081E-10	6.390E-10	4.736E-10	3.649E-10
E	1.036E-07	3.502E-08	1.798E-08	8.550E-09	3.071E-09	1.523E-09	8.967E-10	5.872E-10	4.132E-10	3.062E-10	2.360E-10
ESE	1.196E-07	4.045E-08	2.077E-08	9.875E-09	3.547E-09	1.759E-09	1.036E-09	6.782E-10	4.772E-10	3.537E-10	2.725E-10
SE	1.294E-07	4.375E-08	2.246E-08	1.068E-08	3.836E-09	1.902E-09	1.120E-09	7.335E-10	5.161E-10	3.825E-10	2.947E-10
SSE	1.524E-07	5.153E-08	2.646E-08	1.258E-08	4.518E-09	2.241E-09	1.219E-09	8.639E-09	6.079E-09	4.505E-09	3.472E-10

TABLE 2.3.5-7 (Continued)

DIRECTION FROM	DISTANCE IN MILES										
SITE	5.00	7.500	10.00	15.00	20.00	25.00	30.00	35.00	40.00	45.00	50.00
S	3.791E-10	1.684E-10	1.020E-10	5.157E-11	3.121E-11	2.093E-11	1.499E-11	1.126E-11	8.754E-12	6.993E-12	5.708E-12
SSW	3.383E-10	1.503E-10	9.104E-11	4.601E-11	2.785E-11	1.867E-11	1.338E-11	1.005E-11	7.812E-12	6.240E-12	5.093E-12
SW	2.825E-10	1.255E-10	7.603E-11	3.843E-11	2.326E-11	1.559E-11	1.117E-11	8.391E-12	6.524E-12	5.211E-12	4.254E-12
WSW	2.274E-10	1.010E-10	6.120E-11	3.093E-11	1.872E-11	1.255E-11	8.995E-12	6.754E-12	5.252E-12	4.195E-12	3.424E-12
W	1.807E-10	8.029E-11	4.864E-11	2.458E-11	1.488E-11	9.976E-12	7.148E-12	5.368E-12	4.173E-12	3.334E-12	2.721E-12
WNW	1.537E-10	6.828E-11	4.136E-11	2.090E-11	1.265E-11	8.483E-12	6.079E-12	4.564E-12	3.549E-12	2.835E-12	2.314E-12
NW	1.482E-10	6.581E-11	3.987E-11	2.015E-11	1.220E-11	8.177E-12	5.860E-12	4.400E-12	3.421E-12	2.733E-12	2.231E-12
NNW	2.095E-10	9.305E-11	5.637E-11	2.849E-11	1.724E-11	1.156E-11	8.284E-12	6.221E-12	4.837E-12	3.864E-12	3.154E-12
N	3.018E-10	1.341E-10	8.123E-11	4.106E-11	2.485E-11	1.666E-11	1.194E-11	8.964E-12	6.970E-12	5.568E-12	4.544E-12
NNE	4.035E-10	1.792E-10	1.086E-10	5.488E-11	3.321E-11	2.227E-11	1.596E-11	1.198E-11	9.317E-12	7.442E-12	6.074E-12
NE	3.632E-10	1.613E-10	9.772E-11	4.939E-11	2.990E-11	2.004E-11	1.436E-11	1.079E-11	8.386E-12	6.699E-12	5.468E-12
ENE	2.899E-10	1.288E-10	7.802E-11	3.943E-11	2.387E-11	1.600E-11	1.147E-11	8.610E-12	6.695E-12	5.348E-12	4.365E-12
E	1.875E-10	8.328E-11	5.044E-11	2.550E-11	1.543E-11	1.035E-11	7.414E-12	5.567E-12	4.329E-12	3.458E-12	2.882E-12
ESE	2.165E-10	9.618E-11	5.826E-11	2.945E-11	1.782E-11	1.195E-11	8.563E-12	6.430E-12	5.000E-12	3.994E-12	3.260E-12
SE	2.342E-10	1.040E-10	6.301E-11	3.185E-11	1.928E-11	1.292E-11	9.261E-12	6.954E-12	5.407E-12	4.319E-12	3.525E-12
SSE	2.758E-10	1.225E-10	7.422E-11	3.751E-11	2.271E-11	1.522E-11	1.091E-11	8.191E-12	6.369E-12	5.087E-12	4.152E-12

TABLE 2.4.1-1 ESTIMATED MONTHLY AVERAGE FLOWS OF BUCKHORN CREEK (CFS) (AVERAGE 1924 – 1981 = 87.2) DRAINAGE AREA = 79.5 SQ MI.

WATER YEAR	OCTOBER	NOVEMBER	DECEMBER	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	MEAN FOR WATER YEAR
1922				21.7	62.1								-
1923				92.6	97.5	129.0	96.5	74.9	42.4	89.6	43.3	39.4	-
1924	15.8	30.5	50.2	90.6	94.6	94.6	99.5	86.7	50.2	74.9	39.4	86.7	72.8
1925	48.3	33.5	74.9	135.9	88.7	82.7	57.1	59.1	18.4	18.4	20.7	8.3	53.8
1926	6.5	14.2	24.0	90.0	98.0	87.0	84.2	24.0	26.6	60.1	38.4	12.8	47.1
1927	3.9	26.6	93.6	51.2	98.5	97.5	49.3	34.5	55.2	88.7	65.0	33.5	68.9
1928	101.5	44.3	108.4	65.0	96.5	84.7	110.3	96.5	75.8	50.2	96.5	141.8	78.2
1929	55.2	32.5	33.5	39.4	114.3	115.2	97.5	85.7	81.8	82.7	46.3	24.6	81.6
1930	107.4	92.6	91.6	98.5	95.5	92.6	86.7	32.0	126.1	26.6	9.9	9.9	53.1
1931	6.9	11.8	41.4	70.9	49.3	56.1	124.1	157.6	37.4	136.9	25.0	58.1	85.9
1932	15.8	10.8	65.0	108.4	112.3	134.0	69.0	46.3	57.1	13.8	13.8	3.0	64.5
1933	16.7	53.2	146.8	157.6	163.5	91.6	122.1	36.4	10.3	11.2	24.0	15.2	70.7
1934	7.3	4.3	10.3	9.3	17.2	79.2	136.0	34.9	123.1	81.8	121.2	108.4	61.1
1935	28.6	49.3	212.8	172.4	96.5	137.9	141.8	95.5	25.6	82.7	16.7	104.4	86.5
1936	19.7	65.0	79.8	291.6	290.6	217.7	258.1	30.5	106.4	86.7	105.4	37.4	157.3
1937	109.3	119.2	235.4	273.8	262.0	152.7	230.5	68.0	37.4	85.7	115.2	75.8	117.6
1938	28.6	34.5	45.3	78.8	49.3	52.2	93.6	40.3	170.4	90.6	36.4	153.7	78.9
1939	41.4	44.3	94.6	108.4	285.7	225.6	96.5	71.9	61.1	252.2	208.8	80.8	123.5
1940	28.6	31.7	40.8	68.0	124.6	138.2	132.5	52.1	20.4	7.9	36.2	12.5	60.0
1941	5.7	18.1	30.6	46.5	45.3	100.8	150.6	20.4	18.1	253.7	31.7	10.2	61.1
1942	9.1	6.8	53.3	31.7	71.4	148.3	64.6	60.0	68.0	37.4	146.1	118.9	68.0
1943	111.0	74.8	124.6	246.9	151.7	141.6	96.3	35.1	111.0	202.7	26.1	36.2	113.2
1944	14.7	28.3	70.3	207.2	201.6	336.3	222.0	66.9	18.1	17.0	43.0	15.9	103.0
1945	117.8	48.7	118.9	90.6	165.3	124.6	66.9	39.6	12.5	26.0	186.8	320.5	109.8
1946	59.0	46.5	234.1	268.4	203.8	99.7	134.8	134.8	41.9	65.7	44.2	40.8	114.4
1947	74.8	78.1	70.3	155.1	64.6	88.4	93.0	41.9	32.8	21.5	14.7	71.4	66.9
1948	32.8	164.2	70.3	114.4	368.0	205.0	125.7	35.1	19.3	10.2	11.3	14.7	96.3
1949	54.4	168.7	208.4	139.3	185.7	103.0	76.0	216.3	96.3	89.5	329.5	94.0	147.2
1950	37.4	73.6	61.1	90.6	80.5	82.7	43.0	63.5	23.8	109.8	13.6	15.9	57.8
1951	27.2	26.0	55.5	49.9	54.4	72.5	90.6	24.9	10.2	11.3	13.6	3.4	39.6
1952	2.3	6.8	19.3	47.6	109.8	346.5	79.3	44.2	20.4	18.1	116.6	234.4	86.0
1953	35.1	113.2	80.5	193.6	243.5	115.5	139.3	49.9	38.5	17.0	11.3	5.7	94.0
1954	3.4	7.9	72.5	390.7	180.1	163.1	132.5	80.5	19.3	14.7	6.8	1.1	86.1
1955	15.9	19.3	52.1	61.2	115.5	88.4	78.2	14.7	10.2	26.0	130.2	479.0	86.1
1956	47.6	52.1	39.6	38.5	164.2	185.7	132.5	78.2	52.1	40.8	20.4	21.5	72.5
1957	91.8	116.6	122.3	70.3	135.9	178.9	77.0	79.3	177.8	22.7	45.3	47.6	97.5
1958	103.0	223.1	223.1	205.0	202.7	175.5	157.4	325.0	45.3	38.5	96.3	19.3	150.6
1959	66.9	48.7	101.7	109.8	167.6	155.1	314.8	61.1	85.0	129.1	77.0	174.4	123.4
1960	207.6	134.8	103.0	169.9	382.8	231.0	200.4	108.7	27.2	60.0	104.2	46.4	152.9
1961	55.5	32.8	53.2	72.5	266.1	168.7	166.5	132.5	47.6	31.7	62.3	14.7	89.5
1962	9.1	18.1	73.6	178.9	128.0	173.3	201.6	27.2	32.8	171.0	44.2	14.7	88.3

TABLE 2.4.1-1 ESTIMATED MONTHLY AVERAGE FLOWS OF BUCKHORN CREEK (CFS) (AVERAGE 1924 – 1981 = 87.2) DRAINAGE AREA = 79.5 SQ MI.

1963	13.6	207.2	104.2	168.7	172.1	220.8	73.6	53.2	28.3	17.0	17.0	15.9	90.6
1964	10.2	118.9	140.4	165.3	191.4	171.0	195.9	35.1	15.9	9.1	23.8	58.9	94.0
1965	268.4	55.8	168.7	89.5	166.5	199.3	83.8	61.1	160.8	465.4	183.4	31.7	158.5
1966	31.7	27.2	24.9	69.1	192.5	188.0	61.1	122.3	63.4	14.7	14.7	19.3	67.9
1967	15.9	22.6	40.8	61.1	130.2	71.3	39.6	34.0	123.4	40.8	228.7	63.4	72.5
1968	22.6	28.3	158.5	189.1	63.4	90.6	62.3	31.7	23.8	31.7	4.5	1.1	57.8
1969	11.3	35.1	40.8	55.5	138.2	185.7	71.3	27.2	35.1	19.3	152.9	39.6	66.8
1970	60.0	35.9	64.9	66.4	140.9	106.4	125.1	43.6	11.4	20.3	34.1	5.1	59.0
1971	10.7	41.2	36.2	159.6	226.6	230.5	104.4	15.3	14.4	21.0	34.1	14.7	80.0
1972	207.0	86.9	76.3	96.8	209.8	99.5	100.5	83.5	33.3	35.9	24.9	22.5	89.7
1973	47.8	173.4	230.2	174.4	443.3	175.4	236.4	76.2	158.6	41.2	17.4	8.6	147.8
1974	6.1	4.6	45.0	91.5	175.4	111.3	80.5	78.1	35.1	10.9	106.4	82.3	67.9
1975	20.3	28.6	95.2	269.9	236.4	265.0	93.8	37.5	17.0	185.2	16.4	28.7	107.4
1976	24.9	41.9	92.5	143.8	106.4	75.2	34.7	35.3	74.4	14.1	4.4	5.9	54.3
1977	33.4	30.7	128.1	122.2	57.4	207.0	76.8	22.1	9.1	1.8	9.3	42.2	61.4
1978	41.7	75.5	72.6	272.8	92.9	183.2	252.2	164.5	53.6	40.1	37.2	8.5	108.4
1979	13.9	23.9	54.0	141.0	218.0	173.0	129.0	158.0	90.1	25.9	15.9	122.0	96.1
1980	51.2	209.0	74.4	133.0	108.0	223.0	120.0	44.0	38.8	17.6	5.2	5.5	85.6
1981	18.9	20.5	40.0	34.1	123.0	44.4	24.1	11.2	6.0	4.4	27.7	9.5	29.7

*Estimated values based on data from USGS records of:

Deep River, 1924-30 (DA = 346 sq. mi.)

Little River, 1930-39 (DA = 229 sq. mi.)

Middle Creek, 1939-81 (DA = 80.7 sq. mi.)

TABLE 2.4.1-2

COMPARISON OF MONTHLY AVERAGE FLOW BETWEEN ESTIMATED* AND ACTUAL FLOW OF BUCKHORN CREEK

(Drainage Area - 74.2 Sq. Mi. at Gage Station)

WATER YEAR	FLOW	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEPT
		(CFS)											
1971	ACTUAL									18.9	17.2	9.3	8.1
TO 1972	ESTIMATED									31.1	33.5	23.3	21.0
1972	ACTUAL	15.8	147.0	221.0	135.0	344.0	158.0	178.0	35.1	153.0	32.5	15.4	4.8
TO 1973	ESTIMATED	44.6	161.8	233.5	162.7	413.8	163.7	220.7	71.1	148.0	38.4	16.3	8.0
1973	ACTUAL	2.8	4.9	25.9	110.0	176.0	110.0	62.7	181.0	37.4	8.3	95.5	64.6
TO 1974	ESTIMATED	5.7	4.3	42.0	85.4	163.7	103.9	75.1	91.6	32.7	10.2	99.3	76.8
1974	ACTUAL	10.3	12.0	68.8	299.0	188.0	209.0	52.1	24.4	12.7	191.0	8.5	34.4
TO 1975	ESTIMATED	18.9	24.8	88.8	251.9	220.7	247.3	87.5	35.0	18.8	204.5	15.3	26.8
1975	ACTUAL	24.7	48.5	95.3	153.0	82.5	68.4	24.4	20.5	28.6	5.1	3.2	4.3
TO 1976	ESTIMATED	23.3	39.1	86.3	134.2	99.3	70.2	32.4	32.9	69.4	13.2	4.1	5.5
1976	ACTUAL	14.1	13.9	89.8	150.0	40.2	279.0	66.1	8.5	4.9	1.7	3.5	12.4
TO 1977	ESTIMATED	31.2	28.2	119.5	114.0	53.6	188.5	71.7	20.6	8.5	1.7	8.7	39.4
1977	ACTUAL	22.8	46.7	57.5	387	64.7	213	229	143	50.7	18.1	12.7	2.4
TO 1978	ESTIMATED	38.9	70.4	67.8	254.7	86.7	171.0	235.4	153.5	50.0	37.4	34.8	7.9

Data Source - USGS

*Estimated values based on data from USGS records of Middle Creek near Clayton, N.C., by drainage area relationship.

TABLE 2.4.1-3 ESTIMATED MONTHLY AVERAGE FLOW IN CAPE FEAR RIVER AT BUCKHORN DAM IN CFS DRAINAGE AREA = 3196 SQ. MI.

WATER YEAR	OCTOBER	NOVEMBER	DECEMBER	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	MEAN FOR WATER YEAR
1924	-	-	-	3920	5083	4013	4424	3073	1539	4480	2390	4201	-
1925	4187	1309	2275	13612	3995	3048	1058	1598	503	570	446	308	2879
1926	142	492	693	2630	7483	4606	3886	511	678	1583	847	266	1950
1927	83	279	2043	1236	4043	5816	1794	699	1078	3252	2343	1006	1828
1928	3517	1148	6902	1246	3543	2547	7560	4104	2639	1808	5357	21327	5116
1929	2238	889	862	1305	6471	15366	4254	3876	3682	4233	2357	1014	3872
1930	12665	5271	4380	4263	4995	2243	1808	979	1530	988	376	154	3315
1931	94	368	1863	2399	1082	1991	5073	4867	754	1332	6829	526	2284
1932	181	209	2130	6686	3807	6278	2276	1358	2861	409	477	334	2256
1933	2955	3613	8681	4902	5221	2583	3150	973	494	309	1170	746	2897
1934	123	131	223	395	1188	4095	5698	1681	4829	2408	1520	4785	2247
1935	985	1887	5236	4217	3511	6245	7013	2806	751	1101	282	2809	3166
1936	298	1672	1803	13872	10817	8753	12748	814	2705	1996	2493	726	4881
1937	3482	989	5926	13695	6199	3680	5533	1912	925	1044	3604	2458	4121
1938	1431	1149	1169	2975	1489	2431	2944	1246	2882	6558	1287	722	2218
1939	322	1628	2912	3168	12999	7812	3173	2733	1120	2330	8086	992	3889
1940	467	596	1001	1993	6302	3716	3587	1479	1600	669	3544	587	2111
1941	155	3054	1680	2550	1821	3908	5281	649	1214	2857	423	344	2030
1942	110	99	578	528	3083	5412	1429	2349	2216	671	1894	1915	1714
1943	1207	1608	6688	7334	4096	6113	4632	1265	1983	6184	608	791	3376
1944	195	361	870	4999	7325	10445	7406	2203	471	3694	1556	1495	3418
1945	5944	1890	3040	3314	7649	3570	1971	1426	416	2866	1766	20083	4448
1946	1831	1091	7723	6991	9666	2554	2608	4745	2462	3681	3045	1028	3928
1947	1817	1990	1585	8976	1950	4278	3984	876	543	636	477	3028	2529
1948	1495	6382	1898	3735	12135	6239	5038	1738	1694	876	1426	473	3559
1949	1974	8531	7969	4876	5802	3266	3307	5013	945	2967	5514	2392	4473
1950	3066	4408	2096	2785	2325	3799	1399	4492	1416	4201	850	721	2640
1951	835	584	2025	1337	1880	2980	5471	856	1160	550	690	158	1538
1952	107	328	3040	4096	5799	13946	3653	2253	919	475	2340	6112	3602
1953	561	3984	2409	7923	8965	8684	3695	1562	1568	485	236	487	3344
1954	152	162	1675	9842	2617	4603	3908	1770	641	346	259	139	2027
1955	4524	930	3002	1797	6284	3101	3482	910	444	1318	4349	4355	2878
1956	1919	945	623	639	6226	5373	3505	2097	1002	1908	618	1896	2212
1957	3242	1668	3636	1688	8917	6307	2823	1593	3283	1301	2084	1863	3162
1958	2203	7410	5178	7831	6066	5110	9193	7990	1184	1467	1007	277	4564
1959	642	501	2446	2879	5155	3221	9888	1598	1987	3522	1777	2703	3001
1960	6062	2832	3340	4743	15942	9415	8802	2656	924	990	1803	971	4841
1961	798	526	855	1517	9271	5802	6626	3355	1927	1124	2227	412	2790
1962	166	290	2395	9294	6265	6370	8086	891	3429	2032	808	548	3369
1963	459	4322	4077	4975	5532	9313	1649	1325	919	615	399	437	2824

TABLE 2.4.1-3 ESTIMATED MONTHLY AVERAGE FLOW IN CAPE FEAR RIVER AT BUCKHORN DAM IN CFS DRAINAGE AREA = 3196 SQ. MI.

WATER YEAR	OCTOBER	NOVEMBER	DECEMBER	JANUARY	FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY	AUGUST	SEPTEMBER	MEAN FOR WATER YEAR
1964	352	1929	2727	6299	7112	4734	5920	933	696	935	2083	3629	3091
1965	6980	1173	4756	2447	6983	8881	2617	1350	4129	8209	2162	866	4251
1966	1125	619	506	1920	8294	8108	1469	3295	1023	394	900	815	2339
1967	459	536	1071	1576	4931	1567	1110	1486	550	504	4067	810	1537
1968	366	409	4825	7009	1531	4304	1483	1492	1146	918	299	89	2004
1969	497	1374	1426	2454	5966	6943	3774	1136	2541	1144	2752	1796	2626
1970	1686	507	2177	1810	5501	4130	5088	2025	530	651	2489	349	2332
1971	513	2251	1499	5080	7728	6252	3582	4741	1084	676	2837	1153	3177
1972	8739	2699	2820	3800	7602	2425	3098	4819	4371	1204	1219	615	3626
1973	1633	5265	9721	5852	10121	8195	10622	2827	5629	3258	1490	401	5378
1974	249	291	2239	5945	6434	3163	3827	4491	1636	776	2110	3954	2902
1975	545	626	3746	11420	6424	12804	3824	2952	1710	11346	1155	4047	5066
1976	1885	2230	2576	5304	3504	2128	1331	1359	3353	460	274	205	2059
1977	1438	706	4593	4360	1586	7707	3472	650	433	235	357	2016	2309
1978	1890	1494	2792	14252	3779	7093	4787	9870	2282	1702	2053	773	4425
1979	386	560	2382	8827	10377	9847	5546	4161	3572	932	569	5588	4352
1980	1822	5837	1653	5934	3183	9392	3920	2182	1964	1253	302	342	3150
1981	493	764	762	700	4129	1498	1053	601	792	2327	1817	1490	1350

*Estimated values based on data from USGS records of the Cape Fear River at Lillington, N.C. by drainage area relationship.

TABLE 2.4.1-4

DEVELOPMENT OF WATER RESOURCES FOR CAPE FEAR RIVER BASIN

ITEM	UNITS	B. EVERETT JORDAN (NEW HOPE) PROJECT	RANDLEMAN PROJECT	HOWARD MILL PROJECT
Status		Final Construction Stage	Early Design Stage	Authorized
Drainage area above damsite	sq mi	1,690	169	639
Location of Dam		2.5 mi. north of Moncure, N.C. on Haw River	5 mi. north of Randleman, N.C. on Deep River	3 mi. below Randolph County Line on Deep River
Dam:				
Type	-	Earth and Rockfill	Rolled Earth	Concrete
Total Length	feet	1,330	2,400	2,700
Height above stream bed	feet	113	108	101
Storage Capacity	acre-feet	778,000	108,000	233,000
Spillway:				
Type	-	Uncontrolled Side Channel	Uncontrolled Rock Saddle	Gated Concrete
Length of crest	feet	800	400	200
Number and size of gates	-	None	None	5-40'x36'
Outlet Works:				
Type	-	Concrete conduits located in structures of all dams		
Diameter of sluice	feet	19	12.8	8

REFERENCES:

1. Design Memorandum 2, New Hope Project, Cape Fear Basin, N.C., Corps of Engineers, 1967
2. Design Memorandum 2, Randleman Lake Project, Cape Fear Basin, N.C., Corps of Engineers, 1975
3. Design Memorandum 2, Howards Mill Lake Project, Deep River, N.C., Corps of Engineers, 1975

TABLE 2.4.1-5

CAPE FEAR RIVER
INDUSTRIAL WATER WITHDRAWALS
DOWNSTREAM OF BUCKHORN DAM

Industry	Location	Approximate River Drainage Area (sq mi)	Withdrawal (mgd)	Discharge (mgd)
DuPont	Fayetteville	4330	7.0	6.7
Rohm & Haas	Fayetteville	4330	4.0	2.6
Cape Fear Feed	Fayetteville	4330	1.8	1.5
Federal paper	Acme	5280	46.0	43.0
CP&L Sutton Plant	Wilmington	7050	6.0	0.0
DuPont	Wilmington	7050	9.3	6.0
Swift Agriculture Chemicals	Wilmington	7050	1.2	1.1
U.S. Steel Agriculture Chemicals	Wilmington	7050	1.6	1.6
Wright Chemical	Wilmington	7050	1.5	1.1

NOTE: This information was developed from a telephone survey of known users obtained from N.C. State records.

TABLE 2.4.1-6

CAPE FEAR RIVER
MUNICIPAL WATER WITHDRAWALS DOWNSTREAM OF BUCKHORN DAM

Municipality	Approximate River Drainage Area (sq. mi.)	Average Withdrawal (mgd)
Lillington	3440	0.2
Dunn	3470	1.8
Fayetteville	4330	10.8
Wilmington	5190	8.7

TABLE 2.4.2-1

ESTIMATED MAXIMUM FLOOD PEAKS FOR BUCKHORN CREEK AT THE CAPE FEAR RIVER DRAINAGE AREA = 79.5 Sq. Mi.

Water Year	Date Maximum Flood Occurred	Momentary Maximum (cfs)
1940	Apr. 22, 1940	568
41	July 15, 1941	1330
42	Sept. 8, 1942	1409
43	July 14, 1943	2226
44	Mar. 21, 1944	1576
1945	Sept. 18, 1945	3408
46	April 27, 1946	936
47	Sept. 22, 1947	837
48	Feb. 14, 1948	1527
49	May 11, 1949	3212
1950	May 17, 1950	420
51	April 9, 1951	355
52	Sept. 1, 1952	4039
53	Feb. 16, 1953	1084
54	Jan. 23, 1954	3024
55	Sept. 4, 1955	5320
56	Mar. 17, 1956	1310
57	June 9, 1957	2000
58	May 7, 1958	3665
59	Sept. 4, 1959	1734
1960	Oct. 24, 1959	1665
61	Feb. 21, 1961	788
62	July 5, 1962	1950
63	Nov. 11, 1962	1537
64	Nov. 7, 1963	1113
65	July 28, 1965	5103
66	May 19, 1966	1970
67	June 19, 1967	1773
68	Jan. 15, 1968	699
69	Aug. 5, 1969	1724
1970	Oct. 3, 1970	656
71	March 4, 1971	1704
72	Oct. 7, 1971	809
73	Feb. 3, 1973	8383
74	Aug. 8, 1974	471
75	March 20, 1975	1862
76	June 27, 1976	803
77	Sept. 8, 1977	968
78	Apr. 27, 1978	4413

NOTE: Estimated values are derived from USGS records of Middle Creek near Clayton, North Carolina, (DA = 80.7 Sq. Mi.) by drainage area relationship. (D.A. for Buckhorn Creek = 79.5 Sq. Mi.)

TABLE 2.4.2-2

ESTIMATED* AND MEASURED** MAXIMUM FLOOD PEAKS FOR BUCKHORN CREEK AT USGS GAGE STATION NEAR CORINTH, N.C. (D.A. = 74.2 Sq. Mi.)

WATER YEAR	DATE MAXIMUM FLOOD OCCURRED		MOMENTARY MAXIMUM (cfs)	
1973	February 3, 1973	(February 2)	7820*	(6920)**
1974	August 8, 1974	(August 7)	440*	(1410)**
1975	March 20, 1975	(July 16)	1740*	(2300)**
1976	June 27, 1976	(June 28)	750*	(1060)**
1977	September 8, 1977	(March 14)	900*	(2520)**
1978	April 27, 1978	(April 26)	4120*	(4660)**

*Estimated values are derived from USGS records of Middle Creek near Clayton, North Carolina, (D.A. = 80.7 Sq. Mi.) by drainage area relationship.

**USGS Gage Station on Buckhorn Creek near Corinth, North Carolina, established in June, 1972.

TABLE 2.4.2-3

MAXIMUM FLOOD FLOW OF THE CAPE FEAR RIVER AT BUCKHORN DAM

Water Year	Date Occurred	Momentary Max. Flow (O) (cfs)
1924	Sept. 30, 1924	48680
5	Jan. 12, 1925	42920
6	Jan. 19, 1926	25360
7	Mar. 7, 1927	30850
8	Sept. 20, 1929	78040
9	Mar. 1, 1929	62900
1930	Oct. 2, 1929	99410
1	Aug. 21, 1931	27130
2	Mar 7, 1932	47290
3	Oct. 18, 1932	27130
4	Apr. 10, 1934	37160
5	Dec. 2, 1934	38090
6	Apr. 7, 1936	68010
7	Jan. 29, 1937	32330
8	July 27, 1938	43670
9	Feb. 10, 1939	44130
1940	Feb. 8, 1940	29730
1	Nov. 15, 1940	29360
2	Feb. 18, 1942	28620
3	July 14, 1943	38000
4	Sept. 30, 1944	43200
5	Sept. 19, 1945	139370
6	Feb. 11, 1946	50540
7	Jan. 14, 1947	36790
8	Feb. 15, 1948	46360
9	Nov. 29, 1948	49610
1950	May 15, 1950	33910
1	Apr. 9, 1951	32330
2	Mar. 5, 1952	71630
3	Feb. 16, 1953	40970
4	Jan. 23, 1954	52490
5	Oct. 17, 1954	46360
6	Mar. 17, 1956	43200
7	Feb. 2, 1957	38840
8	Nov. 26, 1957	43200
9	April 20, 1959	37160

Water Year	Date Occurred	Momentary Max. Flow (O) (cfs)
1960	Apr. 6, 1960	44130
1	Mar. 22, 1961	33910
2	Jan. 7, 1962	52490
3	Mar. 7, 1963	39300
4	Apr. 9, 1964	39300
5	July 28, 1965	53420
6	Mar. 1, 1966	45900
7	Aug. 25, 1967	24530
8	Jan. 15, 1968	33910
9	Feb. 3, 1969	29080
1970	Feb. 18, 1970	34560
1	Mar. 4, 1971	38370
2	Oct. 25, 1971	40790
3	Feb. 3, 1973	49990
4	Jan. 29, 1974	21280
5	July 16, 1975	44130
6	Jan. 28, 1976	17750
7	Jan. 10, 1977	23230
8	Apr. 27, 1978	34190

NOTE:Maximum flood flow derived from data of USGS Gaging Station at Lillington by the drainage area ratio relationship.

TABLE 2.4.2-4

PLANT AREA WATER ACCUMULATION FOR DESIGN PMP CONDITIONS*

Time (HR)	Incremental Rainfall (in.)	Incremental Plus Accumulated Rainfall (in.)	Utilized Plant** Drainage (in.)	Net Accumulated Water (in.) Depth
0	-	-	-	-
1	1.5	1.5	1.5	0
2	2.1	2.1	2.1	0
3	2.3	2.3	2.3	0
4	18.8	18.8	4.0	14.8
5	4.0	18.8	4.0	14.8
6	1.25	15.05	4.0	12.05
9	2.95	15.0	12.0	3.0
12	2.5	5.5	5.5	0
15	1.7	1.7	1.7	0
18	1.2	1.2	1.2	0
21	1.1	1.1	1.1	0
24	0.7	0.7	0.7	0
27	0.7	0.7	0.7	0
30	0.6	0.6	0.6	0
33	0.6	0.6	0.6	0
36	0.6	0.6	0.6	0
	42.6			

References:

"Probable Maximum Precipitation Estimates, United States East of the 105 Meridian," Hydrometeorological Report No. 51, NOAA & Corps of Engineers, June 1978.

"Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian," Hydrometeorological Report No. 52 NOAA & Corps of Engineers, August 1982.

* 1 sq. mi. (or point) PMP intensity.

** Designed drainage is 4 in/hr.

TABLE 2.4.3-1

PROBABLE MAXIMUM PRECIPITATION

	Drainage Basin of Buckhorn Creek at Cape Fear River (D.A. = 79.5 Sq. Mi.)*		Drainage Basin of Buckhorn Creek at Main Dam (D.A. = 71.0 Sq. Mi.)*		Drainage Basin of Tom Jack Creek at Auxiliary Dam (D.A. = 2.43 Sq. Mi.)**	
Time (hr.)	Depth of PMP (in.)	1-hr. Increm. (in.)	Depth of PMP (in.)	1-hr. Increm. (in.)	Depth of PMP (in.)	1-hr. Increm. (in.)
1		10.95		11.10		14.68
2		3.25		3.29		4.49
3		2.20		2.24		3.29
4		2.19		2.21		2.70
5		1.66		1.69		2.40
6	21.9	1.65	22.2	1.67	29.95	2.39
7		0.50		0.50		0.65
8		0.30		0.40		0.60
9		0.30		0.40		0.50
10		0.30		0.30		0.40
11		0.30		0.30		0.33
12	23.9	0.30	24.35	0.25	32.73	0.30
13		0.20		0.25		0.27
14		0.20		0.20		0.25
15		0.20		0.20		0.24
16		0.20		0.20		0.21
17		0.20		0.20		0.20
18		0.20		0.20		0.20
19		0.20		0.20		0.15
20		0.20		0.20		0.15
21		0.20		0.20		0.15
22		0.20		0.20		0.15
23		0.15		0.15		0.14
24	26.2	0.15	26.7	0.15	34.98	0.14
25						0.12
26						0.12
27						0.12
28						0.12
29						0.12
30						0.11
31						0.11
32						0.11
33						0.11
34						0.10
35						0.10
36					36.32	0.10
48	29.6		29.8		38.10	

*A reduction of 10 percent in the PMP intensity for basin correction is included.

**PMP for a drainage area of 10 sq. mi. without basin correction.

TABLE 2.4.3-2

TIME DISTRIBUTION OF PROBABLE MAXIMUM PRECIPITATION

Time (hr.)	Drainage Basin of Buckhorn Creek at Cape Fear River (D.A. = 79.5 Sq. Mi.) Incremental Rainfall (in.)	Drainage Basin of Buckhorn Creek at Main Dam (D.A. = 71.0 Sq. Mi.) Incremental Rainfall (in.)	Drainage Basin of Tom Jack Creek at Auxiliary Dam (D.A. = 2.43 Sq. Mi.) Incremental Rainfall (in.)
1	0.20	0.20	2.40
2	0.20	0.20	2.70
3	0.20	0.20	3.29
4	0.20	0.20	14.68
5	0.20	0.20	4.49
6	0.20	0.30	2.39
7	0.30	0.40	0.65
8	0.50	0.50	0.60
9	1.65	1.67	0.50
10	1.66	1.69	0.40
11	10.95	11.10	0.33
12	3.25	3.29	0.30
13	2.20	2.24	0.27
14	2.19	2.21	0.25
15	0.30	0.40	0.24
16	0.30	0.30	0.21
17	0.30	0.25	0.20
18	0.30	0.25	0.20
19	0.20	0.20	0.15
20	0.20	0.20	0.15
21	0.20	0.20	0.15
22	0.20	0.20	0.15
23	0.15	0.15	0.14
24	0.15	0.15	0.14
25			0.12
26			0.12
27			0.12
28			0.12
29			0.12
30			0.11
31			0.11
32			0.11
33			0.11
34			0.10
35			0.10
36			0.10
TOTAL	26.20	26.70	36.32

TABLE 2.4.3-3 SNYDER AND LOSS PARAMETERS BUCKHOR CREEK BASIN

GEOMETRIC & HYDROLOGIC PARAMETERS OF DRAINAGE BASIN	NATURAL CONDITION AT CONFLUENCE WITH CAPE FEAR	NATURAL CONDITION AT CORINTH RECORDING STATION	CONDITIONS WITH RESERVOIRS CONSTRUCTED – BELOW MAIN RESERVOIR SUB-BASIN I	BELOW MAIN RESERVOIR SUB-BASIN II	MAIN RESERVOIR SUB-BASIN III	MAIN RESERVOIR SUB-BASIN IV	MAIN RESERVOIR SUB-BASIN V
GEOMETRIC PARAMETERS							
Drainage Area (D.A.) - Sq. Mi.	79.5	74.2	6.25	2.25	2.13	14.10	4.67
Length Longest Water Course (L)-Mi.	19.0	15.6	4.0	1.7	1.2	6.4	4.4
Length to Center of Gravity of Basin (L_{CA})-Mi.	10.0	6.6	2.1	0.4	0.4	2.7	2.5
SNYDER PARAMETERS							
Lag Time (t_p)-Hr	18.27	15.97	7.32	4.05	3.24	9.03	7.91
C_T	3.91	3.91	3.91	3.91	3.91	3.91	3.91
C_p	0.75	0.75	0.75	0.75	0.75	0.75	0.75
HEC-1 LOSS PARAMETERS							
STRKR	0.05		0.05	0.05	0.05	0.05	0.05
DLTKR	0.17		0.17	0.17	0.17	0.17	0.17
RTIOL	1.00		1.00	1.00	1.00	1.00	1.00
ERAIN	0.35		0.35	0.35	0.35	0.35	0.35

GEOMETRIC & HYDROLOGIC PARAMETERS OF DRAINAGE BASIN	CONDITIONS WITH RESERVOIRS CONSTRUCTED – MAIN RESERVOIR SUB-BASIN VIA	MAIN RESERVOIR SUB-BASIN VIB	MAIN RESERVOIR SUB-BASIN VII	MAIN RESERVOIR SUB-BASIN VIII	AUXILIARY RESERVOIR SUB-BASIN IX
GEOMETRIC PARAMETERS					
Drainage Area (D.A.)- Sq. Mi.	13.31	4.10	6.24	1.82	1.73
Length Longest Water Course (L)-Mi.	6.4	3.6	4.8	1.4	1.2
Length to Center of Gravity of Basin (L_{CA})-Mi.	3.4	1.9	2.1	0.8	0.7

TABLE 2.4.3-3 SNYDER AND LOSS PARAMETERS BUCKHOR CREEK BASIN

GEOMETRIC & HYDROLOGIC PARAMETERS OF DRAINAGE BASIN	SUB-BASIN VIA	SUB-BASIN VIB	SUB-BASIN VII	SUB-BASIN VIII	SUB-BASIN IX
SNYDER PARAMETERS					
Lag Time (t_p)-Hr	9.65	6.90	7.72	4.14	3.79
C_T	3.91	3.91	3.91	3.91	3.91
C_p	0.75	0.75	0.75	0.75	0.75
HEC-1 LOSS PARAMETERS					
STRKR	0.05	0.05	0.05	0.05	0.05
DLTKR	0.17	0.17	0.17	0.17	0.17
RTIOL	1.00	1.00	1.00	1.00	1.00
ERAIN	0.35	0.35	0.35	0.35	0.35

TABLE 2.4.5-1

WAVE RUNUP PARAMETERS FOR STRUCTURES PROTECTED BY RIPPAP

Fetch [1]	Safety Related Structure	Maximum Still Water Level (ft.) MSL	Wind Speed (mph)	Wind Direction	Effective Fetch Length (ft.)	Average Water Depth (ft.)	Significant Wave Height (ft.)	Maximum Wave Height (ft.)	Wave Length (ft.)	Wave Period (sec)	Significant Wave Runup (ft.)	Maximum Wave Runup (ft.)	Wind Setup (ft.)	Maximum Water Level (ft.) MSL
(PMF - WATER LEVEL IN THE RESERVOIR)														
1	Main Dam	238.9[2]	50.4	N	4720	30	2.4	4.0	40.1	2.8	3.3	4.1	0.1	243.1[3]
2	Auxiliary Dam	256.0[2]	52.9	NW	2120	15	1.6	2.7	24.8	2.2	1.8	1.9	0.1	258.0[3]
3	Auxiliary Dam	238.9[2]	50.2	S	5930	30	2.7	4.5	46.1	3.0	3.2	3.6	0.2	242.7[3]
(NORMAL OPERATION W.L. IN RESERVOIRS)														
7	Auxiliary Dam	252	123	NW	1285	10	3.2	5.4	50.8	3.2	3.6	3.8	0.4	256.2[3]

Notes:

(1) See Figure 2.4.5-1

(2) See Section 2.4.3.5

(3) Maximum Water Level = Maximum Still Water Level + Maximum Wave Runup + Wind Setup

(4) Top of Main Dam = 260 Ft. MSL

(5) Top of Auxiliary Dam 260 Ft. MSL

TABLE 2.4.5-2

WAVE RUNUP PARAMETERS FOR PLANT ISLAND

Fetch [1]	Safety Related Structure	Maximum Still Water Level (ft.) MSL	Wind Speed (mph)	Wind Direction	Effective Fetch Length (ft.)	Average Water Depth (ft.)	Significant Wave Height (ft.)	Maximum Wave Height (ft.)	Wave Length (ft.)	Wave Period (sec)	Significant Wave Runup (ft.)	Maximum Wave Runup (ft.)	Wind Setup (ft.)	Maximum Water Level (ft.) MSL
					(PMF - WATER LEVEL IN THE RESERVOIR)									
4	Natural	256.0[2]	54.0	NNW	1410	17	1.3	2.2	19.5	2.0	1.1	1.3	0.1	257.4[3]
5	Sacrificial Spoil Fill	238.9[2]	51.8	SSE	4060	29	2.2	3.7	37.3	2.7	1.0	1.2	0.1	240.2[3]
6	Natural	256.0[2]	54.4	W	2000	19	1.6	2.6	23.9	2.2	1.4	1.6	0.1	257.7[3]
8	Sacrificial Spoil Fill	238.9[2]	50.9	S	3740	29	2.1	3.5	34.6	2.6	0.9	1.1	0.1	240.1[3]
					(NORMAL OPERATION W.L. IN RESERVOIRS)									
5	Sacrificial Spoil Fill	220	123	SSE	1970	16	4.0	6.7	62.7	3.5	1.8	2.0	1.0	222.5[3]
6	Natural	252	123	W	710	15	2.8	4.7	40.1	2.8	2.4	2.7	0.2	254.9[3]

NOTES:

(1) Figure 2.4.5-1

(2) See Section 2.4.3.5

(3) Maximum Water Level = Maximum Still Water Level + Maximum Wave Runup + Wind Setup

(4) Plant Grade = 260 Ft. MSL

TABLE 2.4.11-2

NORMAL MONTHLY METEOROLOGICAL CONDITIONS AT SITE

Month	Average Dry Bulb(1) Temp. (F)	Wind(1) (Mph)	Average Wet Bulb(1) Temp. (F)	Average Monthly Air Vapor Pressure (mm Hg)	Average Monthly Atmospheric Pressure(2) (mm Hg)
Jan.	41.6	8.5	37.6	4.4	764.8
Feb.	43.0	9.1	38.0	4.2	763.9
Mar.	49.5	9.6	43.0	5.5	763.1
Apr.	59.3	9.4	51.6	8.5	762.9
May	67.6	7.8	60.6	12.5	761.7
June	75.1	7.0	68.1	16.5	762.8
July	77.9	6.7	70.9	19.0	763.1
Aug.	76.9	6.6	70.4	18.6	764.4
Sept.	71.2	7.0	64.7	14.9	763.8
Oct.	60.5	7.2	54.8	10.0	764.9
Nov.	50.0	7.8	44.5	6.3	765.2
Dec.	41.9	7.9	36.9	4.7	764.4

(1) Raleigh-Durham Weather Service Data 1887-1964.

(2) Raleigh-Durham Airport Data 1972-1978, adjusted to sea level elevation.

TABLE 2.4.11-3

WORST MONTHLY COINCIDENT METEOROLOGICAL CONDITIONS FOR PERIOD OF RECORD (1931-1970) FOR DETERMINATION OF CRITICAL MONTHLY EVAPORATION RATES

MONTH	H_c (BTU/SF/DAY)	S	T_a (F)	BRUNT COEFF.	W (MPH)	P (mmHg)	DP (F)
JAN 1932	1105.	0.42	51.0	0.732	9.2	760	42.0
FEB 1932	1456.	0.51	51.0	0.723	8.8	760	37.0
MAR 1938	2190.	0.61	57.2	0.723	9.8	760	44.0
APR 1954	2565.	0.64	63.6	0.721	8.5	760	52.0
MAY 1953	2900.	0.76	73.5	0.724	6.5	760	61.0
JUNE 1943	2965.	0.71	80.4	0.725	7.5	760	69.0
JULY 1932	2950.	0.74	81.6	0.727	8.2	760	65.0
AUG 1938	2745.	0.83	80.7	0.724	7.5	760	68.0
SEPT 1933	2350.	0.55	77.5	0.730	8.4	760	65.0
OCT 1941	1810.	0.71	68.2	0.720	8.5	760	54.0
NOV 1931	1390.	0.69	58.4	0.718	6.2	760	42.0
DEC 1931	1070.	0.52	51.2	0.722	7.3	760	40.0

WHERE:

H_c = Clear Sky Solar Radiation, Btu/Ft.^{2}/Day

S = Fraction Possible Sunshine

T_a = Monthly Average Ambient Air Temperature, F

W = Monthly Average Windspeed, MPH

P = Normal Atmospheric Pressure, mmHg

DP = Monthly Average Dew Point Temperature, F

TABLE 2.4.11-10

ALLOCATION OF MAIN RESERVOIR VOLUME

Purpose	Volume (ac.-ft.)
1 - Storage below Elevation 205.7 ft. MSL	24,800
2 - Storage Elevation 205.7 ft. MSL to Elevation 220 ft. MSL	47,350
3 - Flood Storage Elevation 220 ft. MSL to Elevation 236.2 ft. MSL	81,000
Total Storage	153,150

NOTE: Main Reservoir Elevation 236.2 ft. MSL, stillwater level, results from a PMF without antecedent flow.

TABLE 2.4.11-12

CALCULATED MINIMUM FLOW OF BUCKHORN CREEK AT THE MAIN DAM

(DA=71.0 sq. mi.)

Water Year	Date Minimum Flow Occurred	Daily Minimum* (cfs)
1940	Aug. 6, 1940	1.8
1	Oct. 8, 1940	3.2
2	Oct. 25, 26, 1941	2.7
3	Sept. 14, 15, 1943	8.3
4	Sept. 6, 7, 10, 11, 1944	6.0
5	June 7, 1945	4.4
6	Sept. 10, 1946	7.8
7	Aug. 16, 1947	6.2
8	Sept. 20, 21, 1948	1.1
9	July 30, Aug. 14, 1949	10.6
1950	Aug. 30, 1950	4.8
1	Sept. 14, 1951	1.0
2	Oct. 8-10, 1951	0.5
3	Sept. 22-24, 1953	0.4
4	Sept. 15-18, 26-30, 1954	0.1
5	Oct. 11-13, 1954	0
6	Sept. 19, 20, 1956	4.0
7	Aug. 15, 1957	3.2
8	Sept. 30, 1958	8.6
9	Oct. 1, 1959	9.7
1960	July 26, 1960	8.1
1	Sept. 26, 1961	6.6
2	Sept. 15, 1962	5.5
3	Aug. 19, 1963	4.9
4	July 3, 1964	4.5
5	June 7, 1965	14.1
6	Sept. 11, 12, 13, 1966	4.4
7	June 16, 17, 1967	6.4
8	Sept. 25, 26, 1968	0.4
9	Oct. 2, 3, 1968	0.5
1970	Sept. 26, 1970	1.7
1	Oct. 9, 1970	2.2
2	Sept. 15, 1972	6.0
3	Sept. 30, 1973 (Sept. 29, 30, 1973)**	5.0 (3.0)**
4	Nov. 7, 1973 (Oct. 18, 19, 1973)**	3.1 (1.0)**
5	June 26, 1975 (Aug. 30, Sept. 7, 1975)**	6.0 (3.6)**
6	Sept. 1, 1976 (Sept. 2, 1976)**	1.4 (0.04)**
7	Sept. 6, 1977 (July 31, 1977)**	0.1 (0.07)**
8	Sept. 29, 1978 (Oct. 1, 1977)**	3.0 (1.2)**

*Calculated data, based upon actual Middle Creek Data, as adjusted by drainage area ratios (Middle Creek DA=80.7 sq. mi.) (Main Dam DA = 71.0 sq. mi.)

**Calculated data, based upon actual Buckhorn Creek data, as adjusted by drainage area ratios (Buckhorn Creek at gage station DA=74.2 sq. mi.)

TABLE 2.4.11-18

RESERVOIR ANALYSIS
NORMAL OPERATION
CRITICAL PERIOD MAY 1980 - MAY 1982

						AUXILIARY RESERVOIR OPERATION			MAIN RESERVOIR OPERATION									
Year	Mo.	CREEK INFLOW DA=79.5 CFS	NAT. EVAP In.	DIR RAIN In.	NET EVAP. In.	CREEK INFLOW DA.RATIO CFS	NET EVAP. Ac.Ft.	RWL@ END OF MONTH FT. MSL	CREEK INFLOW DA.RATIO CFS	PUMP TO AUX RES. CFS	ALLOW FOR SEEP. CFS	TOTAL AVAIL WATER Ac. Ft	AVERAGE RES.SURF. AREA Ac	FORCED EVAP. Ac.Ft	NET EVAP. Ac. Ft	INCR. STOR. USE Ac. Ft	TOTAL STOR. USE Ac. Ft	RWL @ END OF MONTH FT. MSL
1980	M	44.0	7.70	2.75	4.95	1.07	132	250.0	34.80	1.08	5	1766	3636	1090	1500	824	14274	216.1
	J	38.8	8.21	3.37	4.84	0.94	129	250.0	30.70	1.23	5	1456	3620	1090	1460	1094	15368	215.8
	J	17.6	9.23	2.12	7.11	0.43	190	250.0	14.00	2.66	5	390	3544	1140	2100	2850	18218	214.9
	A	5.23	8.51	0.76	7.75	0.13	207	250.0	4.15	3.23	5	-251	3422	1130	2210	3591	21809	213.8
	S	5.54	6.30	3.62	2.68	0.13	72	250.0	4.42	1.07	5	-98	3322	1080	742	1920	23729	213.2
	O	18.9	4.64	2.19	2.45	0.46	65	250.0	15.10	0.60	5	584	3267	1070	667	1153	24882	212.8
	N	20.5	2.99	2.38	0.61	0.50	16	250.0	16.30	0	5	672	3246	988	165	481	25363	212.7
	D	40.0	1.54	1.70	-0.16	0.97	-4	250.3	31.90	0	5	1654	3225	959	-43	-738	24625	212.9
1981	J	34.1	1.42	1.04	0.38	0.82	10	250.4	27.20	0	5	1365	3253	953	103	-309	24316	213.0
	F	123	2.44	3.53	-1.09	2.97	-30	251.0	98.30	0	5	5182	3358	872	-350	-4615	19701	214.5
	M	44.4	4.41	1.33	3.08	1.06	86	251.0	35.40	0	5	1869	3436	1000	882	13	19714	214.5
	A	24.1	6.28	1.04	5.24	0.58	144	250.6	19.20	0	5	845	3412	1020	1490	1665	21379	213.9
	M	11.2	7.70	2.37	5.33	0.27	144	250.2	9.86	0	5	299	3332	1090	1480	2271	23650	213.2
	J	6.0	8.21	1.13	7.08	0.15	189	250.0	4.8	2.13	5	-139	3237	1090	1910	3139	26789	212.2
	J	4.38	9.23	2.90	6.33	0.11	169	250.0	3.51	2.64	5	-254	3128	1140	1650	3044	29833	211.2
	A	27.7	8.51	5.25	3.26	0.67	87	250.0	22.2	0.74	5	1012	3055	1130	830	948	30781	210.9
	S	9.53	6.30	1.61	4.69	0.23	125	250.0	7.66	1.87	5	47	2994	1080	1170	2203	32984	210.2
	O	21.4	4.64	3.91	0.73	0.52	20	250.0	17.2	0	5	750	2959	1070	180	500	33484	210.0
	N	9.6	2.99	0.98	2.01	0.23	54	250.0	7.61	0.46	5	128	2919	988	487	1349	34833	209.4
	D	42.6	1.54	4.44	-2.90	1.03	-78	250.5	34.30	0	5	1802	2926	959	-707	-1550	33283	210.1
1982	J	176.3	1.42	4.39	-2.97	4.25	-83	251.5	142	0	5	8424	3099	953	-767	-8238	25045	212.8
	F	152.7	2.44	3.97	-1.53	3.62	-45	252.0	122	0	5	6498	3349	872	-427	-6053	18992	214.7
	M	137.9	4.41	2.87	1.54	3.24	46	252.0	110	0	5	6456	3553	1000	456	-5000	13992	216.2
	A	57.2	6.28	3.32	2.96	1.35	87	251.7	45.3	0	5	2398	3657	1020	902	-476	13516	216.3
	M	63.3	7.70	2.34	5.36	1.50	156	251.6	50.1	0	5	2773	3672	1090	1640	-43	13473	216.4

NOTES:

(1) Worst monthly evaporation rates used.

(2) Starting Level = 216.3 FT MSL for Main Reservoir
= 250.0 FT MSL for Auxiliary Reservoir

The Main Reservoir starting level is the lowest reached in a seven-year (October 1973 through September 1980) simulation of the reservoir operation under normal evaporation conditions as described in the SHNPP Environmental Report, Section 2.4.2.3.2.2.3 (e.g. NSSS power = 2785 Mw and Cap. Factor = 75%)

(3) All creek inflows estimated from USGS records for Middle Creek at Clayton by drainage area ratio.

(4) Preliminary, unpublished USGS flow records used for October 1981 through May 1982

(5) On-site rainfall records used.

(6) Tech Spec 3.7.5 limits the Main Reservoir to ≥ 206' for normal operation

TABLE 2.4.11-19

RESERVOIR ANALYSIS
NORMAL OPERATION
100-YEAR DROUGHT

					AUXILIARY RESERVOIR OPERATION			MAIN RESERVOIR OPERATION									
Mo.	CREEK INFLOW DA=79.5 CFS	NAT. EVAP In.	DIR RAIN In.	NET EVAP. In.	CREEK INFLOW DA.RATIO CFS	NET EVAP. Ac.Ft.	RWL@ END OF MONTH FT. MSL	CREEK INFLOW DA.RATIO CFS	PUMP TO AUX RES. CFS	ALLOW FOR SEEP. CFS	TOTAL AVAIL WATER Ac. Ft	AVERAGE RES.SURF. AREA Ac	NET EVAP. Ac. Ft	FORCED EVAP. Ac.Ft	INCR. STOR. USE Ac. Ft	TOTAL STOR. USE Ac. Ft	RWL @ END OF MONTH FT. MSL
M	51.6	7.70	2.16	5.54	1.25	148	250	40.8	1.15	5	2131	3660	1690	1090	649	14099	216.1
J	12.5	8.21	1.17	7.04	0.30	188	250	9.92	2.85	5	123	3580	2100	1090	3067	17166	215.2
J	12.5	9.23	1.72	7.51	0.30	200	250	9.95	2.95	5	123	3467	2170	1140	3187	20353	214.3
A	12.5	8.51	4.88	3.63	0.30	97	250	9.97	1.27	5	228	3736	1020	1130	1922	22275	213.6
S	4.1	6.30	0.67	5.63	0.10	150	250	3.25	2.42	5	-248	3282	1540	1080	2868	25143	212.7
O	4.1	4.64	0.72	3.92	0.10	105	250	3.25	1.60	5	-206	3184	1040	1070	2316	27459	212.0
N	4.1	2.99	1.20	1.79	0.10	48	250	3.26	0.70	5	-145	3124	466	988	1599	29058	211.5
D	4.1	1.54	0.75	0.79	0.10	21	250	3.26	0.24	5	-122	3084	203	959	1284	30342	211.0
J	51.6	1.42	1.44	-0.02	1.25	-1	250.3	41.50	0	5	2244	3080	-5	953	-1296	29046	211.5
F	51.6	2.44	2.51	-0.07	1.25	-2	250.5	41.40	0	5	2022	3120	-18	872	-1168	27879	211.9
M	51.6	4.41	1.63	2.78	1.24	75	250.5	41.40	0	5	2238	3147	729	1000	-509	27369	212.0
A	51.6	6.28	4.33	1.95	1.24	53	250.6	41.40	0	5	2166	3163	514	1020	-632	26737	212.2

NOTES:

(1) Worst monthly evaporation rates used.

(2) Starting Level = 216.3 FT MSL for Main Reservoir

= 250.0 FT MSL for Auxiliary Reservoir

The Main Reservoir starting level is the lowest reached in a seven-year (October 1973 through September 1980) simulation of the reservoir operation under normal evaporation conditions as described in the SHNPP Environmental Report, Section 2.4.2.3.2.2.3 (e.g. NSSS power = 2785 Mw and Cap. Factor = 75%)

(3) Rainfall Data = Monthly rainfall of 1933-1934 (drought period) x $\frac{\text{Ave.Min. 12 Month flow (100 yr.drought)}}{\text{Ave. 12 Month flow (1933−1934)}}$

(4) Tech Spec 3.7.5 limits the Main Reservoir to ≥ 206' for normal operation

TABLE 2.4.11-20

MONTHLY MAIN RESERVOIR RELEASE-AVERAGE AND 100-YEAR DROUGHT CONDITIONS

Month	Average (in CFS)	100-Year Drought (in CFS)
January	76.7	0
February	102.7	0
March	119.7	0
April	66.5	0
May	54.6	0
June	11.0	0
July	14.7	0
August	2.2	0
September	21.3	0
October	19.1	0
November	21.2	0
December	15.2	0

TABLE 2.4.12-1

CONCENTRATIONS OF NUCLIDES CONTAINED IN THE REFUELING WATER STORAGE TANK*

NUCLIDE	CONCENTRATION IN TANK $(\mu ci/cc)$	μ RESTRICTED AREA MPC_w $(\mu ci/cc)$	C/MPC_w
I-131	1.5E-03	3.0E-07	5000
Cs-134	3.5E-03	9.0E-06	390
Cs-137	1.8E-02	2.0E-05	900

CONCENTRATION AND C/MPC IN THE CAPE FEAR RIVER WATER AT LILLINGTON FOLLOWING A TANK RUPTURE*

ISOTOPE	Σ RWST ACTIVITY $(\mu ci/cc)$	LILLINGTON ACTIVITY (RWST/1.5E6)	LILLINGTON 10CFR20 LIMITS $(\mu ci/cc)$	C(i)/MPC(i)
Fe-55	1.63E1	1.09E-5	8E-4	1.36E-2
Co-60	3.23E0	2.15E-6	5E-5	4.31E-2
I-131	3.21E-2	2.14E-8	3E-7	7.13E-2

TOTAL Σ C(i)/MPC(i) = 1.42E-1

i = Isotopes listed with C/MPC > 1E-3

Total includes non-listed isotopes

CONCENTRATION AND C/MPC IN THE GROUNDWATER AT CORINTH FOLLOWING A TANK RUPTURE

ISOTOPE	HALF LIFE (YRS.)	TRAVEL TIME (YRS.)	CONC. RWST $(\mu ci/cc)$	CONC. CORINTH $(\mu ci/cc)$	10CFR20 $(\mu ci/cc)$	$\frac{C(i)}{MPC\ (i)}$
H-3	12.28	144	3.54E0	1.0466E-3	3E-3	3.49E-1
C-14	5730	144	1.45E-4	1.425E-4	8E-4	1.78E-1
Co-60	5.271	144	3.23E0	1.93E-8	5E-5	3.87E-4

TOTAL Σ C(i)/MPC(i) = 5.27E-1

*Except where noted, analysis and terms used in this section are based on "pre-1993 10 CFR 20" (see Section 12.0).

TABLE 2.4.13-1

PUBLIC WELLS WITHIN A 10-MILE RADIUS OF THE PLANT (AS REGISTERED WITH N.C. DIVISION OF ENVIRONMENTAL MANAGEMENT)

Location***	Owner	Maximum Pumpage Rate in gpm	Water* Level in ft.	Type of Use
1	A. Town of Fuquay-Varina	225	145	Municipal
	B. Town of Fuquay-Varina	370	104	Municipal
	C. Town of Fuquay-Varina	63	308	Municipal
	D. Town of Fuquay-Varina	300	87	Municipal
	E. Town of Fuquay-Varina	205	135	Municipal
	F. Town of Fuquay-Varina	34	100	Municipal
	G. Town of Fuquay-Varina	65	230	Municipal
	H. Town of Fuquay-Varina	90	55	Municipal
2.	A. Town of Holly Springs	50	160	Municipal
	B. Town of Holly Springs	55	105	Municipal
	C. Holly Springs School	0	15	School (standby)
	D. Pleasant Grove (near Holly Springs)	15	135	Trailer Park
	E. Town of Holly Springs	75	**	Trailer Park
3.	Darwood Thomas	**	**	Trailer Park
4.	James Pierson	**	**	Trailer Park
5.	A. Noah Jones	10	25	Trailer Park
	B. Noah Jones	10	25	Trailer Park
6.	McCoy-Thomas	**	**	Trailer Park
7.	New Hope Trailer Park	**	**	Trailer Park
8.	Moncure School	8	22	School
9.	Moncure Community Health Center	18	30	Clinic
10.	City of Moncure	125	114	Municipal
11.	Pleasant Hill Baptist Church	1	30	Public Supply
12.	U.S. Army Corps of Engineers, B. Everett Jordan Dam	2.5	120	Public Supply
13.	Frank Dickens (for Church)	2	30	Church
14.	Green Level Baptist Church	9	50	Public Supply
15.	A. M. Council Community Store	35	**	Public Supply
16.	Salem Baptist Church	3	25	Public Supply
17.	D. R. Allen & Son, Inc.	1.5	75	Office
18.	Reichhold Chemical Co.	30	70	Office
19.	Jack O. Farrell	5	127	Trailer Park
20.	Deep River Restaurant	10	237	Restaurant
21.	Lutheran Church Camp	12	270	Recreation
22.	Harris Energy and Environmental Center	9	144	Office
23.	Harris Energy and Environmental Center	18	208	Office
24.	Harris Energy and Environmental Center	15	120	Office

* Depth below ground surface.

TABLE 2.4.13-1 (Continued)

** Not available

*** For location, see Figure 2.4.13-4.

TABLE 2.4.13-2

LOCATION OF SITE WELLS AND PIEZOMETERS

	Piezometer Number	Plant Coordinate	Top of PVC Pipe Elevation	Ground Elevation	Well Number	Plant Coordinate	Top of Casing Elevation	Ground Elevation
(1)	PZ-2	N 1414 W 1172	264.54	263.15	1	N 2956 W 3357	262.21	259.84
	PZ-G	N 1871 W 1172	263.93	262.91	2	N 2651 W 3030	261.42	260.12
	LP-1	N 3271 W 3377	264.27	260.30	3	N 2746 W 3564	261.88	260.19
	LP-2	N 3495 W 1748	261.69	258.20	4A	N 1385 W 3305	260.71	257.54
	LP-3	N 3019 W 0492	265.25	261.75	5	N 4020 W 0977	245.05	244.06
	LP-4	N 2594 W 1958	265.13	260.94	5A	N 1178 W 3980	266.88	264.48
	LP-5	N 1905 W 3643	264.35	260.45	6	S 0840 W 0720	221.93	217.88
	LP-6	N 1886 W 2442	264.97	261.15	7	N 3694 W 0170	254.04	253.26
	LP-7	N 2118 W 1046	265.39	261.25	7A	N 2461 W 3400	261.68	260.19
(3)	LP-8	N 2050 W 0089	263.95	259.96	8	N 1714 W 0907	260.34	258.18
	LP-9	N 0861 E 0091	258.94	254.71	8A	N 1007 W 4186	261.02	259.57
	LP-10	N 0804 W 0898	264.76	261.10	9	N 2467 E 0101	259.74	258.73
(2)	LP-11	N 0911 W 1657	266.85	262.95	9A	S 0427 E 0106	233.65	231.60
	LP-12	N 0825 W 2980	264.44	259.61	10	N 2534 E 0060	262.38	259.75
	LP-13	N 0389 W 3288	262.90	259.03	11	N 3810 W 3758	256.11	255.03
	LP-14	N 0544 W 2295	264.89	260.71	12	N 0381 W 3241	260.46	258.46
(4)	LP-15	N 0509 W 1851	265.51	261.65	13	N 3684 W 0099	247.65	247.15
	LP-16	N 0219 W 0898	263.30	259.35	14	N 3239 W 3673	271.50	270.25
					15	N 3914 W 0080	241.83	240.00

(1) Abandoned 1-23-81

(2) Abandoned 5-07-80

TABLE 2.4.13-2 (Continued)

(3) Destroyed 1980

(4) Abandoned 1980

Note: Monitoring wells and piezometers listed above are for historical record. Current site groundwater monitoring locations are maintained in the ODCM.

TABLE 2.4.13-3

GROUNDWATER CONSUMPTION IN THOUSANDS OF GALLONS

MONTH	BATCH PLANT USE*	OTHER USE**	TOTAL
March, 1980	769.70	580	1349.7
April	1838.80	560	2398.8
May	1770.20	580	2350.2
June	1725.60	560	2285.6
July	3525.80	580	4105.8
August	2230.60	580	2810.6
September	2065.60	560	2625.6
October	1493.50	580	2073.5
November	1839.30	560	2399.3
December	1581.50	580	2161.5
January, 1979	966.10	580	1546.1
February	643.10	520	1163.1
March	1590.00	580	2170.0
April	1175.30	560	1735.3
May	753.60	580	1333.6
June	1147.40	560	1707.4
July	767.00	580	1327.0
August	1106.50	580	1686.5
September	752.70	560	1312.7
October	812.70	580	1392.7
November	1178.20	560	1738.2
December	1683.30	580	2263.3
January, 1980	907.40	580	1487.4
February	1641.90	540	2181.9

*Estimate based upon reported concrete volumes and measured consumption rate of 100 gallons per cubic yard (March, 1980). This includes water for mixing, aggregate washing, and cleanup.

**Estimate based upon measured consumption of 18,600 gallons per day (January, 1980 - March, 1980).

TABLE 2.4.13-4

ESTIMATED GROUNDWATER USE

	CONCRETE		CONSTRUCTION		OPERATION		TOTAL
YEAR	PRODUCTION (yd^3)	WATER USE*	PERSONNEL	WATER USE**	STAFF	WATER USE***	WATER USE
1980	1.43 x 105	39,178	4076	20,380	0	0	59,558
1981	1.43 x 105	39,178	3510	17,550	86	860	57,588
1982	1.43 x 105	39,178	2899	14,495	203	2030	55,703

NOTE: All water uses are in gallons per day.

*Estimate based upon projected concrete production and observed consumption rate of 100 gallons per cubic yard.

** Estimate based upon observed consumption rate of 5 gallons per person per day.

*** Estimate based upon projected consumption rate of 10 gallons per person per day.

TABLE 2.4.13-5

GROUNDWATER LEVELS IN SITE WELLS

Date Measured	Well Number 1	2	3	5	5A	6	7	7A	8	8A	9	9A	11
11/15/79	118.28*	196.84	--	240.22	198.55*	--	199.37	--	--	135.84*	173.41*	202.48	--
11/21/79	114.63*	197.59	157.38*	240.05	--	200.10	201.37	185.85*	--	136.09*	173.41*	202.15	--
11/29/79	114.16	197.50	157.21*	240.05	--	200.10	201.29	185.76*	--	136.01*	173.45*	202.15	--
12/07/79	114.71*	197.42	157.30*	239.97	198.55*	200.18	201.12	185.68*	192.94*	135.84*	173.41*	201.98	--
12/19/79	--	--	--	--	--	--	--	--	--	--	--	--	--
12/24/79	185.51	195.62	195.78	240.45	176.08	200.73	203.04	195.88	197.44	171.62	187.94	202.55	--
12/27/79	200.13*	195.34	200.05*	239.88	172.55*	200.68	201.54	199.84*	196,84*	160.06*	169.74*	202.32	--
01/03/80	208.21	199.09	206.96*	240.05	175.63*	200.26	202.25	207.22*	195.80*	168.27*	179.24*	202.32	235.03
01/10/80	--	--	--	240.13	153.13*	--	202.04	--	--	128.44*	179.49*	204.82	--
01/14/80	213.29	203.42	211.96*	--	--	202.01	--	212.26*	200.26*	--	--	--	236.03
01/25/80	211.38	201.25	205.46*	241.05	164.55*	203.68	202.21	210.85*	202.34*	155.60*	178.51*	204.48	240.94
02/01/80	186.38	200.09	182.96*	242.05	166.63*	202.01	201.46	156.68*	201.84*	161.19*	175.16*	203.82	234.61
02/11/80	186.46	195.92	148.30*	240.72	171.30*	202.43	198.62	171.60*	199.76*	159.44*	166.57*	205.98	233.78
12/18/80	182.54	193.84	146.38*	240.72	169.21*	201.60	196.62	156.35*	204.76*	142.69*	165.91*	203.57	232.69
12/26/80	194.13	192.34	148.80*	241.30	164.13*	201.85	194.96	135.18*	201.59*	127.94*	166.41*	203.65	232.36

Note: Wells 4A and 10 are not accessible for monitoring.

TABLE 2.4.13-6

GROUNDWATER LEVELS IN SITE PIEZOMETERS

Date Measured	Piezometer Number																	
	PZ-2	PZ-G	LP-1	LP-2	LP-3	LP-4	LP-5	LP-6	LP-7	LP-8	LP-9	LP-10	LP-11	LP-12	LP-13	LP-14	LP-15	LP-16
11/15/79	201.29	217.76	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
11/21/79	201.54	210.68	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
11/29/79	201.44	210.76	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
12/07/79	201.42	210.68	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
12/19/79	202.71	216.14	229.31	257.11	219.67	230.50	--	233.72	150.29	183.16	208.02	200.26	209.14	205.36	202.98	205.14	223.68	200.22
12/24/79	203.14	214.53	230.57	256.79	220.05	230.13	143.45	233.67	160.49	190.65	208.44	200.76	210.35	201.44	199.60	218.39	223.71	200.80
12/27/79	208.71	214.91	230.94	256.69	219.33	229.30	145.18	232.80	165.47	191.49	208.19	199.84	209.81	199.27	197.82	222.39	223.09	200.13
01/03/80	203.04	214.01	233.19	256.77	219.50	228.71	155.18	232.80	177.89	188.99	208.23	199.97	209.68	197.77	197.07	225.39	222.43	200.05
01/10/80	--	--	--	257.94	--	--	--	--	--	--	--	--	--	197.40	--	--	--	--
01/14/80	206.37	217.01	234.44	--	224.33	233.13	169.85	235.05	179.89	193.28	210.94	203.22	215.35	--	208.57	226.89	223.51	203.13
01/25/80	209.37	215.60	233.94	258.61	220.08	235.13	164.52	235.97	195.89	188.53	211.44	204.63	210.10	200.61	199.73	224.39	225.68	200.13
02/01/80	202.71	219.18	230.44	259.19	220.25	230.13	175.60	233.47	199.39	188.45	208.77	201.43	210.85	199.69	194.23	228.39	222.93	203.13
02/11/80	202.46	221.18	230.10	258.36	224.83	229.55	176.35	232.47	201.47	181.45	206.69	200.01	209.27	197.19	192.65	226.72	222.43	201.80
12/18/80	201.46	219.10	228.77	257.86	219.67	229.44	172.60	232.14	203.14	178.70	207.19	201.01	208.68	197.36	193.32	226.64	221.84	201.05
12/26/80	201.21	218.36	229.77	258.77	219.25	229.63	172.27	231.89	201.22	181.20	206.37	200.68	208.18	197.44	192.90	226.39	221.51	200.88

TABLE 2.4.13-7

PERMEABILITY OF PLANT SITE MATERIALS BASED ON DOWN-HOLE PRESSURE TESTS*

Material	No. of Tests	Permeability Range ft./yr.	Permeability Range Centimeters/Second
Fine sandstone	8	0.47-2.54**	4.7×10^{-7}-2.37×10^{-4}
Shaley siltstone	3	6.71-12.93‡	6.7×10^{-6}-4.20×10^{-4}
Siltstone	5	1.31-2.91	1.31×10^{-6}-2.91×10^{-6}

* Down-hole pressure tests may yield high permeabilities.

** One test produced results of 237 ft./yr.

‡ One test produced results greater than 520 ft./yr.

Tests performed in borings BP-62, BP-68, and BP-70.

TABLE 2.4.13-8

CHEMICAL QUALITY OF SITE GROUNDWATER

ANALYSIS PARAMETER	WELL NO. 2	WELL NO. 4A	WELL NO. 7A
Color	3	0	0
pH	7.3	7.9	7.9
Alkalinity $CaCO_3$	107	134	140
Total Hardness	72	106	137
Iron	0.13	0.35	0.95
Manganese	0.24	0.38	0.29
Turbidity SiO_2	1	2	1
Acidity $CaCO_3$	11	3	5
Chloride	23	22	21
Sodium	35	30	19
Potassium	2.0	1.6	1.1
Fluoride	<0.10	<0.10	<0.10
Arsenic	<0.01	<0.01	<0.01
Cadmium	<0.01	<0.01	<0.01
Chromium^{+6}	<0.05	<0.05	<0.05
Copper	<0.05	<0.05	<0.05
Lead	<0.05	<0.05	<0.05
Zinc	0.40	<0.05	<0.05
Calcium	14.8	21.0	26.5
Magnesium	7.5	11.0	15.4

Note: Analyses performed during March 1973 by N. C. Board of Health, Laboratory Division, Raleigh, North Carolina. All results are expressed in parts per million except the parameters of color and pH.

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
FEB 1698				32.9	80.0			GAB	202
7 FEB 1757				32.9	80.0			BOV	202
23 NOV 1766				32.9	80.0			BOV	202
21 FEB 1774	14	0	0.0	37.3	77.4			BOL	140
22 FEB 1774				37.3	76.7			BOL	166
NOV 1776				32.9	80.0			BOV	202
19 NOV 1789	6	0	0.0	38.3	77.5			BOL	196
15 JAN 1791	5	0	0.0	37.6	77.4			BOL	156
11 FEB 1795	20	0	0.0	38.0	78.5			BOL	160
11 APR 1799	3	20	0.0	34.3	80.6	V		GAB	134
11 APR 1799	16	55	0.0	34.3	80.6			GAB	134
10 FEB 1801	21	0	0.0	37.4	79.2			BOL	118
23 AUG 1802	5	0	0.0	37.6	77.4			BOL	156
2 FEB 1812	9	30	0.0	37.6	77.4			BOL	156
22 APR 1812				37.6	77.4			BOL	156
30 DEC 1816				32.9	80.0			BOV	202
31 DEC 1816	13	0	0.0	36.8	76.3			BOL	166
8 JAN 1817	4	0	0.0	32.9	80.0	V		BOV	202
3 SEP 1820	3	0	0.0	33.2	79.1			BOV	172
9 AUG 1826	21	0	0.0	37.6	77.4			BOL	156
10 AUG 1826	12	0	0.0	37.6	77.4			BOL	156
9 MAR 1828	22	0	0.0	37.7	78.5	V		GAB	140
27 AUG 1833	6	0	0.0	37.7	78.0	V		GAB	147
11 APR 1843				34.3	80.6			BOV	134
17 OCT 1850				37.4	78.4			BOL	121
29 APR 1852	13	0	0.0	36.6	81.6	VI		BOL	160
3 MAY 1852	3	0	0.0	36.7	82.0			BOL	183
18 SEP 1852	3	0	0.0	36.7	82.0			BOL	183
2 NOV 1852	18	35	0.0	36.7	78.0	VI		GAB	147
22 NOV 1854	16	0	0.0	37.1	81.5			BOL	171
2 FEB 1855	8	0	0.0	37.0	78.6	V		EQH	91
21 MAR 1856	9	0	0.0	37.7	78.8			BOL	138
22 MAR 1859				37.2	81.5			BOL	175
19 JAN 1860	18	0	0.0	32.9	80.0	V		BOV	202
19 DEC 1860				32.9	80.0			BOV	202

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
31 AUG 1861	5	22	0.0	36.6	78.5	VI		GAB	66
1869				32.9	80.0			BOV	202
FEB 1872				36.8	79.4			BOL	79
1 MAR 1872				36.8	79.4	III		GAB	79
4 JUN 1872				37.6	77.4	III		BOL	156
3 OCT 1873				37.2	78.2			BOL	111
10 FEB 1874				35.7	82.1	V		GIT	177
17 APR 1874				35.7	82.1			GIT	177
10 MAR 1875				37.7	78.0			BOL	147
23 DEC 1875				37.7	78.0			BOL	147
23 DEC 1875	4	45	0.0	37.6	78.5	VII		EQH	133
26 DEC 1875	12	0	0.0	37.6	77.9			BOL	143
2 JAN 1876	21	30	0.0	37.6	77.9	III		GAB	143
21 DEC 1876	10	30	0.0	36.9	81.1			BOL	145
22 DEC 1876	23	45	0.0	37.4	77.5			BOL	142
2 JAN 1878	19	0	0.0	37.8	77.8			BOL	158
26 OCT 1879				34.4	81.1			GIT	151
13 DEC 1879	7	0	0.0	35.2	80.8	V		EQH	109
8 JAN 1882	17	10	0.0	34.6	76.5			GAB	158
2 APR 1882				38.6	78.6			BOL	200
18 JAN 1884	13	0	0.0	34.3	78.0	V		EQH	110
2 FEB 1885	7	10	0.0	36.9	81.1	IV		BOL	145
6 AUG 1885	13	0	0.0	36.2	81.6	V		EQH	152
10 OCT 1885	4	35	0.0	37.7	78.8	VI		EQH	138
27 AUG 1886	1	30	0.0	33.0	80.2			VIS	199
27 AUG 1886	5	30	0.0	33.0	80.2			VIS	199
28 AUG 1886	5	30	0.0	33.0	80.2			VIS	199
29 AUG 1886				33.0	80.2			VIS	199
31 AUG 1886	21	51	0.0	32.9	80.0	X		GAB	202
31 AUG 1886	21	59	0.0	32.9	80.0			GAB	202
1 SEP 1886	22	0	0.0	38.1	78.5			BOL	167
3 SEP 1886	24	0	0.0	36.9	81.1			BOL	145
24 SEP 1886	21	56	0.0	36.9	81.1			BOL	145
22 OCT 1886	5	20	0.0	32.9	80.0	VI		GAB	202
22 OCT 1886	14	45	0.0	32.9	80.0	VII		GAB	202

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
5 NOV 1886	12	20	0.0	32.9	80.0	VI		GAB	202
28 AUG 1889	21	0	0.0	34.3	81.6			VIS	178
3 MAY 1897	17	18	0.0	37.1	80.7	VI		EQH	137
9 MAY 1897				33.5	81.3			VIS	202
31 MAY 1897	18	58	0.0	37.3	80.7	VIII		EQH	147
28 JUN 1897				37.3	79.9			BOL	122
4 SEP 1897				36.9	81.1			BOL	145
22 OCT 1897	3	20	0.0	37.0	81.0	V		EQH	145
27 NOV 1897				37.7	77.5			BOL	159
18 DEC 1897	23	45	0.0	37.7	77.5	V		EQH	159
5 FEB 1898	20	0	0.0	37.0	80.7	VI		EQH	132
25 NOV 1898	15	0	0.0	37.0	81.0	V		BOL	145
20 JAN 1899				34.2	81.7			VIS	187
13 FEB 1899	9	30	0.0	37.0	81.0	V		EQH	145
3 MAR 1899				36.8	76.3			BOL	166
19 DEC 1899				34.3	81.4			VIS	169
1 OCT 1901				34.2	81.7			VIS	187
1 DEC 1901	20	0	0.0	34.1	80.7			VIS	148
17 MAY 1902				37.3	80.7			BOL	147
24 MAY 1902	1	15	0.0	34.3	81.4			VIS	169
24 MAY 1902	9	2	0.0	34.3	81.4			VIS	169
13 MAR 1904	22	30	0.0	34.5	82.0			VIS	191
30 APR 1904				34.0	81.7			VIS	195
29 APR 1905				37.4	79.6			BOL	122
18 APR 1906				34.1	81.3			VIS	173
5 AUG 1906	1	2	0.0	33.0	80.2			GIT	199
10 FEB 1907	19	30	0.0	37.8	78.5	III		BOL	146
11 FEB 1907	13	22	0.0	37.7	78.4	VI		EQH	141
15 JAN 1908	14	0	0.0	33.0	80.2	IV		GIT	199
3 MAR 1908	16	6	0.0	33.0	80.2	IV		GIT	199
7 MAR 1908	1	50	0.0	33.0	80.2	IV		GIT	199
23 AUG 1908	9	30	0.0	37.5	77.9	V		EQH	137
25 OCT 1908	23	10	0.0	33.0	80.2	III		GIT	199
28 OCT 1908	6	24	0.0	33.0	80.2	IV		GIT	199
8 MAY 1910	21	10	0.0	37.7	78.4	V		EQH	141

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
10 FEB 1911	5	22	0.0	36.6	79.4			BOL	66
12 JUN 1912	10	30	0.0	33.0	90.2	VII		EQH	199
7 AUG 1912	20	0	0.0	37.7	78.4	IV		BOL	141
7 DEC 1912				34.7	81.7	IV		GIT	170
1 JAN 1913	18	28	0.0	34.7	81.7	VII		EQH	170
5 MAR 1914	16	0	0.0	35.1	81.4			VIS	144
6 MAR 1914	20	20	0.0	34.2	79.8	IV		GIT	114
19 JUN 1914	3	3	0.0	33.0	80.2	III		GIT	199
13 JUL 1914	20	53	0.0	33.0	80.2	III		GIT	199
14 JUL 1914	3	0	0.0	33.0	80.2	II		GIT	199
22 SEP 1914	7	4	0.0	33.0	80.2	V		EQH	199
23 DEC 1914	6	55	0.0	33.0	80.2	II		GIT	199
12 DEC 1915	19	55	0.0	32.8	79.0	IV		GIT	199
21 FEB 1916	17	39	0.0	35.5	82.5	VII		GAB	200
16 APR 1916	6	56	0.0	33.0	80.2	II		GIT	199
30 APR 1916	1	46	0.0	33.0	80.2	II		GIT	199
25 JUN 1916	7	5	0.0	33.0	80.2	III		GIT	199
14 JUL 1916	13	18	0.0	33.0	80.2			GIT	199
26 AUG 1916	19	36	0.0	36.0	81.0	V		EQH	116
24 SEP 1916	4	42	0.0	33.0	80.2	II		GIT	199
11 APR 1917	14	1	0.0	33.0	80.2	II		GIT	199
19 APR 1918			0.0	36.8	76.3			BOL	166
1 AUG 1920	6	53	0.0	33.0	80.2	III		GIT	199
19 APR 1921	18	45	0.0	33.0	80.2	III		GIT	199
23 APR 1921	18	48	0.0	33.0	80.2	III		GIT	199
15 JUL 1921				36.6	82.3	VI		BOL	197
7 AUG 1921	6	30	0.0	37.8	78.4	V		EQH	148
8 AUG 1922	4	25	0.0	33.0	80.2	III		GIT	199
22 MAR 1923	23	25	0.0	33.0	80.2	III		GIT	199
28 OCT 1923	11	15	0.0	34.9	80.1	III		GIT	85
14 FEB 1924	11	6	0.0	33.0	80.2	III		GIT	199
3 JUN 1924	10	4	0.0	33.0	80.2	III		GIT	199
13 NOV 1924	5	30	0.0	36.6	82.2	V		GAB	192
26 DEC 1924	4	30	0.0	37.3	79.9	V		EQH	122
15 MAY 1925	20	30	0.0	37.3	77.5			GAB	136

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
14 JUL 1925	16	20	0.0	37.6	77.4	IV		GAB	156
8 JUL 1926	9	50	0.0	35.9	82.1	VI		EQH	177
10 JUN 1927	7	16	0.0	38.0	79.0	V		EQH	158
30 OCT 1928	11	45	0.0	37.5	77.5			USE	148
19 NOV 1928	22	45	0.0	35.8	82.3			GAB	188
22 DEC 1928	21	30	0.0	35.3	80.8			GAB	107
3 JAN 1929	12	5	0.0	33.9	80.3			USE	145
27 DEC 1929	2	56	0.0	38.1	78.5	VI		USE	167
3 SEP 1930	1	30	0.0	33.0	80.2			USE	199
15 SEP 1930	7	40	0.0	37.5	77.5			USE	148
26 DEC 1930	3	24	0.0	34.5	80.3			GIT	112
5 OCT 1931	21	15	0.0	37.7	78.3			BOL	142
4 JAN 1932	23	5	0.0	37.6	78.6	III		BOL	132
6 JAN 1932	12	35	0.0	33.0	80.2			GIT	199
13 JAN 1932	12	40	0.0	33.0	80.2			GIT	199
25 DEC 1932				37.3	77.4			BOL	140
26 JAN 1933	22	0	0.0	37.3	77.4	III		BOL	140
23 JUL 1933	10	0	0.0	37.6	78.5	III		BOL	133
26 JUL 1933	2	34	0.0	33.0	80.2			GIT	199
19 DEC 1933	9	12	0.0	33.0	80.2	V		GIT	199
23 DEC 1933	9	40	0.0	33.0	80.2			GIT	199
2 APR 1934	21	5	0.0	37.3	77.4			BOL	140
9 DEC 1934	10	0	0.0	33.0	80.2			GIT	199
6 FEB 1935	12	36	0.0	33.0	80.2			GIT	199
10 FEB 1935	18	45	0.0	37.3	77.4			BOL	140
20 OCT 1935	16	20	0.0	33.0	80.2			GIT	199
APR 1936	7	42	0.0	38.1	78.5			BOL	167
30 DEC 1936	3	50	0.0	33.0	80.2			GIT	199
2 FEB 1937	20	26	0.0	37.7	78.6	IV		BOL	139
25 OCT 1937	19	1	0.0	33.0	80.2			GIT	199
5 JAN 1940	13	45	0.0	33.0	80.2			GIT	199
8 OCT 1940	3	20	0.0	33.0	80.2			GIT	199
27 OCT 1940	22	20	0.0	33.0	80.2			VIS	199
27 DEC 1940	9	32	0.0	33.0	80.2			GIT	199
6 OCT 1942	22	15	0.0	37.6	78.6			BOL	132

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
1 NOV 1942	3	20	0.0	34.4	81.1			GIT	151
28 DEC 1943	15	25	0.0	33.0	80.2	IV		GIT	199
28 JAN 1944	18	30	0.0	33.0	80.2			GIT	199
30 JAN 1945	21	20	0.0	33.0	80.2			GIT	199
5 JUN 1945	13	10	0.0	34.0	81.0			GIT	165
26 JUL 1945	10	32	15.0	34.5	81.5	VII*	5.6PAS	G-R	166
26 JUL 1945	10	32	30.0	34.3	81.4	V		USE	169
10 OCT 1945	15	43	0.0	37.6	78.5			BOL	133
12 OCT 1945	15	0	0.0	37.6	78.5			BOL	133
29 OCT 1945	21	29	0.0	37.6	78.5			BOL	133
8 FEB 1946	18	9	0.0	33.0	80.2			GIT	199
24 MAY 1946	14	40	0.0	38.0	78.5			BOL	160
2 NOV 1947	4	30	0.0	33.0	80.2			GIT	199
4 JAN 1948	21	45	0.0	37.6	78.6			BOL	132
26 MAR 1948	18	48	0.0	38.1	78.5			BOL	167
2 FEB 1949	10	52	0.0	33.0	80.2			GIT	199
8 MAY 1949	6	1	0.0	37.6	77.9			BOL	143
27 JUN 1949	6	53	0.0	33.0	80.2			GIT	199
26 NOV 1950	2	45	0.0	37.7	78.4			BOL	141
4 MAR 1951	2	55	0.0	33.0	80.2			GIT	199
8 MAR 1951	0	20	0.0	33.0	80.2			GIT	199
9 MAR 1951	2	0	0.0	37.6	77.4			BOL	156
10 MAR 1951	8	18	0.0	33.0	80.2			GIT	199
30 MAR 1951	7	55	0.0	33.0	80.2			GIT	199
7 SEP 1952	12	32	0.0	33.0	80.2			GIT	199
10 SEP 1952	22	15	0.0	38.1	78.5	IV		BOL	167
7 FEB 1953	3	0	0.0	37.7	78.2	IV		BOL	144
6 JAN 1955	15	30	0.0	36.6	82.2	IV		BOL	192
17 JAN 1955	7	37	0.0	37.3	78.5	IV		BOL	113
28 SEP 1955	2	2	0.0	36.6	81.3	V		BOL	145
13 MAY 1957	9	24	58.0	35.8	82.1	VI		GAB	177
5 MAR 1958	6	53	43.0	34.3	77.8	V		GAB	116
23 APR 1959	20	58	41.0	37.5	80.5	VI		USE	151
7 JUL 1959	18	17	0.0	37.4	80.7	IV		BOL	152
3 AUG 1959	6	8	30.0	33.0	79.5	VI		CGS	188

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
21 AUG 1959	12	20	0.0	37.4	80.7	IV		BOL	152
26 OCT 1959	21	7	28.0	34.5	80.2	VI		GAB	108
3 JAN 1960	2	30	0.0	36.0	82.0			GAB	172
12 MAR 1960	12	47	40.0	33.0	79.0	V		CGS	186
4 SEP 1960	18	40	0.0	37.4	79.3	IV		BOL	118
20 MAY 1961	15	43	0.0	33.0	80.2	III		GIT	199
18 OCT 1961	0	35	0.0	33.0	80.2	III		GIT	199
11 APR 1963	17	45	0.0	34.8	82.3	IV		GIT	199
28 OCT 1963	22	38	35.0	36.7	81.0	V		CHC	133
20 APR 1964	19	4	46.6	34.0	81.0	V		GIT	164
25 NOV 1964	2	50	5.0	37.4	81.5	V*	4.5	CGS	184
26 APR 1965	15	26	21.5	37.3	81.6			CGS	184
9 SEP 1965	9	42	20.0	34.7	81.3			GAB	149
10 SEP 1965	2	32	0.0	34.7	81.3			GAB	149
12 SEP 1965	13	25	2.0	34.7	81.3			GIT	149
31 MAY 1966	6	19	2.1	37.6	78.0	V	3.1	CGS	141
23 OCT 1967	9	4	10.0	33.4	80.7	V	3.8	USE	187
16 DEC 1967	12	23	37.4	37.4	81.6	IV*	3.5	CGS	188
8 MAR 1968	5	38	15.2	37.3	80.8	IV	3.9	BLA	151
22 SEP 1968	21	41	18.5	34.0	81.5	IV	3.7	USE	186
25 NOV 1968	20	0	0.0	34.1	77.9	IV		GAB	125
20 NOV 1969	1	0	9.0	37.4	81.0	VI	4.8CGS	USE	163
11 DEC 1969	23	44	39.2	37.8	77.4	V		USE	168
30 JUL 1970	8	48	51.5	37.0	82.2	IV*	3.8	CGS	204
30 JUL 1970	15	15	16.3	37.0	82.2	V*	4.0	CGS	204
10 SEP 1970	1	41	10.0	36.1	81.4	V		USE	140
1 APR 1971	5	5	11.0	37.4	81.6			NOS	188
19 MAY 1971	12	54	3.4	33.3	80.6	V	3.4	NOS	186
31 JUL 1971	20	16	55.6	33.4	80.7	III		ERL	187
11 AUG 1971	3	50	0.0	33.4	80.8			GIT	188
12 SEP 1971	0	6	27.1	38.1	77.4	V		ERL	183
22 OCT 1971	21	55	0.0	36.0	82.0			GIT	172
9 JAN 1972	23	24	29.1	37.4	81.6			ERL	186
3 FEB 1972	23	11	8.4	33.5	80.4	V	4.5	ERL	175
25 APR 1972	10	26	0.0	36.2	79.9			GIT	63

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
25 APR 1972	11	46	0.0	36.2	79.9			GIT	63
20 MAY 1972	19	39	6.4	37.0	82.2			ERL	204
19 DEC 1973	10	16	8.7	33.0	80.3			GS	201
30 MAY 1974	21	28	37.2	37.4	80.4	V	3.6BLA	GS	141
22 NOV 1974	5	25	55.5	32.9	80.1	VI	4.7	GS	204
11 NOV 1975	8	10	39.3	37.2	80.8	IV	3.0SLM	GS	147
16 NOV 1975	1	1	3.5	34.3	80.6			GS	135
19 JUN 1976	5	54	13.9	37.4	81.6	III*	3.0BLA	GS	187
3 JUL 1976	20	53	46.5	37.2	81.1			GS	158
13 SEP 1976	18	54	37.1	36.6	80.8	VI	3.3BLA	GS	121
18 JAN 1977	18	29	13.5	33.1	80.2	III*	3.0BLA	GS	194
27 FEB 1977	20	5	34.6	37.9	78.6	V	2.4BLA	GS	152
31 MAY 1977	23	50	13.2	33.0	80.2	II*	2.3 GS	GS	203
23 AUG 1977	13	45	0.0	32.9	80.2	II*	2.3GS	SEU	202
25 AUG 1977	4	20	7.5	33.4	80.7	III*	2.8GS	SEU	189
1 SEP 1977	21	5	32.5	33.4	80.7	I*	1.8GS	SEU	185
23 OCT 1977	7	51	41.7	37.0	82.0	III*	2.8BLA	GS	193
10 NOV 1977	11	25	0.1	33.4	80.7		0.8GS	SEU	187
15 DEC 1977	7	15	55.2	33.0	80.3	II*	2.0GS	SEU	201
15 DEC 1977	19	16	43.6	32.9	80.2	II*	2.6GS	SEU	202
20 DEC 1977	23	41	23.3	33.1	80.2	I*	1.8GS	SEU	195
25 JAN 1978	3	29	38.7	34.3	81.3	III*	2.8USC	SEU	164
28 JAN 1978	23	13	23.3	37.2	80.8		1.0BLA	SEU	146
25 FEB 1978	3	53	27.7	36.2	79.3	II*	2.2BLA	SEU	39
17 MAR 1978	18	26	34.5	36.8	80.7	III*	2.8BLA	SEU	123
22 MAR 1978	15	52	26.7	36.2	81.7	III*	2.9BLA	SEU	159
22 APR 1978	6	36	24.3	34.2	81.3	II*	2.6BLA	SEU	165
10 MAY 1978	4	19	12.8	37.4	80.7		0.4NAV	SEU	151
25 MAY 1978	8	30	27.0	37.1	80.8		1.0NAV	SEU	141
22 JUN 1978	6	42	29.0	36.8	82.4		1.0NAV	SEU	204
4 AUG 1978	8	48	42.1	35.3	82.5	II*	2.3BLA	SEU	204
14 AUG 1978	4	50	5.0	38.0	80.9		1.1BLA	SEU	189
7 SEP 1978	22	53	23.0	33.1	80.2	II*	2.6GS	SEU	195
30 OCT 1978	9	15	6.5	33.0	80.2			SEU	195
30 OCT 1978	9	15	13.0	33.0	80.2	I*	1.9GS	SEU	195

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
30 OCT 1978	9	16	14.9	33.0	80.2	II*	2.4GS	SEU	195
30 OCT 1978	10	4	22.0	33.1	80.2		1.3GS	SEU	195
8 JAN 1979	1	4	33.3	34.4	81.3	II*	2.3GS	SEU	163
22 JAN 1979	17	11	29.6	34.0	81.1	I*	1.7GS	SEU	168
27 JAN 1979	23	55	15.7	33.1	80.2	III*	2.8	SEU	195
1 FEB 1979	1	25	48.4	34.3	81.3	II*	2.6USC	SEU	164
16 FEB 1979	14	37	8.4	34.3	81.4	III*	3.3GS	SEU	165
20 FEB 1979	23	20	44.4	34.4	81.4	II*	2.4GS	SEU	165
24 FEB 1979	9	31	42.3	34.3	81.3	II*	2.0GS	SEU	165
22 MAR 1979	3	17	46.1	34.4	81.3	II*	2.0GS	SEU	159
22 APR 1979	2	4	57.2	34.4	81.3	II*	2.1GS	SEU	161
24 APR 1979	9	49	55.9	34.3	81.3	I*	1.6GS	SEU	163
4 MAY 1979	12	13	8.9	34.3	82.0	II*	2.6GS	SEU	194
7 MAY 1979	10	10	17.2	34.5	81.4	I*	1.7GS	SEU	160
5 JUN 1979	9	37	44.2	34.3	81.3	II*	2.4USC	SEU	165
5 JUN 1979	9	40	2.1	34.3	81.3	II*	2.3USC	SEU	165
11 JUN 1979	16	21	4.5	34.3	81.3	I*	1.7GS	SEU	163
30 JUN 1979	1	35	8.5	34.3	81.3	II*	2.3GS	SEU	164
4 JUL 1979	11	53	22.2	33.0	80.2		1.3GS	SEU	197
12 JUL 1979	18	31	44.2	34.7	81.6	II*	2.1GS	SEU	164
13 JUL 1979	19	25	38.9	34.8	81.7	II*	2.2GS	SEU	168
6 AUG 1979	15	21	27.6	34.4	81.5			SEU	169
7 AUG 1979	19	32	16.9	34.2	81.3	III*	3.0	SEU	167
11 AUG 1979	2	11	56.6	33.0	80.2	II*	2.5GS	SEU	200
14 SEP 1979	0	45	31.4	34.3	81.3	III*	2.7USC	SEU	164
3 OCT 1979	3	43	10.0	34.4	81.3	II*	2.4GS	SEU	161
8 OCT 1979	8	53	52.8	36.4	82.0	IV*	3.6GS	SEU	179
14 OCT 1979	8	23	57.3	34.3	81.4	III*	2.9GS	SEU	166
15 OCT 1979	23	54	51.8	34.3	81.3	II*	2.4GS	SEU	164
21 OCT 1979	7	10	28.6	32.9	80.2	I*	1.6GS	SEU	204
6 NOV 1979	4	4	50.8	37.4	78.3		1.3	SEU	126
11 NOV 1979	7	21	52.9	37.7	77.5		1.2	SEU	161
20 NOV 1979	15	49	2.8	34.2	80.7	II*	2.5GS	SEU	141
27 NOV 1979	22	36	5.3	34.4	82.2	II*	2.2GS	SEU	204
7 DEC 1979	5	43	34.9	33.0	80.2	III*	2.8GS	SEU	198

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
17 DEC 1979	17	34	56.9	33.5	80.6		1.1GS	SEU	178
6 JAN 1980	13	50	55.7	36.6	81.6		2.3BLA	SEU	159
17 JAN 1980	21	8	35.7	34.6	82.3	II*	2.0GS	SEU	203
18 FEB 1980	3	58	55.3	37.4	80.6		0.3BLA	SEU	150
10 MAR 1980	22	11	30.6	34.3	81.3	II*	2.3GS	SEU	164
7 APR 1980	22	14	33.6	34.3	82.1	I*	1.9GS	SEU	204
9 APR 1980	20	47	24.0	34.8	79.9	III*	2.8GS	SEU	81
10 APR 1980	22	33	15.7	37.5	81.1		0.5BLA	SEU	171
22 APR 1980	3	14	4.6	36.4	80.6	II*	2.5BLA	SEU	104
24 APR 1980	6	16	56.6	34.3	81.3	III*	3.0GS	SEU	164
26 APR 1980	3	59	54.8	37.8	77.6	II*	2.4BLA	SEU	161
8 MAY 1980	21	13	22.1	34.4	82.2	II*	2.0GS	SEU	203
9 MAY 1980	15	41	40.7	35.0	81.9			SEU	173
18 MAY 1980	3	31	19.9	37.6	77.9	I*	1.6BLA	SEU	141
18 MAY 1980	22	33	55.4	38.0	78.1		0.7BLA	SEU	164
22 JUN 1980	20	33	6.2	33.0	80.2	II*	2.1GS	SEU	197
22 JUN 1980	23	35	26.5	33.0	80.2	I*	1.6GS	SEU	197
1 JUL 1980	23	33	19.8	33.4	80.7	I*	1.6GS	SEU	187
29 JUL 1980	1	10	22.7	34.4	81.4	III*	3.2 GS	SEU	165
4 AUG 1980	10	13	32.7	38.1	77.8		0.7BLA	SEU	175
1 SEP 1980	5	44	42.2	33.0	80.2	III*	2.9GS	SEU	200
21 SEP 1980	10	2	46.3	38.2	80.1	I*	1.4BLA	SEU	181
26 SEP 1980	1	31	57.8	38.1	77.8	II*	2.0BLA	SEU	176
26 SEP 1980	5	4	15.7	38.1	77.7		0.1BLA	SEU	177
26 SEP 1980	16	41	19.0	33.7	79.8			SEU	145
1 OCT 1980	22	24	9.7	34.1	80.5	II*	2.1GS	SEU	142
9 OCT 1980	1	47	1.1	37.2	80.8			SEU	147
11 OCT 1980	22	40	28.5	38.1	77.8		0.7BLA	SEU	178
14 OCT 1980	1	20	4.6	37.1	80.2	I*	1.7BLA	SEU	118
16 OCT 1980	3	48	7.6	38.1	80.2		1.1BLA	SEU	177
5 NOV 1980	21	48	14.7	38.2	79.9	III*	2.8BLA	SEU	178
8 NOV 1980	20	49	54.0	33.7	80.9	II*	2.2GS	SEU	174
20 NOV 1980	16	19	15.3	34.7	82.1	II*	2.0GS	SEU	190
25 NOV 1980	7	44	4.0	38.1	80.1		0.6BLA	SEU	177
2 DEC 1980	7	47	38.2	37.4	80.5		0.4BLA	SEU	147

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
27 DEC 1980	8	40	26.7	34.3	81.3	II*	2.5GS	SEU	164
19 JAN 1981	21	54	19.3	37.7	78.4		0.1BLA	SEU	143
21 JAN 1981	16	29	58.1	37.8	78.4		1.2BLA	SEU	145
11 FEB 1981	13	44	16.4	37.7	78.4	IV*	3.4BLA	SEU	142
11 FEB 1981	13	50	31.4	37.7	78.4	III*	3.2BLA	SEU	144
11 FEB 1981	13	51	38.6	37.7	78.5	III*	2.9BLA	SEU	142
12 FEB 1981	10	41	59.0	37.7	78.4			SEU	143
21 FEB 1981	4	48	26.5	33.6	81.2	II*	2.0	SEU	192
4 MAR 1981				35.8	79.7	III*	2.8BLA	SEU	42
19 MAR 1981	4	33	55.4	33.0	80.2	II*	2.3GS	SEU	201
20 MAR 1981	4	2	3.0	37.5	77.7		0.6BLA	SEU	144
26 MAR 1981	9	12	50.9	33.0	80.2	I*	1.4GS	SEU	201
9 APR 1981	7	10	31.6	35.5	82.1	III*	3.2TEI	SEU	175
9 APR 1981	7	12	54.4	37.5	77.8	II*	2.1BLA	SEU	137
9 APR 1981	7	34	36.0	37.5	77.9		0.4BLA	SEU	135
9 APR 1981	12	2	37.4	35.5	82.1	III*	2.7TEI	SEU	178
9 APR 1981	12	2	53.8	34.2	81.8	II*	2.4GS	SEU	190
10 APR 1981	6	4	59.8	35.5	82.1	II*	2.5TEI	SEU	175
11 APR 1981	15	29	25.7	38.2	79.8			SEU	180
12 APR 1981	13	7	30.6	34.3	81.3	II*	2.1GS	SEU	167
16 APR 1981	13	49	20.5	37.6	78.2		0.1BLA	SEU	137
3 MAY 1981	4	5	39.5	33.0	80.2		1.0GS	SEU	201
5 MAY 1981	21	21	57.9	35.3	82.4	III*	3.3TEI	SEU	197
19 MAY 1981	9	46	11.3	35.2	82.1	II*	2.3GS	SEU	182
3 JUN 1981	20	54	22.9	36.2	81.7	II*	2.3TEI	SEU	156
6 JUN 1981	8	5	58.7	38.2	79.5		0.7BLA	SEU	175
8 JUN 1981	5	10	46.7	32.9	80.2		1.2GS	SEU	203
19 JUN 1981	20	1	6.6	34.3	81.1	I*	1.8GS	SEU	158
30 JUL 1981	11	59	48.5	38.2	78.1	I*	1.4VPI	SEU	178
15 AUG 1981	16	38	25.4	33.9	81.6		0.9GS	SEU	193
24 AUG 1981	11	50	11.2	36.9	80.7		1.0VPI	SEU	131
9 SEP 1981	19	30	45.3	34.3	81.6	II*	2.1GS	SEU	177
14 SEP 1981	20	43	25.0	34.3	81.3	I*	1.8GS	SEU	163
15 SEP 1981	17	56	43.9	34.4	81.3	I*	1.7GS	SEU	156
3 OCT 1981	9	56	25.4	36.0	79.3		1.1VPI	SEU	31

TABLE 2.5.2-1
EARTHQUAKE LIST (SITE LOCATION 35.7N 79.0W)

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES
12 NOV 1981	6	24	10.0	37.2	80.8		0.7VPI	SEU	145
23 NOV 1981	13	14	51.0	38.2	79.0	II*	2.1VPI	SEU	175
29 NOV 1981	12	3	8.2	34.7	81.6	I*	1.5GIT	SEU	165
29 NOV 1981	20	39	40.0	34.6	81.6	I*	1.5GIT	SEU	164
4 DEC 1981	2	35	56.4	37.0	80.7	II*	2.0VPI	SEU	134

NOTES TO TABLE 2.5.2-1

1. The magnitude listed is chosen in the following order. M_L, CGS M_S, CGS M_B, Other M_S. Blank if no magnitude information.
2. When maximum intensity is not given, an estimated intensity is calculated by using the relation:

   $$M = 1 + 2/3\ I$$

   The calculated intensity is indicated by an asterisk.

3. The following abbreviations are used:

BLA	Blacksburg, Va.
BOL	Bollinger (Reference 2.5.2-29)
BOV	Bollinger and Visvanathan (Reference 2.5.2-30)
CGS	Coast and Geodetic Survey
CHC	Chapel Hill, N. C.
EQH	Earthquake History of the United States, Coffman and Von Hake (Reference 2.5.2-31)
ERL	Environmental Research Laboratories (NOAA)
G-R	Gutenberg and Richter (Reference 2.5.2-32)
GAB	Bollinger (Reference 2.5.2-1)
GIT	Georgia Institute of Technology
GS	U.S. Geological Survey
MB	Body Wave Magnitude
ML	Local Magnitude
MS	Surface Wave Magnitude
NAV	Narrows, Virginia
NOS	National Ocean Survey (NOAA)
PAS	Pasadena, Ca.
SEU	Southeastern U.S. Seismic Network
SLM	St. Louis, Mo.
TEI	Tennessee Earthquake Information Center
USC	University of South Carolina
USE	U.S. Earthquakes, Yearly Publication (NOAA)
VIS	Visvanathan (Reference 2.5.2-33)
VPI	Virginia Polytechnic Institute and State University
WGP	Weston Geophysical

4. Earthquakes listed in this table are those that have occurred within about 200 mi. of the SHNPP site through 1981.

TABLE 2.5.2-2

COMPARISON OF SELECTED RESERVOIRS - DEPTHS AND VOLUMES

Dam or Reservoir	Depth of Reservoir (m)	Volume of Reservoir (m^3)
Hoover Dam, Lake Meade, Nevada	(Approx.) 200	$4,010 \times 10^7$
Marathon Lake, Greece	120	4×10^7
Koyna Dam, India	98	288×10^7
Kariba Dam & Reservoir, Africa	110	$16,000 \times 10^7$
Kremasta Dam, Greece	120	0.475×10^7
Hsin Feng Kiang Dam & Reservoir, China	80	$1,150 \times 10^7$
Keban Dam & Reservoir, Turkey	212	$3,000 \times 10^7$
Shearon Harris Nuclear Power Plant Reservoirs:		
Main	19	8.9×10^7
Auxiliary	13	0.54×10^7
Clark Hill, S. C.	60	250×10^7
Jocassee, S. C.	110	143×10^7
Keowee, S. C.	60	143×10^7
Monticello, S. C.	40	49×10^7

TABLE 2.5.2-3

RESULTS OF SEISMIC REFRACTION SURVEY

Depth (ft.)	Material	Shear Wave Velocity (ft. per sec.)	Compressional Wave Velocity (ft. per sec.)	Poisson's Ratio
0-8	residual soil	500*	1,500	.44
8-16	weathered and fractured rock	2500	5,500	.37
Below 16	sound bedrock	5600	12,000	.35

*This velocity was assumed on the basis of previous experiences under similar conditions.

TABLE 2.5.2-4

Ambient Vibration Measurements

Location	Frequency Hz	Gain x100	Trace in Inches Trans.	Trace in Inches Vert.	Trace in Inches Long.	Ground Motion Velocity, in./sec. x 10^{-3} Trans.	Ground Motion Velocity, in./sec. x 10^{-3} Vert.	Ground Motion Velocity, in./sec. x 10^{-3} Long.
Intersection Trenches No. 3 & No. 4	100, 55-1/2, 25	V20	0.28	0.46	0.22	0.14	0.23	0.11
	100, 55-1/2, 25	V5	0.06	0.10	0.06	0.12	0.20	0.12
Intersection Trenches No. 1 & No. 2	55-1/2, 50, 25	V20	0.42	0.32	0.42	0.21	0.16	0.21
	55-1/2, 50, 25	V5	0.12	0.10	0.10	0.24	0.20	0.20
Boring P-6	100, 55-1/2, 25	V20	0.58	--	--	0.29	--	--
	100, 55-1/2, 25	V5	0.18	0.16	0.22	0.36	0.32	0.44
Boring P-7	100, 55-1/2	V20	0.44	--	--	0.22	--	--
		V5	0.10	0.24	0.10	0.20	0.48	0.20
NW End of Trench No. 1	100, 55-1/2	V20	0.14	0.10	0.06	0.07	0.05	0.03
		V5	--	--	--	--	--	--

Note:
V = velocity
Trans. = transverse
Vert. = vertical
Long. = longitudinal

TABLE 2.5.4-1

RESULTS OF COMPRESSIONAL WAVE VELOCITY AND DENSITY OF ROCK MATERIAL - PLANT SITE

BORING NO.	SAMPLE DEPTH (ft.)	ROCK DESCRIPTION	DENSITY (lbs./ft.^{3})	COMPRESSIONAL WAVE VELOCITY (ft./sec.)
BP-1	31.0	Sandstone	153.8	5,720
BP-5	45.0	Sandy Siltstone	162.8	10,300
BP-8	26.5	Silty Sandstone	163.2	9,080
BP-13	26.9	Silty Conglomeritic Sandstone	161.2	9,580
BP-13	35.1	Silty Sandy Shale	168.1	8,380
BP-17	44.5	Sandy Siltstone	165.1	11,000
BP-21	21.0	Silty Shale	160.3	6,300
BP-24	29.5	Sandstone	140.3	6,340
BP-31	33.0	Sandy Siltstone	162.0	9,580
BP-35	27.5	Fine Grained Sandstone	164.4	8,730
BP-38	41.0	Silty Sandstone	163.0	9,900
BP-46	91.3	Fine Grained Claystone	161.7	8,550
BP-58	29.0	Fine Grained Sandy Claystone	171.7	5,430
BP-58	52.0	Sandy Siltstone	161.8	9,700
BP-59	54.0	Sandstone	163.4	9,500
BP-59	64.0	Medium to Fine Grained Sandstone	165.1	8,550
BP-62	63.0	Silty Sandstone	157.9	9,780
BP-62	76.8	Fine Grained Sandstone	164.7	9,000
BP-63	86.0	Fine Grained Sandstone	164.8	10,000
BP-64	72.5	Sandstone	165.3	8,270
BP-65	82.0	Fine Grained Sandstone	164.3	7,470
BP-66	80.0	Fine Grained Sandstone	164.7	7,100
BP-68	85.5	Fine Grained Sandstone	162.9	9,040
BP-69	78.5	Fine Grained Sandstone	165.2	8,800
BP-70	79.5	Siltstone	164.8	9,700
BP-71	84.0	Med. to Fine Grained Sandstone	161.1	8,500
BP-72	66.0	Sandy Siltstone	165.4	7,370
BP-73	85.0	Sandy Siltstone	164.0	8,100
BP-73	119.5	Silty Claystone	165.1	9,500
BP-74	111.4	Sandy Siltstone	166.9	8,030
BP-75	90.5	Fine Grained Sandstone	159.7	8,850
BP-76	103.0	Fine Grained Sandstone	166.5	9,530
BP-77	34.5	Silty Sandstone	160.2	8,970

Average Value 162.8 lbs./ft.^{3}

TABLE 2.5.4-2 ROCK QUALITY DESIGNATION (RQD) PLANT FOUNDATION AREA

Boring No.	Average RQD (Percent) (Measured Below Elevation 235 ft.)
BP - 1	94.7
BP - 2	98.
BP - 3	92.8
BP - 4	80.1
BP - 5	80.4
BP - 6	96.
BP - 7	93.
BP - 8	89.
BP - 9	86.2
BP - 10	95.3
BP - 11	100.
BP - 12	94.5
BP - 13	89.3
BP - 14	96.3
BP - 15	86.9
BP - 16	83.9
BP - 17	95.4
BP - 18	87.
BP - 19	91.1
BP - 20	87.5
BP - 21	97.2
BP - 22	94.3
BP - 23	90.9
BP - 24	94.1
BP - 25	89.4
BP - 26	97.5
BP - 27	91.
BP - 28	95.
BP - 29	99.1
BP - 30	90.4
BP - 31	97.9
BP - 32	94.6
BP - 33	99.
BP - 34	90.1
BP - 35	100.
BP - 36	95.
BP - 37	75.7
BP - 38	93.7
BP - 39	93.2
BP - 40	94.1
BP - 41	92.9
BP - 42	91.4
BP - 43	97.6
BP - 44	97.2
BP - 45	95.4
BP - 46	95.6
BP - 47	75.9
BP - 48	99.7
BP - 49	93.3
BP - 50	90.
BP - 51	90.6
BP - 52	96.
BP - 53	90.2
BP - 54	96.3
BP - 55	98.9
BP - 56	87.8
BP - 57	91.2
BP - 58	96.6
BP - 59	88.4

TABLE 2.5.4-2 ROCK QUALITY DESIGNATION (RQD) PLANT FOUNDATION AREA

Boring No.	Average RQD (Percent) (Measured Below Elevation 235 ft.)
BP - 60	91.6
BP - 61	92.5
BP - 62	94.2
BP - 63	95.6
BP - 64	92.
BP - 65	92.
BP - 66	95.
BP - 67	96.3
BP - 68	95.6
BP - 69	95.6
BP - 70	97.7
BP - 71	91.9
BP - 72	83.
BP - 73	92.3
BP - 74	94.4
BP - 75	90.7
BP - 76	91.6
BP - 77	98.6

Average Value 92.7 percent

TABLE 2.5.4-3

Young's Modulus (E_t) and Ultimate Stress (q_u)

-Rock Material

Boring No.	Sample Depth (ft.)	Young Modulus (E_t) - psi	Ultimate Stress (q_u) - psi
BP - 35	27.5	1.27×10^6	6,740
BP - 46	91.3	1.36×10^6	4,200
BP - 58	29.0	0.38×10^6	2,980
BP - 62	63.0	$3. \times 10^6$	8,850
BP - 63	86.0	1.96×10^6	12,150
BP - 64	72.5	1.29×10^6	7,735
BP - 66	80.0	1.4×10^6	3,970
BP - 68	85.5	2.4×10^6	3,700
BP - 69	78.5	1.86×10^6	10,700
BP - 70	79.5	2.03×10^6	9,200
BP - 74	111.4	1.33×10^6	9,500
P - 7	117.	2.5×10^6	15,350
P - 7	85.	1.43×10^6	9,800
P - 6	64.	1.4×10^6	9,200
Average Value		1.69×10^6 psi	8,148 psi

TABLE 2.5.4-4

TRIAXIAL COMPRESSION TEST RESULTS

Boring Number	Depth (feet)	Ultimate Compressive Strength (psi)	Confining Pressure (psi)	Strain at Failure	Shear Angle (From Vertical) (degrees)
P-6	64	22,200	4000	.010	40
P-7	85	35,900	6000	.012	40
P-7	117	49,600	6000	.015	20

TABLE 2.5.4-5

Poisson's Ratio (μ) for Rock Material

(Based on Unconfined Compression Test Results)

Boring No.	Sample Depth (ft.)	Rock Type	Poisson's Ratio (μ)
BP - 35	27.5	Sandstone	0.173
BP - 46	91.3	Claystone	0.203
BP - 58	29.0	Claystone	0.291
BP - 62	63.0	Sandstone	0.442
BP - 63	86.0	Sandstone	0.328
BP - 64	72.5	Sandstone	0.125
BP - 66	80.0	Sandstone	0.092
BP - 68	85.5	Sandstone	0.138
BP - 69	78.5	Sandstone	0.25
BP - 70	79.5	Siltstone	0.45
BP - 74	111.4	Siltstone	0.17
Average Value			0.242

Notes:

Poisson's ratio values were determined at a stress level equal to one-half of failure stress.

TABLE 2.5.4-6

SHOCKSCOPE TEST RESULTS

Boring	Depth (ft.)	Velocity of Compressional Wave Propagation (ft/ per second)
P - 7	92	9000
P-6	109 (2 samples)	7400
		7700
P - 6	125	4500
P-6	161 (2 samples)	7400
		7600
P-6	193 (3 samples)	7000
		7000
		8200
P - 6	222	10000

TABLE 2.5.4-7

PIEZOMETRIC LEVEL FLUCTUATIONS AT THE SITE

	BC-106	BD-110*	BX-28	PZ-1**	PZ-2**	RAB	Remarks
Location	Old Intake Channel	East Aux. Dike	East Aux. Dam	Light Tower	3 ft. North of PZ-1		
Ground Elev. above MSL	246.18	256.57	245.07	262.0	262.0	194.0	
Length of Pipe	1'-9"	3'-1"	1'-9"	2'-1"	8"	5"	
Date							
10 Nov. 78	236.7	245.3	230.3				
22 Nov. 78				230.6		191.7	
28 Nov. 78				231.3		193.1	Rained Nov. 27, 1978
1 Dec. 78	234.1	245.3		249.7		193.1	
4 Dec. 78	237.0	245.3					
5 Dec. 78				235.7		192.2	
9 Dec. 78	239.2	245.3					
11 Dec. 78				225.9		193.2	
15 Dec. 78				212.9		191.7	
16 Dec. 78	238.9	245.3					
22 Dec. 78	238.8	245.3			212.5	191.4	
26 Dec. 78	242.9	245.3			213.3	191.5	
27 Dec. 78	242.9	245.3			211.9	192.1	
28 Dec. 78	242.9	245.3			211.9	191.8	
29 Dec. 78	212.9	245.3			211.9	191.3	
5 Jan. 79	244.7	248.1			214.5	191.3	
15 Jan. 79	245.1	253.6			211.0	191.8	
26 Jan. 79					215.7	191.5	
29 Jan. 79					214.6	191.1	
31 Jan. 79					214.0	191.0	
2 Feb. 79					213.5	190.9	
5 Feb. 79					213.2	190.7	
12 Feb. 79					212.7	191.4	

BD-110 was probably dry through December 29, 1978.

** PZ-1 was drilled on November 14, 1978, and gradually filled with silt. PZ-1 was abandoned and PZ-2 was drilled 100 feet deep on December 7, 1978.

TABLE 2.5.4-8

Spring Constants Formulae for Foundations

Note: $G = \frac{E}{2(1+v)}$

Table 10.14. Spring Constants for Rigid Rectangular Footing Resting on Elastic Half-Space

Motion	Spring Constant	Reference
Vertical	$k_z = \frac{G}{1-v} \beta_z \sqrt{4cd}$	Barkan (1962)
Horizontal	$k_x = 4(1 + v)G\beta_x \sqrt{cd}$	Barkan (1962)
Rocking	$k_\psi = \frac{G}{1-v} \beta_\psi 8cd^2$	Gorbunov-Possadov (1961)

Notes:

1.) Values for β_z, β_x, and β_ψ are given on Fig. 10-16 for various values of d/c

2.) Table and Figure are from Richart, Hall and Woods,1970 (Reference 2.5.4-8)

TABLE 2.5.4-9

Average Building Settlements

	Building	Size of Mat ft.	Shape Factor m	Total Load on MAT in 1000 Kips D.L.	D+L.	Area, ft.^{2}	Avg.- Settlement After Completion ft.
1)	Cont. Bldg.	R=74	.96	144.8	158.7	17,200	.018
2)	Fuel Handling Bldg.	534 x 200	.90	464.7	489	72,400	.025
3)	Aux. Bldgs.	207 x 187	.95	180.6	228	30,100	.019
4)	Waste Process Bldg.	349 x 200	.93	336	452	66,000	.02
5)	Turbine Bldg.	340 x 169	.92	186.7	366	57,400	.01
6)	Diesel Generator Bldg.	155 x 123	.945	69	77.4	19,000	.008
7)	Emergency Service Water Intake Structure	184 x 103	.944		186.3	19,000	.013
8)	Tank Bldg.	142 x 74	.925	48	54	10,000	.012
9)	Diesel Fuel Oil Storage Tank	96 x 86	.95	14.6	15.6	8,200	.001
10)	Emergency Service Water Screen Structure	89 x 57	.94	15.1	15.1	5,000	.002(1)
11)	Emergency Service Water Discharge Structure	53 x 29	.93	2.7	2.7	1,500	.001(1)

Notes: (1) These values are rounded off to the nearest whole integer in the third decimal place.

TABLE 2.5.6-1

MAIN DAM - MINIMUM FACTORS OF SAFETY

		Minimum Factor of Safety		
		Static	Pseudo-Static	
Case	Circle No.		OBE .075g H, .05g V	SSE .15g H, 10g V
Normal Operation	9A	1.64	1.44	1.29
Downstream Drawdown*	9A	1.79	1.60	1.45

NOTE: For complete Table of Factors of Safety for all circles investigated, see Figures 2.5.6-19 and 2.5.6-20.

*This case refers to the original seismic analysis of the Main Dam where the postulated failure of the Afterbay Dam resulted in a drawdown on the downstream face of the Main Dam. This condition no longer exists with the final design of the reservoir system. Appendix 2.5D presents the original analysis and the effects of later design changes on the original analysis.

TABLE 2.5.6-2

AUXILIARY DAM - MINIMUM FACTORS OF SAFETY

			Minimum Factor of Safety	
		Static	Pseudo-Static	
Case	Circle		OBE .075g H, .05g V	SSE .15g H, 10g V
Normal Operation	1A	1.70	1.24	1.23
Downstream Drawdown	8A	1.69	1.45	1.50

NOTE: The values given are minimum or near minimum. For the complete Table of Factors of Safety for all circles investigated see Figure 2.5.6-21.

TABLE 2.5.6-3

AUXILIARY SEPARATING DIKE - MINIMUM FACTORS OF SAFETY

Case	Circle	Static	Minimum Factor of Safety Pseudo-Static OBE .075g H, .05g V	SSE .15g H, 10g V
Normal Operation	7A	1.92	1.40	1.42

NOTE: The values given are minimum or near minimum. For the complete Table of Factors of Safety for all circles investigated see Figure 2.5.6-22.

TABLE 2.5B-1

MAIN DAM AND SPILLWAY TESTING PROGRAM

Boring No.	Sample Depth (ft.)	Grain Size Analysis	Compressional Wave Velocity	Unconfined Compression
BM-2	1.5	*		
BM-2	3.0	*		
BM-2	5.0	*		
BM-3	1.0	*		
BM-3	5.0	*		
BM-3	10.0	*		
BM-3	15.0	*		
BM-3	20.0	*		
BM-3	25.0	*		
BM-3	30.0	*		
BM-8	74.5		*	*
BM-9	5.0	*		
BM-9	7.5	*		
BM-9	43.0		*	
BM-10	1.5	*		
BM-10	4.0	*		
BM-10	22.7		*	*
BM-11	37.5		*	
BM-12	82.5		*	
BM-13	57.5		*	
BM-14	1.0	*	*	*
BM-14	3.5	*		
BM-14	8.5	*		
BM-14	52.5			
BM-15	1.0	*		
BM-15	3.5	*		
BM-15	8.5	*		
BM-15	39.5		*	*
BM-16	51.5		*	
BM-34	33.5		*	*
BM-37	76.5		*	
BM-39	83.5		*	*
BM-41	83.5		*	*
BM-44	57.0		*	*
BM-47	59.0		*	
BM-49	64.5		*	
BM-51	25.5		*	

TABLE 2.5B-2

AUXILIARY DAM AND SEPARATING DIKE TESTING PROGRAM

Boring No.	Sample Depth (ft.)	Grain Size Analysis	Compressional Wave Velocity Determined	Unconfined Compression Test
BX-1			*	*
BX-2	34.0		*	*
BX-4	10.0	*		
BX-5	5.0	*		
BX-5	10.0	*		
BX-6	5.0	*		
BX-7	1.0	*		
BX-7	19.6		*	*
BX-9	5.0	*		
BX-9	10.0	*		
BX-9	17.0		*	
BX-9	30.0		*	
BX-10	13.0		*	
BX-11	32.7		*	*
BX-12	10.0	*		
BX-13	3.5	*		
BX-14	1.0	*		
BX-14	3.5	*		
BX-14	17.0		*	
BX-14	27.3		*	
BX-15	3.5	*		
BX-15	4.0		*	
BX-15	18.0		*	*
BX-16	1.0	*		
BX-16	3.5	*		
BD-18	5.0	*		
BD-19	3.5	*		
BD-19	6.0	*		

TABLE 2.5B-3

RESULTS OF COMPRESSIONAL WAVE VELOCITY AND DENSITY TESTS
MAIN DAM AND SPILLWAY

Boring No.	Sample Depth (ft.)	Rock Description	Density (lbs./ft^3)	Compressional Wave Velocity (ft./sec.)
BM-8	74.5	Hornblende - Mica Gneiss	183.8	20,100
BM-9	43.0	Hornblende - Mica Gneiss	184.6	18,600
BM-10	22.7	Hornblende - Mica Gneiss	183.4	17,500
BM-11	37.5	Hornblende - Mica Gneiss	187.7	20,600
BM-12	82.5	Granitic Gneiss	187.3	17,600
BM-13	57.5	Granite	168.8	14,800
BM-14	52.5	Mica Gneiss	184.9	20,500
BM-15	39.5	Granitic Gneiss	169.1	15,300
BM-16	51.5	Mica Gneiss	176.8	11,300
BM-34	33.5	Weathered Granitic Gneiss	179.9	8,900
BM-37	76.5	Granitic Gneiss	167.5	17,750
BM-39	83.5	Micaceous Granitic Gneiss	166.2	14,200
BM-41	83.5	Hornblende - Mica Gneiss	184.4	20,550
BM-44	57.0	Mica Gneiss	185.5	16,050
BM-47	59.0	Hornblende - Mica Gneiss	181.5	19,350
BM-49	64.5	Hornblende - Mica Gneiss	182.0	20,200
BM-51	25.5	Granitic Gneiss	168.8	18,150

TABLE 2.5B-4

RESULTS OF COMPRESSIONAL WAVE VELOCITY AND DENSITY TESTS
AUXILIARY DAM SITE

Boring No.	Sample Depth (ft.)	Rock Description	Density (lbs./ft^3)	Compressional Wave Velocity (ft./sec.)
BX-1		Fine Grained Sandstone	165.3	4,530
BX-2	34.0	Fine Grained Sandstone	162.7	10,100
BX-7	19.6	Coarse Grained Sandstone	156.7	5,970
BX-9	17.0	Fine to Coarse Sandstone	154.3	7,300
BX-9	30.0	Siltstone	164.3	7,700
BX-10	13.0	Silty Fine Sandstone	161.9	5,800
BX-11	32.7	Siltstone	177.6	7,520
BX-14	17.0	Fine to Coarse Sandstone	150.7	2,800
BX-14	27.3	Siltstone	162.8	9,020
BX-15	4.0	Fine to Coarse Sandstone	151.0	3,500
BX-15	18.0	Sandstone	156.0	9,980

TABLE 2.5C.2-1

MATERIAL M - RESULTS OF ISOTROPICALLY-CONSOLIDATED DRAINED (CID) TRIAXIAL TESTS

Parameter	Magnitude
Friction angle φ, degrees	30
Cohesion C, lb./ft.^{2}	200
Modulus No. K	160
Modulus exponent, n	0.7
Failure ratio, R_f	0.76
Poisson's ratio parameters:	
G	0.2
F	0.14
D	8.6

Degree of compaction - 100% standard

TABLE 2.5c2-2 – MATERIAL M – RESULTS OF CYCLIC TRIAXIAL STRESS CONTROL TESTS

Degree of Compaction	K_c	Confining Pressure $\sigma 3$ lb./ft.[2]	Cyclic Stress Ratio $\pm \frac{\overline{Vd}}{2\sigma_3}$	No. of Cycles Causing 5% Strain	No. of Cycles Causing 10% Strain
95%	1	4000	0.373	2.9	
			0.363		7.1
			0.263	11.4	
			0.262		26
			0.203	59	113
			0.369	5.1	
			0.329		14.2
			0.212	65	
			0.207		111
			0.277	19	
			0.276		35
		8000	0.301	4.7	
			0.298		9.5
			0.226	26.3	56
			0.348	9.5	
			0.355		20.8
			0.302	6	
			0.288		14
100%	1	2000	0.454	53	160
			0.494	45	150
			0.412	155	410
		4000	0.390	11	32
			0.445	6.9	
			0.436		25
			0.319	35	
			0.313		200
		8000	0.404	5.5	
			0.403		11
100%	1	8000	0.292	39.5	
			0.291		76
			0.451	2.5	
			0.433		7
		12000	0.278	15.1	
			0.275		27.5
			0.409	3.1	
			0.398		7
			0.322	5.7	
			0.310		13.5
			0.240	40	
			0.235		78
100%	1.5	2000	0.573	4.2	
			0.581		13
			0.467	8	
			0.468		28
			0.414	78	>1000
		4000	0.485	4.1	
			0.487		13
			0.453	34	
			0.457		69
			0.557	1.4	
			0.567		4.8
			0.413	14.1	
			0.414		44
		8000	0.347	4.6	

TABLE 2.5c2-2 – MATERIAL M – RESULTS OF CYCLIC TRIAXIAL STRESS CONTROL TESTS

Degree of Compaction	K_c	Confining Pressure $\sigma 3$ lb./ft.^{2}	Cyclic Stress Ratio $\pm \frac{\overline{Vd}}{2\sigma_3}$	No. of Cycles Causing 5% Strain	No. of Cycles Causing 10% Strain
			0.352		17
			0.395	2	
			0.399		7
	1	8000	0.309	16	
			0.313		100
100%	2.0	2000	0.656	11.1	
			0.675		79
			0.507	87	
			0.629	15	
			0.638		159
			0.682	6.2	
			0.677		42
		8000	0.423	1.6	
			0.440		8
			0.519	<1	
			0.527		2.5
			0.366	4.6	
			0.374		37
			0.330	15	
			0.335		700

TABLE 2.5C.2-3

MATERIAL M - RESULTS OF STRAIN-CONTROLLED CYCLIC TRIAXIAL TESTS

Test No.	Confining Pressure σ_3, lb./ft.^{2}	Platten to Platten Axial Strain ε, (%)	Deviator Stress σ_d, lb./ft.^{2}	Apparent Young's Modulus E, k/ft.^{2}	Damping λ, (%)
1	2000	0.0206	477	2315	22.3
2	2000	0.0800	1040	1300	21.7
3	8000	0.0210	1264	6025	16.9
4	8000	0.0807	2630	3256	17.9

Degree of compaction - 100% standard

TABLE 2.5C.2-4

MATERIAL M - RESULTS OF CYCLIC TORSION TESTS

Degree of Compaction	Confining Pressure σ_3 lb./ft.^{2}	Average Shear Strain %	Damping %	Shear Modulus G, K/in.^{2}
100% Standard	2000	4.8×10^{-3}	8.1	27.5
	4000	3.7×10^{-3}	7.0	45.1
	1000	5.2×10^{-3}	8.0	27.7
	2000	4.6×10^{-3}	7.4	35.8
	4000	3.9×10^{-3}	7.0	52.0
	500	9.5×10^{-3}	12.0	11.0
	1000	7.5×10^{-3}	9.6	17.3
	2000	6.8×10^{-3}	9.2	20.8
	500	8×10^{-3}	10.8	13.4
	1000	6×10^{-3}	9.7	18.5
	2000	3.8×10^{-3}	10.0	30.6
	4000	3.4×10^{-3}	8.3	48.2

TABLE 2.5C.2-5

MATERIAL Z - RESULTS OF ISOTROPICALLY-CONSOLIDATED DRAINED (CID) TRIAXIAL TESTS

Parameter	Magnitude
Friction angle φ, degrees	30
Cohesion C, lb./ft.^{2}	100
Modulus No. K	160
Modulus exponent, n	0.3
Failure ratio, R_f	0.80
Poisson's ratio parameters:	
G	0.21
F	0.084
D	6.0

Degree of compaction - 97% standard

TABLE 2.5C.2-6

MATERIAL Z - RESULTS OF CYCLIC TRIAXIAL STRESS CONTROL TESTS

Degree of Compaction	K_c	Confining Pressure σ_3 lb./ft.^{2}	Cyclic Stress Ratio ± σ_d $2\sigma_3$	No of Cycles Causing 5% Strain	10% Strain
95%	1	2500	0.375	9.7	33
			0.280	43	130
			0.445	6.3	23
97%	1	1250	0.439	80	210
			0.449	24	70
			0.512	23	80
		2500	0.527	3.9	
			0.494		9.3
			0.418	13.3	
			0.417		34
			0.295	122	
		5000	0.370	4.8	
			0.365		20
			0.293	20.5	68
			0.421	1.8	
			0.416		6
	1.5	1250	0.550	4	
			0.558		10.6
			0.471	8.5	
			0.486		30
			0.425	15	
			0.427		57
		2500	0.386	6	
			0.392		26
			0.492	3.7	9.6
		5000	0.215	89	>1000
			0.373	3.2	
			0.367		6.9
			0.272	20	
			0.271		58
97%	2.0	1250	0.574	12.5	
			0.578		31
			0.762	6.1	
			0.780		9.6
			0.601	20	
			0.610		56
		2500	0.309	13.6	
			0.310		47
			0.244	50	
			0.245		390
			0.485	1.8	
			0.491		4.5
100%	1.0	2500	0.390	58	190
			0.491	19	83
			0.321	100	370

TABLE 2.5C.2-7

MATERIAL Z - RESULTS OF STRAIN-CONTROLLED CYCLIC TRIAXIAL TESTS

Test No.	Confining Pressure σ_3, lb./ft.^{2}	Platten to Platten Axial Strain ε, (%)	Deviator Stress σ_d, lb./ft.^{2}	Apparent Young's Modulus E, k/ft.^{2}	Damping λ, (%)
S-9	1156	0.0198	380	1915	14.1
S-16	1160	0.0769	691	899	20.1
S-19	3945	0.0838	1400	1670	18.3
S-20	4000	0.0192	751	3915	13.1

Degree of compaction - 97% standard

TABLE 2.5C.2-8

MATERIAL Z - RESULTS OF CYCLIC TORSION TESTS

Degree of Compaction	Confining Pressure σ_3 lb./ft.^{2}	Average Shear Strain %	Damping %	Shear Modulus G, K/in.^{2}
97% Standard	500	2.8×10^{-2}	7.0	4.5
	1000	1.9×10^{-2}	5.9	9.6
	2000	1.7×10^{-2}	6.2	11.6
	1000	1.5×10^{-2}	5.1	13.4
	2000	1.3×10^{-2}	4.4	19.4
	4000	1×10^{-2}	3.9	30.3

TABLE 2.5C.2-9

AUXILIARY DAM AND DIKE

UNDISTURBED FOUNDATION SOILS - RESULTS OF CYCLIC TRIAXIAL (STRESS-CONTROL) TESTS

Degree Compaction	K_c	Confining Pressure σ_3	Stress Ratio ± $\sigma_d \; 2\sigma_3 c$	No. of Cycles Causing 5% Strain	No. of Cycles Causing 10% Strain	Remarks
Undisturbed sample TPA 1 S-1A at depth 2.5 ft.	1	500	0.449			0.1% strain at 1000 cycles
Undisturbed sample TPA 1 at depth 3.1 ft.	1	1500	0.476			0.6% strain at 1000 cycles
Undisturbed sample TPA 2	1	1450	0.457	700*		*Extrapolated
Undisturbed Sample TPA 1	1.5	3000	0.712	94	225	
Undisturbed Sample TPA 1	1.7	500				>0.1% strain at 1000 cycles
Undisturbed Sample TPA 2	1	3000	0.455			1.9% strain at 1000 cycles
Undisturbed Sample TPA 1	1	500	0.440			<0.1% strain at 1000 cycles
Undisturbed Sample TPA 1	1	1500	0.425			0.8% strain at 1000 cycles
Undisturbed Sample TPA 1	1	3000	0.475			1.1% strain at 1000 cycles
Undisturbed Sample TPA 2	1.5	500	0.480			0.15% strain at 1000 cycles
Undisturbed Sample TPA 2	1.5	3000	0.661	310	556	

TABLE 2.5C.2-10

AUXILIARY DAM, SUMMARY OF MEASUREMENTS IN RESIDUAL SOIL

Impact-Receiver Spacing, ft.	Time, msec.* P Wave	Time, msec.* S Wave	Velocity, ft./sec. V_P	Velocity, ft./sec. V_S
15	11	21-5	1360	715
"	11	19.5	1360	770
"	10	20	1500	750
"	10	22	1500	680
"	11	20	1360	750
"	9-9.5	19	1670 - 1580	790
"	10	20	1500	750
"	9	20	1670	750
"	9	19	1670	790
			(1505 avg.)	(750 avg.)
20	14	30	1430	666
"	15	30	1330	666
"	13	28 - 29	1540	715 - 690
"	-	-	-	-
"	14	27	1430	740
"	13	27 - 28	1540	740 - 715
"	14	28	1430	715
"	16	28	1250	715
			(1420 avg.)	(705 avg.)

*msec. - millisecond
Note: Measurements made in 2-ft. - deep trench into clayey silt, approximately 100 ft. east of TPA1.

Geophone Spacing, ft.	Frequency, hertz	Phase* Difference	Wave Length, ft.	V_R (ft./sec.)
10.2	30	-	No stability on channel I below 50 hz below 50 hz	
"	60 ± 5	0	10.20	610
"	120 ± 5	0	5.10	610
"	185 ± 5	0	3.40	630
"	250 ± 10	0	2.55	640
"	305 ± 5	0	2.04	620
"	360	-		(620 avg.)
6.1	No stability on channel I below 50 hz and on channel II below 30 hz			
"	50 ± 5	π	12.20	610
"	100 ± 5	0	6.10	610
"	150 ± 5	π	4.07	610
"	200 ± 5	0	3.05	610
"	245 ± 5	π	2.44	600
"	295 ± 5	0	2.03	600
				(605 avg.)

*0 = in-phase response
π = 180°out-of-phase response
Note: Measurements made approximately 100 ft. east of TPA1

TABLE 2.5C.2-11

AUXILIARY DAM, SUMMARY OF MEASUREMENTS IN TRANSITIONAL MATERIAL

Impact-Receiver Spacing, ft.	Time, msec.*		Velocity, ft./sec	
	P Wave	S Wave	V_P	V_S
10	5	10	2000	1000
"	5	10	2000	1000
"	5.5	10	1820	1000
"	6	10	1670	1000
"	5.5	10	1820	1000
"	5-5.5	9.5	2000 - 1820	1050
"	5	9.5	2000	1050
"	6	9.5	1670	1050
"	6	11.5	1670	870
			(1804 avg.)	(1000 avg.)
20	8	14.5	2500	1380
"	8.5	17	2360	1180
"	8.5	14.5	2360	1380
"	8.5	16	2360	1250
"	8	14	2500	1430
"	8	15	2500	1330
"	8.5	14.5	2360	1380
"	8.5	14	2360	1430
"	8	14	2500	1430
"	9	16	2220	1250
"	9	16	2220	1250
			(2385 avg.)	(1335 avg.)
22	7	-	3140	-
"	7	-	3140	-
"	7	-	3140	-
"	7	-16	3140	~1380
"	8	-	2750	-
			(3060 avg.)	
12.7	5	12	2540	1060
"	5-6	12	2540 - 2110	1060
"	5	13	2540	980
"	6	11	2110	1150
"	5	12	2540	1060
"	5	11	2540	1150
"	5	12	2540	1060
"	5.5	12.5	2300	1020
"	6	12	2110	1060
			(2395 avg.)	(1065 avg.)

*msec. = millisecond

Notes:
(1) Measurements made in 3-ft .deep trench into approximately 30 ft. north of TPA2
(2) Measurements made in 8 ft. deep trench 50 ft. east of TPA2 in highly weathered, medium hard to moderately soft, red brown clayey sandy siltstone.

TABLE 2.5C.2-12

AUXILIARY DAM, SUMMARY OF MEASUREMENTS IN WEATHERED ROCK

	Time, msec*		Velocity, ft./sec	
Spacing, ft.	P Wave	S Wave	V_P	V_S
26	8	18	3250	1445
"	8	18	3250	1445
12	5	9	2400	1335
"	5	9	2400	1335
"	5	9	2400	1335

Geophone Spacing, ft.	Frequency, hertz	Phase* Difference	Wave Length, ft.	V_R (ft./sec.)
16.7	40	π	33.40	1340
11.2	55	π	22.40	1230
11.2	120	0	11.20	1340
4.5	150 ± 10	π	9.00	1350 (1315 avg.)
4.5	350 ± 10	0	4.50	1570

*msec. = millisecond

**0 = in-phase response

π = 180° out of phase response

Note: Measurements made on near surface, weathered, medium hard, buff to red brown fine to medium grained sandstone layer approximately 100 ft. east of TPA2. Sandstone layer is approximately 3 ft. thick and is underlain by clayey silt.

TABLE 2.5C.2-13

AUXILIARY DIKE, SUMMARY OF MEASUREMENTS

Impact-Receiver Spacing, ft.	Time, msec*		Velocity, ft./sec	
	P Wave	S Wave	VP	VS
		Residual Soil		
25	-	35	-	715
"	-	35	-	715
"	18	35	1390	715
"	20	35	1250	715
"	20	35	1250	715
			(1300 avg.)	(715 avg.)
		Weathered Rock		
20	5	9-9.5	4000	2220 - 2100
"	5.5	9	3640	2220
"	5.5-6	10	3330 - 3640	2000
"	5	8.5	4000	2350
"	6	10	3330	2000
"	6	11	3330	1820
"	5	8.5	4000	2350
"	6	13	3330	1540
			(3640 avg.)	(2055 avg.)

TABLE 2.5C.2-14

AUXILIARY DAM AND AUXILIARY DIKE,
K_2,max BASED ON SHEAR WAVE VELOCITIES

Auxiliary Dam	d, ft.	$d^{1/2}$	VS (ft./sec.)	V_S^2	K_2, max
Residual Soil					$K_2,max = 4.22\ 10^{-4}\ V_S^2/d^{1/2}$
	7.5	2.74	750	$56.2\ 10^4$	86
	10	3.16	705	49.6 "	66
	5.1	2.26	680	46.4 "	87
	3	1.73	665	44.3 "	108
Transitional Material					$K_2,max = 4.22\ 10^{-4}\ V_S^2/d^{1/2}$
	5	2.24	1000	100 "	194
	10	3.16	1335	179 "	245
	11	3.32	1380	191 "	250
	6.3	2.51	1065	114 "	198
Weathered Rock					$K_2,max = 4.44\ 10^{-4}\ V_S^2/d^{1/2}$
	3	1.73	1335	179 "	460
	3	1.73	1445	209 "	540
	3	1.73	1445	209 "	540
	2.3	1.51	1725	297 "	875
Residual Soil					$K_2,max = 4.22\ 10^{-4}\ V_S^2/d^{1/2}$
	12.5	3.54	715	51 "	61
Weathered Rock					$K_2,max = 4.44\ 10^{-4}\ V_S^2/d^{1/2}$
	10	3.16	2055	420 "	591

TABLE 2.5C.2-15

MAIN DAM, EXPECTED CONSTRUCTED MATERIAL PROPERTIES FOR DYNAMIC ANALYSIS

Material	Unit Weight, γ, lb./ft.^{3} Moist	Saturated	$\overline{K_o}$*	Poisson's Ratio μ	Shear Modulus Parameter K_2, max **
Core*** Material M	137	142	0.7	0.35	120
Inner Filler	130	135	0.6	0.35	60
Outer Filler	135	140	0.6	0.35	120
Rockfill Shell	130	145	0.6	0.30	250
Weathered Rock	150	150	0.6	0.35	700

* $\overline{K_o}$ = ratio of horizontal effective stress to vertical effective stress

** G_{max} = K_2, max $\bar{\sigma}_o^{1/2}$

$\bar{\sigma}_o$ = Mean normal effective stress in lb./ft.^{2}

G_{max} = Shear modulus in k/ft.^{2}

*** 100% standard compaction at optimum water content

TABLE 2.5C.2-16

AUXILIARY DAM AND AUXILIARY RESERVOIR SEPARATING DIKE, EXPECTED CONSTRUCTED MATERIAL PROPERTIES FOR DYNAMIC ANALYSIS

	Unit Weight, γ, lb./ft.^{3}				
Material	Moist	Saturated	$\overline{K_o}$*	Poisson's Ratio μ	Shear Modulus Parameter K_2, max **
Core*** Material Z	128	135	0.7	0.35	100
Filter	135	140	0.6	0.35	90
Random Rockfill	130	140	0.6	0.30	150
In-Situ Soil	135	135	0.6	0.30	100+ 190++ 90+++
Weathered Rock	150	150	0.6	0.35	700

* $\overline{K_o}$ = ratio of horizontal effective stress to vertical effective stress

** G_{max} = K_2, max $\bar{\sigma}_o^{1/2}$

$\bar{\sigma}_o$ = Mean normal effective stress in lb./ft.^{2}

G_{max} = Shear modulus in k/ft.^{2}

*** 97% standard compaction at optimum water content

\+ at west section of Auxiliary Dam

++ at east section of Auxiliary Dam

+++ at west abutment of Auxiliary Reservoir Separating Dike

TABLE 2.5C.2-17

MATERIAL M, SHEAR MODULUS AND DAMPING RATIO
BASED ON CYCLIC TORSION TESTS

Test No.	Initial Effective Confining Pressure σ_{3c}, k/ft.^{2}	Average Shear Strain* γ in 10^{-6}	Damping Ratio λ	Shear Modulus G, k/ft.^{2}	G/G_{max}**	Maximum Shear Modulus G_{max}, k/ft.^{2}
M 1.1	2	48	0.081	3960	0.86	4605
M 1.2	4	37	0.070	6494	0.89	7295
M 1.3	8	31	0.061	7805	0.92	8485
M 2.1	1	52	0.080	3989	0.84	4750
M 2.2	2	46	0.074	5155	0.86	5995
M 2.3	4	39	0.070	7488	0.89	8415
M 3.1	0.5	95	0.120	1584	0.62	2555
M 3.2	1	75	0.096	2491	0.72	3460
M 3.3	2	68	0.092	2995	0.76	3940
M 6.1	0.5	80	0.108	1930	0.70	2755
GT 1.1	1	60	0.097	2664	0.92	2660
GT 1.2	2	38	0.100	4406	0.955	4615
GT 1.3	4	34	0.083	6941	0.96	7230
LM 1.1	0.5	46	0.113	1634	0.86	1900
LM 1.2	1	43	0.086	2850	0.88	3240
LM 1.3	2	45	0.081	3670	0.87	4220
LM 2.1	1	32	0.079	3660	0.91	4020
LM 2.2	2	30	0.077	4100	0.92	4455
LM 2.3	4	25	0.069	6050	0.94	6440

* Average shear strain is 0.7 measured maximum shear strain

** From page 2.5C.2-31

Note: Reconstituted specimens of material M compacted at 100% standard compaction at optimum water content

TABLE 2.5C.2-18

MATERIAL M, SHEAR MODULUS AND DAMPING RATIO
BASED ON STRAIN-CONTROLLED CYCLIC TRIAXIAL TEST RESULTS

Test No.	Initial Effective Confining Pressure σ_{3c} k/ft.^{2}	Measured Axial Strain* ε in 10^{-3}	Corrected Axial Strain* ε_c in 10^{-3}	Axial Deviator Stress σ_d k/ft.^{2}	Young's Modulus E** k/ft.^{2}	Shear Modulus G*** k/ft.^{2}	$\frac{G}{G_{max}}$****	Maximum Shear Modulus G_{max} k/ft.^{2}	Damping Ratio λ	Shear Strain***** γ in 10^{-3}
1	2	0.206	0.137	0.477	3482	1290	0.35	3686	0.223	0.185
2	2	0.800	0.533	1.040	1951	723	0.16	4520	0.217	0.72
3	8	0.210	0.140	1.264	9029	3344	0.35	9555	0.169	0.19
4	8	0.807	0.538	2.630	4889	1811	0.16	11320	0.179	0.73

* ε_c = $\varepsilon/1.5$, account for non-uniform strain of test specimen at strains smaller than 10^{-3}

** E = σ_d/ε_c

*** G = $E/2(1+\mu)$; $\mu = 0.35$ (selected)

**** From page 2.5C.2-31

***** γ = $\varepsilon_c\,(1+\mu)$

Note: Reconstituted specimens of material M compacted at 100% standard compaction at optimum water content

TABLE 2.5C.2-19

MATERIAL M, TYPICAL SHEAR MODULUS AND DAMPING RATIO BASED ON STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS

Test No.	Initial Effective Confining Pressure σ_{3c} k/ft.^{2}	Axial Strain* ε in 10^{-3}	Axial Deviator Stress σ_d k/ft.^{2}	Young's Modulus E** k/ft.^{2}	Shear Modulus G*** k/ft.^{2}	Damping Ratio λ	Shear Strain***** γ in 10^{-3}
1	2	18.7	1.81	96.5	35.8	0.253	25.3
2A	4	24.6	3.6	146.3	54.2	0.273	33.2
3A	8	20.0	3.14	158.9	58.9	0.260	26.7
4A	4	16.7	2.65	157.9	58.5	0.259	22.6
4A	4	35.0	2.55	72.9	27.0	0.267	47.3

* $E = \sigma_d/\varepsilon$

** $G = E/2(1+\mu)$; $\mu = 0.35$ (selected)

*** $\gamma = \varepsilon(1+\mu)$

Notes:

1. Reconstituted specimens of material M compacted at 100% standard compaction at optimum water content.
2. This table presents the results of five typical tests among thirty one tests which were conducted to determine the cyclic strength characteristics.

TABLE 2.5C.2-20

MATERIAL Z, SHEAR MODULUS AND DAMPING RATIO BASED ON CYCLIC TORSION TESTS

Test No.	Initial Effective Confining Pressure σ_{3c}, k/ft.^{2}	Average Shear Strain* in 10^{-5}	Damping Ratio λ	Shear Modulus G, k/ft.^{2}	G/G_{max}**	Maximum Shear Modulus G_{max}, k/ft.^{2}
Z1.1	0.5	28	0.07	648	0.28	2315
Z1.2	1	19	0.059	1382	0.40	3455
Z1.3	2	17	0.062	1670	0.45	3710
Z2.1	1	15	0.051	1930	0.54	3575
Z2.2	2	13	0.044	2794	0.60	4655
Z2.3	4	10	0.039	4363	0.71	6145
Z3.1	2	10	0.039	4349	0.71	6125
Z3.2	4	10	0.037	5904	0.71	8315

* Average shear strain is 0.7 maximum shear strain which is measured

** From page 2.5C.2-36

Note: Reconstituted specimens of material Z compacted at 97% standard compaction at optimum water content

TABLE 2.5C.2-21

MATERIAL Z, SHEAR MODULUS AND DAMPING RATIO
BASED ON STRAIN-CONTROLLED CYCLIC TRIAXIAL TEST RESULTS

Test No.	Initial Effective Confining Pressure σ_{3c} k/ft.^{2}	Measured Axial Strain* ε in 10^{-3}	Corrected Axial Strain* ε_c in 10^{-3}	Axial Deviator Stress σ_d k/ft.^{2}	Young's Modulus E** k/ft.^{2}	Shear Modulus G*** k/ft.^{2}	$\frac{G}{G_{max}}$****	Maximum Shear Modulus G_{max} k/ft.^{2}	Damping Ratio λ	Shear Strain***** γ in 10^{-3}
S-9	1.156	0.198	0.132	0.380	2979	1066	0.41	2600	0.141	1.8
S-16	1.16	0.769	0.513	0.691	1347	499	0.16	3120	0.201	6.9
S-19	3.94	0.838	0.559	1.400	2500	926	0.15	6175	0.183	7.6
S-20	4.0	0.192	0.128	0.751	5867	2173	0.42	5175	0.132	1.75

* ε_c = $\varepsilon/1.5$, account for non-uniform strain of test specimen at strains smaller than 10^{-3}

** E = σ_d/ε_c

*** G = $E/2(1+\mu)$; $\mu = 0.35$ (selected)

**** From page 2.5C.2-36

***** γ = $\varepsilon_c\,(1+\mu)$

Note: Reconstituted specimens of material Z compacted at 97% standard compaction at optimum water content

TABLE 2.5C.2-22

MATERIAL Z, TYPICAL SHEAR MODULUS AND DAMPING RATIO BASED ON STRESS-CONTROLLED CYCLIC TRIAXIAL TEST

Test No.	Initial Effective Confining Pressure σ_{3c} k/ft.^{2}	Axial Strain* ε in 10^{-3}	Axial Deviator Stress σ_d k/ft.^{2}	Young's Modulus E** k/ft.^{2}	Shear Modulus G*** k/ft.^{2}	Damping Ratio λ	Shear Strain***** γ in 10^{-3}
4	5	24.0	3.715	154.8	57.3	0.260	32.4
7	2.520	19.5	2.130	109.2	40.5	0.244	26.3
8	2.475	13.8	1.473	106.7	39.5	0.244	18.6

* $E = \sigma_d/\varepsilon$

** $G = E/2(1+\mu)$; $\mu = 0.35$ (selected)

*** $\gamma = \varepsilon(1+\mu)$

Notes:

1. Reconstituted specimens of material Z compacted at 97% standard compaction at optimum water content.
2. This table presents the results of three typical tests among twenty-four tests which were conducted to determine the cyclic strength characteristics.

TABLE 2.5C.2-23

MAIN DAM, MATERIAL PROPERTY COMBINATIONS
FOR PARAMETRIC STUDIES

Set Identification	Core (Material M)		Fine Filter		Coarse Filter		Rockfill		Weathered Rock	
	K_2, max	Damping	K_2 , max	Damping	K_2, max	Damping	K_2, max	Damping	K_2, max	Damping
Basic Set	120	a	60	c	120	c	180	c	700	c
A	120	a	60	c	120	c	250	c	700	c
B	140	b	60	c	120	c	250	c	700	c
C	120	a	60	c	120	c	150	c	700	c
BC	140	b	60	c	120	c	150	c	700	c
D	120	b	60	d	120	d	250	d	700	d
E	90	a	50	c	90	c	150	c	700	c
AV	120	a	60	c	120	c	250	c	700	c

Notes:

1. All computation sets for main dam, maximum cross section, 105 ft. high, set IV (M-105-IV)
2. AV is set A with horizontal and vertical accelerations acting simultaneously.

a Damping ratio from page 2.5C.2-33

b Damping ratio from page 2.5C.2-33 x 0.90

c Damping ratio from page 2.5C.2-34

d Damping ratio from page 2.5C.2-34 x 0.80

TABLE 2.5C.2-24

AUXILIARY DAM, MATERIAL PROPERTY COMBINATIONS FOR PARAMETRIC STUDIES - 1

Set Identification	Core (Material Z)		Filter		Random Rockfill	
	K_2, max	Damping	K_2, max	Damping	K_2, max	Damping
Set	100	a	90	c	90	c
A	100	a	90	c	150	c
B	125	b	90	c	90	c
C	125	b	120	d	150	d
AV	100	a	90	c	150	c

Notes:

1. All computation sets for auxiliary dam, maximum cross section, 63 ft. high; set IV (A-63-IV)
2. AV is set A with horizontal & vertical accelerations acting simultaneously.

a Damping ratio from page 2.5C.2-37

b Damping ratio from page 2.5C.2-37 x 0.90

c Damping ratio from page 2.5C.2-34

d Damping ratio from page 2.5C.2-34 x 0.80

TABLE 2.5C.2-25

AUXILIARY DAM, MATERIAL PROPERTY COMBINATIONS
FOR PARAMETRIC STUDIES - 2

Set Identification	Core (Material Z)		Filter		Random Rockfill		In-Situ Soil		Weathered Rock	
	K_2, max	Damping	K_2, max	Damping	K_2, max	Damping	K_2, max	Damping	K_2, max	Damping
Basic Set	100	a	90	c	90	c	100	c	700	c
A	100	a	90	c	150	c	100	c	700	c
B	100	a	90	c	150	c	60	c	700	c
C	100	a	90	c	150	c	60	c	250	c

Note: All computation sets for auxiliary dam, cross section, A-44, 44 ft. high.

a Damping ratio from page 2.5C.2-37

c Damping ratio from page 2.5C.2-34

TABLE 2.5C.2-26

AUXILIARY DIKE, MATERIAL PROPERTY COMBINATIONS
FOR PARAMETRIC STUDIES

Set	Core (Material Z)		In-Situ Soil		Random Rockfill	
Identification	K_2, max	Damping	K_2, max	Damping	K_2, max	Damping
Basic Set	100	a	90	c	90	c
A	100	a	90	c	150	c
B	125	b	90	c	90	c
C	125	b	125	d	150	d

Notes: All computation sets for auxiliary dike, maximum cross section, 53 ft. high; set IV (A-53-IV)

a Damping ratio from page 2.5C.2-37

b Damping ratio from page 2.5C.2-37 x 0.90

c Damping ratio from page 2.5C.2-34

d Damping ratio from page 2.5C.2-34 x 0.80

TABLE 2.5C.3-1

BORROW AREA Y
SHRINKAGE FACTORS

Boring	Sample Number	Sample Depth	Shrinkage Factor	Shrinkage Ratio	Test Initial Moisture Content (%)
BB 101	S-1	0' - 5'	19.1	1.70	34.3
BB 101	S-2	5' - 10'	20.2	1.68	32.8
BB 101	S-6	25' - 30'	12.5	1.81	28.1
BB 103	S-1	0.33' - 2.5'	19.7	1.66	48.1
BB 103	S-2	2.5' - 7'	19.6	1.67	32.6
BB 103	S-4	12' - 15'	19.6	1.68	30.0
BB 105	S-1	0.5' - 5'	16.6	1.75	37.0
BB 105	S-2	5' - 10'	19.3	1.71	34.8
BB 107	S-1	0.5' - 5'	20.2	1.64	37.9
BB 107	S-2	5' - 10'	18.1	1.71	35.7
BB 109	S-1	0.5' - 5'	17.5	1.73	32.2
BB 109	S-4	12' - 17'	16.0	1.80	38.9
BB 110	S-1	0.5' - 2.5'	19.5	1.65	41.7
BB 110	S-2	2.5' - 7'	19.8	1.69	41.0
BB 111	S-1	0.5' - 5'	19.8	1.66	59.6
BB 111	S-2	5' - 10'	16.2	1.87	46.2
BB 113	S-1	4' - 5'	20.7	1.63	51.3
BB 113	S-2	5' - 10'	18.5	1.71	29.1
BB 113	S-3	10' - 14'	21.4	1.68	32.6
BB 114	S-1	0.5' - 5'	18.5	1.70	45.5
BB 114	S-2	5' - 10'	21.7	1.61	43.2
BB 116	S-1	0.5' - 3'	22.7	1.59	53.2
BB 116	S-2	3' - 8'	22.9	1.64	34.9
BB 116	S-4	13' - 18'	21.0	1.63	36.8
BB 117	S-1	0.5' - 4'	19.8	1.63	35.1
BB 117	S-2	4' - 9'	20.3	1.67	33.7
BB 117	S-4	14' - 18'	16.4	1.77	24.4
BB 119	S-1	0.5' - 3'	13.3	1.89	31.1
BB 119	S-2	3' - 8'	18.2	1.73	31.8
BB 121	S-1	0.5' - 5'	15.3	1.83	37.4
BB 121	S-2	5' - 10'	21.6	1.66	35.9
BB 121	S-4	15' - 20'	21.3	1.67	37.9
BB 124	S-2	6' - 10'	17.2	1.74	28.9
BB 124	S-4	15' - 20'	17.3	1.78	27.8
BB 125	S-1	0.5' - 4'	16.4	1.77	32.3
BB 125	S-2	4' - 9'	17.1	1.71	32.6

TABLE 2.5C.3-2

BORROW AREA Z
SHRINKAGE FACTORS

Boring	Sample Number	Sample Depth	Shrinkage Factor	Shrinkage Ratio	Test Initial Moisture Content (%)
BB - 156	S-1	0.5' - 3'	15.2	1.76	35.0
BB - 156	S-2	3' - 8'	14.5	1.84	31.5
BB - 158	S-1	0.5' - 3'	18.3	1.73	32.2
BB - 158	S-2	3'- 8'	15.4	1.83	31.6
BB - 159	S-1	0.5' - 5'	18.3	1.74	32.0
BB - 159	S-3	10' - 15'	14.6	1.82	23.3
BB - 161	S-1	0.5' - 3'	17.0	1.69	34.4
BB - 161	S-3	13' - 17'	16.6	1.75	30.9
BB - 163	S-2	2.5' - 7.5'	16.9	1.76	33.8
BB - 166	S-1	0.5' - 4'	15.9	1.75	28.9
BB - 166	S-2	4' - 9'	20.0	1.64	39.6
BB - 167	S-1	0.5' - 5'	12.2	1.88	24.1
BB - 171	S-1	0.5' - 4'	16.6	1.80	40.9
BB - 171	S-2	4' - 9'	16.0	1.84	27.4
BB - 176	S-1	0.5' - 3'	19.7	1.64	52.8
BB - 176	S-2	3' - 8'	23.6	1.60	69.2
BB - 176	S-3	8' - 13'	19.3	1.68	41.1
BB - 176	S-4	13' - 18'	16.1	1.79	27.5
BB - 177	S-1	0.5' - 3'	16.3	1.70	36.9
BB - 177	S-2	3' - 8'	15.4	1.75	32.9
BB - 178	S-1	0.5' - 3'	16.2	1.74	46.2
BB - 178	S-2	3' - 8'	18.1	1.75	40.8
BB - 178	S-4	13' - 18'	17.3	1.74	32.0
BB - 180	S-1	0.5' - 4'	16.0	1.77	48.1
BB - 180	S-2	4' - 9'	15.1	1.83	27.0
BB - 184	S-1	0.5' - 2'	14.8	1.78	32.5
BB - 184	S-2	2' - 7'	14.6	1.83	27.1
BB - 186	S-1	0.5' - 5'	18.4	1.67	71.0
BB - 186	S-2	5' - 8'	18.9	1.65	43.9

TABLE 2.5C.3-3

MAIN DAM BORROW AREA M
SHRINKAGE FACTORS

Boring	Sample Number	Sample Depth	Shrinkage Factor	Shrinkage Ratio	Test Initial Moisture Content (%)
BB 806	S-1	0.0' 4.0'	15.4	1.77	37.5
BB 806	S-2	4.0' 9.0'	19.0	1.73	47.1
BB 806	S-3	9.0' 14.0'	21.5	1.64	40.8
BB 807	S-1	0.0' 5.0'	20.7	1.65	69.3
BB 807	S-2	0.5' 10.0'	15.9	1.81	40.0
BB 807	S-4	15.0' 20.0'	20.7	1.72	56.0
BB 812	S-3	9.0' 14.0'	11.7	1.92	36.3
BB 813	S-1	0.0' 9.0'	25.5	1.56	68.5
BB 814	S-1	0.0' 2.0'	15.4	1.77	48.6
BB 814	S-2	2.0' 12.0'	23.7	1.64	31.7
BB 814	S-3	12.0' 15.0'	33.9	1.43	45.5
BB 815	S-1	0.0' 2.5'	15.8	1.80	50.3
	S-3 & S-4	12.5' 23.0'	16.3	1.85	23.6
BB 863	S-1	0.5' 2.5'	12.4	1.89	40.4
BB 863	S-2	2.5' 7.0'	20.2	1.71	37.8
BB 864	S-1	0.5' 5.0'	19.8	1.70	34.1
BB 864	S-2	5.0' 10.0'	16.6	1.79	31.1
BB 865	S-1	0.5' 5.0'	18.8	1.71	61.6
BB 866	S-1	0.5' 3.0'	16.8	1.74	29.1
BB 866	S-2	3.0' 8.0'	14.3	1.88	25.2
BB 866	S-4	13.0' 18.0'	14.6	1.87	22.1
BB 867	S-1	0.5' 3.0'	17.3	1.74	31.4
BB 868	S-1	0.5' 4.0'	17.4	1.73	44.7
BB 868	S-2	4.0' 9.0'	15.3	1.75	33.4
BB 869	S-1	0.5' 5.0'	17.3	1.77	30.7
BB 869	S-2	5.0' 7.0'	17.6	1.70	39.5
BB 869	S-3	7.0' 11.0'	16.4	1.74	37.2
BB 806	S-1	0.0' 4.0'	15.4	1.77	37.5
BB 806	S-2	4.0' 9.0'	19.0	1.73	47.1
BB 806	S-3	9.0' 14.0'	21.5	1.64	40.8
BB 807	S-1	0.0' 5.0'	20.7	1.65	69.3
BB 807	S-2	0.5' 10.0'	15.9	1.81	40.0
BB 807	S-4	15.0' 20.0'	20.7	1.72	56.0
BB 812	S-3	9.0' 14.0'	11.7	1.92	36.3
BB 813	S-1	0.0' 9.0'	25.5	1.56	68.5
BB 814	S-1	0.0' 2.0'	15.4	1.77	48.6
BB 814	S-2	2.0' 12.0'	23.7	1.64	31.7
BB 814	S-3	12.0' 15.0'	33.9	1.43	45.5
BB 815	S-1	0.0' 2.5'	15.8	1.80	50.3
	S-3 & S-4	12.5' 23.0'	16.3	1.85	23.6
BB 863	S-1	0.5' 2.5'	12.4	1.89	40.4
BB 863	S-2	2.5' 7.0'	20.2	1.71	37.8
BB 864	S-1	0.5' 5.0'	19.8	1.70	34.1
BB 864	S-2	5.0' 10.0'	16.6	1.79	31.1
BB 865	S-1	0.5' 5.0'	18.8	1.71	61.6
BB 866	S-1	0.5' 3.0'	16.8	1.74	29.1
BB 866	S-2	3.0' 8.0'	14.3	1.88	25.2
BB 866	S-4	13.0' 18.0'	14.6	1.87	22.1
BB 867	S-1	0.5' 3.0'	17.3	1.74	31.4
BB 868	S-1	0.5' 4.0'	17.4	1.73	44.7
BB 868	S-2	4.0' 9.0'	15.3	1.75	33.4
BB 869	S-1	0.5' 5.0'	17.3	1.77	30.7
BB 869	S-2	5.0' 7.0'	17.6	1.70	39.5
BB 869	S-3	7.0' 11.0'	16.4	1.74	37.2

TABLE 2.5D-1

COMPUTED PREDOMINANT PERIODS
AND MAXIMUM CREST ACCELERATIONS

Dam	Case		Predominant Period, sec.	Maximum Crest Acceleration, g
Main Dam	M-105-IV	A	0.34	0.43
		B	0.34	0.47
		C	0.41	0.45
		BC	0.40	0.47
		D	0.35	0.42
		E	0.45	0.40
		AV	0.35	0.43
	M-67-IV	A	0.25	0.44
	M-36-IV	A	0.15	0.45
Auxiliary Dam	A-63-IV	Basic	0.43	0.39
		A	0.37	0.39
		B	0.40	0.44
		C	0.33	0.51
		AV	0.38	0.39
	A-44-IV	A	0.24	0.48
		B	0.26	0.45
		C	0.28	0.45
	A-24-IV	A	0.13	0.45
Auxiliary Dike	D-53-IV	A	0.23	0.45
		B	0.30	0.46
		C	0.22	0.52

TABLE 2.5D-2

MAIN DAM, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF τ_f/τ_d IN CORE

Computed Minimum Value of τ_f/τ_d for Case

Plane at	M-105-IV		M-105-IV A		M-105-IV B		M-105-IV C		M-105-IV BC		M-105-IV D		M-105-IV E	
Elevation	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
237.5	1.92	1.68	1.75	1.53	1.65	1.45	1.97	1.72	1.85	1.62	1.77	1.54	2.20	1.93
222.5	1.79	1.57	1.77	1.55	1.60	1.40	1.80	1.58	1.65	1.44	1.79	1.56	2.20	1.92
207.5	1.82	1.57	1.91	1.65	1.70	1.47	1.77	1.52	1.58	1.37	1.89	1.63	2.25	1.94
192.5	1.96	1.69	2.18	1.88	1.91	1.64	1.87	1.62	1.63	1.41	2.03	1.75	2.36	2.04
177.5	2.02	1.75	2.22	1.92	2.04	1.77	1.94	1.68	1.74	1.51	2.13	1.84	2.46	2.14

TABLE 2.5D-3

MAIN DAM, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF τ_f/τ_d IN UPSTREAM FINE FILTER

Computed Minimum Value of τ_f/τ_d for Case

Plane at	M-105-IV		M-105-IV A		M-105-IV B		M-105-IV C		M-105-IV BC		M-105-IV D		M-105-IV E	
Elevation	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
237.5	1.73	1.55	1.70	1.52	1.65	1.47	1.73	1.55	1.74	1.56	1.69	1.51	1.95	1.74
222.5	2.06	1.84	2.24	2.00	2.12	1.89	1.99	1.76	1.96	1.75	2.20	1.96	2.38	2.12
207.5	2.45	2.19	2.85	2.54	2.62	2.34	2.29	2.04	2.19	1.96	2.70	2.41	2.78	2.48
192.5	3.14	2.80	3.84	3.42	3.65	3.25	2.90	2.58	2.80	2.50	3.72	3.32	3.68	3.28
177.5	3.30	2.95	4.03	3.50	3.95	3.43	3.06	2.66	2.96	2.57	3.97	3.44	4.01	3.48

TABLE 2.5D-4

MAIN DAM, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF τ_f/τ_d IN UPSTREAM COATSE FILTER

Computed Minimum Value of τ_f/τ_d for Case

Plane at	M-105-IV		M-105-IV A		M-105-IV B		M-105-IV C		M-105-IV BC		M-105-IV D		M-105-IV E	
Elevation	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
237.5	1.22*	1.20*	1.46*	1.31*	1.35*	1.21*	1.28*	1.15*	1.21*	1.08*	1.40*	1.25*	1.58*	1.42*
222.5	1.58*	1.41*	1.40*	1.69*	1.71*	1.57*	1.48*	1.33*	1.43*	1.28*	1.80*	1.60*	1.90*	1.71*
207.5	1.62	1.45	2.13	1.90	1.98	1.77	1.47	1.31	1.38	1.23	2.02	1.81	1.96	1.75
192.5	2.06	1.84	2.86	2.55	2.73	2.44	1.83	1.63	1.77	1.58	2.82	2.51	2.58	2.30
177.5	2.02	1.80	2.70	2.34	2.66	2.30	1.82	1.58	1.78	1.54	2.67	2.32	2.67	2.32

*For R_d = 80%
All other values for R_d = 75%

TABLE 2.5D-5

MAIN DAM, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF τ_f/τ_d IN UPSTREAM ROCKFILL SHELL

Computed Minimum Value of τ_f/τ_d for Case

	M-105-IV		M-105-IV A		M-105-IV B		M-105-IV C		M-105-IV BC		M-105-IV D		M-105-IV E	
Plane at Elevation	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
237.5	1.66	1.48	1.66	1.48	1.53	1.36	1.66	1.48	1.53	1.37	1.63	1.45	1.82	1.63
222.5	1.70	1.52	1.56	1.39	1.52	1.35	1.76	1.57	1.76	1.57	1.53	1.36	1.84	1.64
207.5	1.51	1.35	1.30	1.16	1.36	1.21	1.62	1.45	1.63	1.46	1.24	1.10	1.63	1.46
192.5	1.75	1.56	1.54	1.37	1.59	1.41	1.88	1.67	1.91	1.70	1.49	1.33	2.00	1.78
177.5	2.12	1.90	1.88	1.68	1.90	1.70	2.24	2.00	2.22	1.98	1.84	1.64	2.50	2.24

TABLE 2.5D-6

AUXILIARY DAM, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF τ_f/τ_d IN CORE

Computed Minimum Value of τ_f/τ_d for Case

Plane at	A-63-IV		A-63-IV A		A-63-IV B		A-63-IV C	
Elevation	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
245	2.13	1.91	2.24	2.00	1.68	1.51	1.70	1.58
235	1.65	1.46	1.77	1.55	1.30	1.15	1.34	1.18
225	1.54	1.37	1.57	1.39	1.25	1.11	1.29	1.14
215	1.43	1.26	1.46	1.29	1.22	1.08	1.30	1.14
205	1.55	1.38	1.53	1.26	1.30	1.15	1.40	1.24

Note: Values based on 97% standard compaction at optimum water content.

TABLE 2.5D-7

AUXILIARY DAM, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF τ_f/τ_d IN FILTER

Computed Minimum Value of τ_f/τ_d for Case

Plane at	A-63-IV		A-63-IVA		A-63-IV B		A-63-IV C	
Elevation	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
245	1.60*	1.42*	1.75*	1.56*	1.53*	1.36*	1.21*	1.08*
235	1.38*	1.22*	1.53*	1.36*	1.33*	1.19*	1.12*	1.00*
225	1.39*	1.24*	1.68*	1.52*	1.34*	1.20*	1.30*	1.16*
215	1.33	1.19	1.68	1.50	1.33	1.19	1.35	1.21
205	1.50	1.34	1.75	1.56	1.46	1.30	1.45	1.29

*For R_d = 80%
All other values for R_d = 75%

TABLE 2.5D-8

AUXILIARY DAM, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF $\tau f/\tau d$ IN RANDOM ROCKFILL

Computed Minimum Value of τ_f/τ_d for Case

Plane at Elevation	A-63-IV		A-63-IV A		A-63-IV B		A-63-IV C	
	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
245	2.28	2.10	1.70	1.57	2.04	1.88	1.53	1.41
235	1.80	1.67	1.35	1.25	1.75	1.61	1.31	1.21
225	1.56	1.38	1.19	1.08	1.56	1.40	1.22	1.09
215	1.64	1.46	1.25	1.11	1.63	1.45	1.38	1.23
205	1.90	1.67	1.37	1.20	1.78	1.55	1.51	1.33

TABLE 2.5D-9

AUXILIARY SEPARATING DIKE, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF $\tau f/\tau d$ IN CORE

Computed Minimum Value of τ_f/τ_d for Case

Plane at Elevation	D-53-IV A		D-53-IV B		D-53-IV C	
	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
241.5	2.10*	1.93*	1.82*	1.68*	1.80*	1.66*
225.0	2.28*	2.13*	1.71*	1.52*	1.90*	1.69*
217.0	1.74	1.54	1.28	1.13	1.50	1.33
210.0	1.74	1.53	1.34	1.18	1.56	1.38

*For 100% standard compaction; all other values for 97% standard compaction.

TABLE 2.5D-10

AUXILIARY SEPARATING DIKE, MAXIMUM CROSS SECTION, MINIMUM VALUES OF τ_f/τ_d IN RANDOM ROCKFILL

Computed Minimum Value of τ_f/τ_d for Case

Plane at Elevation	D-53-IV A		D-53-IV B		D-53-IV C	
	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
241.5	1.84	1.69	1.84	1.69	1.62	1.48
225.0	1.48	1.34	1.61	1.46	1.43	1.29
217.0	1.57	1.40	1.71	1.53	1.55	1.37
210.0	1.73	1.49	1.89	1.61	1.71	1.46

TABLE 2.5D-11

MAIN DAM, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF τf/τd IN DIFFERENT ZONES
CASE M-105-IVA

Computed Minimum Values of τf/τd

Plane at Elevation	Core		Upstream Fine Filter		Upstream Coarse Filter		Upstream Rockfill	
	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
Loading Condition III								
237.5	*	*	*	*	*	*	*	*
222.5	*	*	*	*	*	*	*	*
207.5	2.32	2.01	3.83	3.46	2.75	2.46	1.30	1.16
192.5	2.62	2.28	4.58	4.12	3.38	3.04	1.54	1.37
177.5	2.58	2.29	4.61	4.16	3.02	2.73	1.88	1.68
Loading Condition I								
237.5	1.75	1.53	1.70	1.52	1.46†	1.31†	1.66	1.48
222.5	1.77	1.55	2.24	2.00	1.90†	1.69†	1.56	1.39
207.5	1.91	1.65	2.85	2.54	2.13	1.90	1.30	1.16
192.5	2.18	1.88	3.84	3.42	2.86	2.55	1.54	1.37
177.5	2.22	1.92	4.03	3.58	2.70	2.34	1.88	1.68

*Not applicable because above phreatic surface

†For relative density = 80%

All other values in filters for relative density = 75%

TABLE 2.5D-12

AUXILIARY DAM, MAXIMUM CROSS SECTION,
MINIMUM VALUES OF τ_f/τ_d IN DIFFERENT ZONES
CASE A-63-IVA

Computed Minimum Values of τ_f/τ_d

Plane at Elevation	Core		Upstream Filter		Downstream Filter		Upstream Random Rockfill		Downstream Random Rockfill	
	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles	5 cycles	10 cycles
Loading Condition III										
245	2.42	2.17	1.75†	1.56†	*	*	1.70	1.57	*	*
235	1.77	1.55	1.53†	1.36†	*	*	1.35	1.25	*	*
225	1.59	1.42	1.68†	1.52†	*	*	1.19	1.08	*	*
215	1.47	1.31	1.68	1.50	2.34	2.12	1.25	1.11	1.66	1.43
205	1.55	1.38	1.75	1.56	2.23	2.03	1.37	1.20	1.64	1.40
Loading Condition I										
245	2.24	2.00	1.75†	1.56†	1.75†	1.56†	1.70	1.57	1.70	1.57
235	1.77	1.55	1.53†	1.36†	1.53†	1.36†	1.35	1.25	1.35	1.25
225	1.57	1.39	1.68†	1.52†	1.68†	1.52†	1.19	1.08	1.19	1.08
215	1.46	1.29	1.68	1.50	1.68	1.50	1.25	1.11	1.25	1.11
205	1.53	1.36	1.75	1.56	1.75	1.56	1.37	1.20	1.37	1.20

*Not applicable because above phreatic surface

†For relative density = 80%

All other values in filters for relative density = 75%

TABLE 2.5D-13

BORROW AREA M, INDEX PROPERTIES OF SOILS IN
REPRESENTATIVE BORINGS AND COMPOSITE SAMPLES FROM TEST PITS

Boring and Test Pit Number	Depth Interval ft.	Natural Water Content %	Liquid Limit	Plastic Limit	Plasticity Index	Fines Content	Classification
BB806	0 - 4.0	25.3	49.9	23.7	26.2	50	Red sandy silty CLAY
	4.0 - 9.0	10.4	31.0	22.2	8.8	46	Brown coarse to fine SAND, some silt, trace gravel and clay
	9.0 - 14.0	4.3	27.5	17.7	9.8	44	Brown SAND and SILT, trace gravel
BB807	0 - 5.0	36.6	71.8	45.5	26.3	85	Red CLAY, some silt, little medium to fine sand
	5.0 - 10.0	20.3	31.1	18.8	12.3	82	Red silty CLAY, some sand
	15.0 - 20.0	47.4	52.0	35.8	16.2	63	Red sandy silty CLAY to clayey sandy SILT
BB814	0 - 2.0	20.9	47.8	26.0	21.8	49	Tan and gray SAND and CLAY
	2.0 - 12.0	9.4 and 15.7	26.9	22.7	4.2	39	Red-brown silty SAND to clayey silty SAND
	12.0 - 15.0	14.4	23.1	19.3	3.8	38	Red-brown silty SAND to clayey silty SAND
	15.0 - 18.0	9.4 and 15.7	27.3	21.8	5.5	27	Red-brown silty SAND to silty clayey SAND
Composite Samples from Test Pits							
TPM1	0 - 8.0		29.0	22.0	7.0	45	
TPM2	0 - 9.0		26.0	20.0	6.0	40	
TPM3A	0 - 8.25		29.0	17.0	12.0	42	
TPM4A	0 - 11.0		21.0	19.0	2.0	36	
TPM5A	0 - 5.5		23.0	19.0	4.0	38	
TPM6	0 - 6.0		28.0	24.0	4.0	42	

TABLE 2.5D-14

PHYSICAL PROPERTIES OF MATERIAL M

Description and Classification	Red-brown silty clayey coarse to fine SAND, with trace of fine gravel; SC
Specific Gravity	2.72
Liquid Limit	33
Plasticity Index	11
Compaction Characteristics	
a) standard compaction maximum dry unit weight, γ_d, lb./ft.^{3}	121.8
optimum water content w, %	12.4
b) modified compaction maximum dry unit weight, γ_d, lb./ft.^{3}	130.2
optimum water contentw, %	9.3
Permeability k, cm./sec.	1×10^{-8}

TABLE 2.5D-15

MATERIAL M -

STATIC STRESS-STRAIN PARAMETERS

Parameter	Magnitude: Determined by CID Tests	Magnitude: Determined by UU Tests
Friction angle φ, degrees	30	18
Cohesion C, k./ft.^{2}	0.2	2.0
Modulus No. K	160	120
Modulus exponent, n	0.7	0.44
Failure ratio, R_f	0.76	0.81
Poisson's ratio parameters:		
G	0.2	0.37
F	0.14	0.0
D	8.6	1.4

Note: 100% standard compaction at optimum water content

For definition of parameters, see Section 2.5D.4.4

TABLE 2.5D-16

MATERIAL M, RESULTS OF STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS ON ISOTROPICALLY CONSOLIDATED SPECIMENS

Degree of Standard Compaction %	Initial Effective Confining Pressure $\bar{\sigma}$ 3c k./ft.^{2}	Water Content %	Dry Unit Weight lb./ft.^{3}	Average Cyclic Stress Ratio to N ± $\frac{\sigma_d}{2\bar{\sigma}3c}$	No. of Cycles, N to Cause Initial Liquefaction	± 5 x 10^{-2} Strain	± 10 x 10^{-2} Strain
95	4	12.4	115.5	0.376	1.5		
				0.373		2.9	
				0.363			7.1
“	“	12.4	115.5	0.377	2.3		
				0.369		5.1	
				0.329			14.2
“	“	12.4	115.5	0.269	6.3		
				0.263		11.4	
				0.262			26
“	“	12.4	115.5	0.277	12	19	
				0.276			35
“	“	12.4	115.5	0.203	38	59	113
“	“	12.4	115.5	0.212	46	65	
				0.207			111
“	8	12.4	115.5	0.302	2.2		
				0.301		4.7	
				0.298			9.5
“	“	12.4	115.5	0.304	3.1		
				0.302		6	
				0.288			14
“	“	12.4	115.7	0.346	3.5		
				0.348		9.5	
				0.355			20.8
		12.4	115.7	0.227	16.8		
				0.226		26.3	56
100	2	12.4	121.9	0.494	10	45	150
“	“	12.4	121.9	0.456	14		
				0.454		56	160
“	“	12.4	121.9	0.411	40		
				0.412		155	410
“	4	12.4	121.8	0.445	3.1	6.9	
				0.436			25
“	“	12.4	121.9	0.390	5.2	11	32
“	“	12.4	121.9	0.323	17.5		
				0.319		35	
				0.313			200
“	8	12.5	121.8	0.452	1.1		
				0.451		2.5	
				0.433			7
“	“	12.7	121.5	0.407	2.7		
				0.404		5.5	
				0.403			11
“	“	12.6	121.6	0.292	15	39.5	
				0.291			76
“	12	12.9	121.2	0.410	1.7		
				0.409		3.1	
				0.398			7
“	“	12.8	121.3	0.323	2.9		
				0.322		5.7	
				0.310			13.5
“	“	13.0	121.4	0.278	8.8	15.1	
				0.275			27.5
“	“	12.8	121.5	0.241	25		
				0.240		40	
				0.235			78

TABLE 2.5D-17

MATERIAL M, RESULTS OF STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS ON ANISOTROPICALLY CONSOLIDATED SPECIMENS

Degree of Standard Compaction %	Initial Effective Confining Pressure $\bar{\sigma}$ 3c k./ft.^{2}	Initial Consolidation Ratio Kc	Water Content %	Dry Unit Weight lb./ft.^{3}	Average Cyclic Stress Ratio to N ± $\frac{\sigma_d}{2\bar{\sigma}3c}$	± 5 x 10^{-2} Strain	± 10 x 10^{-2} Strain
100	2	1.5	12.4	121.9	0.573	4.2	
					0.581		13
"	"	"	12.4	121.9	0.467	8	
					0.468		28
"	"	"	12.1	122.1	0.596	17	
					0.592		40
"	"	"	12.0	122.2	0.500	43	
					0.501		105
"	"	"	12.4	121.9	0.414	78	>1000
"	4	"	12.8	121.4	0.557	1.4	
					0.567		4.8
"	"	"	12.6	121.9	0.485	4.1	
					0.487		13
"	"	"	12.7	121.9	0.413	14.1	
					0.414		44
100	2	1.5	13.0	121.2	0.453	34	
					0.457		69
"	8	"	12.4	121.8	0.395	2	
					0.399		7
"	"	"	12.4	121.9	0.347	4.6	
					0.352		17
"	"	"	12.4	121.9	0.309	16	
					0.313		100
"	2	2.0	12.7	121.8	0.682	6.2	
					0.677		42
"	"	"	12.7	121.6	0.629	15	
					0.638		159
"	"	"	12.8	121.5	0.656	11.1	
					0.675		79
"	"	"	12.7	121.5	0.507	87	>1000
"	8	"	12.5	121.8	0.519	<1	
					0.527		2.5
100	8	2.0	12.8	121.5	0.423	1.6	
					0.440		8
"	"	"	12.8	121.6	0.366	4.6	
					0.374		37
"	"	"	12.7	121.6	0.330	15	
					0.335		700

TABLE 2.5D-18

MATERIAL M, SHEAR MODULUS AND DAMPING RATIO
BASED ON STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS

Series No.	Initial Effective Confining Pressure $\bar{\sigma}_{3c}$ k./ft.^{2}	Measured Axial Strain ε in 10^{-3}	Axial Deviator Stress $\pm\sigma_d$ K./ft.^{2}	Young's Modulus E* k./ft.^{2}	Shear Modulus G** k./ft.^{2}	Damping Ratio λ	Shear Strain*** γ in 10^{-3}
1	2	18.7	1.81	96.5	35.8	0.253	25.3
	4	24.6	3.6	146.3	54.2	0.273	33.2
	8	20.0	3.14	158.9	58.9	0.260	26.7
	4	16.7	2.65	157.9	58.5	0.259	22.6
	4	35.0	2.55	72.9	27.0	0.267	47.3

* $E = \sigma_d/\varepsilon$

** $G = E/2\ (1+\mu)$; $\mu = 0.35$ (selected)

*** $\gamma = \varepsilon(1+\mu)$

Note: Reconstituted specimens of material M compacted at 100% standard compaction at optimum water content

TABLE 2.5D-19

MATERIAL M, SHEAR MODULUS AND DAMPING RATIO BASED ON STRAIN-CONTROLLED CYCLIC TRIAXIAL TESTS

Series No.	Initial Effective Confining Pressure $\bar{\sigma}_{3c}$ k.ft.^{2}	Measured Axial Strain ε in 10^{-3}	Corrected Axial Strain* ε_c in 10^{-3}	Axial Deviator Stress ± σ_d K./ft.^{2}	Young's Modulus E** k./ft.^{2}	Shear Modulus G*** k./ft.^{2}	$\frac{G\ ***}{G_{max}}$	Maximum Shear Modulus G_{max} k./ft.^{2}	Damping Ratio λ	Shear Strain***** γ in 10^{-3}
2	2	0.206	0.137	0.477	3482	1290	0.35	3686	0.223	0.185
	2	0.800	0.533	1.040	1951	723	0.16	4520	0.217	0.72
	8	0.210	0.140	1.264	9029	3344	0.35	9555	0.169	0.19
	8	0.807	0.538	2.630	4889	1811	0.16	11320	0.179	0.73

* $\varepsilon_c = \varepsilon/1.5$, to account for end effects and non-uniform strain of test specimen

** E = σ_d/ε_c

*** G = E/2(1+μ); = 0.35 (selected)

**** From Fig. 2.5D-34

***** $\gamma = \varepsilon_c\ (1+\mu)$

Note: Reconstituted specimens of material M compacted at 100% standard compaction at optimum water content

TABLE 2.5D-20

MATERIAL M, SHEAR MODULUS AND DAMPING RATIO
BASED ON CYCLIC TORSION TESTS

Series No.	Initial Effective Confining Pressure $\bar{\sigma}_{3c}$ k.ft.^{2}	Average Shear Strain* γ in 10^{-5}	Damping Ratio λ	Shear Modulus G*** k./ft.^{2}	$\frac{G***}{G_{max}}$	Maximum Shear Modulus G_{max} k./ft.^{2}
3	2	48	0.081	3960	0.86	4605
	4	37	0.070	6494	0.89	7295
	8	31	0.061	7805	0.92	8485
	1	52	0.080	3989	0.84	4750
	2	46	0.074	5155	0.86	5995
	4	39	0.070	7488	0.89	8415
	0.5	95	0.120	1584	0.62	2555
	1	75	0.096	2491	0.72	3460
	2	68	0.092	2995	0.76	3940
	4	42	0.070	5950	0.88	6770
	0.5	80	0.108	1930	0.70	2755
	1	30	0.097	2664	0.92	2660
	2	19	0.10	4406	0.955	4615
	4	17	0.083	6941	0.96	7230
	0.5	46	0.113	1634	0.86	1900
		99	0.119	1460	0.60	2435
	1	43	0.086	2850	0.88	3240
		72	0.092	2615	0.74	3530
	2	45	0.081	3670	0.87	4220
	1	32	0.079	3660	0.91	4020
		54	0.079	3480	0.88	4190
	2	30	0.077	4100	0.92	4455
		52	0.077	4010	0.84	4775
	4	25	0.069	6050	0.94	6440
		42	0.072	5940	0.88	6750

*Average shear strain is 0.7 x measured shear strain to account for non-uniform torsional strains in the specimen.

**From Figure 2.5D-34

Note: Reconstituted specimens of material M compacted at 100% standard compaction at optimum water content

TABLE 2.5D-21

BORROW AREA Z, INDEX PROPERTIES OF SOILS IN REPRESENTATIVE BORINGS AND COMPOSITE SAMPLES FROM TEST PITS

Boring and Test Pit Number	Depth Interval ft.	Natural Water Content %	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content	Classification
BB159	0.5 - 5.0	15.2	32.5	18.9	13.6	83	Yellow-brown silty CLAY, some sand
	10.0 - 15.0	6.6	20.5	18.7	1.8	58	Red-brown sandy SILT
BB171	0.5 - 4.0	18.8	41.0	20.5	20.5	85	Yellow-brown silty CLAY, some sand
	4.0 - 9.0	6.8	26.3	18.6	7.7	77	Red-brown silty CLAY, some sand
BB176	0.5 - 3.0	18.0	54.6	28.4	26.2	72	Orange-brown CLAY, some sand
	3.0 - 8.0	26.2	67.0	38.1	28.9	88	Orange-brown SILT, trace sand
	8.0 - 13.0	26.2	40.6	27.2	13.4	92	Brown clayey SILT, trace sand
	13.0 - 18.0	8.2	27.5	18.2	9.3	67	Red-brown sandy silty CLAY
Composite Samples from Test Pits							
TPZ1	0 - 9.0		28.0	22.0	6.0	47	
TPZ2	0 - 5.5		34.0	23.0	11.0	79	
TPZ3	0 - 7.5		24.0	20.0	4.0	80	
TPZ4	0 - 10.0		29.0	22.0	7.0	82	

TABLE 2.5D-22

PHYSICAL PROPERTIES OF MATERIAL Z

Description and Classification	Brown silty CLAY with some coarse to fine sand and trace of fine gravel; CL
Specific Gravity	2.73
Liquid Limit	35
Plasticity Index	13
Compaction Characteristics	
a) standard compaction maximum dry unit weight, γ_d, lb./ft.^{3}	114.1
optimum water content w, %	15.5
b) modified compaction maximum dry unit weight, γ_d, lb./ft.^{3}	122.5
optimum water content w, %	10.7
Permeability k, cm./sec.	2×10^{-8}

TABLE 2.5D-23

MATERIAL Z -
STATIC STRESS-STRAIN PARAMETERS

Parameter	Magnitude Determined by CID Tests	Magnitude Determined by UU Tests
Friction angle φ, degrees	30	26
Cohesion C, k./ft.^{2}	0.1	1.2
Modulus No. K	160	110
Modulus exponent, n	0.3	0.35
Failure ratio, R_f	0.80	0.69
Poisson's ratio parameters:		
G	0.21	0.4
F	0.084	0.0
D	6.0	1.0

Note: 97% standard compaction at optimum water content

For definition of parameters, see Section 2.5D.4.4

TABLE 2.5D-24

MATERIAL Z, RESULTS OF STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS ON ISOTROPICALLY CONSOLIDATED SPECIMENS

Degree of Standard Compaction %	Initial Confining Pressure $\bar{\sigma}_{3c}$ k./ft.^{2}	Water Content %	Dry Unit Weight lb./ft.^{3}	Average Cyclic Stress Ratio to N $\frac{\pm \sigma_d}{2\bar{\sigma}_{3c}}$	No. of Cycles, N to Cause: Initial Liquefaction	± 5 x 10^{-2} Strain	± 10 x 10^{-2} Strain
95	2.5	15.5	108.4	0.448	2.4		
				0.445		6.3	23
"	"	15.5	108.4	0.375	3.8	9.7	33
"	"	15.5	108.4	0.281	14.6		
				0.280		43	130
97	1.25	15.5	110.6	0.513	3		
				0.512		23	80
"	"	15.5	110.6	0.450	8.6		
				0.449		24	70
"	"	15.5	110.6	0.442	14		
				0.439		80	210
"	2.5	15.3	110.9	0.538	1.6		
				0.527		3.9	
				0.494			9.3
"	"	15.3	110.9	0.424	6.6		
				0.418		13.3	31
"	"	15.5	110.7	0.298	66		
				0.295		122	-
97	5.0	15.5	110.6	0.419	0.8		
				0.421		1.8	
				0.416			6
"	"	15.5	110.6	0.370	2.1	4.8	
				0.365			20
"	"	15.5	110.6	0.297	9.4		
				0.293		20.5	68
98	2.5	17.2*	111.6	0.462	6.5		
				0.466		20	
				0.463			84
"	"	17.7*	111.1	0.403	19	57	-
100	1.25	15.6	114.2	0.482	208	>1000	-
"	2.5	15.5	114.0	0.496	5.8		
				0.491		19	83
"	"	15.5	114.0	0.394	13		
				0.390		58	190
"	"	15.5	114.0	0.321	29	100	370
"	"	15.4	114.2	0.373	45		
				0.370		156	-
"	5.0	15.6	114.1	0.447	1.7		
				0.440		4.5	
				0.408			16.7
"	"	15.3	114.3	0.286	57		
				0.276		123	
				0.275			700

*Optimum water content plus 2%

TABLE 2.5D-25

MATERIAL Z, RESULTS OF STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS ON ANISOTROPICALLY CONSOLIDATED SPECIMENS

Degree of Standard Compaction %	Initial Effective Confining Pressure $\bar{\sigma}_{3c}$ k./ft.^{2}	Initial Consolidation Ratio K_c	Water Content %	Dry Unit Weight lb./ft.^{3}	Average Cyclic Stress Ratio to N $\frac{\pm \sigma_d}{2\bar{\sigma}_{3c}}$	No. of Cycles, N To Cause $\pm 5 \times 10^{-2}$ Strain	$\pm 10 \times 10^{-2}$ Strain
97	1.25	1.5	15.5	110.6	0.550	4	
					0.558		10.6
“	“	“	15.5	110.6	0.471	8.5	
					0.486		30
“	“	“	15.5	110.6	0.425	15	
					0.427		57
“	2.50	“	15.5	110.6	0.492	3.7	9.6
“	“	“	15.5	110.6	0.386	6	
					0.392		26
“	5.0	“	15.4	110.9	0.373	3.2	
					0.367		6.9
“	“	“	15.1	111.1	0.272	20	
					0.271		58
“	“	“	15.7	110.9	0.215	89	>1000
“	1.25	2.0	15.4	110.9	0.762	6.1	
					0.780		9.6
“	“	“	15.7	110.5	0.574	12.5	
					0.578		31
“	“	“	15.6	110.6	0.601	20	
					0.610		56
“	2.5	“	15.6	110.6	0.485	1.8	
					0.491		4.5
“	“	“	15.6	110.6	0.309	13.6	
					0.310		47
“	“	“	15.5	110.8	0.244	50	
					0.245		390
98	2.5	1.5	17.5*	111.4	0.480	2.5	
					0.488		15.5
“	“	“	17.1*	111.7	0.328	56	>1000
100	1.25	1.5	15.5	114.2	0.686	43	
					0.689		119
“	“	“	15.4	114.5	0.610	58	
					0.612		240
“	2.5	“	15.6	114.2	0.502	17.5	
					0.500		64
“	“	“	15.4	114.3	0.332	100	>1000
“	“	“	15.5	114.0	0.297	>1000	-
“	5.0	“	15.6	114.3	0.395	6.7	
					0.399		28

*Optimum water content plus 2%

TABLE 2.5D-26

MATERIAL Z, SHEAR MODULUS AND DAMPING RATIO
BASED ON STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS

Series No.	Initial Effective Confining Pressure $\bar{\sigma}_{3c}$ k.ft.^{2}	Measured Axial Strain ε in 10^{-3}	Axial Deviator Stress ± σ_d K. ft.^{2}	Young's Modulus E* k./ft.^{2}	Shear Modulus G** k./ft.^{2}	Damping Ratio λ	Shear Strain** γ in 10-^{3}
1	5	24.0	3.715	154.8	57.3	0.260	32.4
	2.5	19.5	2.130	109.2	40.5	0.244	26.3
	2.5	13.8	1.473	106.7	39.5	0.244	18.6

* $E = \sigma_d/\varepsilon$

** $G = E/2\ (1+\mu)$; $\mu = 0.35$ (selected)

*** $\gamma = \varepsilon(1+\mu)$

Note: Reconstituted specimens of material Z compacted at 97% standard compaction at optimum water content

TABLE 2.5D-27

MATERIAL Z, SHEAR MODULUS AND DAMPING RATIO
BASED ON STRAIN-CONTROLLED CYCLIC TRIAXIAL TESTS

Series No.	Initial Effective Pressure $\bar{\sigma}_{3c}$ k./ft.^{2}	Measured Axial Strain ε in 10^{-3}	Corrected Axial Strain* ε in 10^{-3}	Axial Deviator Stress $\pm\sigma_d$ K. /ft.^{2}	Young's Modulus E** k. /ft.^{2}	Shear Modulus G*** k. /ft.^{2}	$\frac{G}{G_{max}}$ ****	Maximum Shear Modulus G_{max} k. /ft.^{2}	Damping Ratio λ	Shear Strain***** γ in 10^{-3}
2	1.156	0.198	0.132	0.380	2879	1066	0.41	2600	0.141	0.18
	1.16	0.769	0.513	0.691	1347	499	0.16	3120	0.201	0.67
	3.94	0.838	0.559	1.400	2500	926	0.15	6175	0.183	0.76
	4.0	0.192	0.128	0.751	5867	2173	0.42	5175	0.132	0.175

* $\varepsilon_c = \varepsilon/1.5$, to account for end effects and non-uniform strain of test specimen

** $E = \sigma_d/\varepsilon_c$

*** $G = E/2(1+\mu)$; $\mu = 0.35$ (selected)

**** From Figure 2.5D-55

***** $\gamma = \varepsilon_c (1+\mu)$

Note: Reconstituted specimens of material Z compacted at 97% standard compaction at optimum water content

TABLE 2.5D-28

MATERIAL Z, SHEAR MODULUS AND DAMPING RATIO
BASED ON CYCLIC TORSION TESTS

Series No.	Initial Effective Confining Pressure $\bar{\sigma}_{3c}$ k./ft.^{2}	Average Shear Strain* γ in 10^{-5}	Damping Ratio λ	Shear Modulus G, k./ft.^{2}	G/G_{max}**	Maximum Shear Modulus G_{max}, k./ft.^{2}
3	0.5	28	0.07	648	0.28	2315
	1	19	0.059	1382	0.40	3455
	2	17	0.062	1670	0.45	3710
	1	15	0.051	1930	0.54	3575
	2	13	0.044	2794	0.60	4655
	4	10	0.039	4363	0.71	6145
	2	14.7	0.053	1900	0.54	3520
	4	10.7	0.037	4090	0.68	6015
	6	9.5	0.034	5570	0.73	7630
	4	10.2	0.034	3880	0.70	5540
	6	9.7	0.037	4460	0.72	6195
	6	10.1	0.038	4620	0.70	6600

*Average shear strain is 0.7 x measured maximum shear strain to account for non-uniform torsional strains in the specimen

** From Figure 2.5D-55

Note: Reconstituted specimens of material Z compacted at 97% standard compaction at optimum water content

TABLE 2.5D-29

MATERIAL Z, RESULTS OF CYCLIC TORSION TESTS
AT 100% STANDARD COMPACTION

Series No.	Dry Unit Weight γ_d lb./ft.^{2}	Water Content %	Initial Effective Confining Pressure $\bar{\sigma}_{3c}$ k./ft.^{2}	Shear Modulus G k./ft.^{2}	Damping Ratio λ 10^{-2}	Shear Strain γ 10^{-5}
4	114.3	15.8	1.0	1820	5.72	4.12
				1640	6.26	15.9
			2.0	2300	5.00	3.70
				2080	5.51	14.5
			4.0	3950	3.02	2.75
				3700	4.24	10.6
	114.4	15.8	2.0	3590	8.88	2.68
				3460	4.00	10.7
			4.0	5000	3.00	2.30
				4790	3.68	9.28
			8.0	6150	7.68	2.91
				5970	3.82	8.25
	113.7	17.8*	1.0	1650	4.31	4.90
				1350	5.87	17.5
			2.0	2830	3.37	3.53
				2620	4.66	14.2
			4.0	3640	3.06	3.23
				3400	4.35	12.1
	113.6	17.6*	2.0	2940	9.25	3.68
				2670	4.82	14.0
			4.0	4370	2.98	2.95
				4220	3.89	11.0
			8.0	6360	2.54	2.09
				6130	3.43	8.47
				5750	3.71	15.0

*Optimum water content plus 2%

TABLE 2.5D-30

FILTER MATERIALS, STATIC PROPERTIES

Material Property or Parameter	Main Dam Fine Filter	Main Dam Coarse Filter	Auxiliary Dam Filter
Saturated unit weight γsat, lb./ft.^{3}	135	140	140
Moist unit weight γm, lb./ft.^{3}	130	135	135
Cohesion C, k./ft.^{2}	0	0	0
Angle of internal friction φ, degrees	35	38	37
Modulus number, K	400	400	400
Modulus exponent, n	0.5	0.4	0.4
Failure ratio, R_f	0.8	0.8	0.8
Poisson's ratio parameters			
G	0.35	0.35	0.3
F	0.1	0.1	0.12
D	5.0	5.0	5.0

TABLE 2.5D-31

MAIN DAM, AUXILIARY DAM,
DYNAMIC PROPERTIES FOR FILTERS

Material	Unit Weight lb./ft.^{3} Moist	Saturated	$\overline{K}_o$*	Poisson's Ratio μ	Shear Modulus Parameter K_2, max**	Damping Ratio λ
Main Dam Fine Filter	130	135	0.6	0.35	60	***
Coarse Filter	135	140	0.6	0.35	120	***
Auxiliary Dam Filter	135	140	0.6	0.35	90	***

* $\overline{K}_o$ = ratio of horizontal effective stress to vertical effective stress

** G_{max} = K_2,max $\bar{\sigma}_o^{1/2}$

where

$\bar{\sigma}_o$ = Mean normal effective stress in lb./ft.^{2}

G_{max} = Maximum shear modulus in k./ft.^{2}

*** as shown in Figure 2.5D-59

TABLE 2.5D-32

MAIN DAM, AUXILIARY DAM, AUXILIARY SEPARATING DIKE,
STATIC PROPERTIES FOR ROCKFILLS

Material Property or Parameter	Main Dam Rockfill	Auxiliary Dam and Dike Random Rockfill
Saturated Unit Weight γsat, lb./ft.^{3}	145	140
Moist Unit Weight γm, lb./ft.^{3}	130	130
Cohesion C, k./ft.^{2}	0	0
Angle of Internal Friction φ, degrees	40	28
Modulus Number, K	400	300
Modulus Exponent, n	0.3	0.5
Failure Ratio, R_f	0.8	0.8
Poisson's Ratio Parameters		
G	0.25	0.3
F	0.1	0.12
D	5.0	5.0

TABLE 2.5D-33

MAIN DAM, AUXILIARY DAM, AUXILIARY SEPARATING DIKE, DYNAMIC PROPERTIES FOR ROCKFILL MATERIAL

Material	Unit Weight lb./ft.^{3} Moist	Unit Weight lb./ft.^{3} Saturated	$\overline{K}_o$*	Poisson's Ratio μ	Shear Modulus Parameter K_2, max	Damping Ratio λ
Main Dam Rockfill	130	145	0.6	0.30	180	***
Auxiliary Dam & Dike Random Rockfill	130	140	0.6	0.30	90	***

* $\overline{K}_o$ = ratio of horizontal effective stress to vertical effective stress

** G_{max} = K_2,max $\bar{\sigma}_o^{1/2}$

where

$\bar{\sigma}_o$ = Mean normal effective stress in lb./ft.^{2}

G_{max} = Maximum shear modulus in k./ft.^{2}

*** as shown in Figure 2.5D-59

TABLE 2.5D-34

MAIN DAM, STATIC AND DYNAMIC PROPERTIES OF WEATHERED ROCK

Material Property or Parameter	Symbol	Value Used in Analysis	Basis of Selection
Material Properties for Static Stress Analyses			
Saturated Unit Weight, lb./ft.^{3}	γ_{sat}	150	Table 2.5B-3
Cohesion, k./ft.^{2}	C	4	Kulhawy et. al. 1969
Friction Angle, degrees	φ	10	"
Modulus Number	K	3000	"
Modulus Exponent	n	0.1	"
Failure Ratio	R_f	0.8	"
Poisson's Ratio Parameters	G	0.2	"
	F	0.01	"
	D	1.0	"
Material Properties for Dynamic Analyses			
Saturated Unit Weight, lb./ft.^{3}	γ_{sat}	150	Table 2.5B-3
Ratio of Horizontal to Vertical Effective Stresses	$\bar{K}_o$	0.6	D'Appolonia et. al. 1969, Lacroix and Horn 1973
Poisson's Ratio	μ	0.35	Leonards 1962, Barkan 1962
Shear Modulus Parameter	$K_{2\ max}$	700	Section 2.5D.13.2.1.2.1
Damping Ratio	λ	Figure 2.5D 59	Seed and Idriss 1970

TABLE 2.5D-35

AUXILIARY DAM, STATIC AND DYNAMIC PROPERTIES OF WEATHERED ROCK

Material Property or Parameter	Symbol	Value Used in Analysis	Basis of Selection
Material Properties for Static Stress Analyses			
Saturated Unit Weight, lb./ft.^{3}	γ_{sat}	150	Table 2.5B-4
Moist Unit Weight, lb./ft.^{3}	γ_m	140	"
Cohesion, k./ft.^{2}	C	4.0	Kulhawy et. al. 1969
Friction Angle, degrees	φ	10	"
Modulus Number	K	3000	"
Modulus Exponent	n	0.1	"
Failure Ratio	R_f	0.8	"
Poisson's Ratio Parameters	G	0.2	"
	F	0.01	"
	D	1.0	"
Material Properties for Dynamic Analyses			
Saturated Unit Weight, lb./ft.^{3}	γ_{sat}	150	Table 2.5B-4
Ratio of Horizontal to Vertical Effective Stresses	$\bar{K}_o$	0.6	D'Appolonia et. al. 1969, Lacroix and Horn 1973
Poisson's Ratio	μ	0.35	Leonards 1962, Barkan 1962
Shear Modulus Parameter	$K_{2\ max}$	700	Compressional Wave Velocity (Table 2.5.2-3)
Damping Ratio	λ	Figure 2.5D 59	Seed and Idriss 1970

TABLE 2.5D-36

AUXILIARY DAM, STATIC AND DYNAMIC PROPERTIES OF IN-SITU RESIDUAL SOILS

Material Property or Parameter	Symbol	Value Used in Analysis	Basis of Selection
Material Properties for Static Stress Analyses			
Saturated Unit Weight, lb./ft.^{3}	γ_{sat}	134	Lab Tests
Moist Unit Weight, lb./ft.^{3}	γ_m	130	"
Cohesion, k./ft.^{2}	C	0.15	"
Angle of Internal Friction, degrees	φ	30	"
Modulus Number	K	600	Kulhawy et. al. 1969
Modulus Exponent	n	0.3	"
Failure Ratio	R_f	0.8	"
Poisson's Ratio Parameters	G	0.22	"
	F	0.05	"
	D	3.0	"
Material Properties for Dynamic Analyses			
Saturated Unit Weight, lb./ft.^{3}	γ_{sat}	135	Lab Tests
Ratio of Horizontal to Vertical Effective Stresses	$\bar{K}_o$	0.6	D'Appolonia et. al. 1969, Lacroix and Horn 1973
Poisson's Ratio	μ	0.30	Leonards 1962, Barkan 1962
Shear Modulus Parameter	$K_{2\ max}$		Compressional Wave Velocity App. 2.5C (pp. 2.5C.2-6 & 2.5C.2-7)
Near Sta 14+60		100	Shear Wave Velocity
Near Sta 34+60		190	App 2.5C (PP. 2.5C.2-6 &2.5C.2-7)
Damping Ratio	λ	Figure 2.5D 59	Seed and Idriss 1970

TABLE 2.5D-37

AUXILIARY DAM AND AUXILIARY SEPARATING DIKE,
IN-SITU RESIDUAL SOIL
$K_{2\,max}$ BASED ON SHEAR-WAVE VELOCITY MEASUREMENTS

Auxiliary Dam	Depth ft.	V_S -ft./sec.	$K_{2,max}$
In-Situ Residual Soil	7.5	750	86
Near Sta 14+60	10	705	66
γ_{sat}= 135 lb./ft.^{3}	5.1	680	87
	3	665	108
Transitional Material	5	1000	194
Near Sta 34+60	10	1335	245
γ_{sat} = 142.5 lb./ft.^{3}	11	1380	250
	6.3	1065	198
Auxiliary Dike			
In-Situ Residual Soil	12.5	715	61
Approx. N 685, 400			
E2, 008,550			
γ_{sat} = 135 lb./ft.^{3}			

Note: Ref. App. 2.5C (Table 2.5C.2-14)

TABLE 2.5D-38

AUXILIARY DAM, IN-SITU RESIDUAL SOILS, RESULTS OF STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS

									No. of Cycles, N, Causing			
Test Pit	Depth ft.	Dry Unit Weight lb./ft.^{3}	Water Content %	Liquid Limit	Plasticity Index	$K_{c}=\frac{\bar{\sigma}_{1c}}{\bar{\sigma}_{3c}}$	Initial Effective Confining Pressure $\bar{\sigma}_{3c}$ k./ft.^{3}	Average Cyclic Stress Ratio to N $\frac{\pm\sigma_d}{2\bar{\sigma}_{3c}}$	2×10^{-2} Strain	5×10^{-2} Strain	10×10^{-2} Strain	Remarks
TPA1	2.5	97.7	23.8	73	42	1.0	0.5	0.449	>100,000[E]	-	-	Strain = 0.1×10^{-2} at 1000 cycles
TPA1	≈ 3.5	100.9	24.3	53	29	1.0	0.5	0.440	>100,000[E]	-	-	Strain = 0.1×10^{-2} at 1000 cycles
TPA1	3.1	100.2	23.8	62	35	1.0	1.5	0.476	>100,000[E]	-	-	Strain = 0.55×10^{-2} at 1000 cycles
TPA2	3.2	120.7	12.4	38	12	1.0	1.5	0.463	72			
								0.461		900	>1,000	
TPA1	≈ 3.5	99.7	22.7	55	30	1.0	1.5	0.439	6,500[E]	-	-	Strain = 0.8×10^{-2} at 1100 cycles
TPA1	≈ 8	99	26.4	49	18	1.0	1.5	0.508	2,200[E]	-	-	Strain = 0.5×10^{-2} at 1000 cycles
TPA1	≈ 8	101	24	48	15	1.0	3.0	0.509	105	650	-	Strain = 5.7×10^{-2} at 1000 cycles
TPA1	≈ 3.5	102	22.5	66	38	1.0	3.0	0.486	4,700[E]	80,000[E]	-	Strain = 1.15×10^{-2} at 1000 cycles
TPA2	3.4	130.9	9.5	34	9	1.0	3.0	0.455	300	>100,000[E]	-	Strain = 2×10^{-2} at 1000 cycles
TPA2	2.9	126.9	11.2	36	11	1.5	0.5	0.480	>100,000[E]	-	-	Strain = 0.17×10^{-2} at 1000 cycles
TPA1	≈ 3.5	103.5	21.3	58	32	1.5	3.0	0.711	37			
								0.712		94	225	
TPA2	2.4	123.2	12.3	39	12	1.5	3.0	0.660	101			
								0.661		310		
								0.662			566	
TPA1	≈ 8	98.6	25.3	52	20	1.5	3.0	0.718	3			
								0.719		4.7		
								0.731			18.2	
TPA1	≈ 3.5	104.1	21.1	61	36	1.7	0.5	0.692	>100,000[E]	-	-	Strain <0.2×10^{-2} at 1000 cycles
PA2	2.3	127.2	9.2	40	12	2	1.5	0.977	22			
								0.984		180	-	Strain = 8.4×10^{-2} at 1000 cycles

Note: E indicates number of cycles extrapolated beyond duration of test

TABLE 2.5D-39

AUXILIARY RESERVOIR SEPARATING DIKE, STATIC AND DYNAMIC PROPERTIES OF IN-SITU RESIDUAL SOIL

Material Property or Parameter	Symbol	Value Used in Analyses	Basis of Selection
		Material Properties for Static Stress Analyses	
Saturated Unit Weight,lb./ft.^{3}	γ_{sat}	135	Lab Tests
Moist Unit Weight, lb./ft.^{3}	γ_m	135	"
Cohesion, k./ft.^{2}	C	0.13	Kulhawy et. al., 1969
Friction Angle, degrees	φ	30	"
Modulus Number	K	3000	"
Modulus Exponent	n	0.1	"
Failure Ratio	R_f	0.8	"
Poisson's Ratio Parameters			
	G	0.3	"
	F	0.01	"
	D	1.0	"
		Material Properties for Dynamic Analyses	
Saturated Unit Weight, lb./ft.3	γ_{sat}	135	Lab Tests
Ratio of Horizontal to Vertical Effective Stresses	$\bar{K}_o$	0.6	D'Appolonia et. al., 1969, Lacroix and Horn 1973
Poisson's Ratio	μ	0.30	Leonards 1962, Barkan 1962
Shear Modulus Parameter	$K_{2,max}$	90	Shear-Wave Velocity App. 2.5C (Table 2.5C.2 14)
Damping Ratio	λ	Figure 2.5D 59	Seed and Idriss 1970

TABLE 2.5D-40

MAIN DAM, MATERIAL PROPERTIES FOR
STATIC STRESS ANALYSES

Material Property or Parameter	Core UU Data	Core CID Data	Fine Filter	Coarse Filter	Rockfill	Weathered Rock
Saturated unit weight γ_{sat}, lb./ft.^{3}	142	142	135	140	145	150
Moist Unit Weight γ_m, lb./ft.^{3}	137	137	130	135	130	150
Cohesion C, k./ft.^{2}	2.0	0.2	0	0	0	4
Angle of internal friction φ, degrees	18	30	35	38	40	10
Modulus number, K	120	160	400	400	400	3000
Modulus exponent, n	0.44	0.7	0.5	0.4	0.3	0.1
Failure ratio, R_f	0.81	0.76	0.8	0.8	0.8	0.8
Poisson's ratio parameters						
G	0.37	0.2	0.35	0.35	0.25	0.2
F	0.0	0.14	0.1	0.1	0.1	.01
D	1.4	8.6	5.0	5.0	5.0	1.0

TABLE 2.5D-41

MAIN DAM, MATERIAL PROPERTY COMBINATIONS
FOR STATIC STRESS ANALYSES

Set Identification	Core	Fine Filter	Coarse Filter	Rockfill	Weathered Rock	No. of Steps for Filling Reservoir
M (2)	*CID Data	*	*	*	*	2
M (3)	*UU Data	*	*	*	*	2

*As given in Table 2.5D-40

M (1) Preliminary run with assumed material properties

TABLE 2.5D-42

AUXILIARY DAM, MATERIAL PROPERTIES
FOR STATIC STRESS ANALYSES

Material Property or Parameter	Core CID Data	Core UU Data	Random Rockfill	Filter	In-Situ Residual Soil	Weathered Rock
Saturated unit weight γ_{sat}, lb./ft.^{3}	135	135	140	140	134	150
Moist Unit Weight γ_m, lb./ft.^{3}	128	128	130	135	130	140
Cohesion C, k./ft.^{2}	0.1	1.2	0	0	0.15	4.0
Angle of internal friction φ, degrees	30	26	28	37	30	10
Modulus number, K	160	110	300	400	600	3000
Modulus exponent, n	0.3	0.35	0.5	0.4	0.3	0.1
Failure ratio, R_f	0.80	0.69	0.8	0.8	0.8	0.8
Poisson's ratio parameters						
G	0.21	0.4	0.3	0.3	0.22	0.2
F	0.084	0.0	0.12	0.12	0.05	0.01
D	6.0	1.0	5.0	5.0	3.0	1.0

TABLE 2.5D-43

AUXILIARY DAM, MAXIMUM SECTION,
MATERIAL PROPERTY COMBINATIONS
FOR STATIC STRESS ANALYSES

Set Identification	Core	Filter	Random Rockfill	No. of Steps For Filling Reservoir
A-II-5	*CID Data	*	* except G=0.25	1
A-II-6	*UU Data	*	* except G=0.25	1
A-II-7	*UU Data for Construction CID Data Below Water Level After Filling Reservoir	*	* except G=0.25	
A-II-8	*CID Data	*	*	1
A-II-9	*UU Data	*	*	1

*As given in Table 2.5D-42

Note:Computations for Set No. A-II-1 through A-II-4 were preliminary runs made with assumed material properties

TABLE 2.5D-44

AUXILIARY SEPARATING DIKE, MATERIAL PROPERTIES
FOR STATIC STRESS ANALYSES

Material Property or Parameter	Core CID Data	Core UU Data	Random Rockfill	In-Situ Residual Soil
Saturated unit weight γ_{sat}, lb./ft.^{3}	135	135	140	135
Moist Unit Weight γ_m, lb./ft.^{3}	128	128	130	135
Cohesion C, k./ft.^{2}	0.1	1.2	0	0.13
Angle of internal friction φ, degrees	30	26	28	30
Modulus number, K	160	110	300	3000
Modulus exponent, n	0.3	0.35	0.5	0.1
Failure ratio, R^f	0.80	0.69	0.8	0.8
Poisson's ratio parameters				
G	0.21	0.4	0.25	0.30
F	0.084	0.0	0.12	0.01
D	6.0	1.0	5.0	1.0

TABLE 2.5D-45

AUXILIARY SEPARATING DIKE, MATERIAL PROPERTY COMBINATIONS FOR STATIC STRESS ANALYSES

Set Identification	Core	Random Rockfill	In-Situ Residual Soil	No. of Steps For Filling Reservoir
D (1)	*UU Data	*	*	1
D (2)	*UU Data During Construction CID Data Below Water Level After Filling Reservoir	*	*	1
D (3)	*UU Data	*	*	2
D (4)	*UU Data During Construction CID Data Below Water Level After Filling Reservoir	*	*	2

*As given in Table 2.5D-44

TABLE 2.5-D-46

MAIN DAM, MATERIAL PROPERTY COMBINATIONS
FOR PARAMETRIC STUDIES

Set Identification	Core (Material M)		Fine Filter		Coarse Filter		Rockfill		Weathered Rock	
	$K_{2,max}$	Damping	$K_{2,max}$	Damping	$K_{2,max}$	Damping	$K_{2,max}$	Damping	$K_{2,max}$	Damping
Basic Set	120	a	60	c	120	c	180	c	700	c
A	120	a	60	c	120	c	250	c	700	c
B	140	b	60	c	120	c	250	c	700	c
C	120	a	60	c	120	c	150	c	700	c
BC	140	b	60	c	120	c	150	c	700	c
D	120	b	60	d	120	d	250	d	700	d
E	90	a	50	c	90	c	150	c	700	c
AV*	120	a	60	c	120	c	250	c	700	c

*AV Set A with horizontal and vertical accelerations acting simultaneously

a Damping ratio from Figure 2.5D-35

b Damping ratio from Figure 2.5D-35 x 0.90

c Damping ratio from Figure 2.5D-59

d Damping ratio from Figure 2.5D-59 x 0.80

Note: All computations refer to maximum cross section of main dam, 105 ft. high, series IV (M-105-IV)

TABLE 2.5D-47

AUXILIARY DAM, MAXIMUM SECTION MATERIAL PROPERTY COMBINATIONS FOR PARAMETRIC STUDIES

Set Identification	Core (Material Z)		Filter		Random Rockfill	
	$K_{2,max}$	Damping	$K_{2,max}$	Damping	$K_{2,max}$	Damping
Basic Set	100	a	90	c	90	c
A	100	a	90	c	150	c
B	125	b	90	c	90	c
C	125	b	120	d	150	d
AV*	100	a	90	c	150	c

*AV Set A with horizontal and vertical accelerations acting simultaneously

a Damping ratio from Figure 2.5D-56

b Damping ratio from Figure 2.5D-56 x 0.90

c Damping ratio from Figure 2.5D-59

d Damping ratio from Figure 2.5D-59 x 0.80

Note: All computations refer to maximum cross section of Auxiliary Dam, 63 ft. high, series IV (A-63-IV)

TABLE 2.5D-48

AUXILIARY DAM, CROSS SECTION A-44 MATERIAL PROPERTY COMBINATIONS FOR PARAMETRIC STUDIES

Set Identification	Core		Filter		Rockfill		In-Situ Residual Soil		Weathered Rock	
	$K_{2,max}$	Damping	$K_{2,max}$	Damping	$K_{2,max}$	Damping	$K_{2,max}$	Damping	$K_{2,max}$	Damping
Basic Set	100	a	90	c	90	c	100	c	700	c
A	100	a	90	c	150	c	100	c	700	c
B	100	a	90	c	150	c	60	c	700	c
C	100	a	90	c	150	c	60	c	250	c

a Damping ratio from Figure 2.5D-56

c Damping ratio from Figure 2.5D-59

Note: All computations refer to Auxiliary Dam, cross section A-44, 44 ft. high, series IV (A-44-IV)

TABLE 2.5D-49

AUXILIARY SEPARATING DIKE, MATERIAL PROPERTY COMBINATIONS FOR PARAMETRIC STUDIES

Set Identification	Core (Material Z)		In Situ Residual Soil		Random Rockfill	
	$K_{2,max}$	Damping	$K_{2,max}$	Damping	$K_{2,max}$	Damping
Basic Set	100	a	90	c	90	c
A	100	a	90	c	150	c
B	125	b	90	c	90	c
C	125	b	125	d	150	d

a Damping ratio from Figure 2.5D-56

b Damping ratio from Figure 2.5D-56 x 0.90

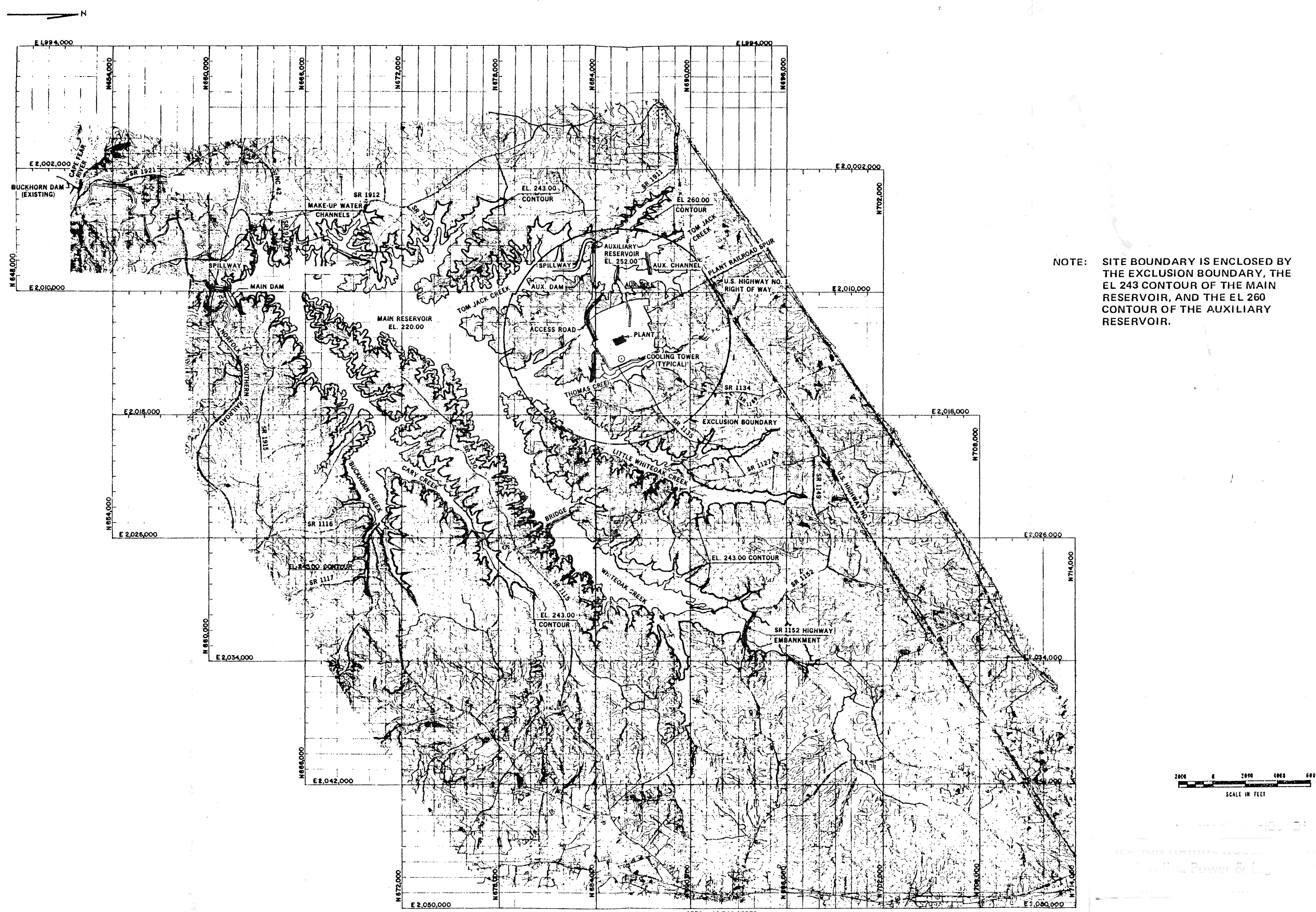
c Damping ratio from Figure 2.5D-59

d Damping ratio from Figure 2.5D-59 x 0.80

Note: All computations refer to maximum cross section of Auxiliary Separating Dike, 53 ft. high, series IV (D-53-IV)

FIGURE 2.1.1-1

MAP OF SITE EXCLUSION BOUNDARY AND SITE BOUNDARY



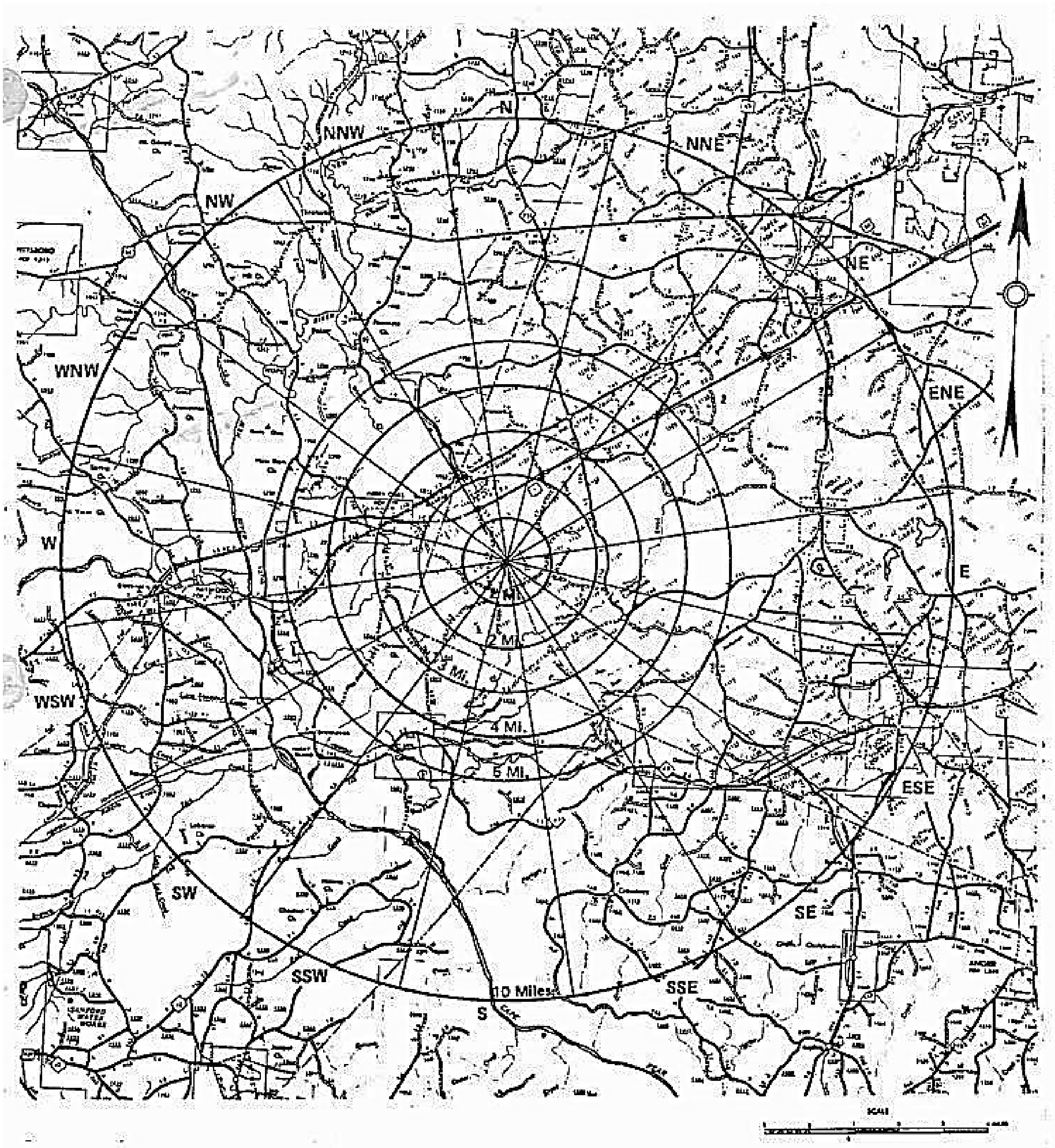
NOTE: SITE BOUNDARY IS ENCLOSED BY THE EXCLUSION BOUNDARY, THE EL 243 CONTOUR OF THE MAIN RESERVOIR, AND THE EL 260 CONTOUR OF THE AUXILIARY RESERVOIR.

FIGURE 2.1.2-1

SHNPP EXCLUSION BOUNDARY PLAN

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.1.3-1



TEN MILE AREA SURROUNDING THE SHEARON HARRIS NUCLEAR POWER PLANT

FIGURE 2.1.3-2

FIFTY MILE RADIAL AREA SURROUNDING
THE SHEARON HARRIS NUCLEAR POWER PLANT

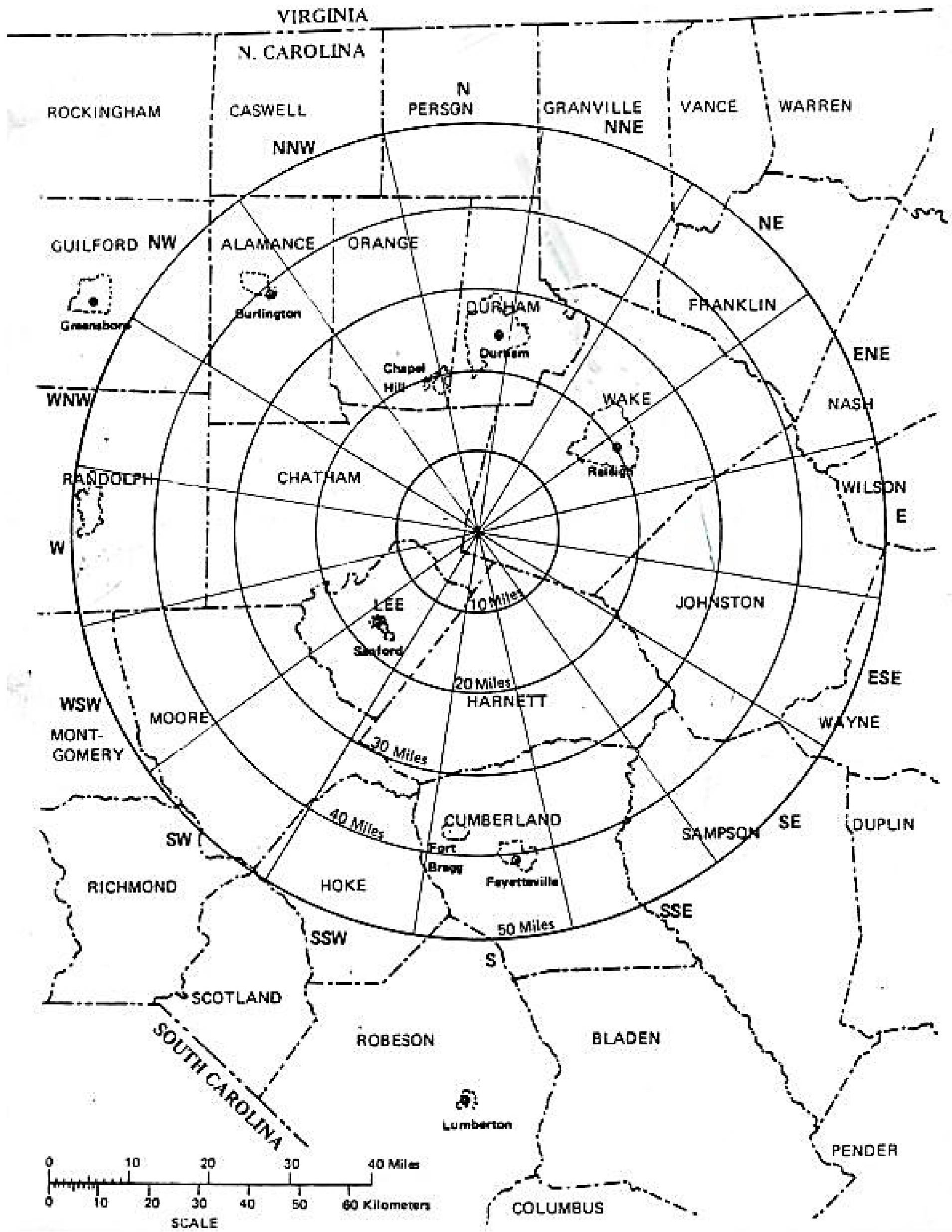


FIGURE 2.1.3-3

PRINCIPAL RECREATIONAL AREAS WITHIN 50 MILES OF THE SHNPP

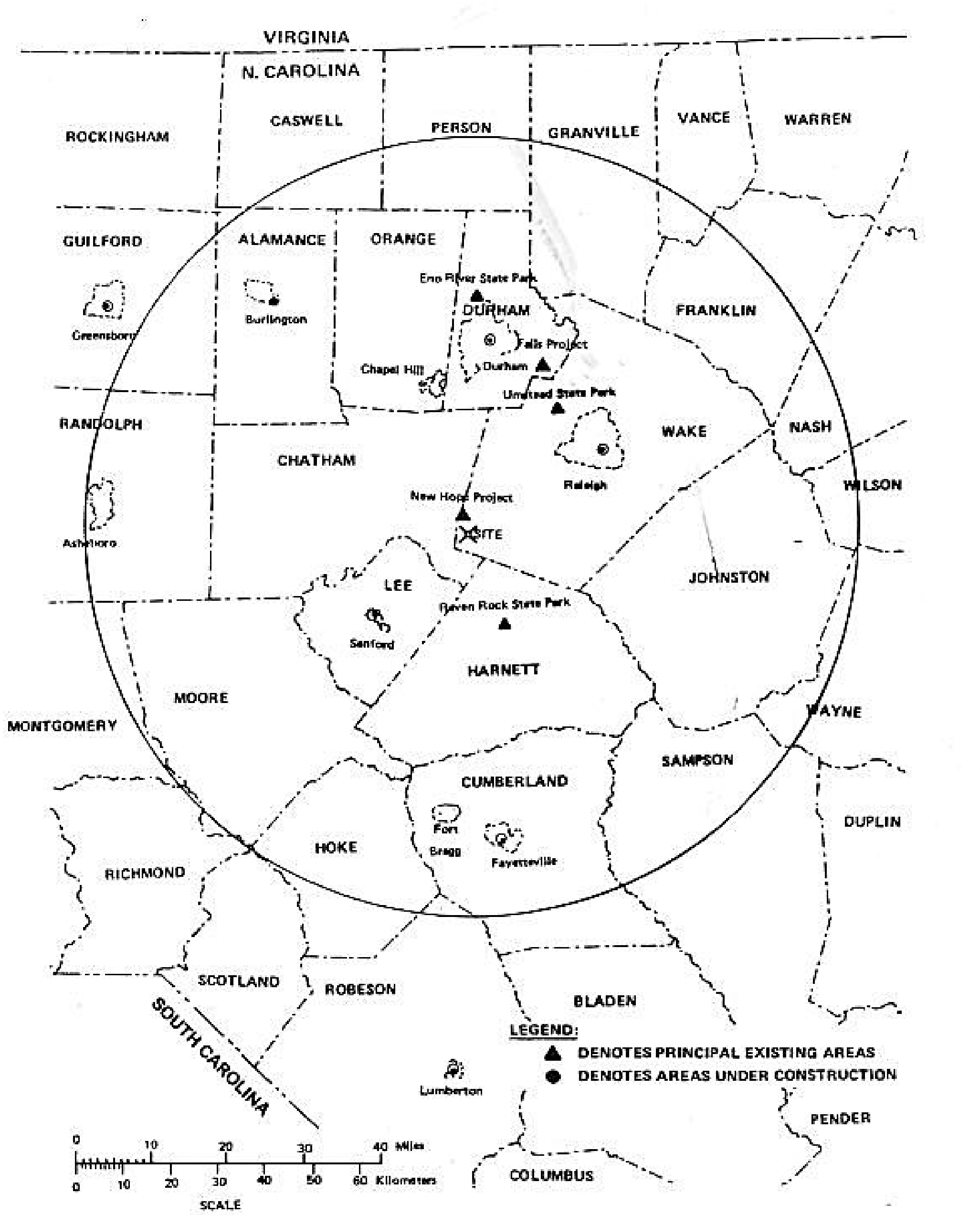


FIGURE 2.1.3-4

LOCAL EMERGENCY PLANNING ZONES AND EVACUATION ROUTES MAP

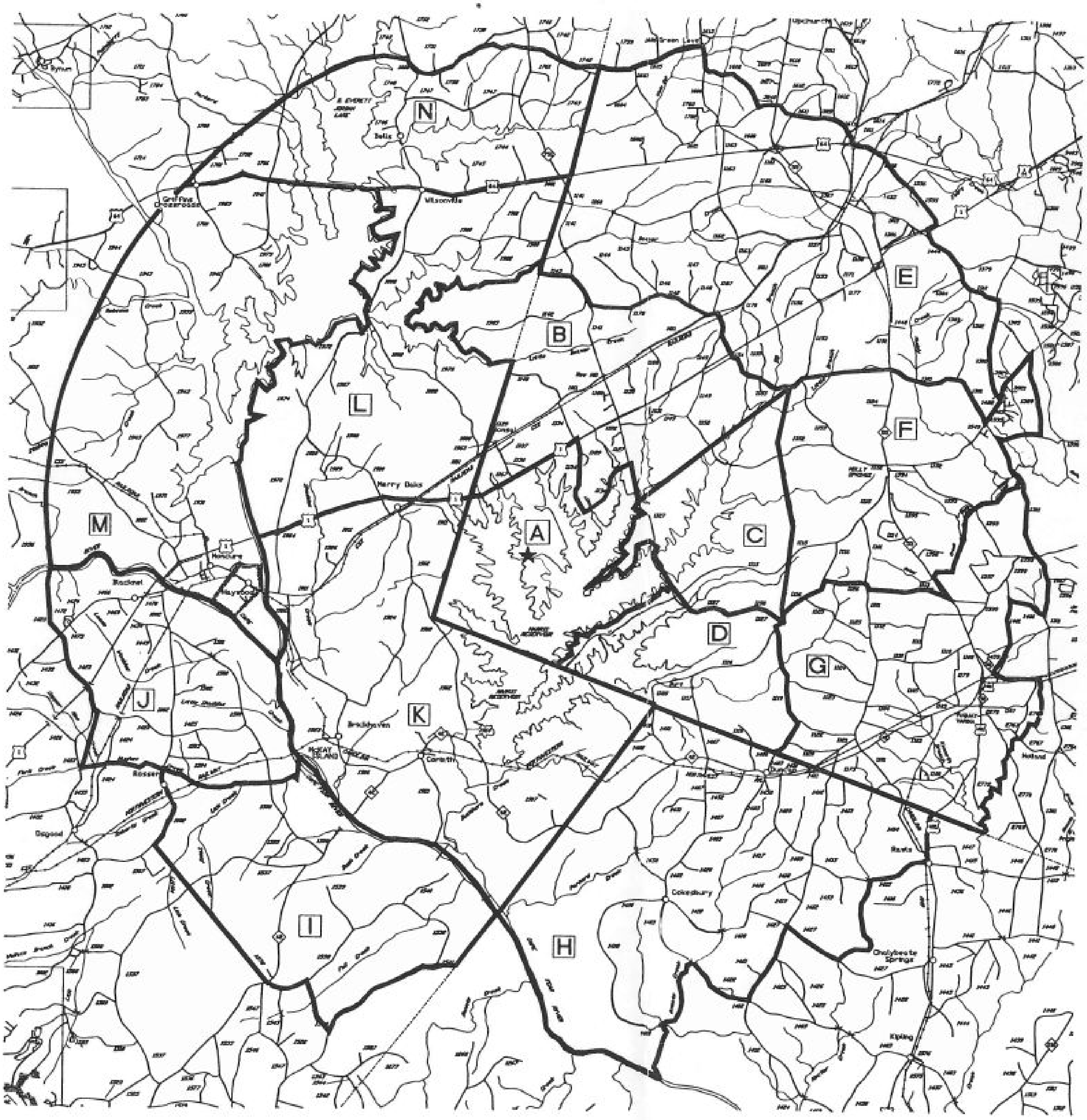
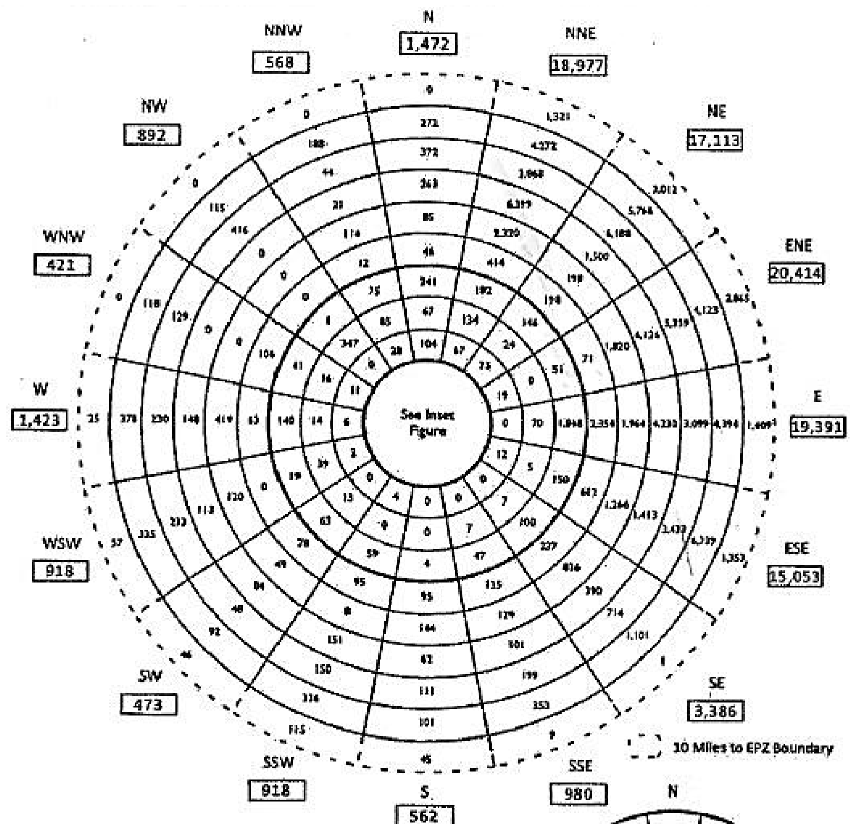


FIGURE 2.1.3-5

PERMANENT RESIDENT POPULATION BY SECTOR



Resident Population

Miles	Subtotal by Ring	Cumulative Total
0 - 1	0	0
1 - 2	65	65
2 - 3	328	393
3 - 4	828	1,221
4 - 5	3,167	4,388
5 - 6	4,586	8,974
6 - 7	9,472	18,446
7 - 8	21,001	39,447
8 - 9	24,573	64,020
9 - 10	28,683	92,703
10 - EPZ	10,258	102,961
	Total	102,961

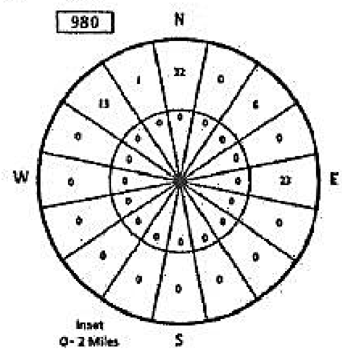
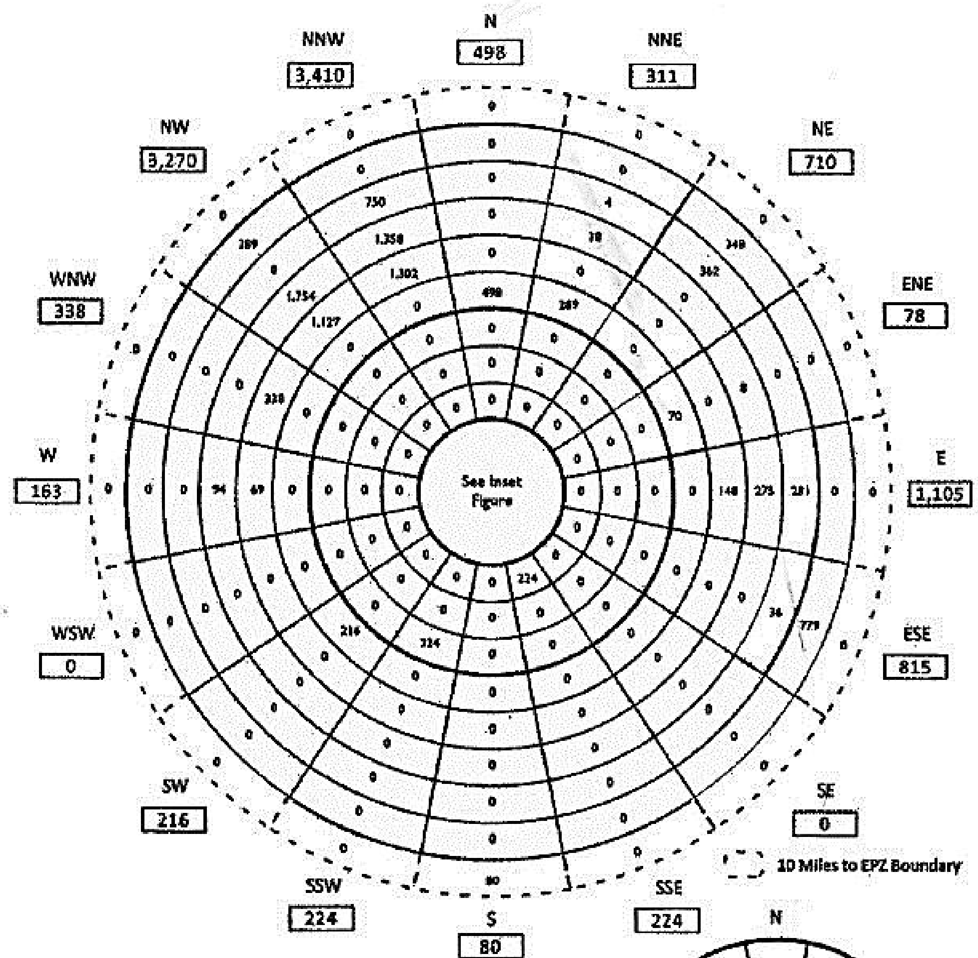


FIGURE 2.1.3-6

TRANSIENT POPULATION BY SECTOR



Transients

Miles	Subtotal by Ring	Cumulative Total
0 - 1	0	0
1 - 2	401	401
2 - 3	224	625
3 - 4	0	625
4 - 5	224	849
5 - 6	1,073	1,922
6 - 7	2,984	4,906
7 - 8	3,507	8,413
8 - 9	1,433	9,846
9 - 10	1,516	11,362
10 - EPZ	80	11,442
	Total:	11,442

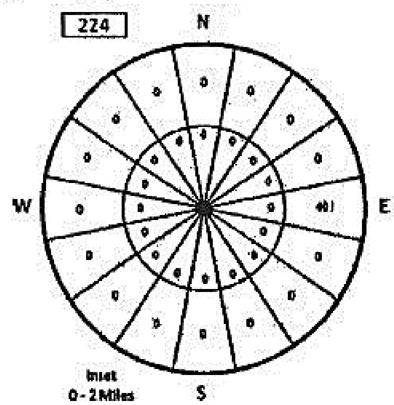


FIGURE 2.2.2-1

LOCATION OF MINING OPERATIONS, GENERAL AVIATION AIRPORTS, AND MAJOR AIRWAYS WITHIN A 10-MILE RADIUS OF THE SHEARON HARRIS NUCLEAR POWER PLANT

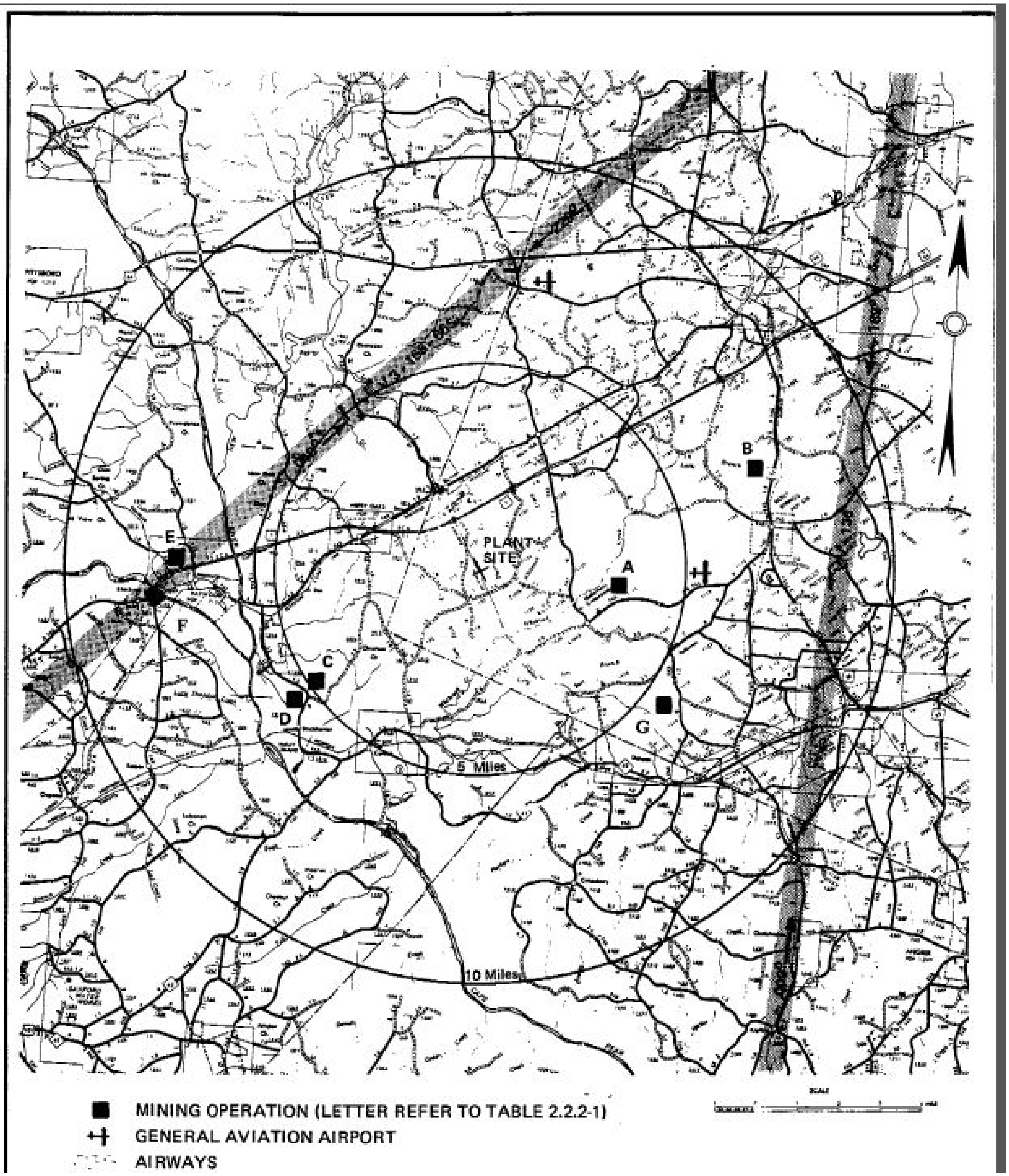
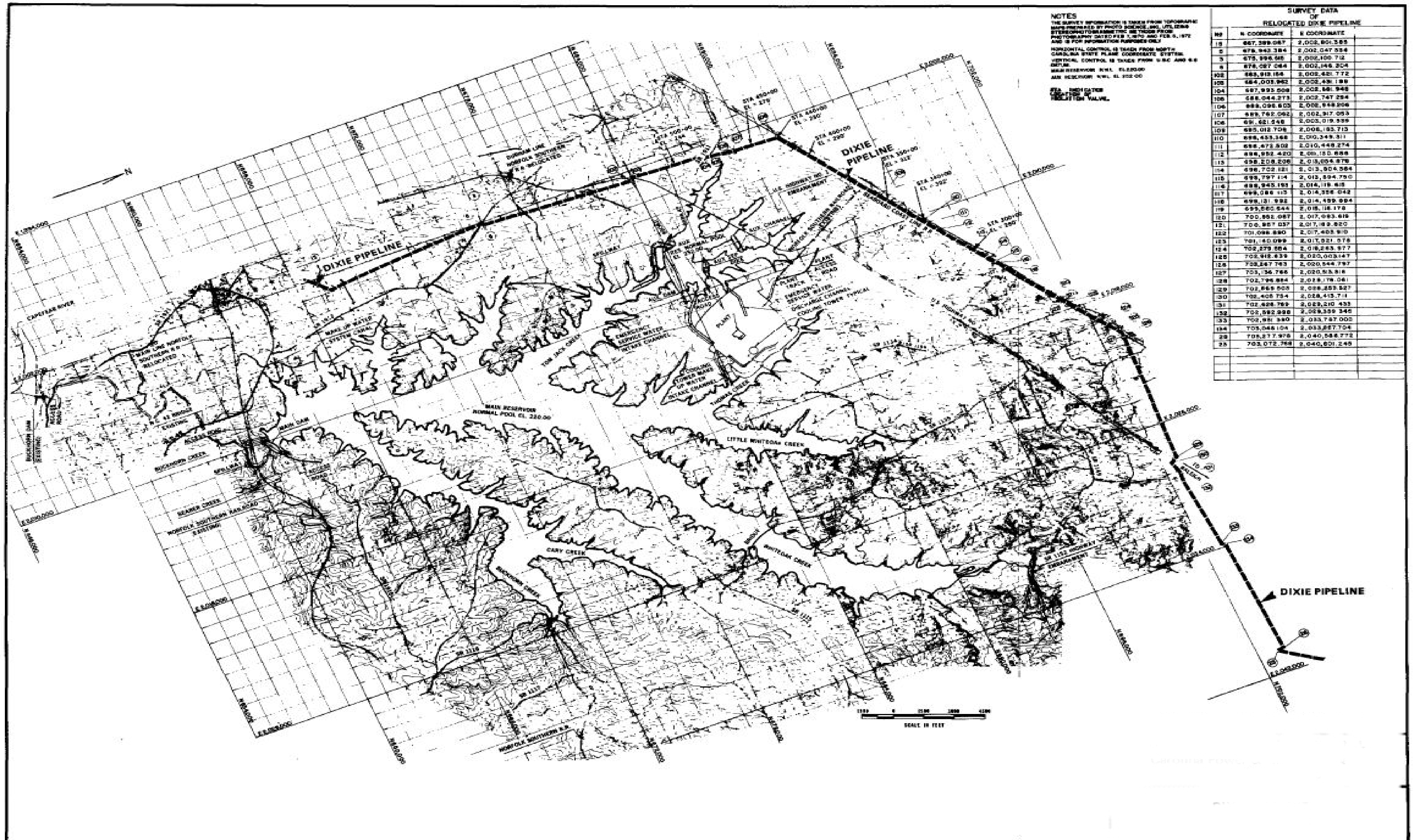


FIGURE 2.2.3-1

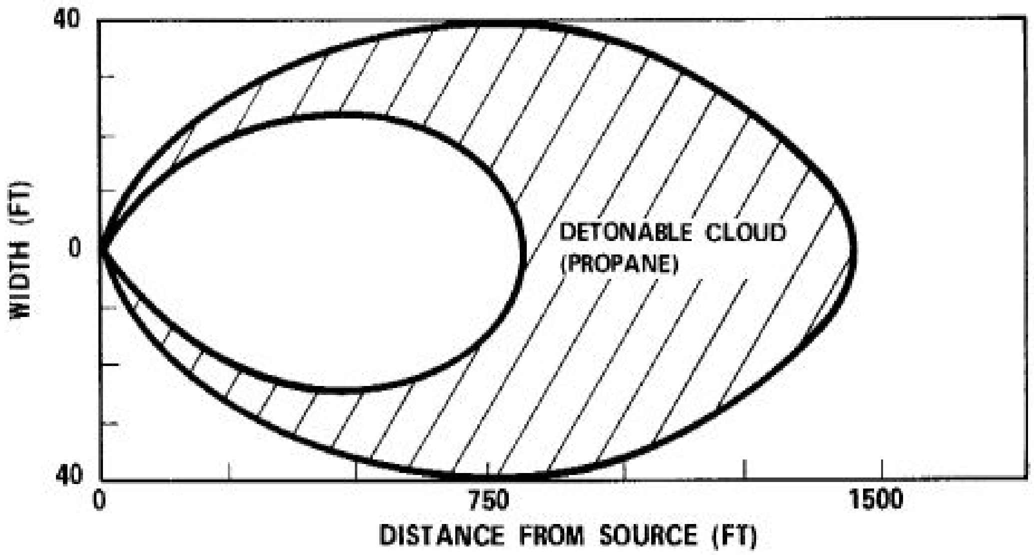
DIXIE PIPELINE



SURVEY DATA OF RELOCATED DIXIE PIPELINE		
NO	N COORDINATE	E COORDINATE
15	667,389.067	2,002,801.385
5	675,942.384	2,002,047.384
3	675,996.615	2,002,100.712
6	676,027.064	2,002,146.204
102	683,913.154	2,002,421.772
103	684,003.962	2,002,431.199
104	687,993.508	2,002,561.948
105	688,044.273	2,002,747.284
106	688,098.603	2,002,848.206
107	689,762.062	2,002,917.053
108	691,621.548	2,003,019.339
109	695,012.706	2,006,193.713
110	698,433.368	2,010,349.311
111	696,472.502	2,010,446.274
112	696,952.420	2,011,130.688
113	698,208.206	2,013,054.878
114	696,702.121	2,013,904.584
115	699,797.114	2,013,594.750
116	698,945.193	2,014,119.615
117	699,086.113	2,014,356.042
118	699,131.992	2,014,459.984
119	699,560.544	2,015,116.178
120	700,862.087	2,017,063.619
121	700,957.037	2,017,189.620
122	701,098.890	2,017,403.910
123	701,140.099	2,017,521.578
124	702,279.884	2,018,263.977
125	702,912.639	2,020,003.147
126	703,247.763	2,020,544.797
127	703,136.766	2,020,515.516
128	702,796.884	2,028,178.061
129	702,865.505	2,028,253.527
130	702,405.734	2,028,413.711
131	702,426.769	2,029,210.433
132	702,582.998	2,029,359.345
133	702,951.380	2,033,757.002
134	703,048.104	2,033,827.704
29	703,277.976	2,040,586.772
25	703,072.768	2,040,601.248

FIGURE 2.2.3-2

DETONABLE CLOUD (ONLY VAPOR FLASHED AT BREAK)



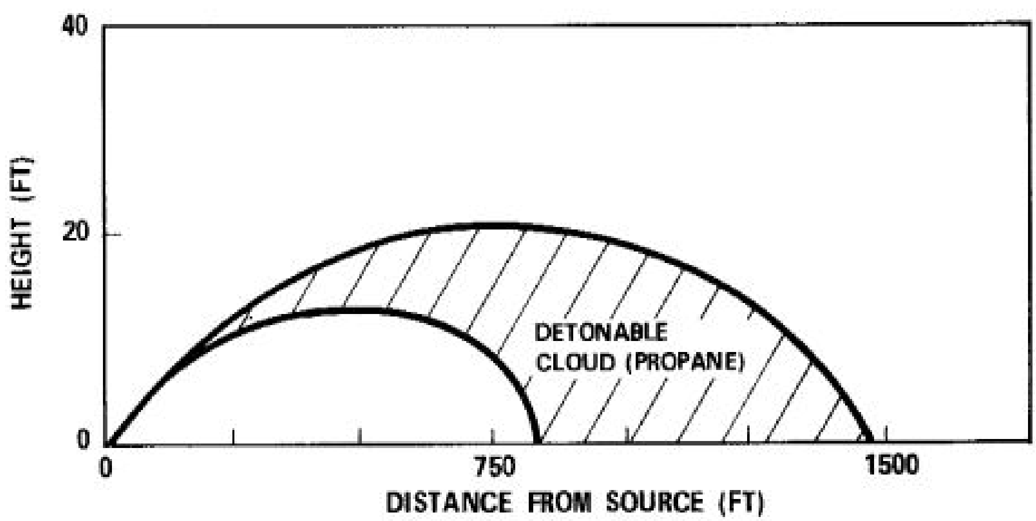


FIGURE 2.2.3-3

MAXIMUM DETONABLE CLOUD (ALL LIQUID VAPORIZED)

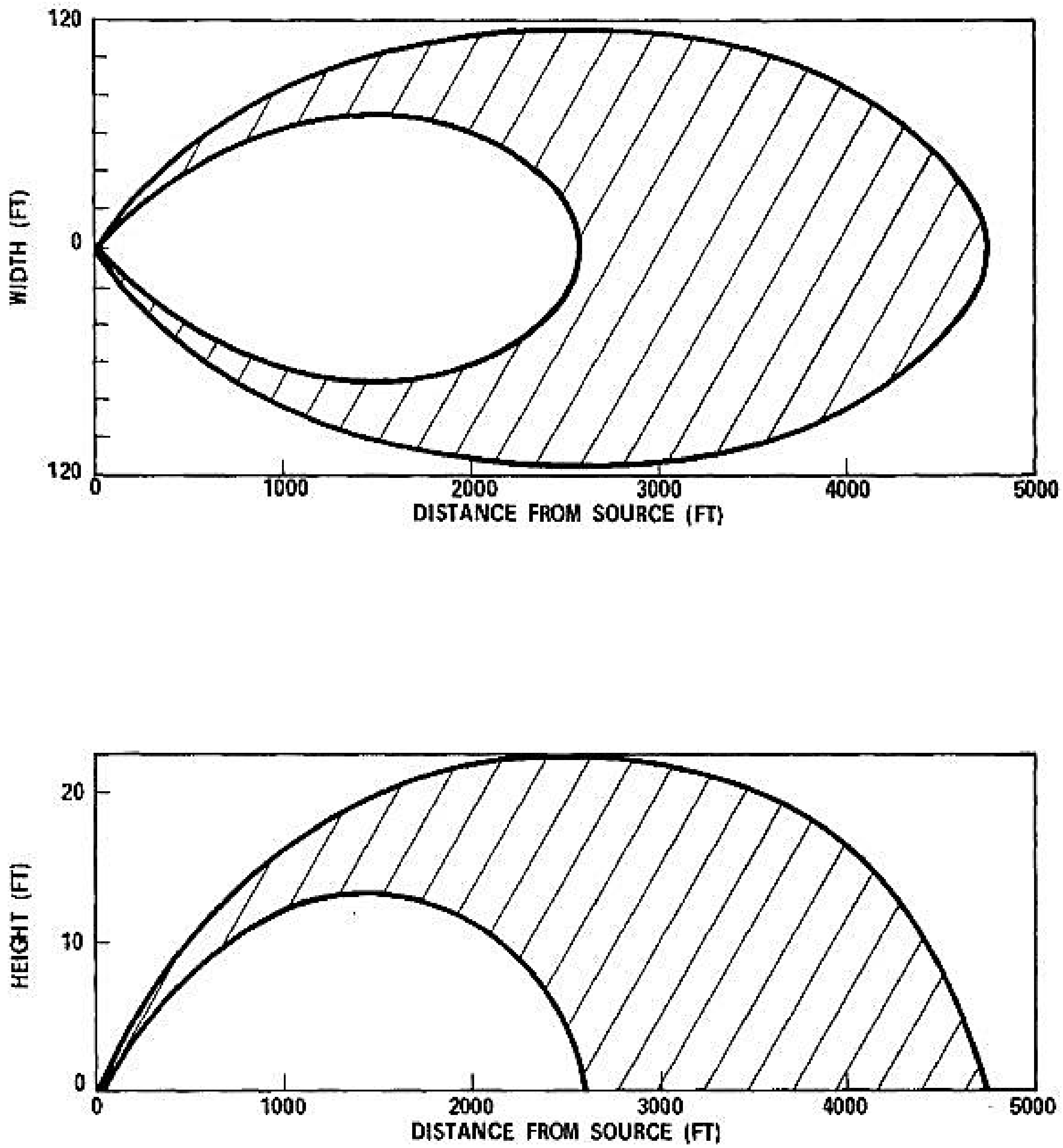


FIGURE 2.2.3-4

BLAST WAVE PARAMETER VS SCALED DISTANCE

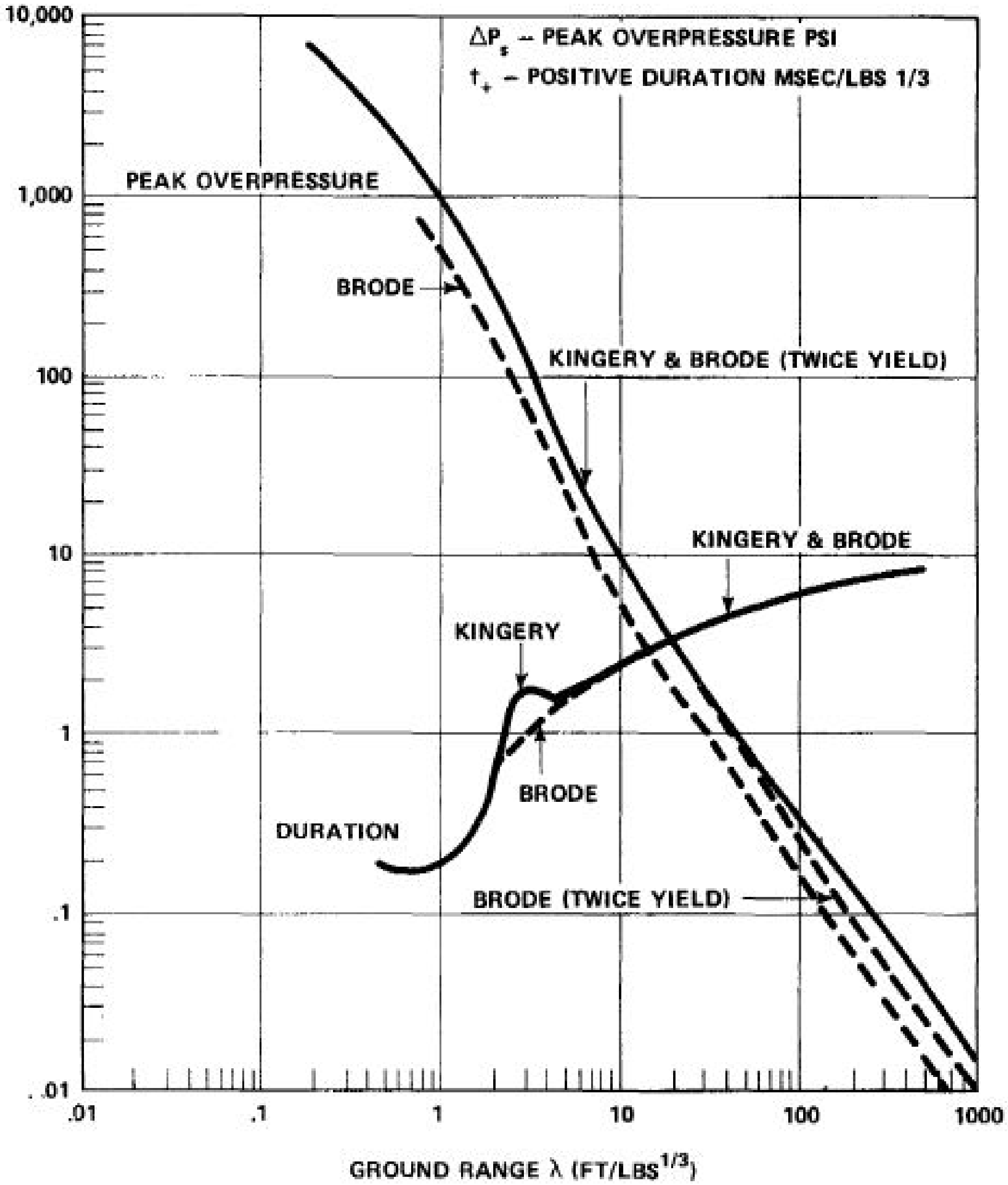


FIGURE 2.2.3-5

ENTRAINMENT COEFFICIENT & US DENSIMETRIC FROUDE NO
(FROM REFERENCE 2.2.3-15)

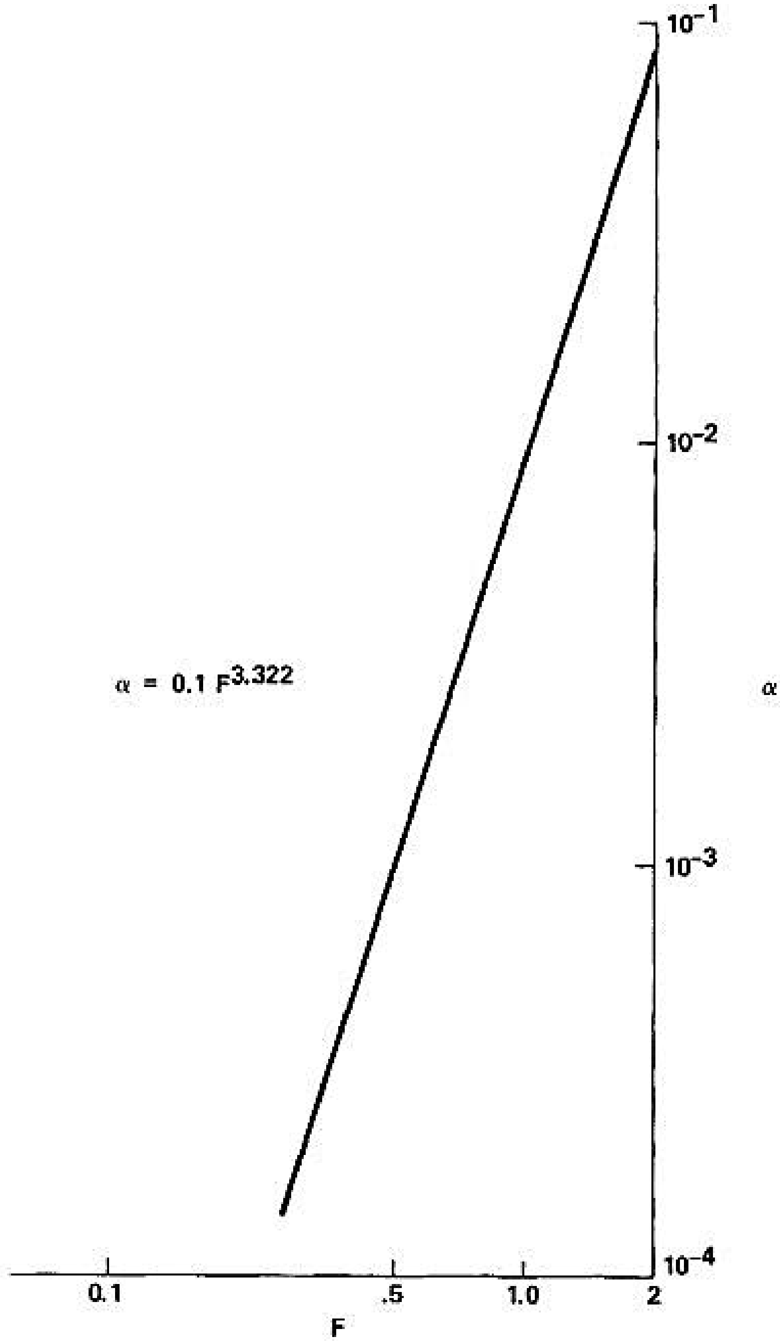


FIGURE 2.2.3-6

100 LB/SEC BREAK GRAVITY SLUMPING MODEL RESULTS

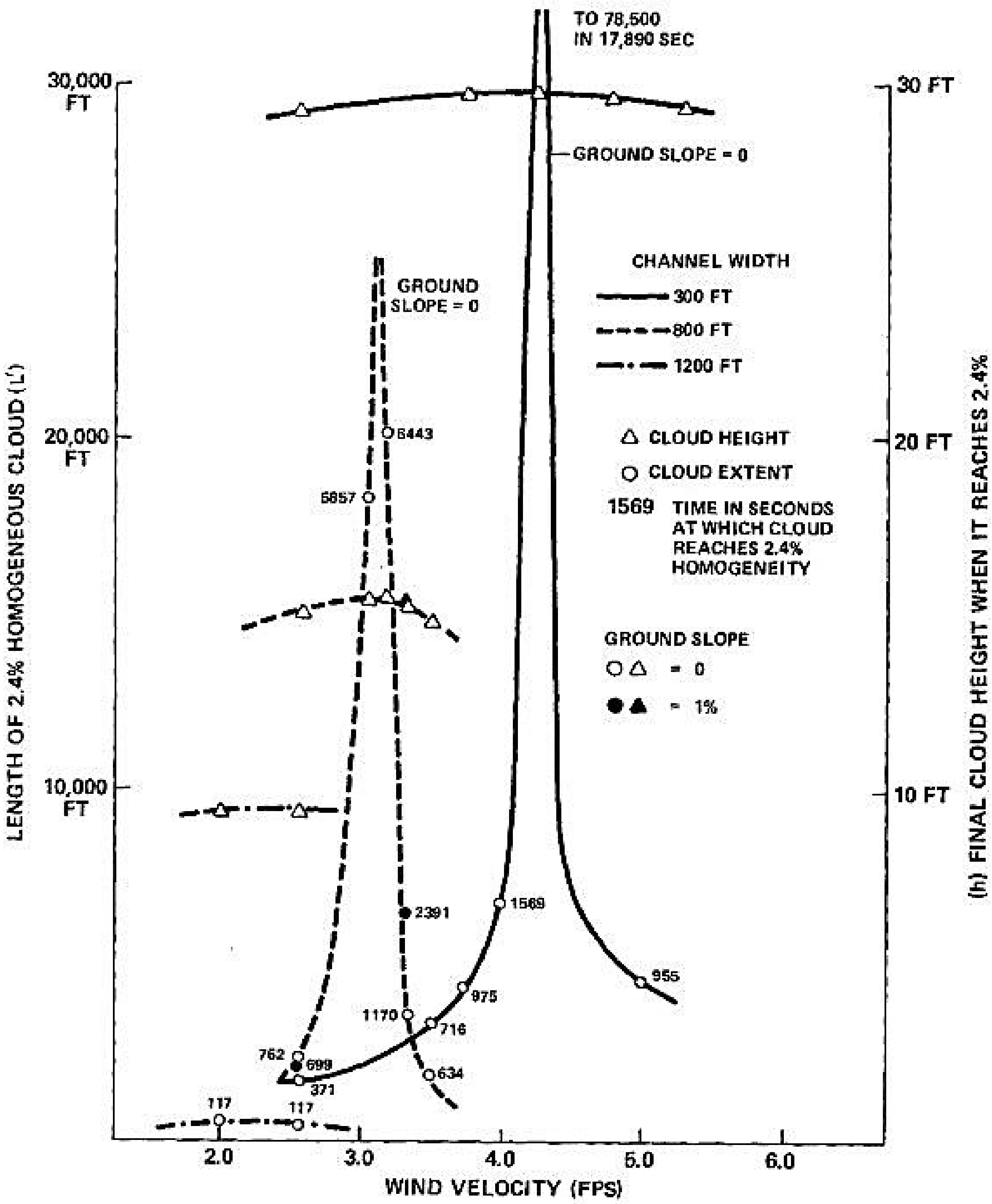


FIGURE 2.2.3-7

UPWIND DISTANCE OF 100 LB/SEC SOURCE OF WIDTH L GIVING 2.4% CONCENTRATION

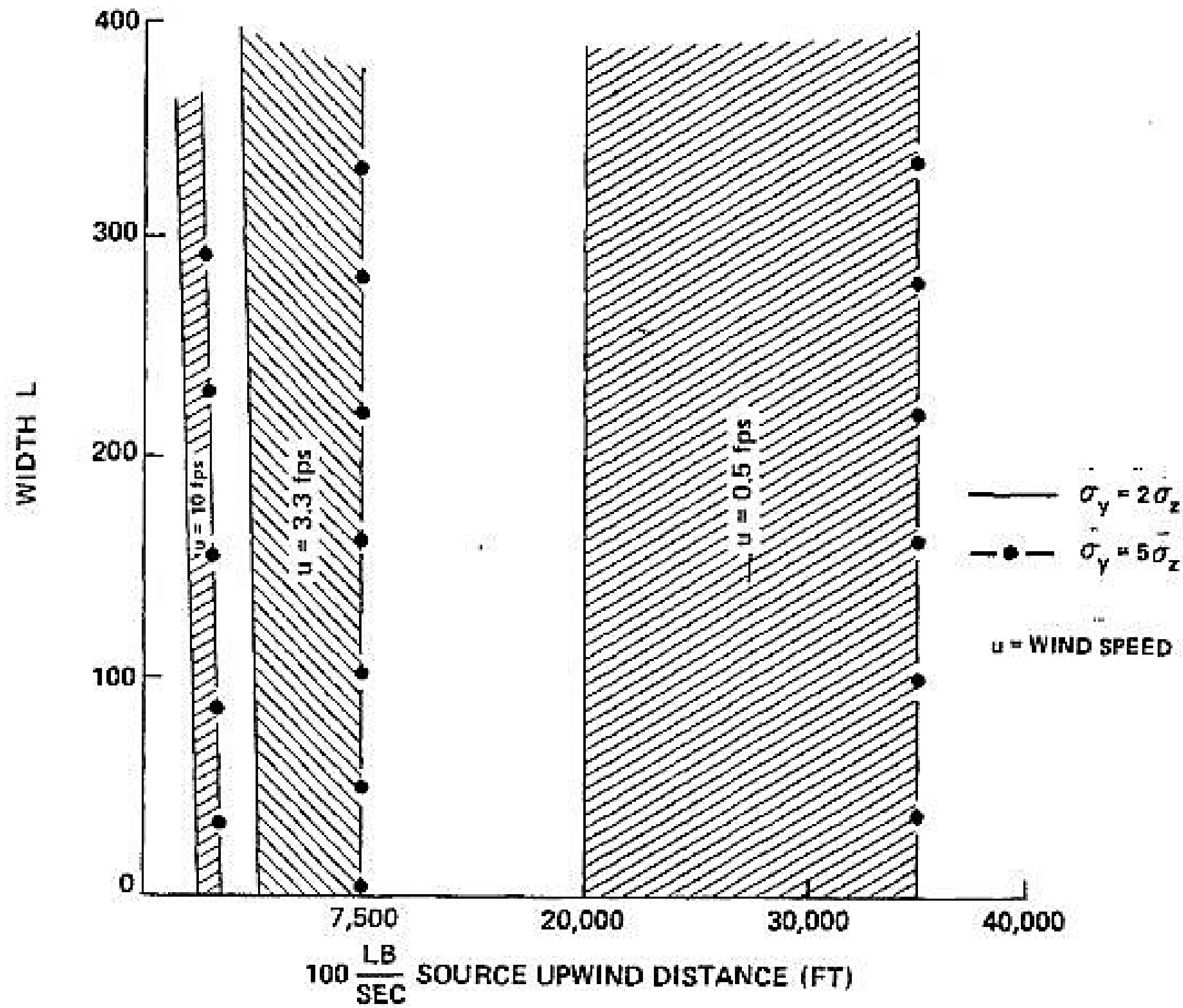
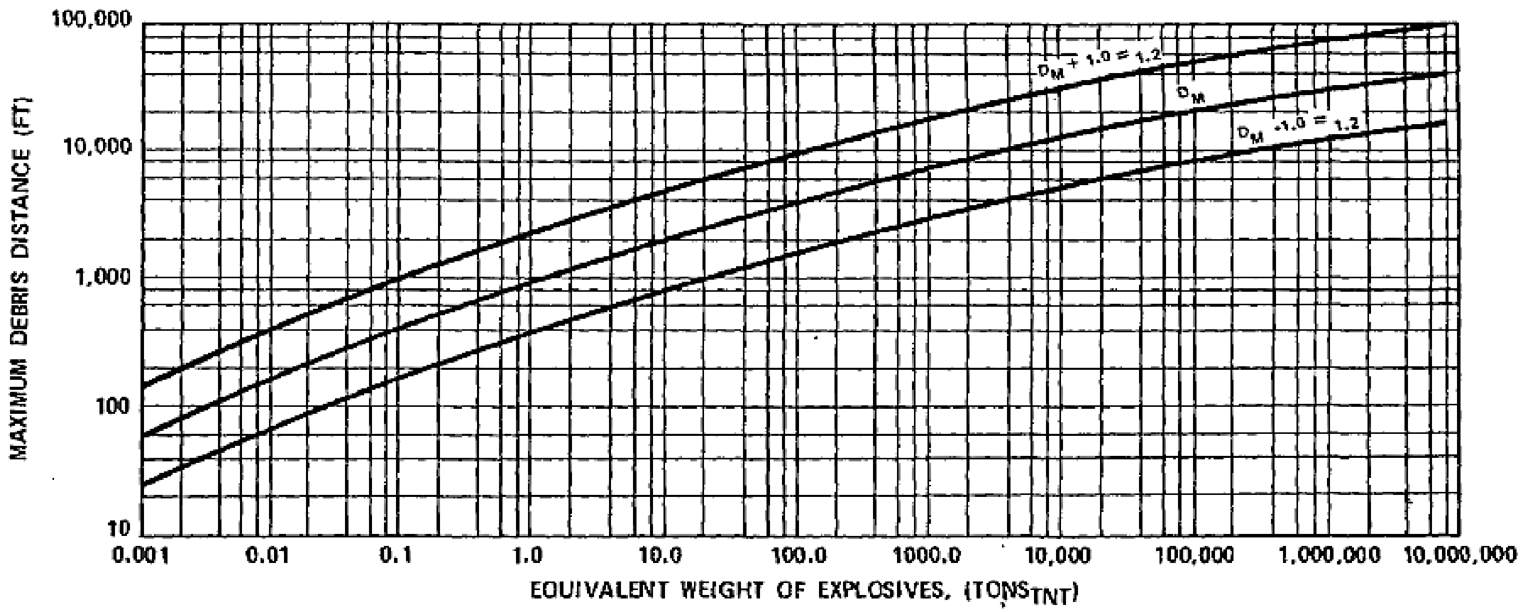


FIGURE 2.2.3-8

QUADRATIC REGRESSION LINE MAXIMUM DEBRIS DISTANCE VS EQUIVALENT YIELD



$$LOG_{10} D_M = 2.960 + 0.347 \; LOG_{10} W - 0.0161 \; (LOG_{10} W)^2$$

STANDARD ERROR = ±0.392 (LOGARITHMIC VALUE)

= ÷2.47 (ARITHMETIC VALUE)

CORRELATION COEFFICIENT = 0.67

FIGURE 2.3.1-1

VERTICAL PROFILE AT THE EXTREME MILE WIND

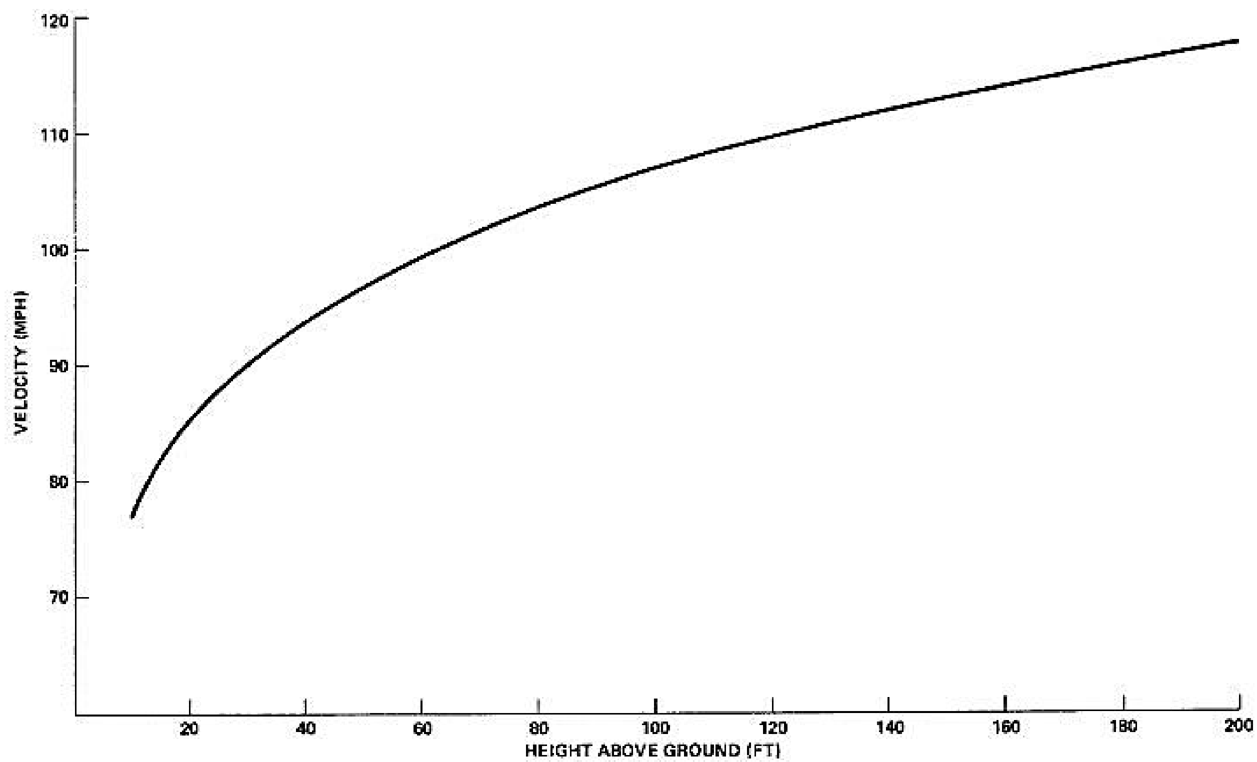


FIGURE 2.3.1-2

RECURRENCE PERIOD OF FASTEST 1 MIN. EXTREME WIND FOR SITE

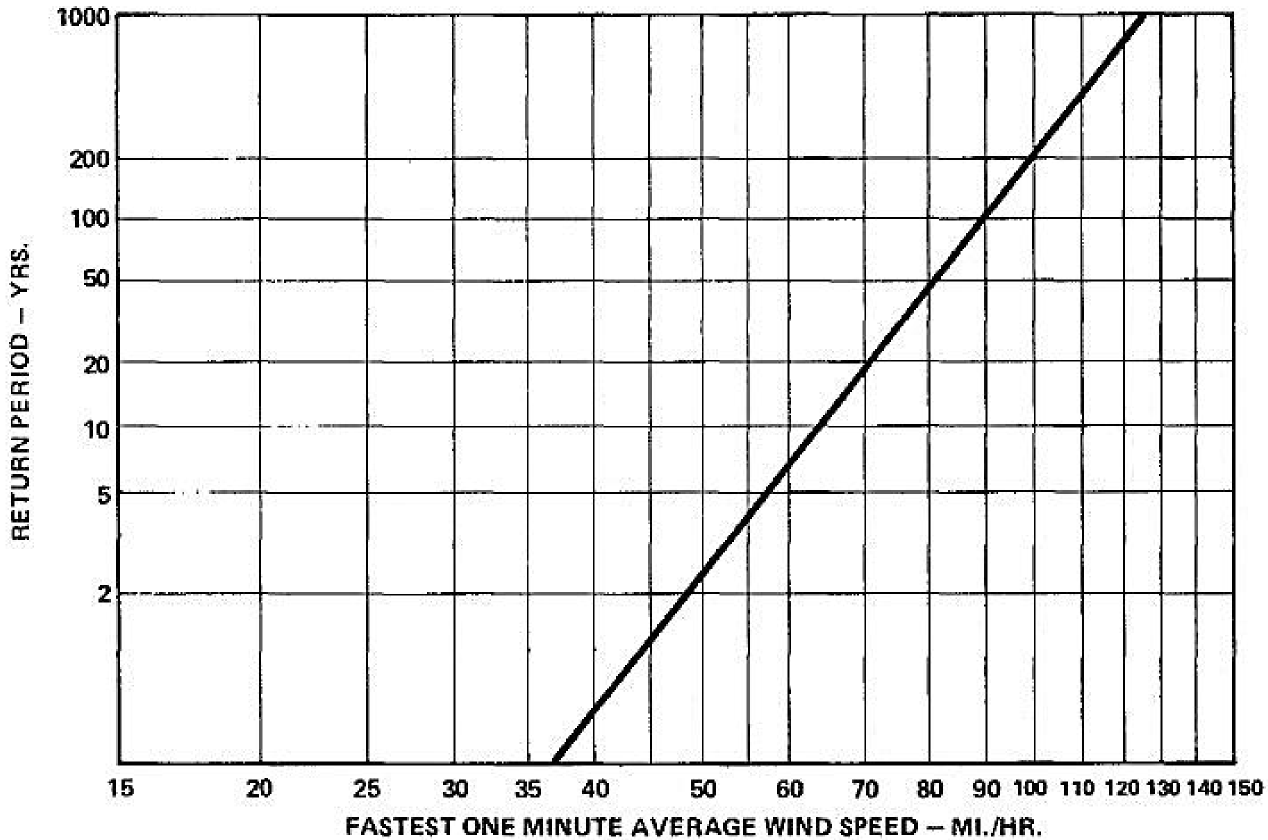
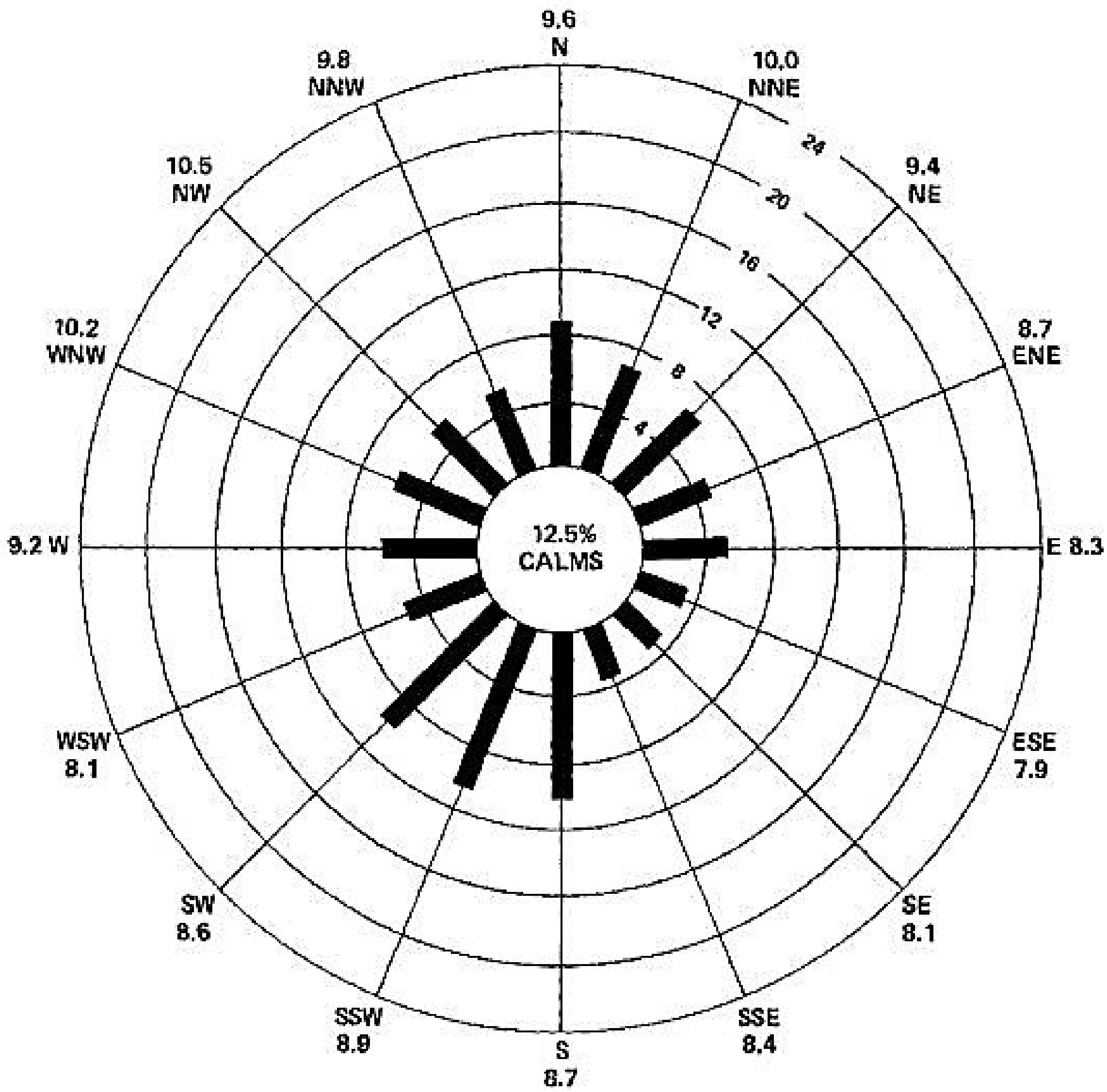


FIGURE 2.3.2-1

RALEIGH-DURHAM WEATHER SERVICE ALL WEATHER WIND ROSE



$\bar{U}$ = 7.9

$\bar{U}$ = ALL DIRECTIONAL AVERAGE WIND SPEED

NOTE: DIRECTIONAL AVERAGE WIND SPEEDS (MPH) ARE DISPLAYED RADIALLY

WIND DIRECTION (%)

1955-1964

FIGURE 2.3.2-2

RALEIGH-DURHAM WEATHER SERVICE SEASONAL WIND ROSE

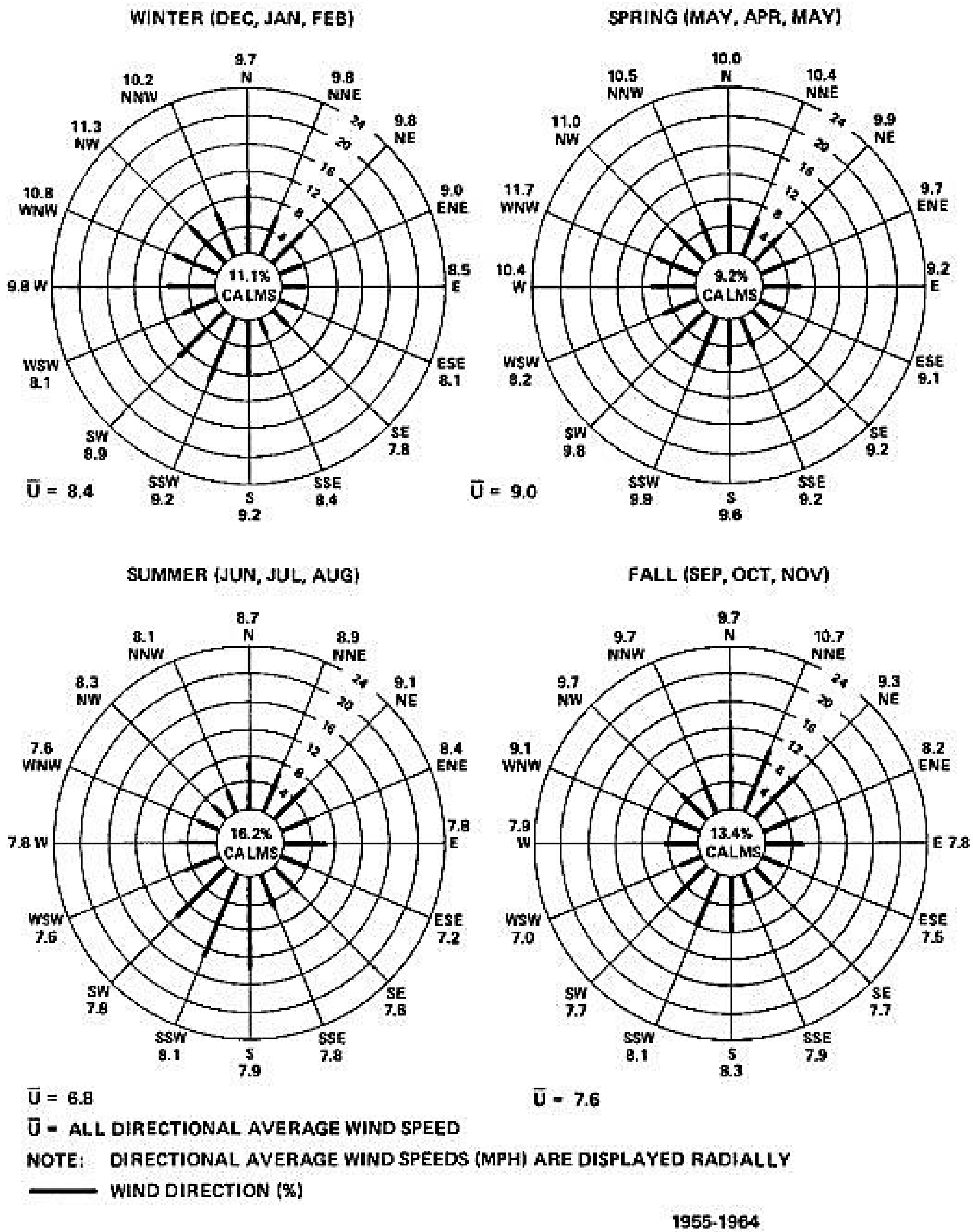
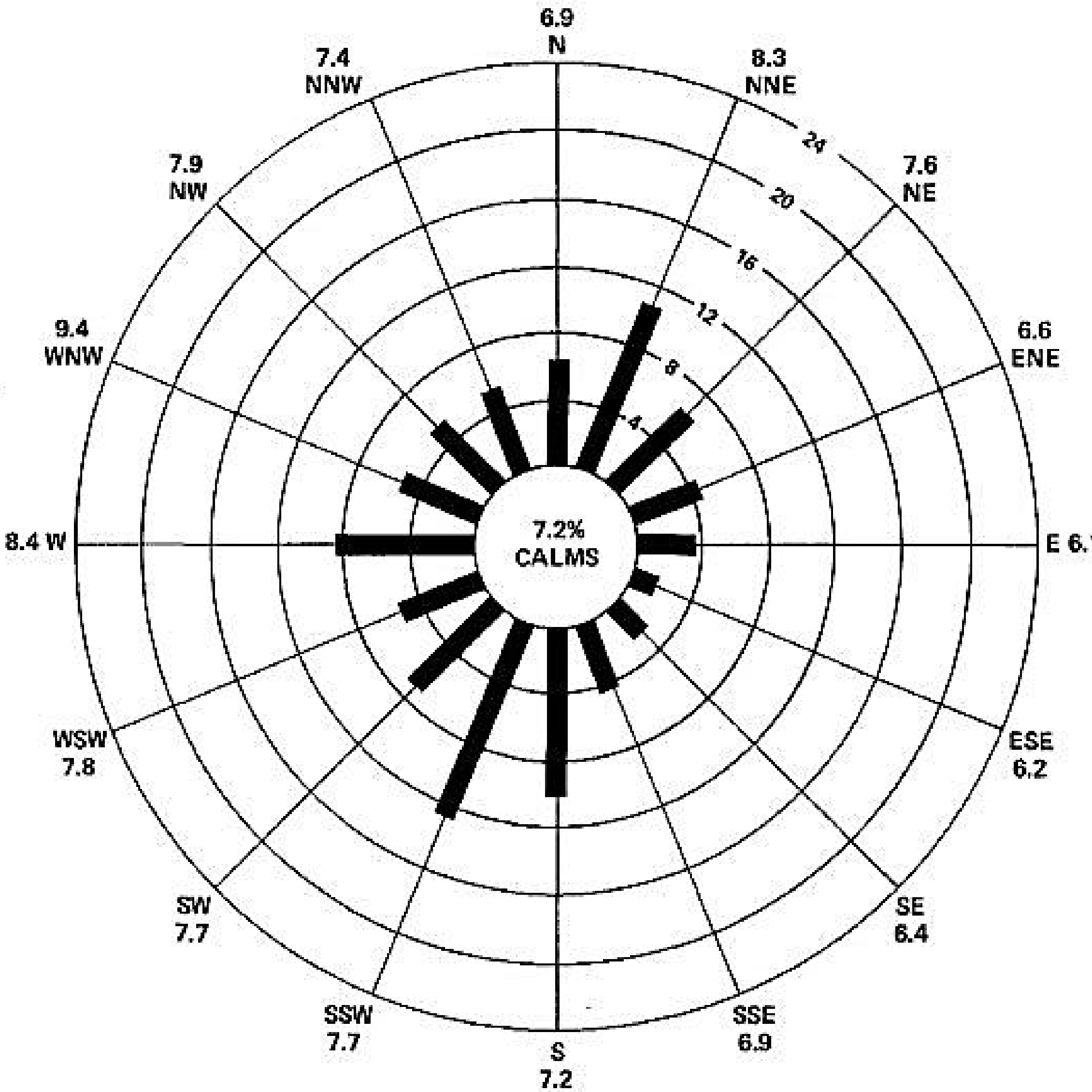


FIGURE 2.3.2-3

GREENSBORO ALL WEATHER WIND ROSE



$\bar{U}$ = 7.0

$\bar{U}$ = ALL DIRECTIONAL AVERAGE WIND SPEED

NOTE: DIRECTIONAL AVERAGE WIND SPEEDS (MPH ARE DISPLAYED RADIALLY

WIND DIRECTION (%)

1960-1970

FIGURE 2.3.2-4

GREENSBORO SEASONAL WIND ROSE

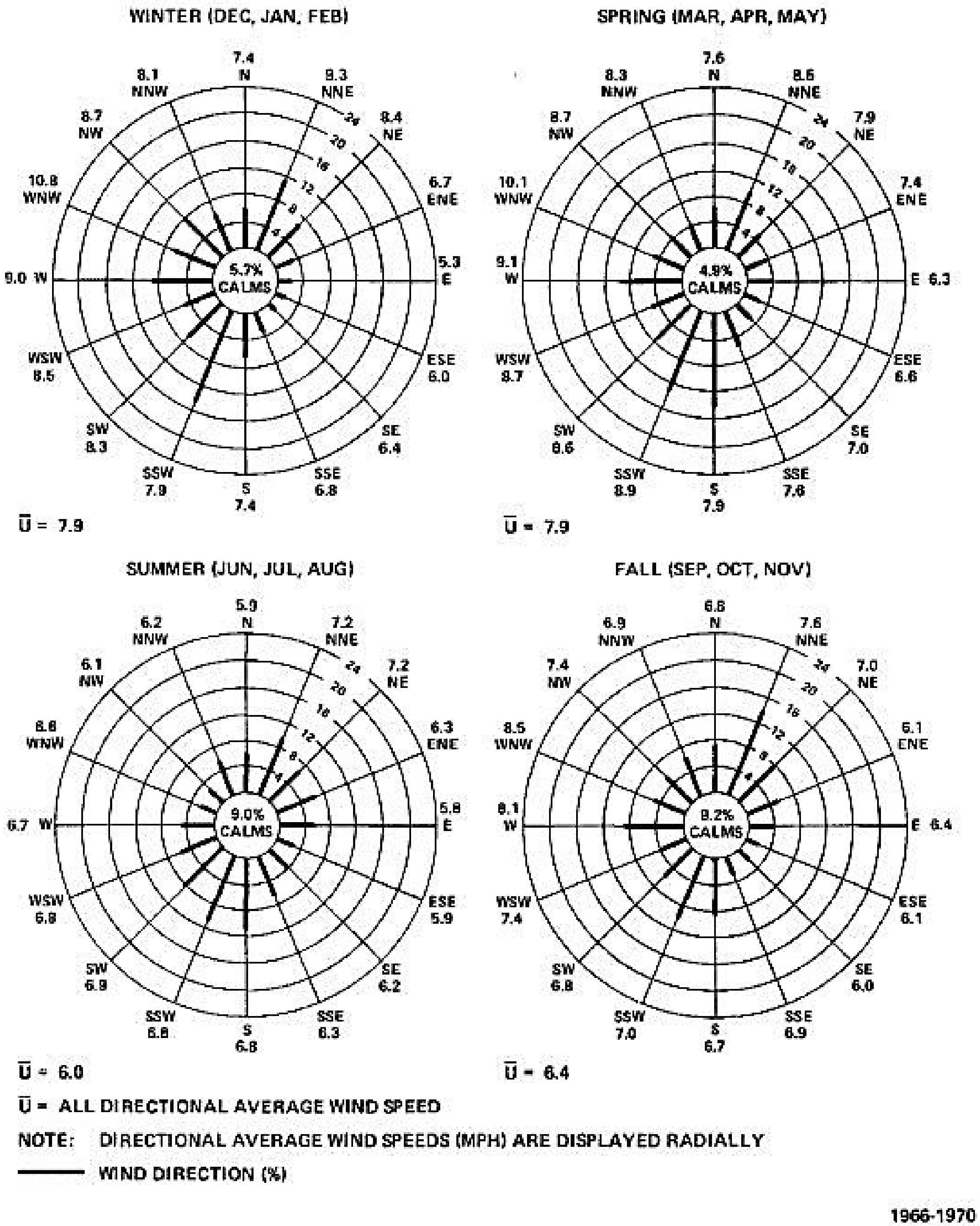
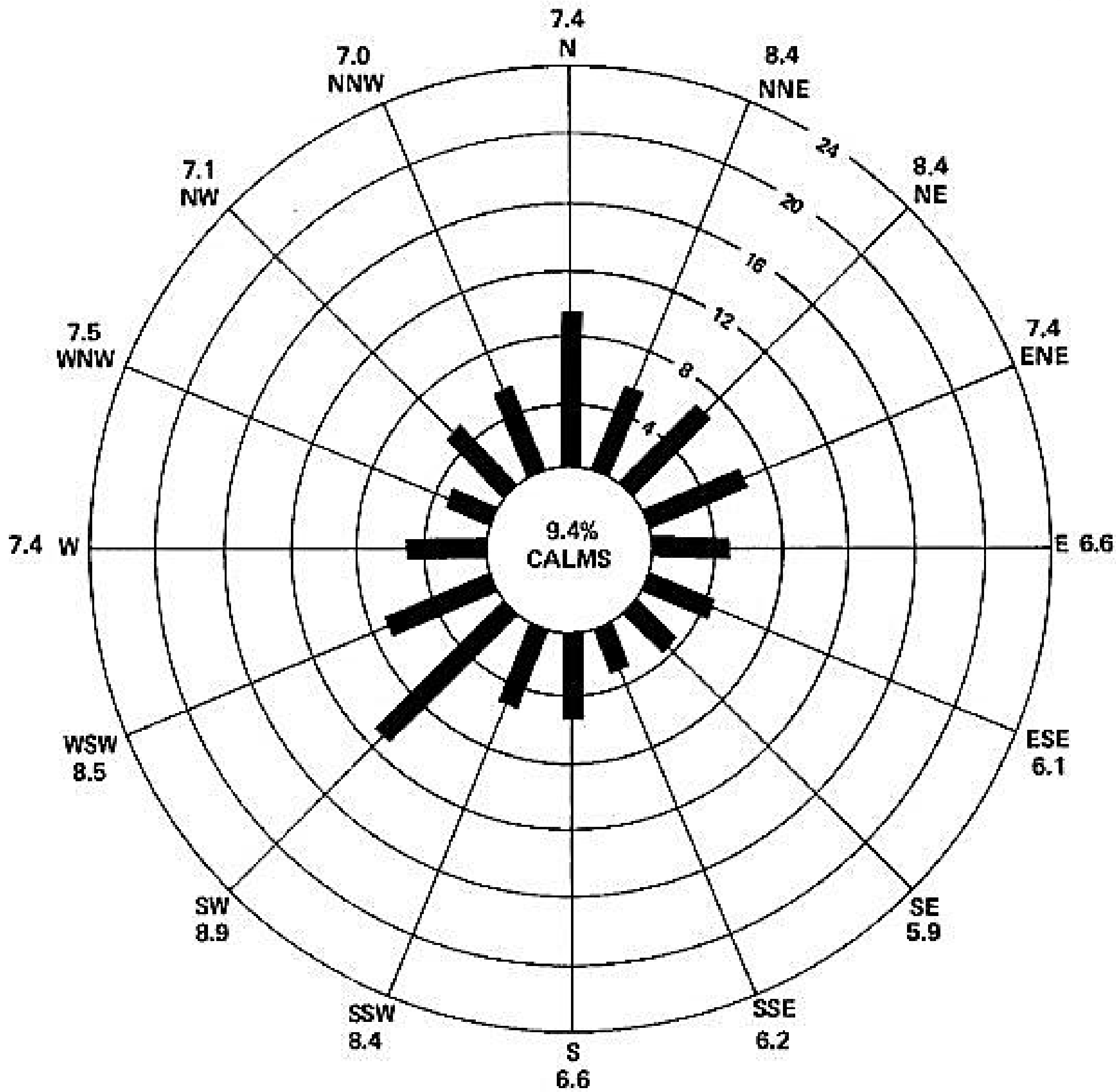


FIGURE 2.3.2-5

CHARLOTTE ALL WEATHER WIND ROSE



$\bar{U}$ = 6.9

$\bar{U}$ = ALL DIRECTIONAL AVERAGE WIND SPEED

NOTE: DIRECTIONAL AVERAGE WIND SPEEDS (MPH) ARE DISPLAYED RADIALLY

WIND DIRECTION %

1966-1970

FIGURE 2.3.2-6

CHARLOTTE SEASONAL WIND ROSE

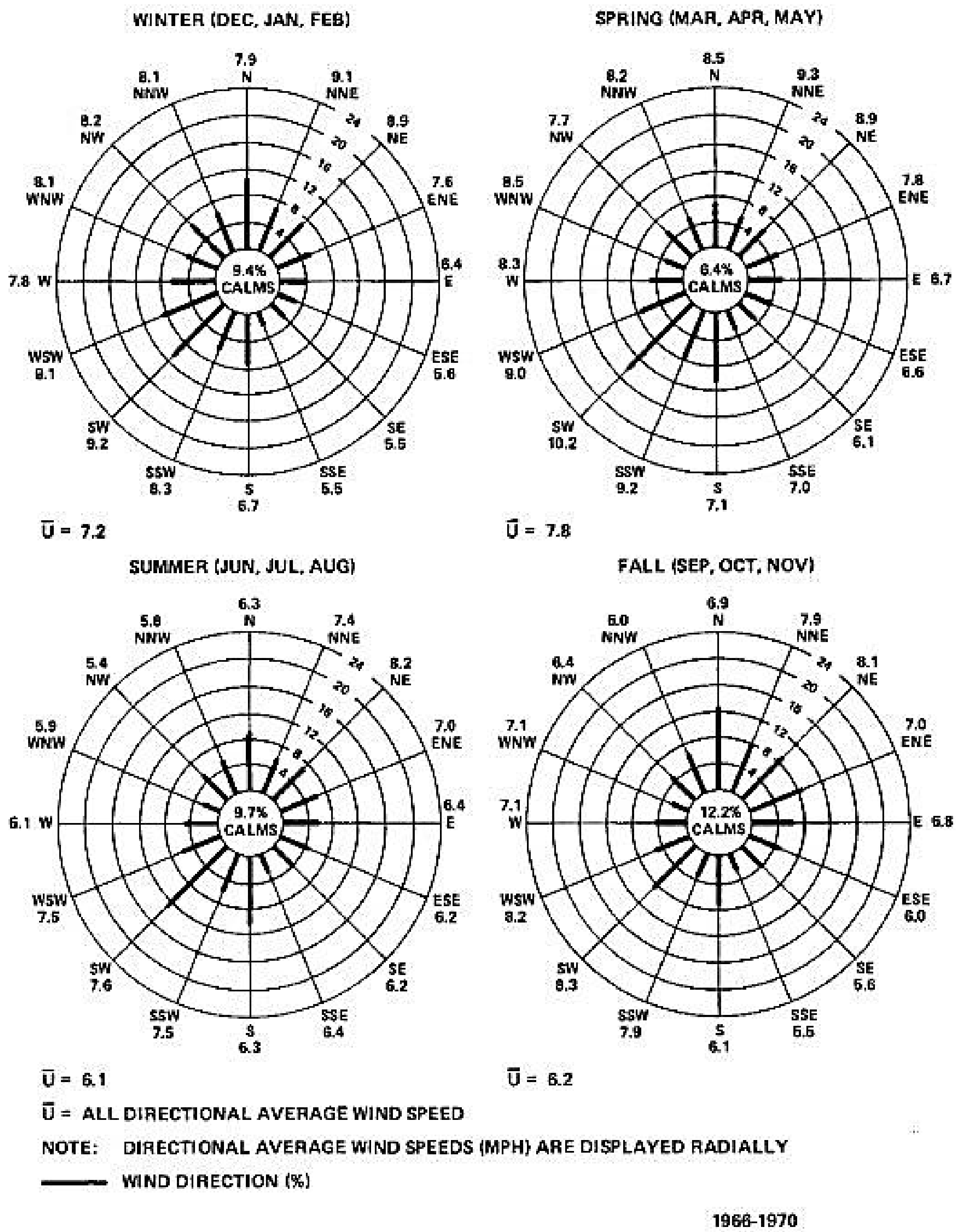
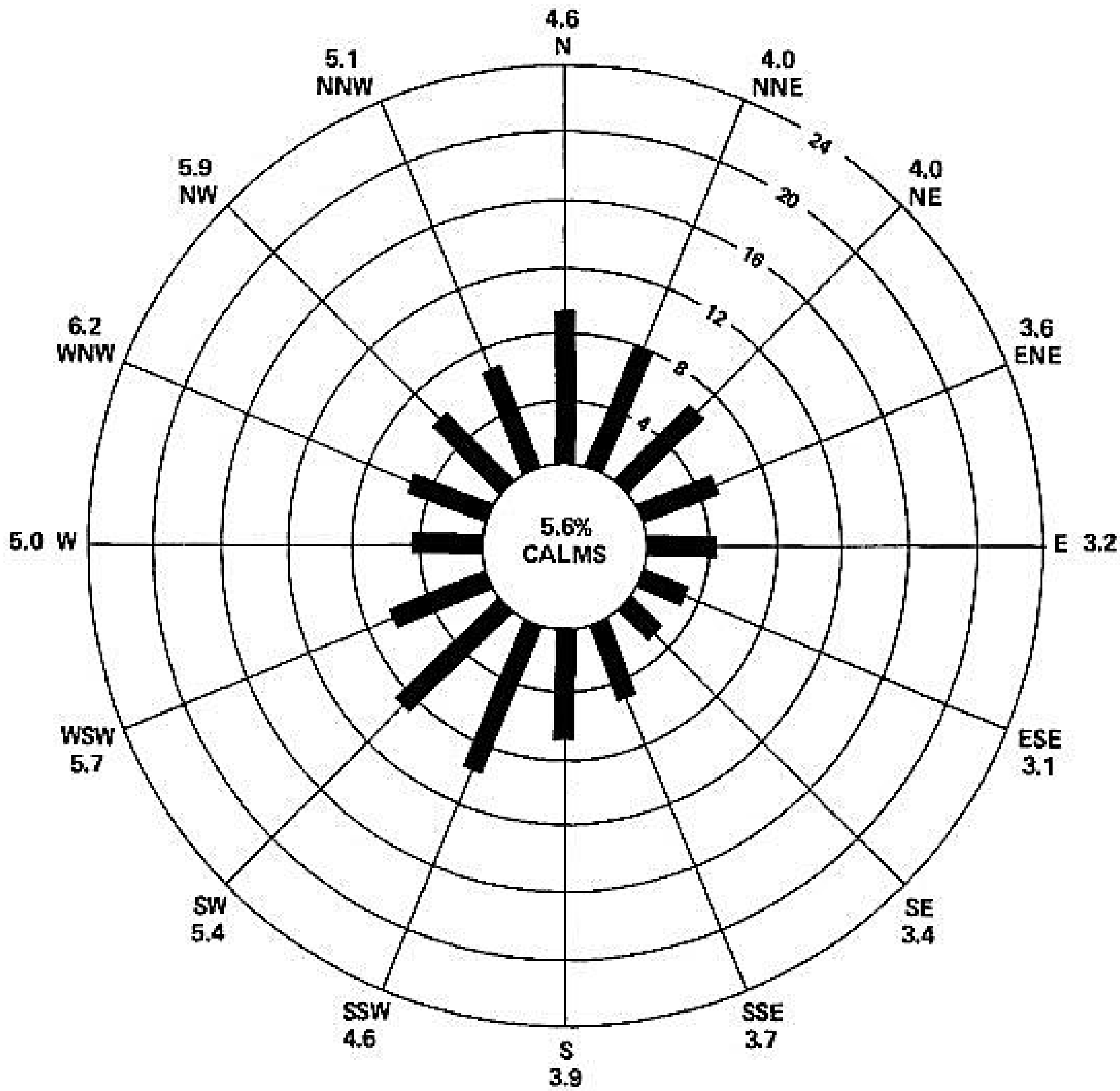


FIGURE 2.3.2-7

SHNPP SITE WIND ROSE – 12.5 METER LEVEL



$\bar{U}$ = 4.6 MPH

$\bar{U}$ = ALL DIRECTION AVERAGE WIND SPEED

NOTE: DIRECTIONAL AVERAGE WIND SPEEDS (MPH) ARE DISPLAYED RADIALLY

WIND DIRECTION (%)

1/14/76 – 12/31/78

FIGURE 2.3.2-8

SHNPP 12.5M LEVEL WIND PERSISTENCE PROBABILITY

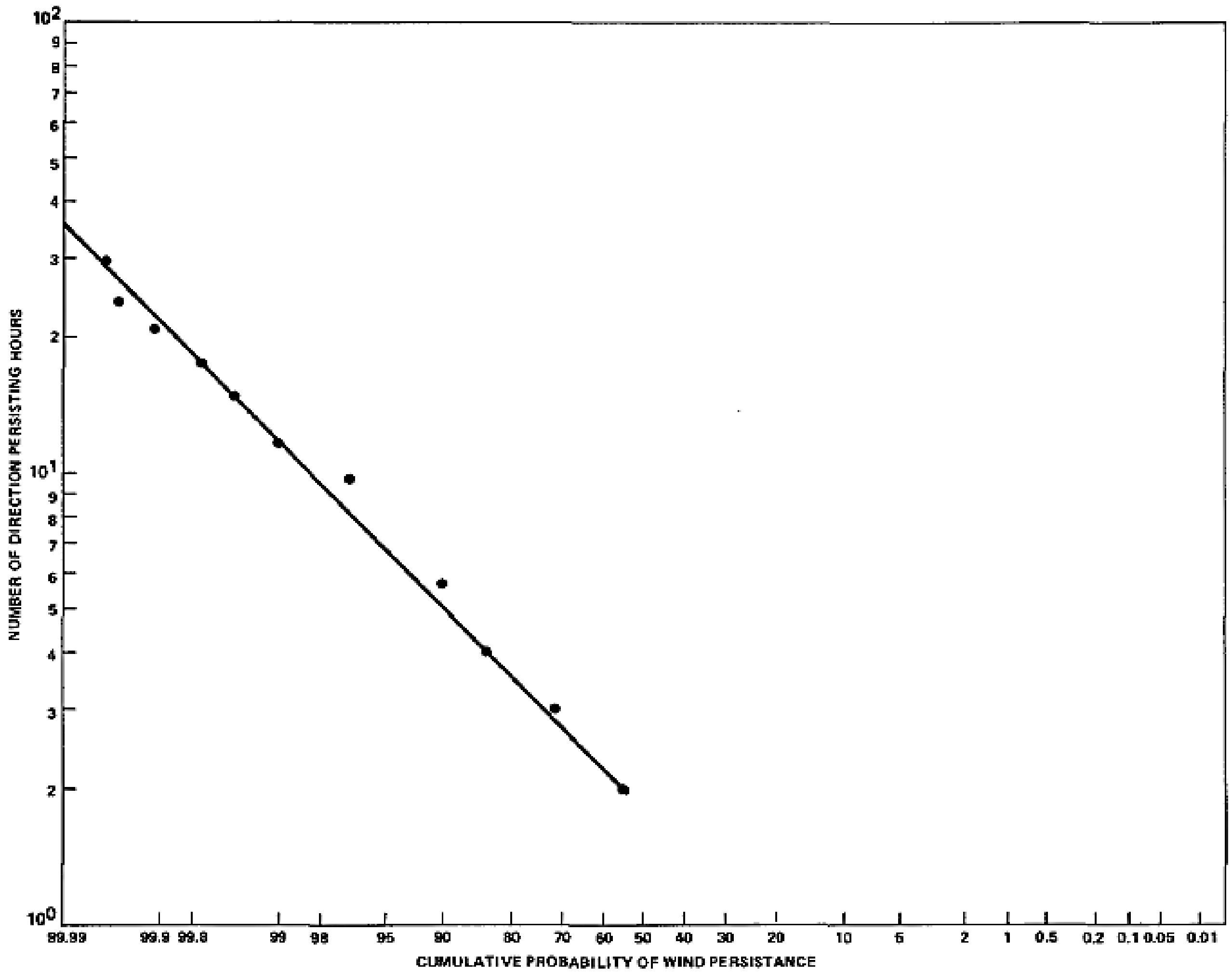
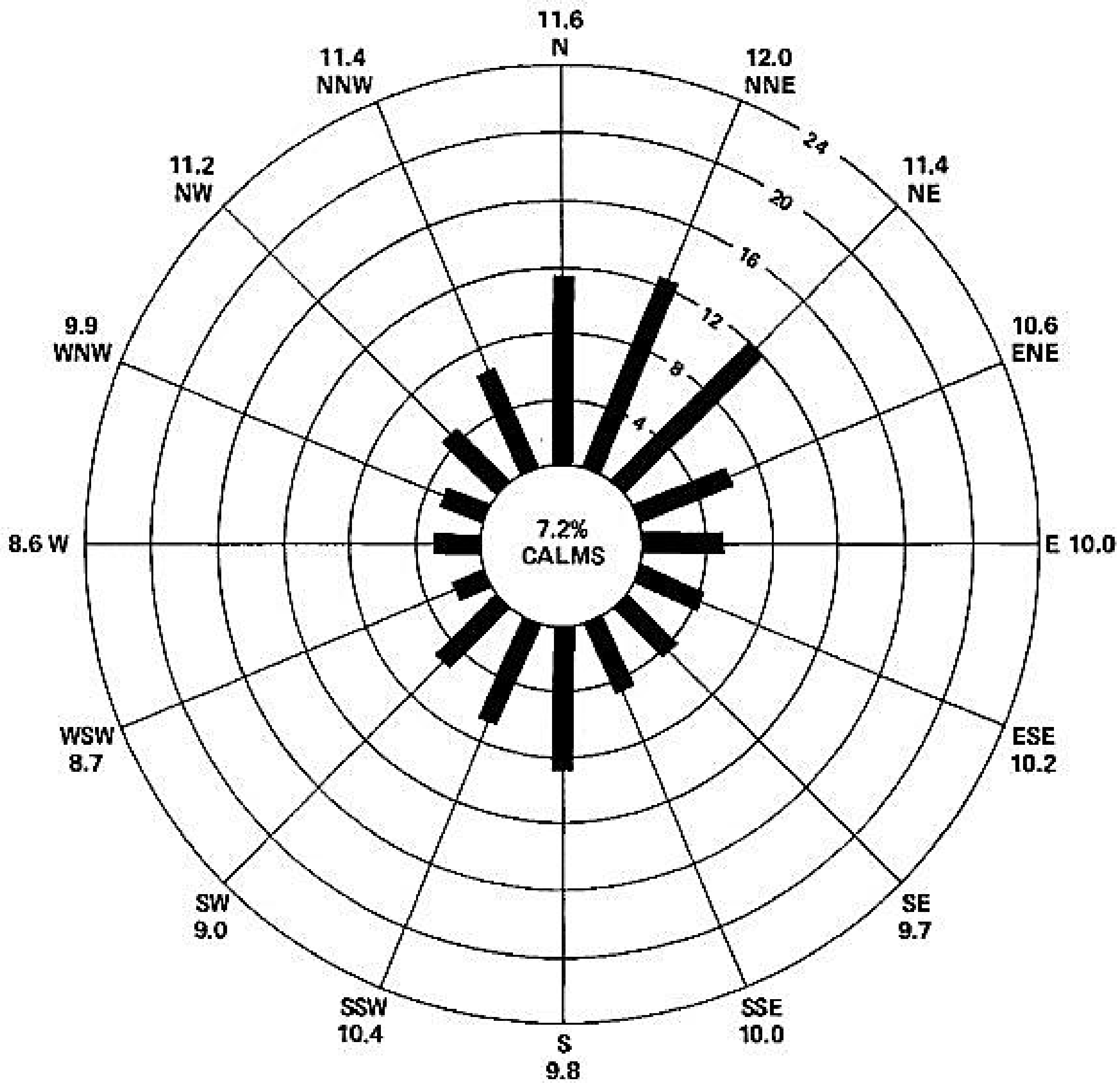


FIGURE 2.3.2-9

RALEIGH-DURHAM WEATHER SERVICE PRECIPITATION WIND ROSE



$\bar{U}$ = 9.9

$\bar{U}$ = ALL DIRECTIONAL AVERAGE WIND SPEED

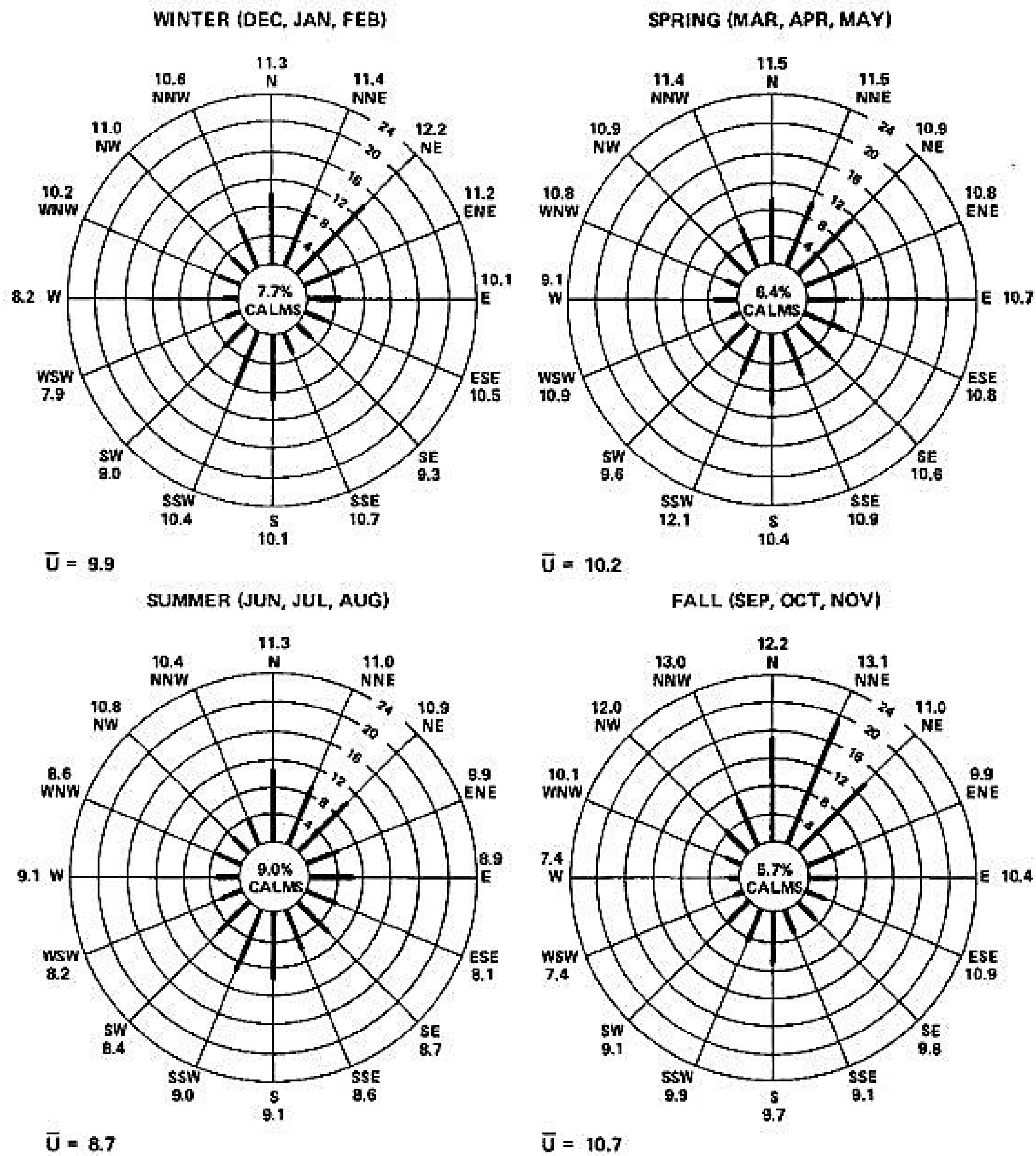
NOTE: DIRECTIONAL AVERAGE WIND SPEEDS (MPH) ARE DISPLAYED RADIALLY

▬ WIND DIRECTION (%)

1955-1964

FIGURE 2.3.2-10

RALEIGH-DURHAM WEATHER SERVICE SEASONAL PRECIPITATION WIND ROSE



U = ALL DIRECTIONAL AVERAGE WIND SPEED

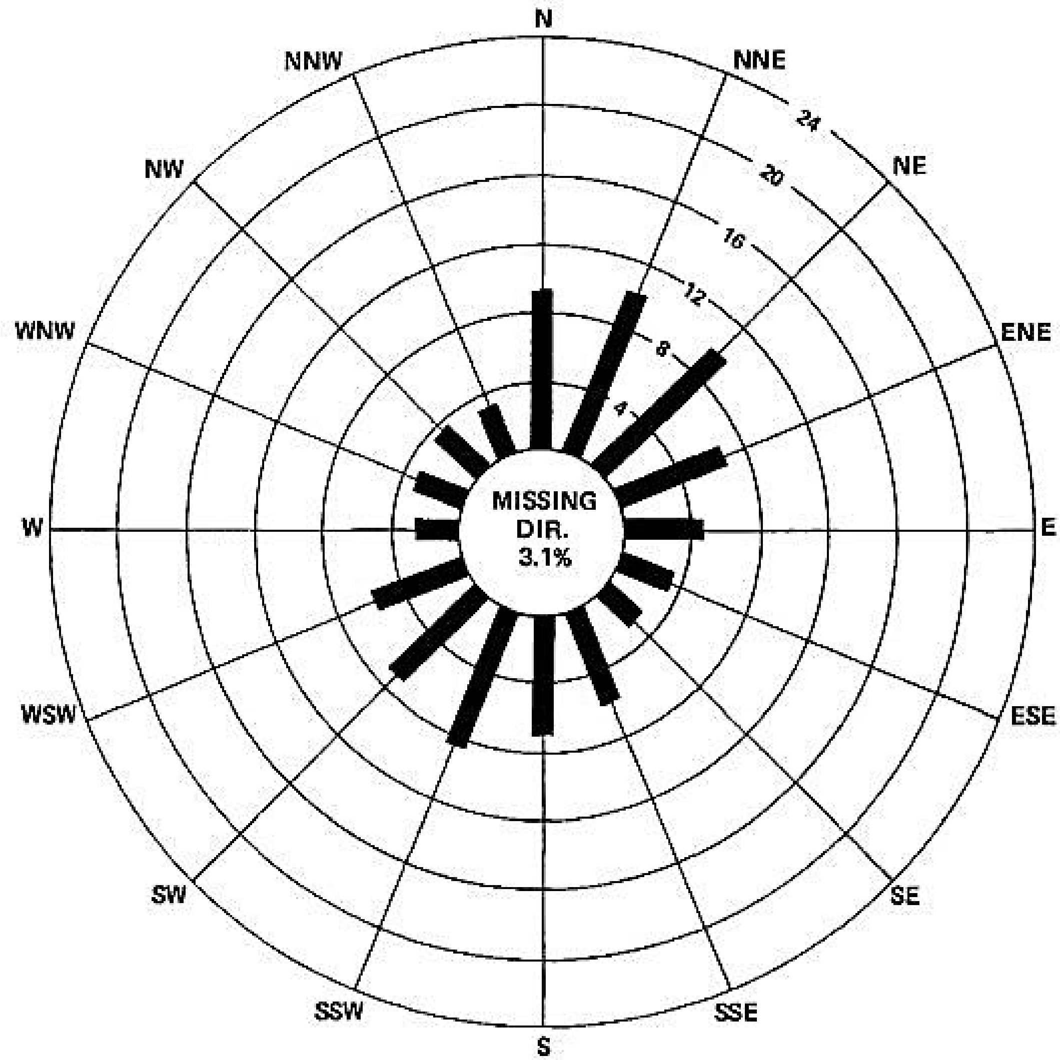
NOTE: DIRECTIONAL AVERAGE WIND SPEEDS (MPH) ARE DISPLAYED RADIALLY

WIND DIRECTION (%)

1955-1964

FIGURE 2.3.2-11

SHNPP PRECIPITATION WIND ROSE 12.5M LEVEL



$\bar{U}$ = NA

$\bar{U}$ = ALL DIRECTIONAL AVERAGE WIND SPEED

NOTE: DIRECTIONAL AVERAGE WIND SPEEDS (MPH) ARE DISPLAYED RADIALLY

WIND DIRECTION (%)

1/14/76 - 12/31/78

FIGURE 2.3.2-12

ANNUAL CUMULATIVE FREQUENCY OF COOLING TOWER PLUME LENGTHS

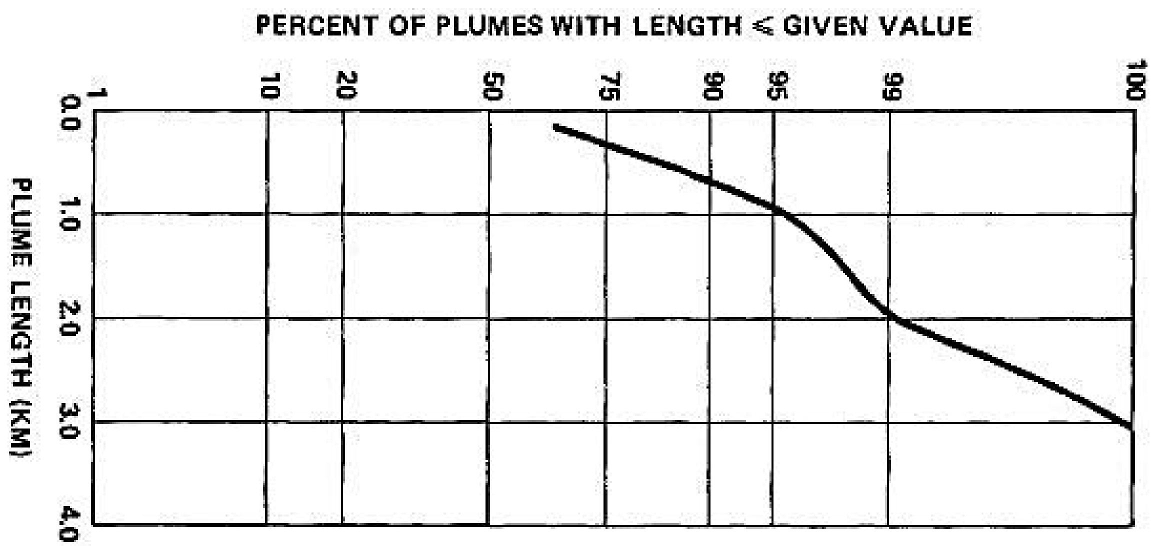


FIGURE 2.3.2-13

ANNUAL HOURLY FREQUENCY OF COOLING TOWER PLUMES

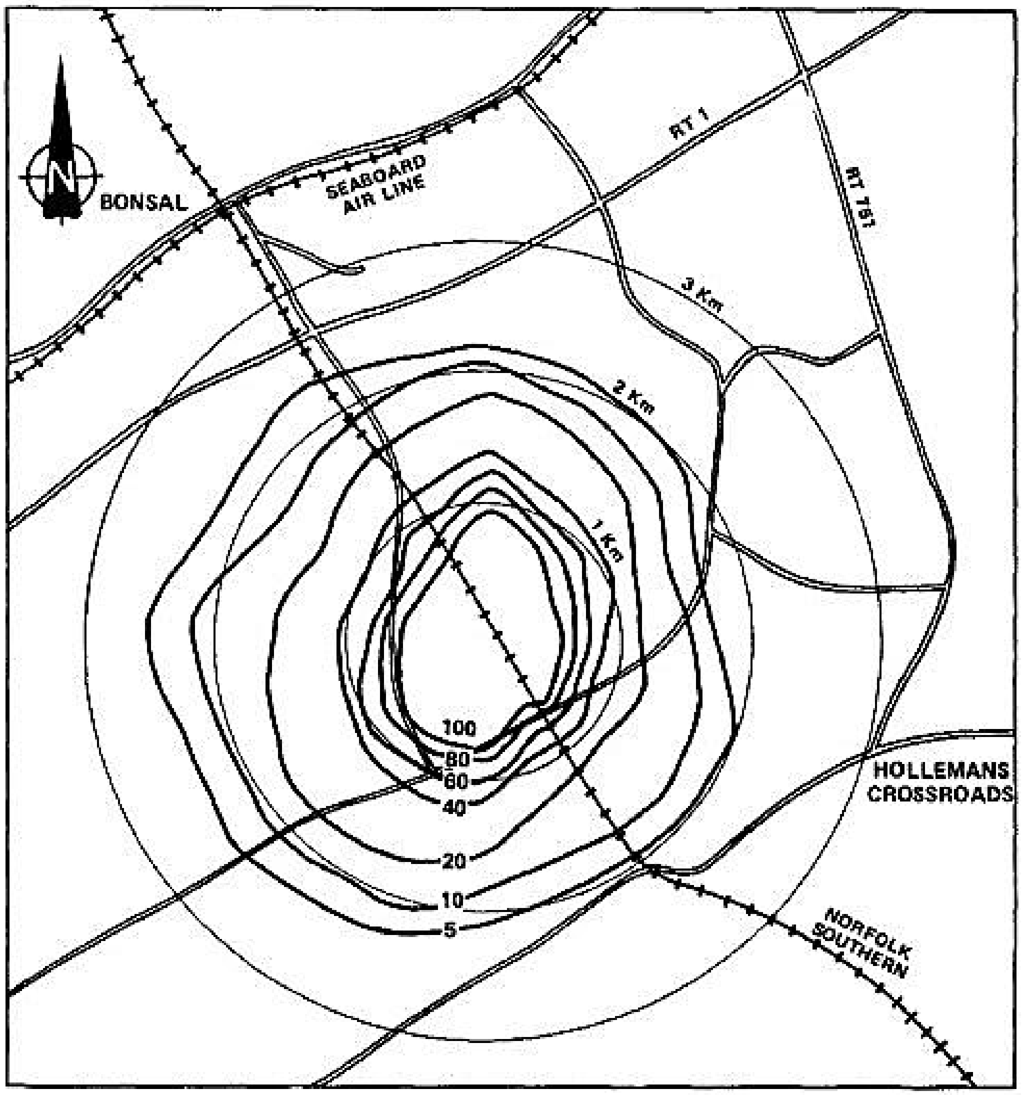


FIGURE 2.3.2-14

MAXIMUM ELEVATION VERSUS DISTANCE FROM THE CENTER OF THE PLANT

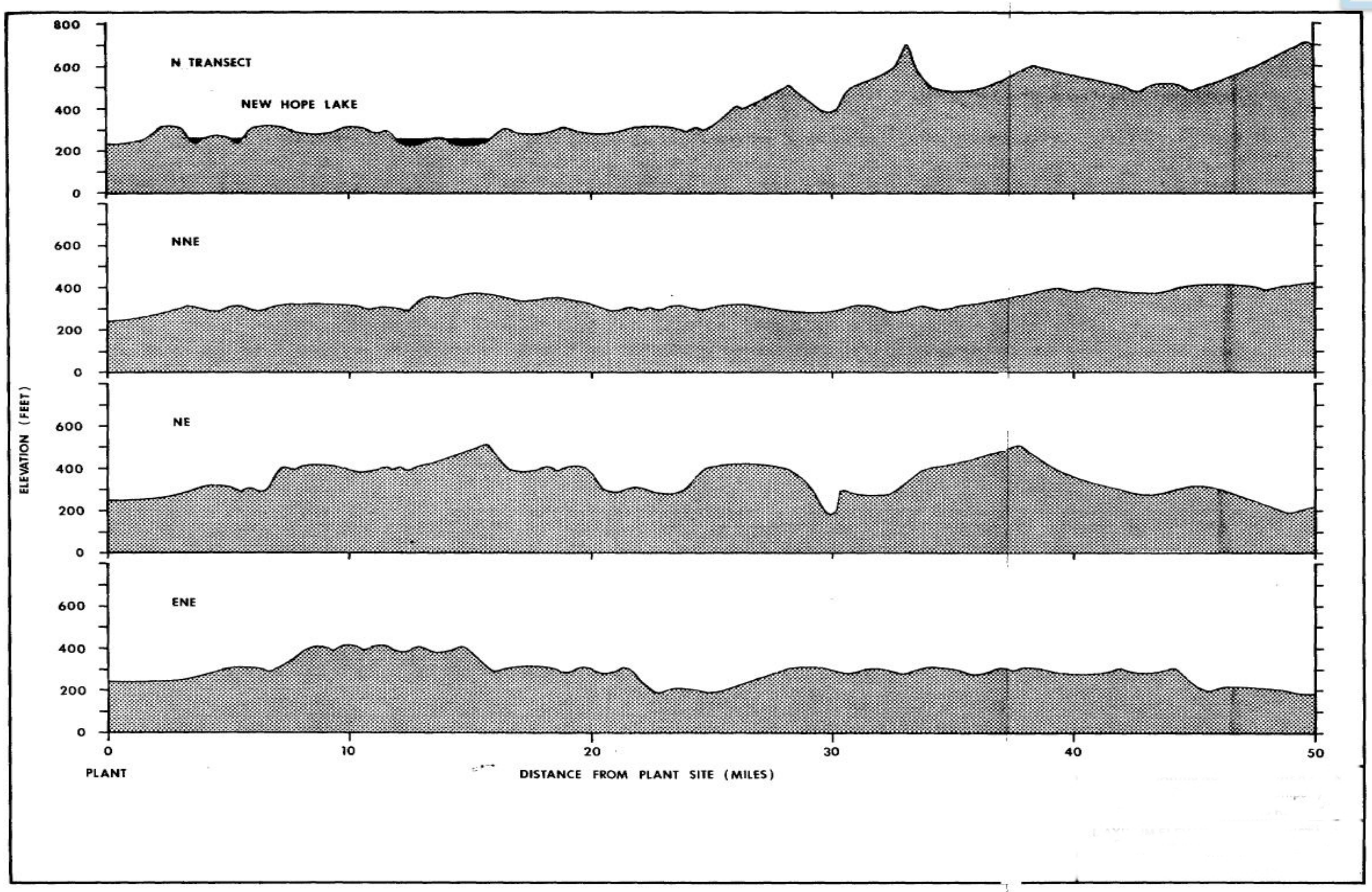


FIGURE 2.3.2-15

MAXIMUM ELEVATION VERSUS DISTANCE FROM THE CENTER OF THE PLANT

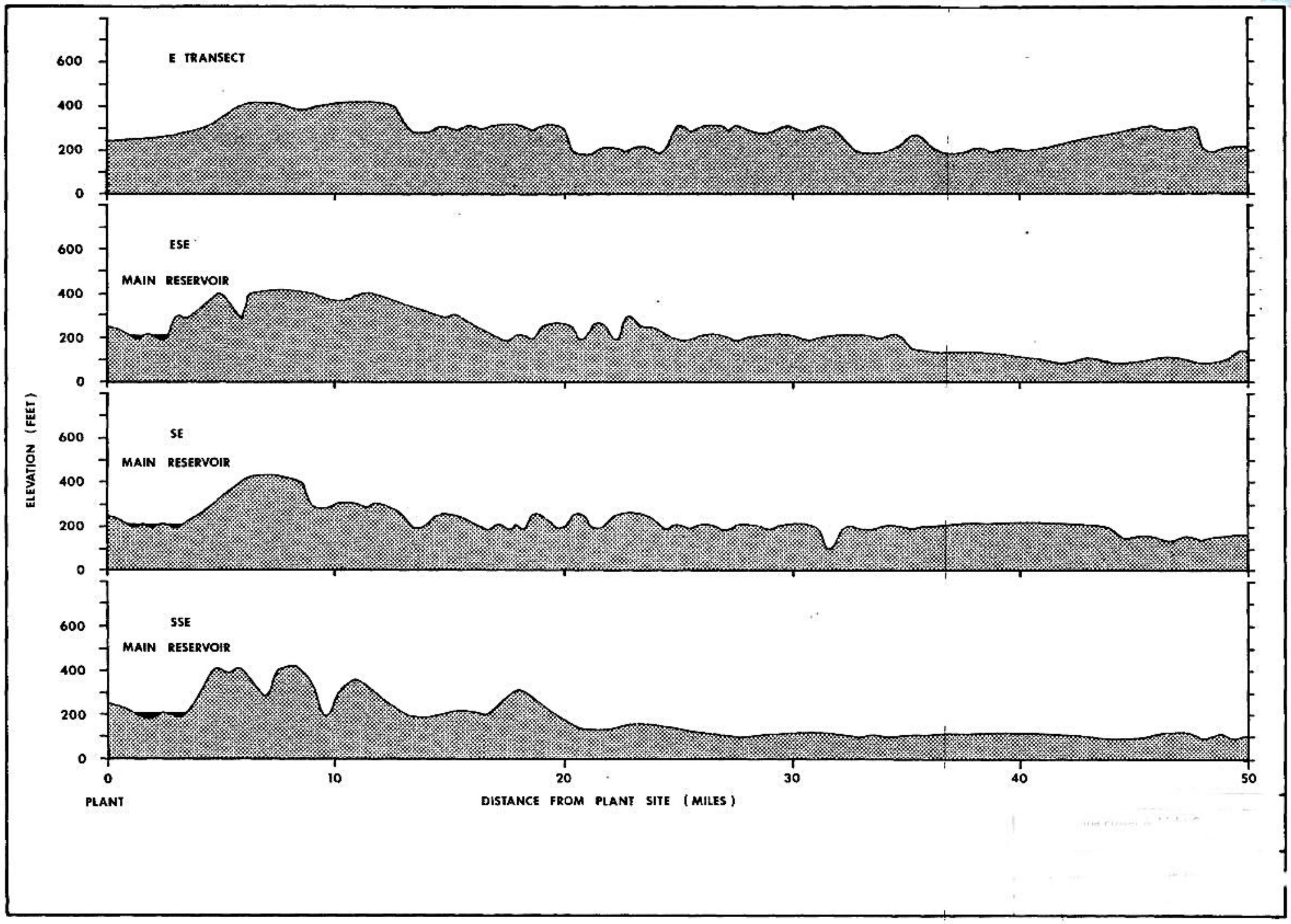


FIGURE 2.3.2-16

MAXIMUM ELEVATION VERSUS DISTANCE FROM THE CENTER OF THE PLANT

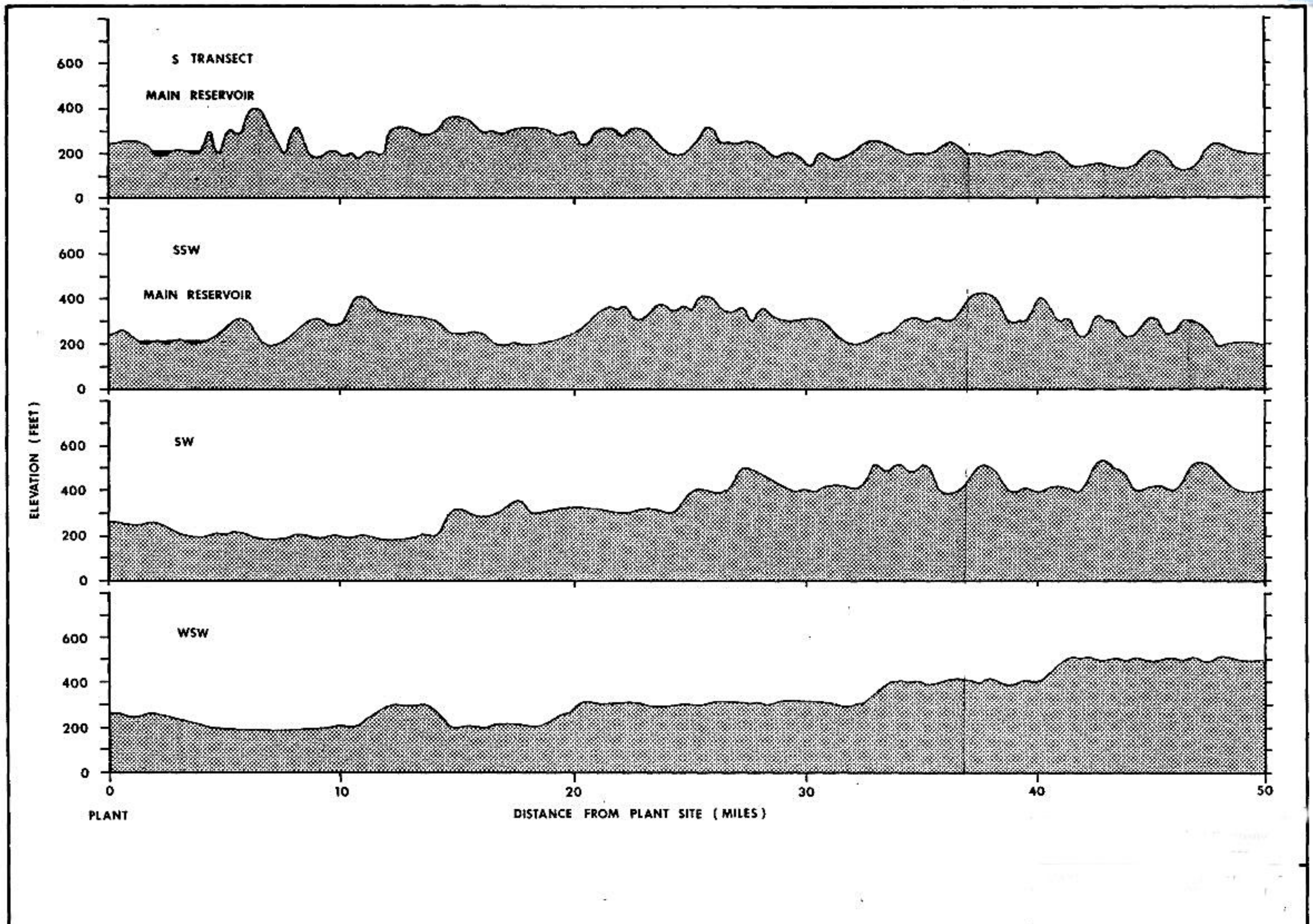


FIGURE 2.3.2-17

MAXIMUM ELEVATION VERSUS DISTANCE FROM THE CENTER OF THE PLANT

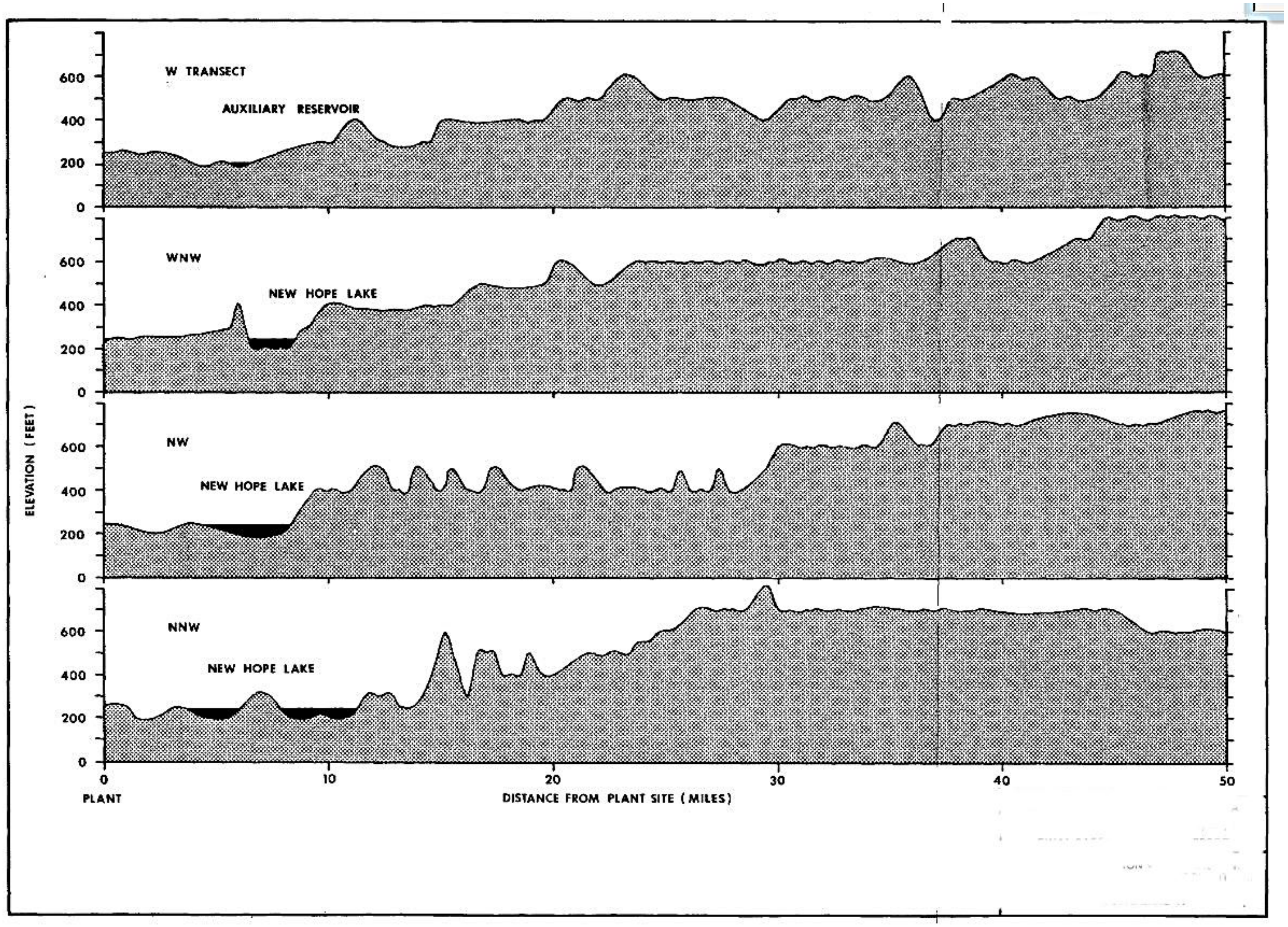


FIGURE 2.3.2-18

TOPOGRAPHIC FEATURES WITHIN A 5-MILE RADIUS OF THE PLANT

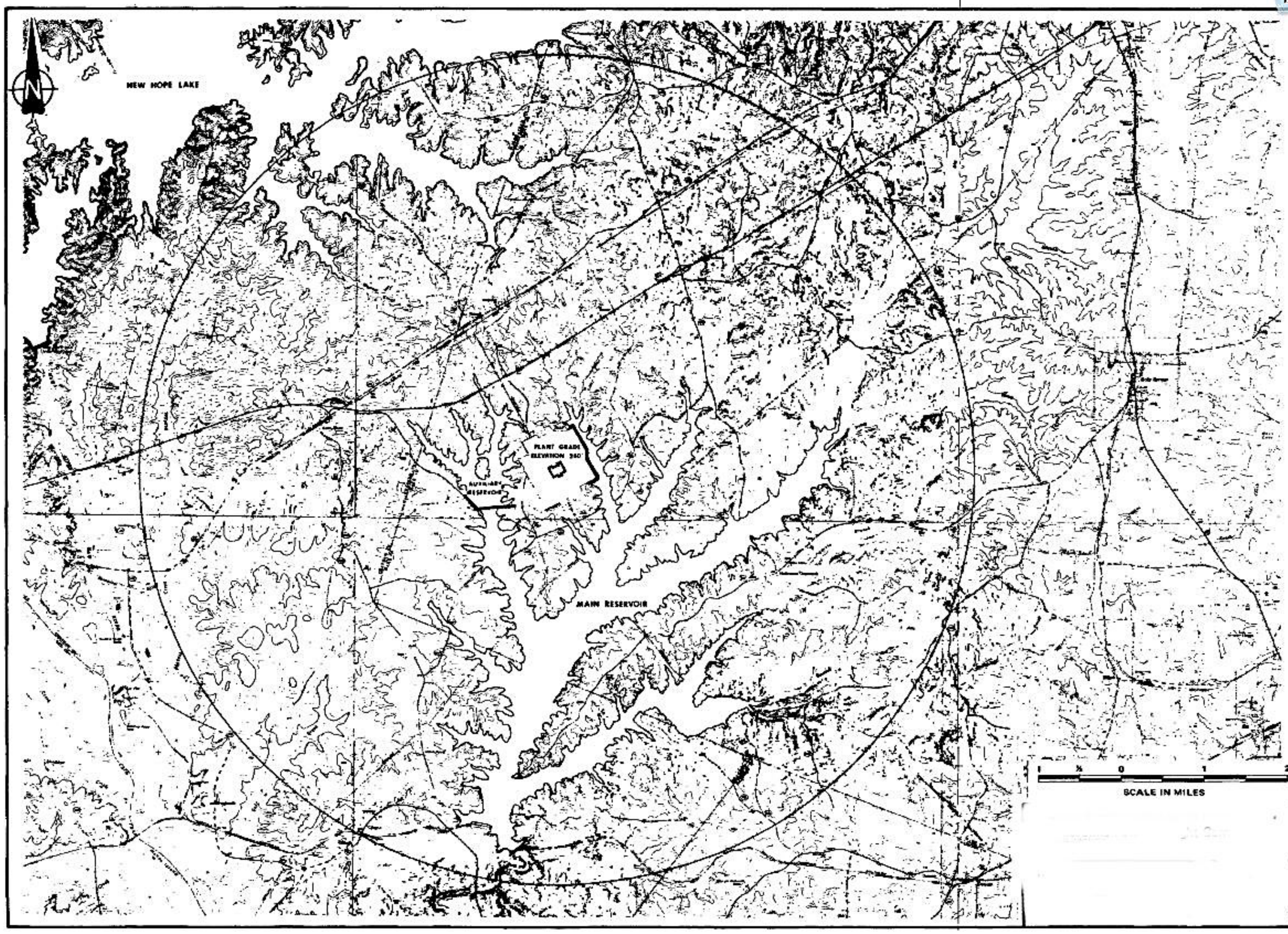


FIGURE 2.3.2-19

TOPOGRAPHIC FEATURES WITHIN A 50-MILE RADIUS OF THE PLANT

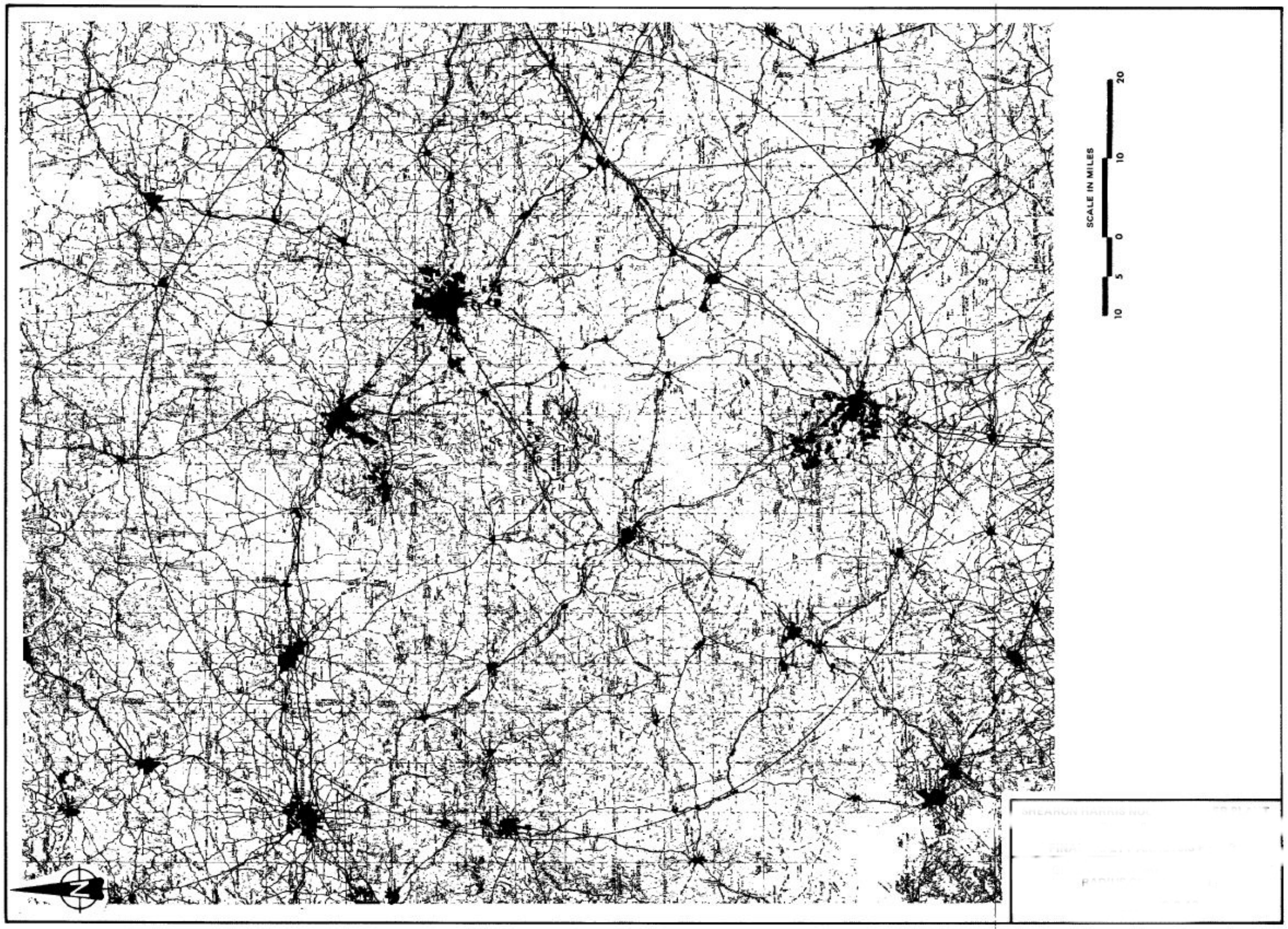


FIGURE 2.3.3-1

METEOROLOGICAL FACILITY

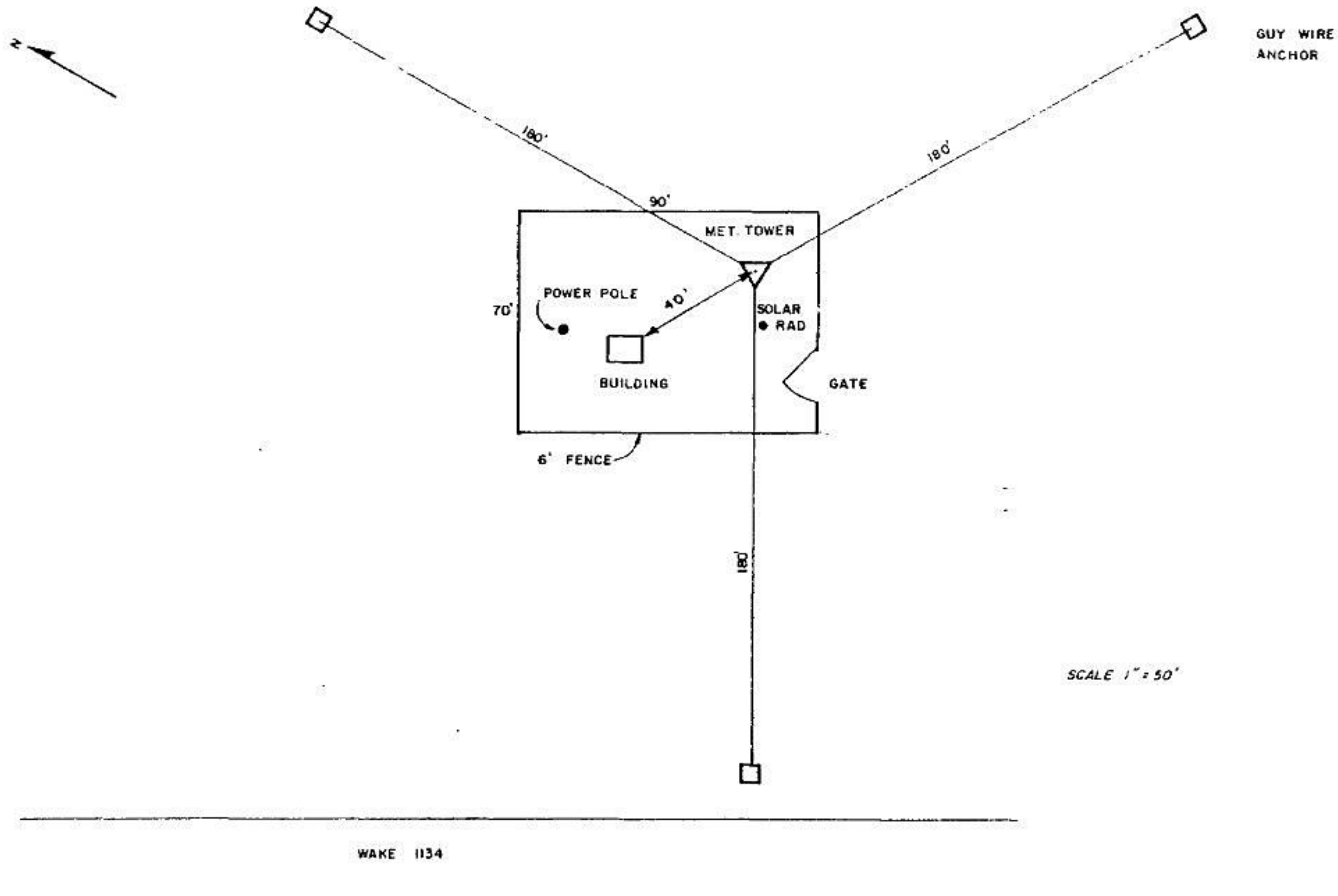
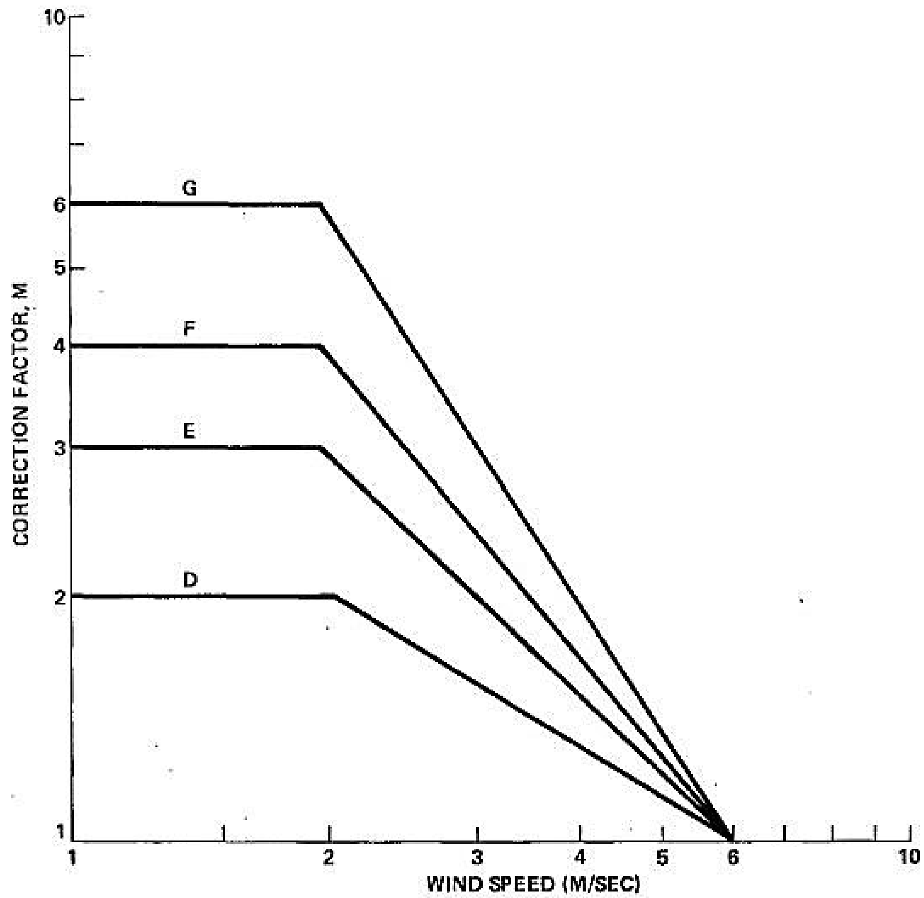


FIGURE 2.3.4-1

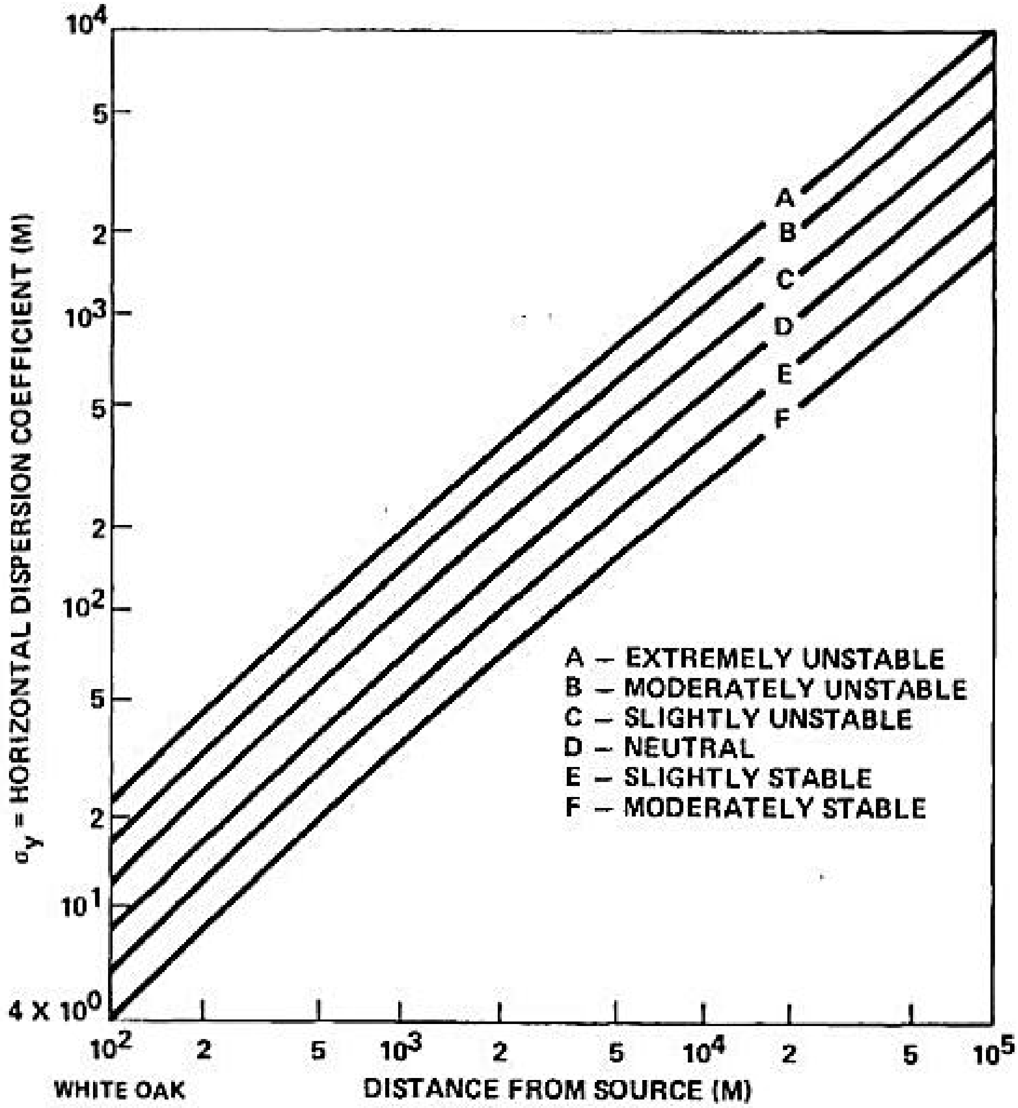
WIND MEANDER CORRECTION FACTORS



CORRECTION FACTORS FOR PASQUILL-GIFFORD σ_y VALUES.

FIGURE 2.3.4-2

HORIZONTAL DISPERSION COEFFICIENTS FOR VARIOUS DISTANCES FROM SOURCE



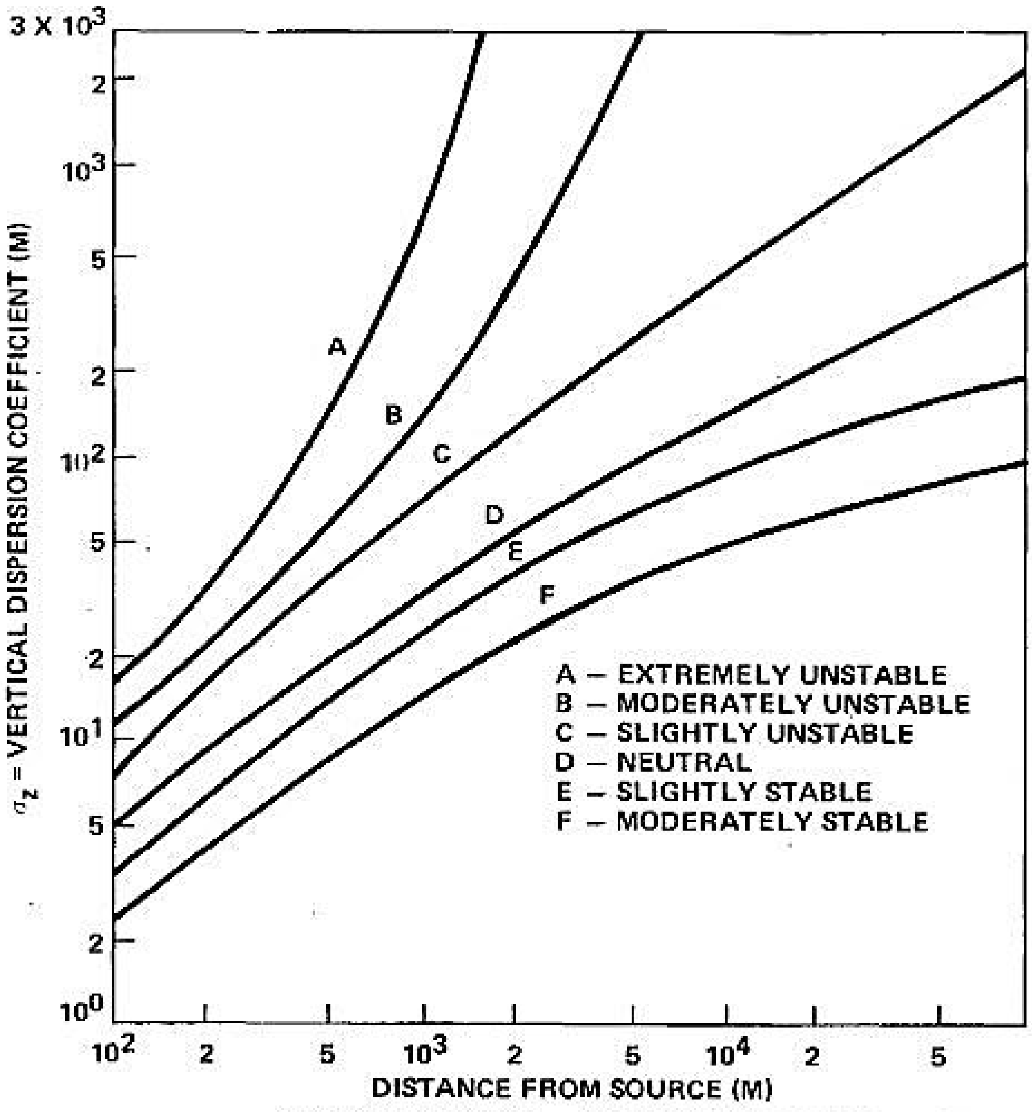
LATERIAL DIFFUSION, σ_y, VS. DOWNWIND DISTANCE FROM SOURCE FOR PASQUILL'S TURBULENCE TYPES.

FOR PURPOSES OF ESTIMATING σ_y DURING EXTREMELY STABLE (G) CONDITIONS, WIHTOUT PLUME MEANDER OR OTHER LATERIAL ENHANCEMENT, THE FOLLOWING APPROXIMATION IS APPROPRIATE:

$$\sigma_y (G) = \frac{2}{3} \sigma_y (F)$$

FIGURE 2.3.4-3

VERTICAL DISPERSION COEFFICIENTS FOR VARIOUS DISTANCES FROM SOURCE



VERTICAL DIFFUSION, σ_y, VS. DOWNWIND DISTANCE FROM SOURCE FOR PASQUILL'S TURBULENCE TYPES

FOR PURPOSES OF ESTIMATING σ_z DURING EXTREMELY STABLE (G) CONDITIONS, THE FOLLOWING APPROXIMATION IS APPROPRIATE:

$$\sigma_z\,(G) = \frac{3}{5}\,\sigma_z\,(F)$$

FIGURE 2.4.1-1

PROJECT SITE PLAN

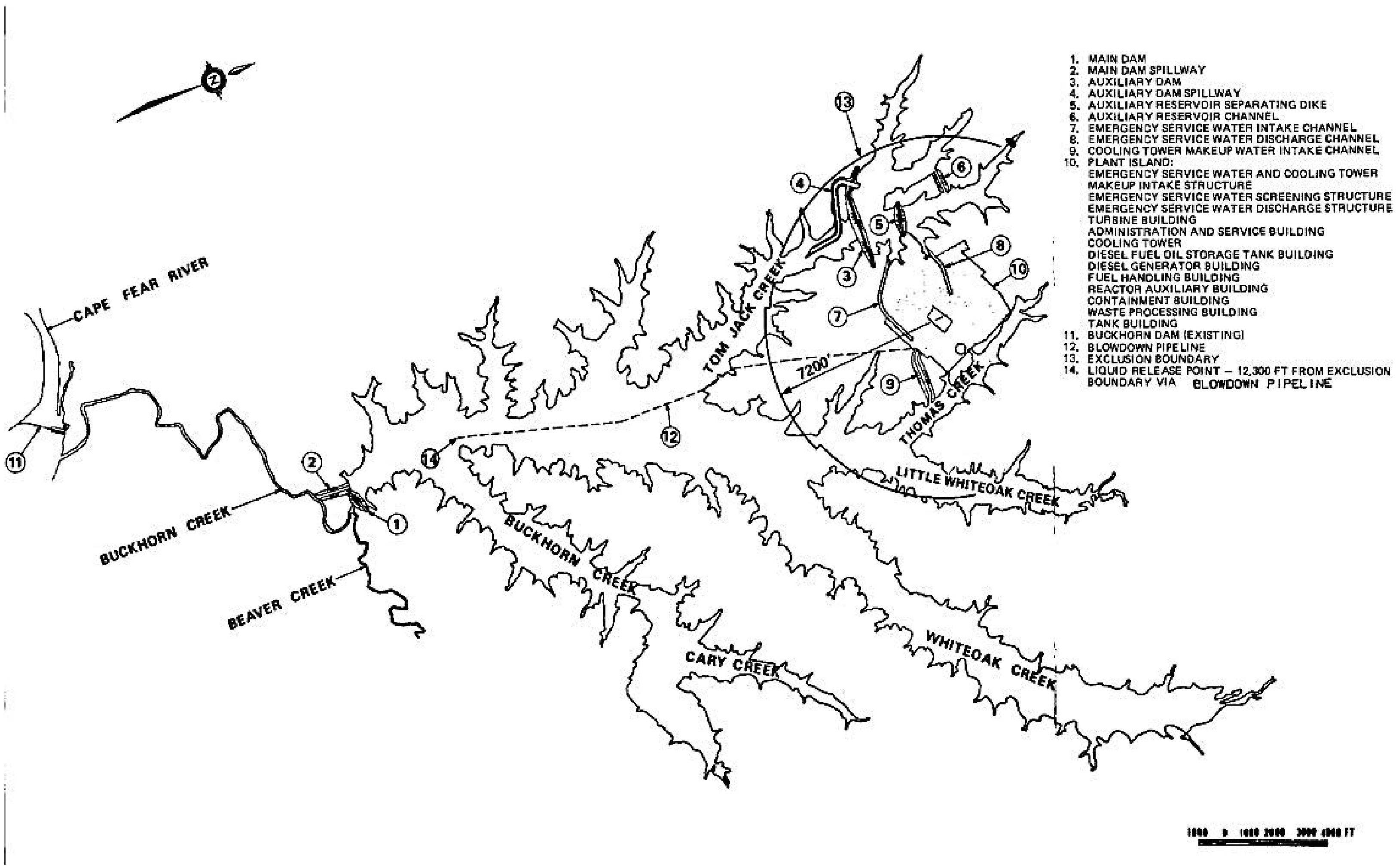


FIGURE 2.4.1-2

FINISHED CONTOURS & DRAINAGE PATTERN

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.4.1-3

LOCATION OF CAPE FEAR BASIN

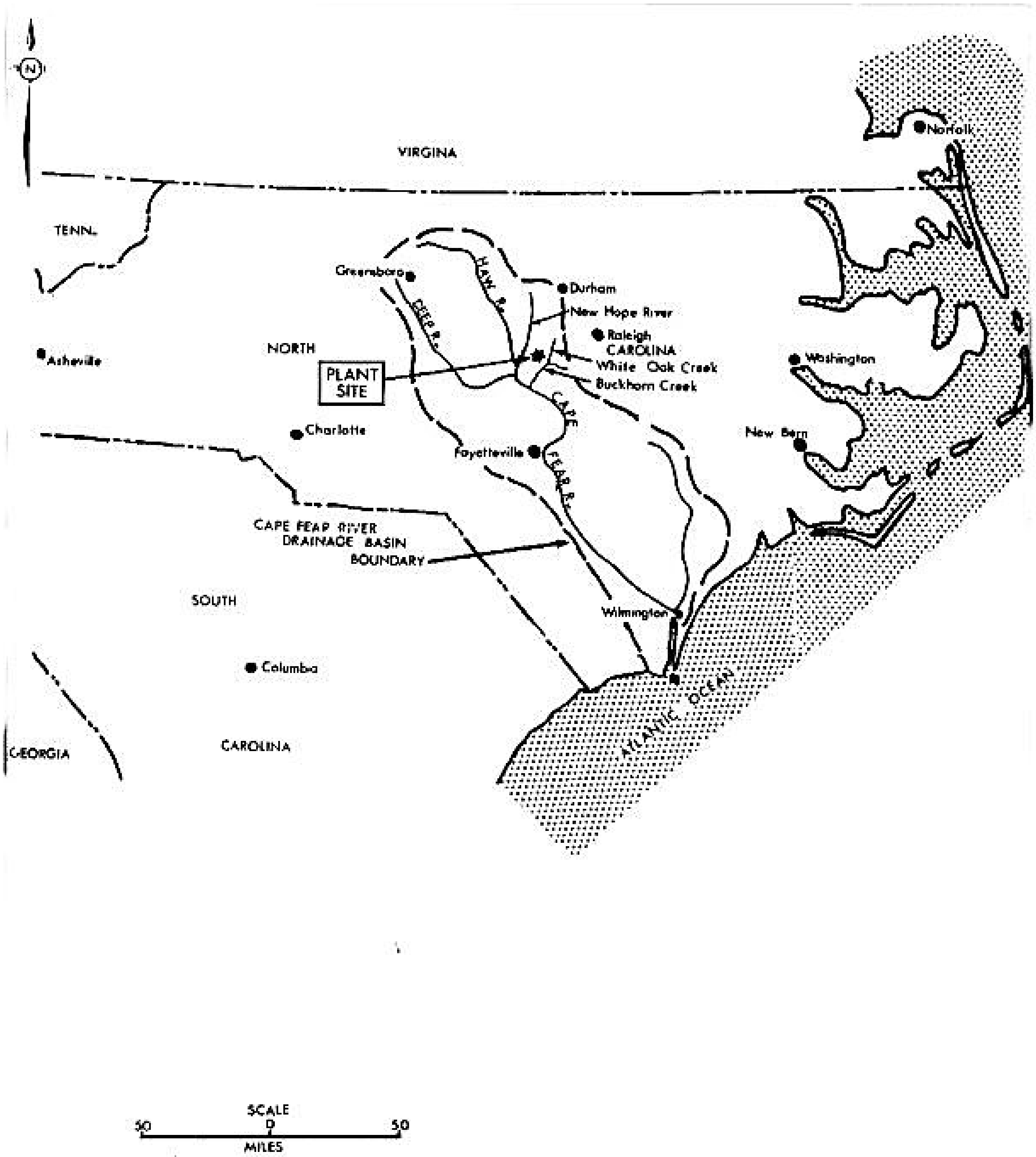


FIGURE 2.4.1-4

CAPE FEAR RIVER BASIN

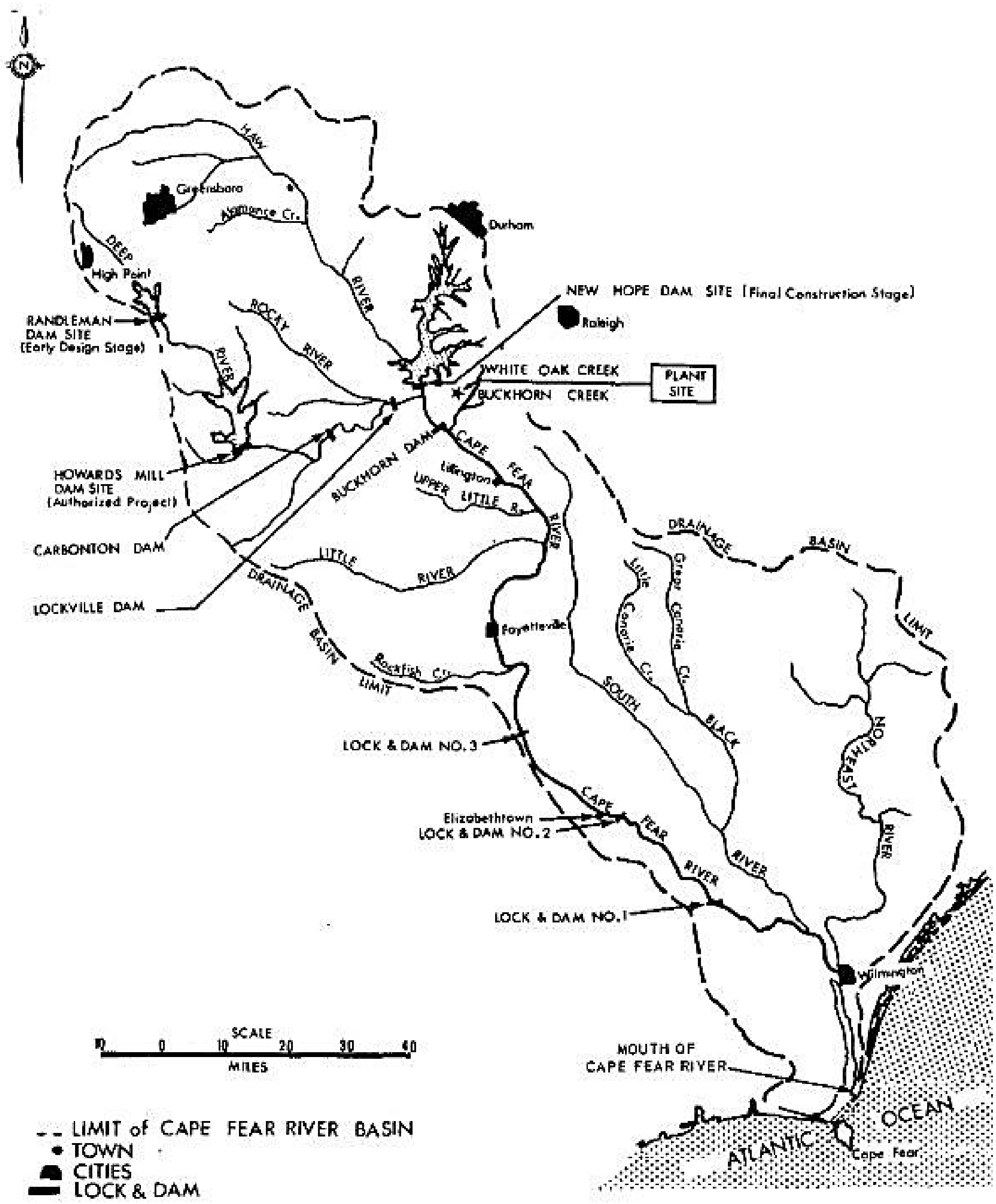
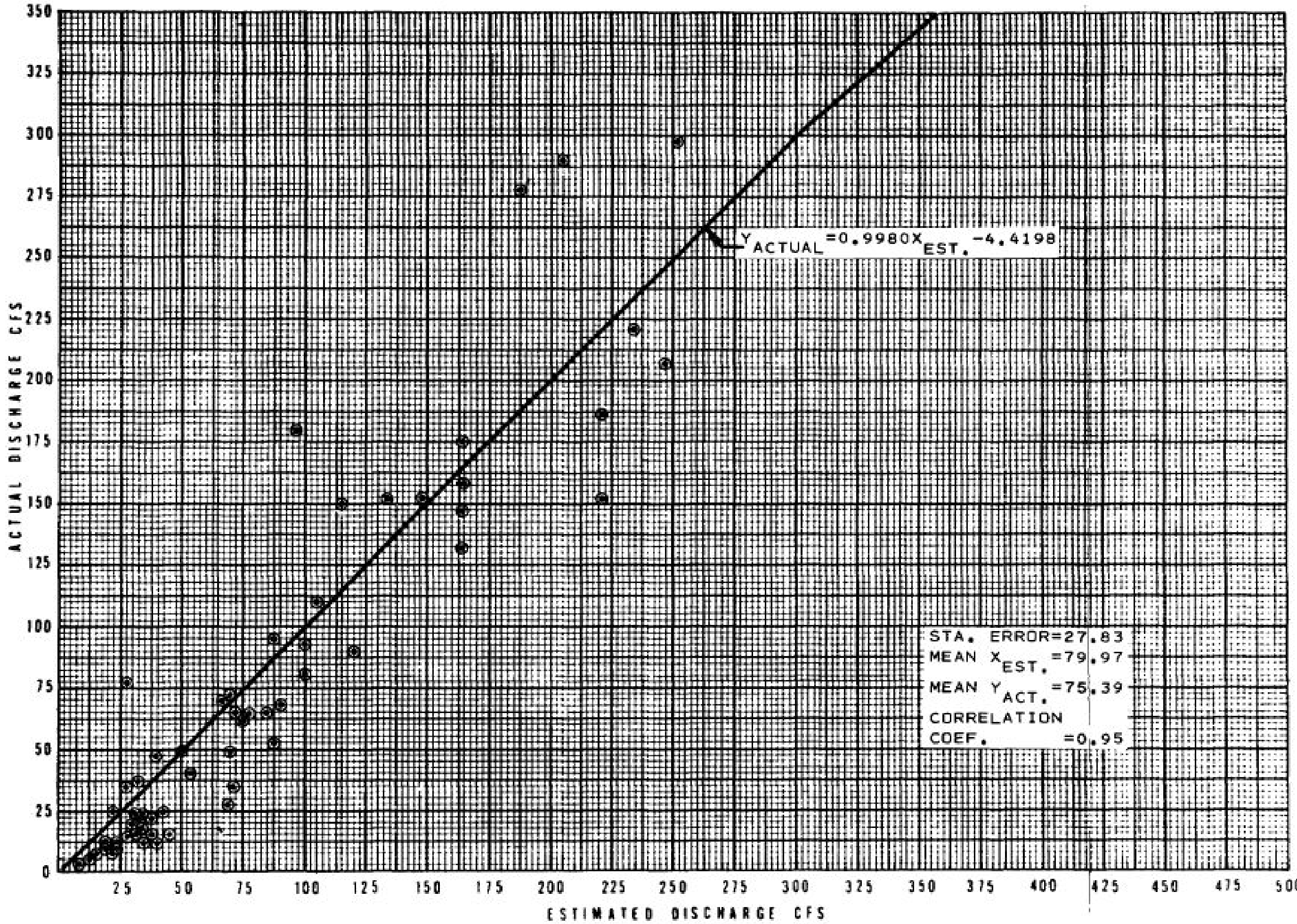


FIGURE 2.4.1-5

CORRELATION OF MONTHLY AVERAGE FLOW BETWEEN ESTIMATED & ACTUAL FLOW OF BUCKHORN CREEK



DATA SOURCE: USGS

FIGURE 2.4.1-6

WATERSHEDS IN THE VICINITY OF THE SITE

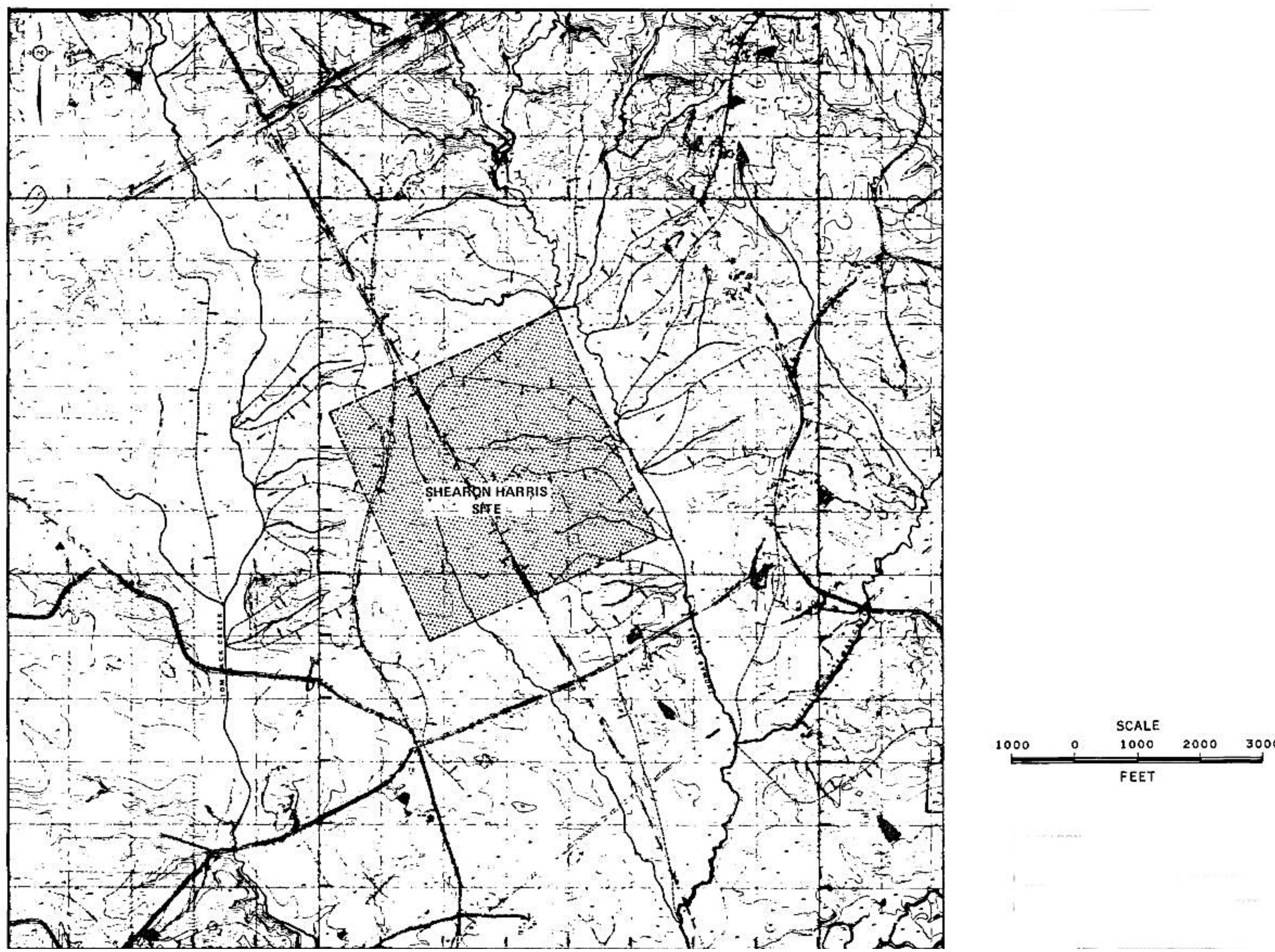


FIGURE 2.4.1-7

MAJOR SURFACE WATER BODIES 5-MILE RADIUS

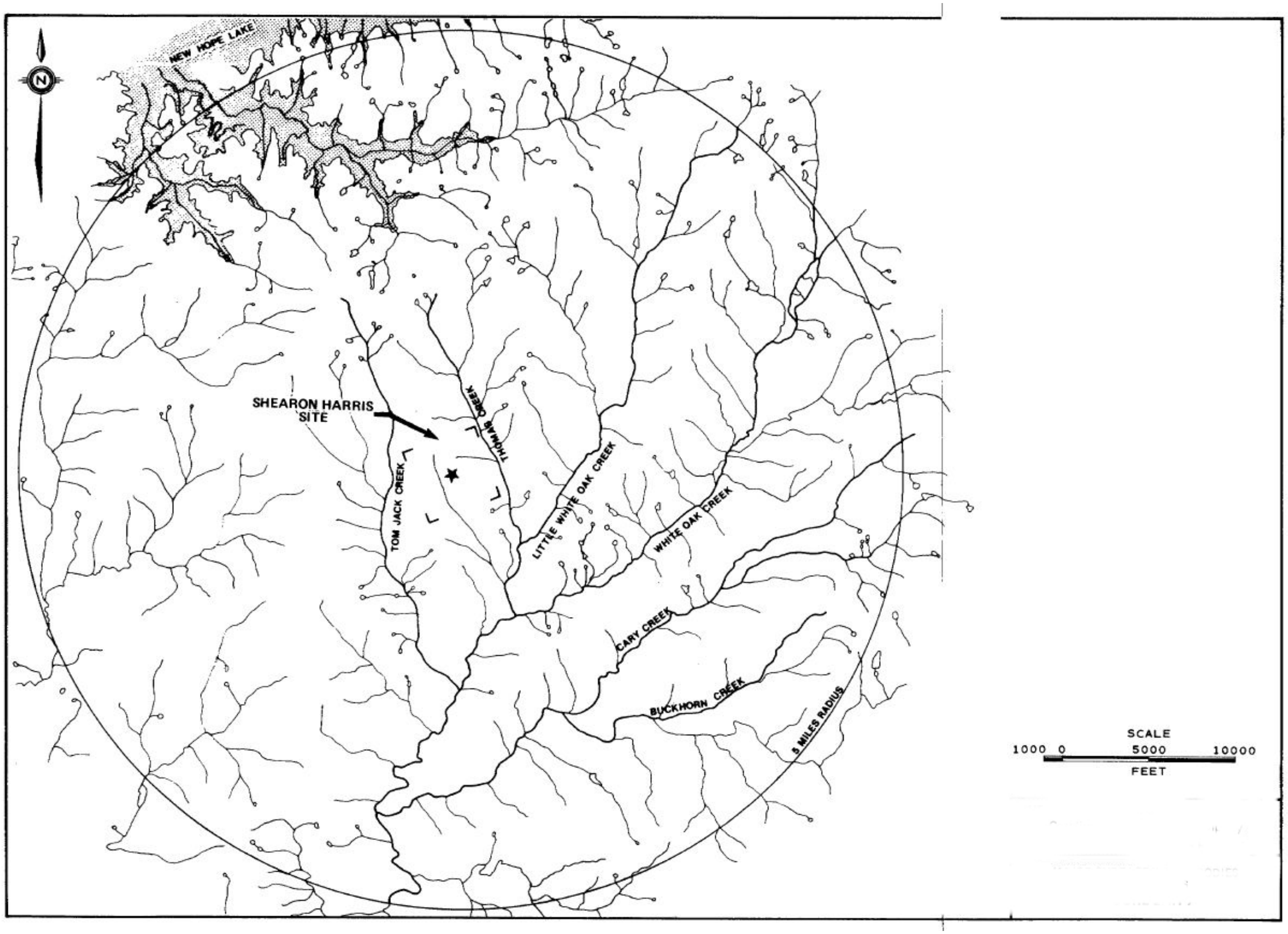


FIGURE 2.4.1-8

MAJOR SURFACE WATER BODIES 25-MILE RADIUS

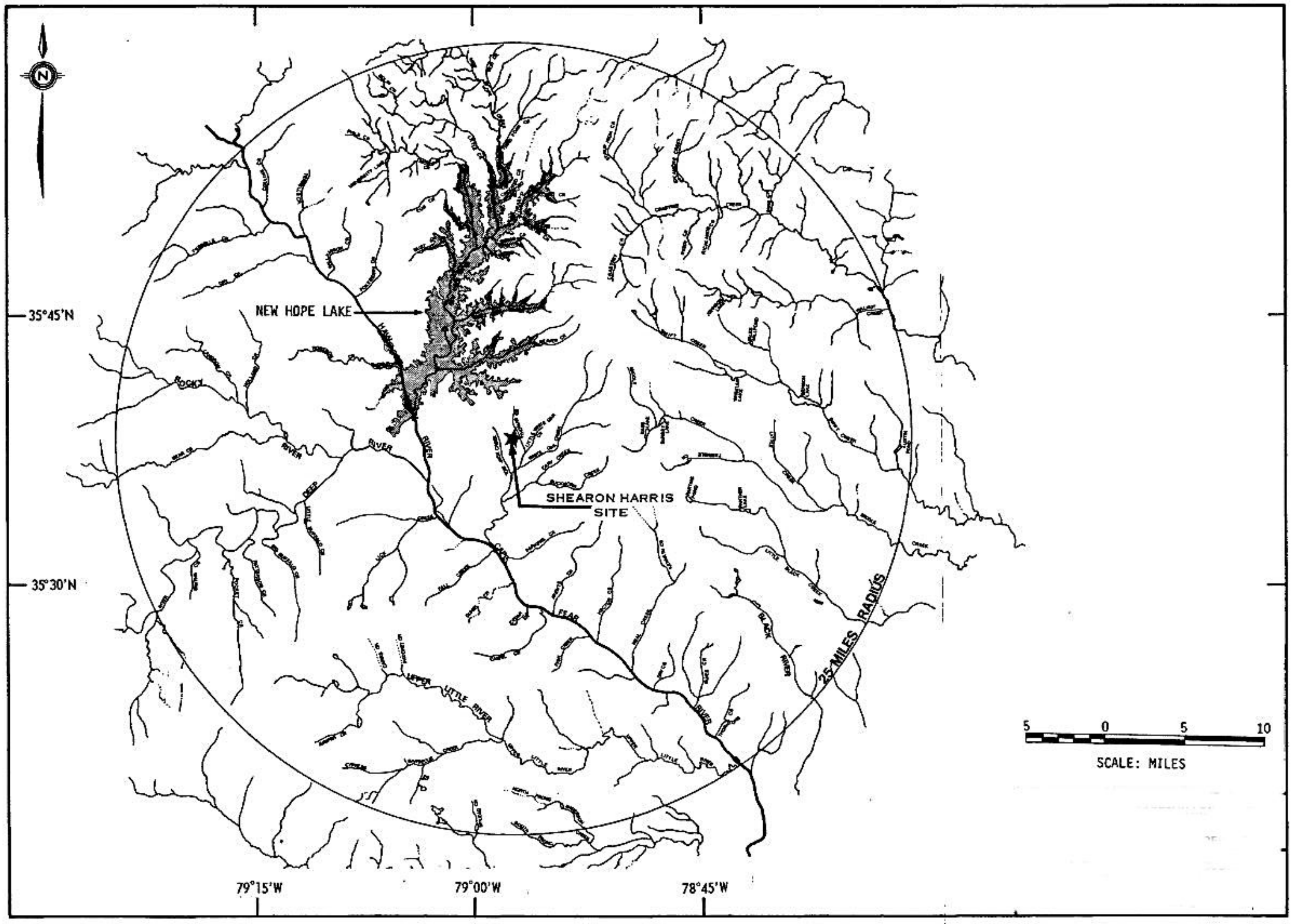


FIGURE 2.4.2-1

BUCKHORN CREEK FLOOD PEAKS FREQUENCY ANALYSIS (LOG PEARSON TYPE III DISTRIBUTION) 1940-1978

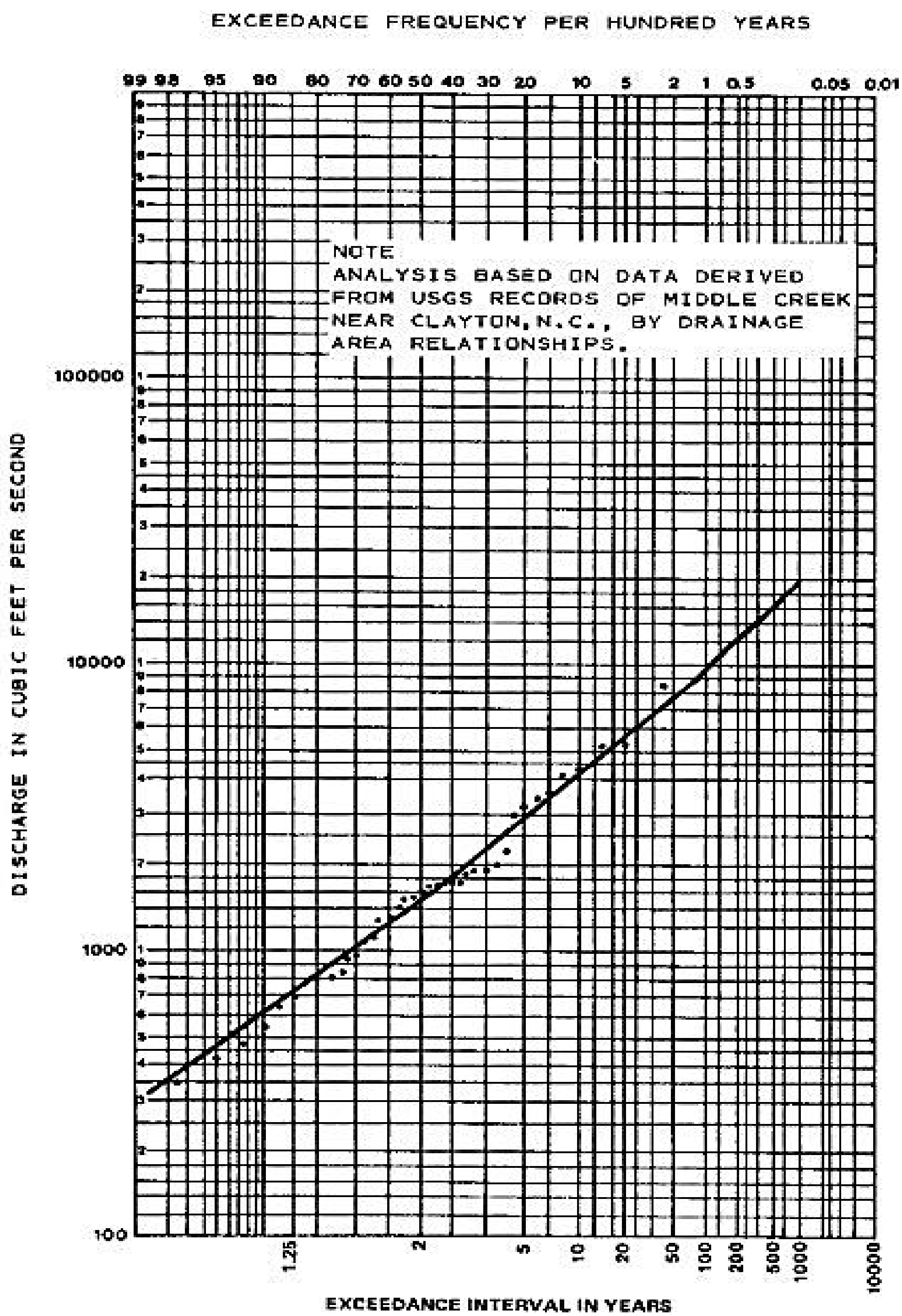


FIGURE 2.4.2-2

CAPE FEAR RIVER (AT BUCKHORN DAM) FLOOD PEAKS FREQUENCY ANALYSIS (LOG PEARSON TYPE III DISTRIBUTION) 1924-1978

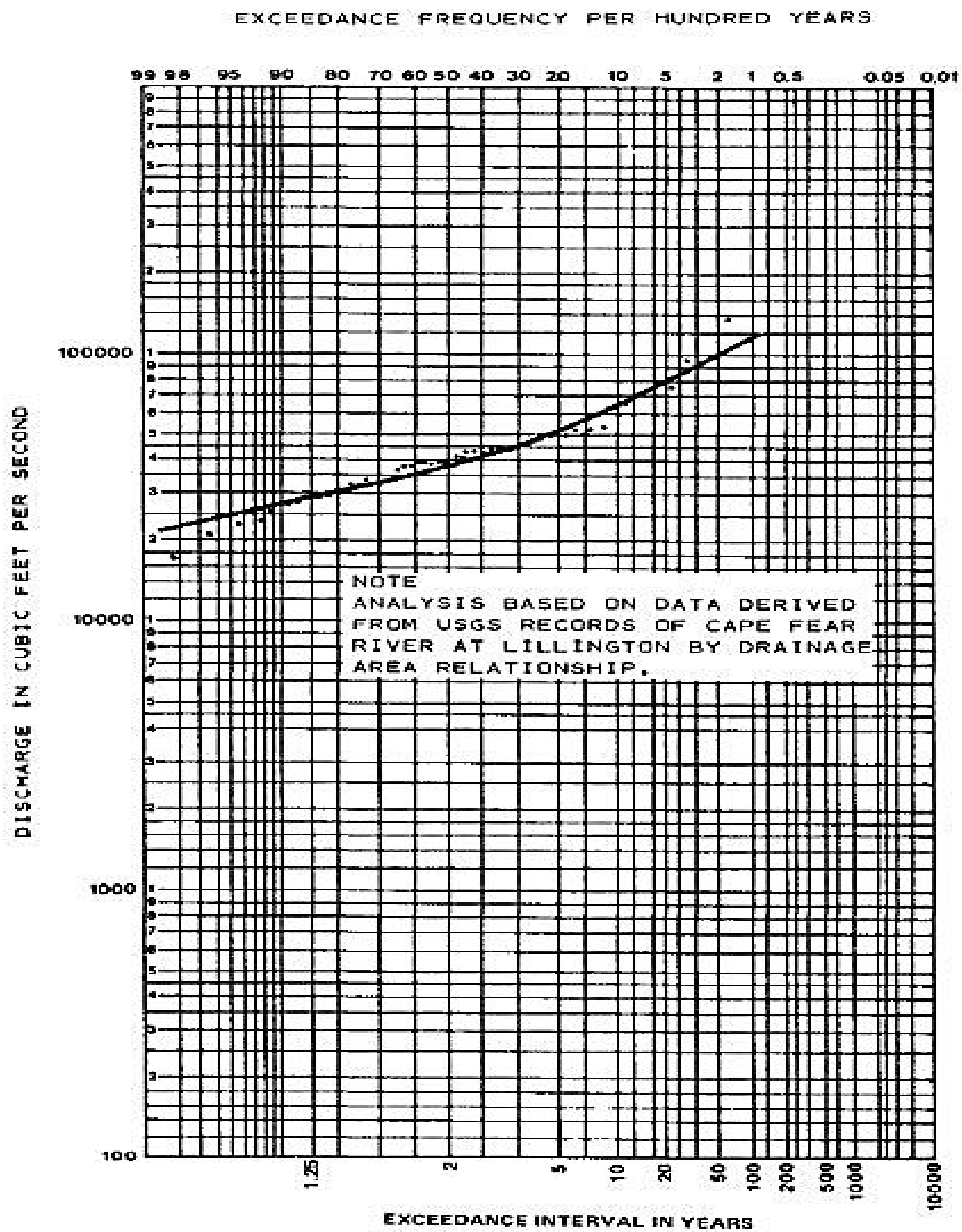


FIGURE 2.4.2-3

FLOOD PLAIN ADJOINING CAPE FEAR RIVER IN THE VICINITY OF BUCKHORN CREEK

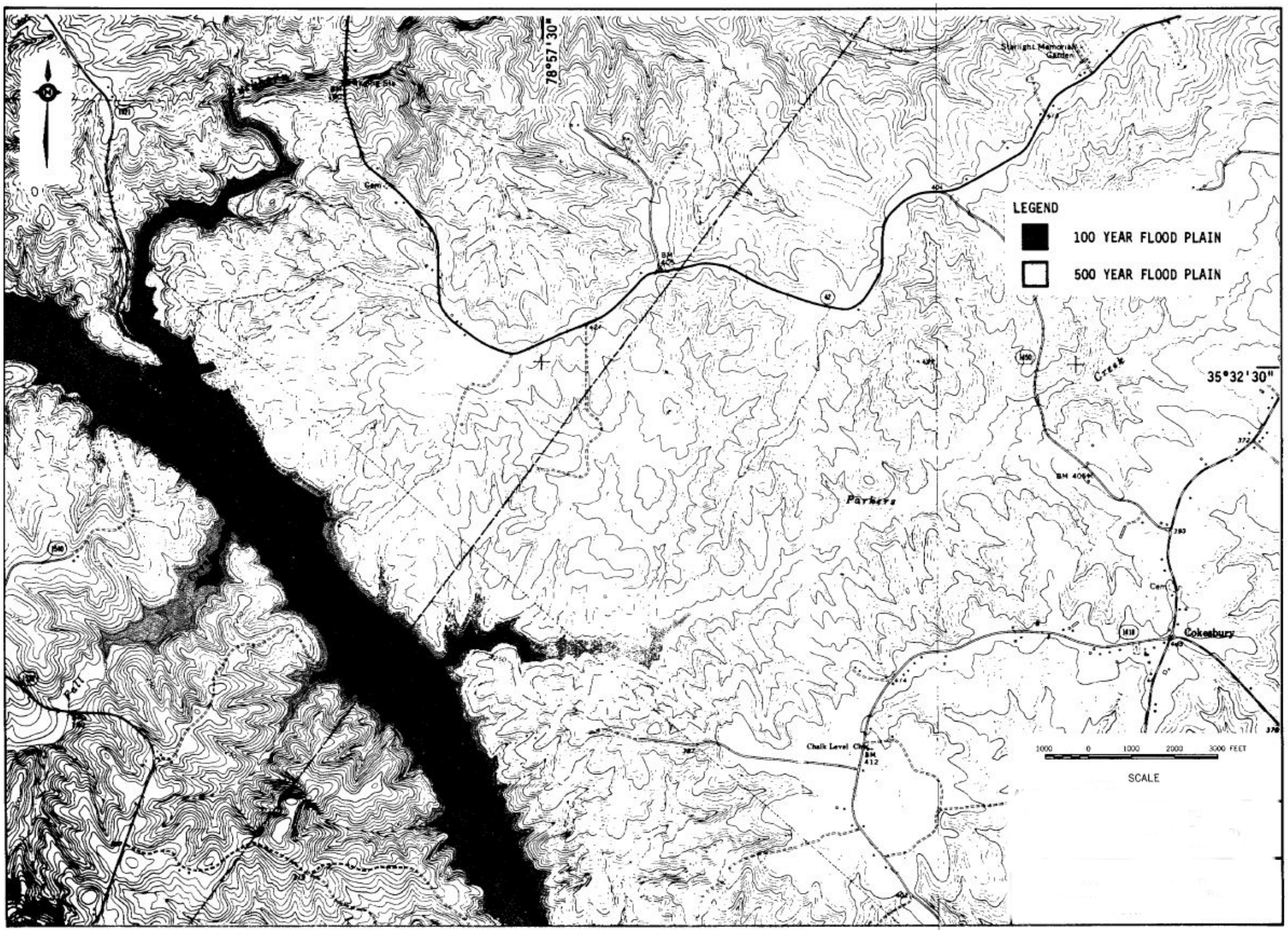


FIGURE 2.4.2-4

FLOOD PLAINS ADJOINING THE BUCKHORN CREEK AND THE MAIN AND AUXILIARY RESERVOIRS

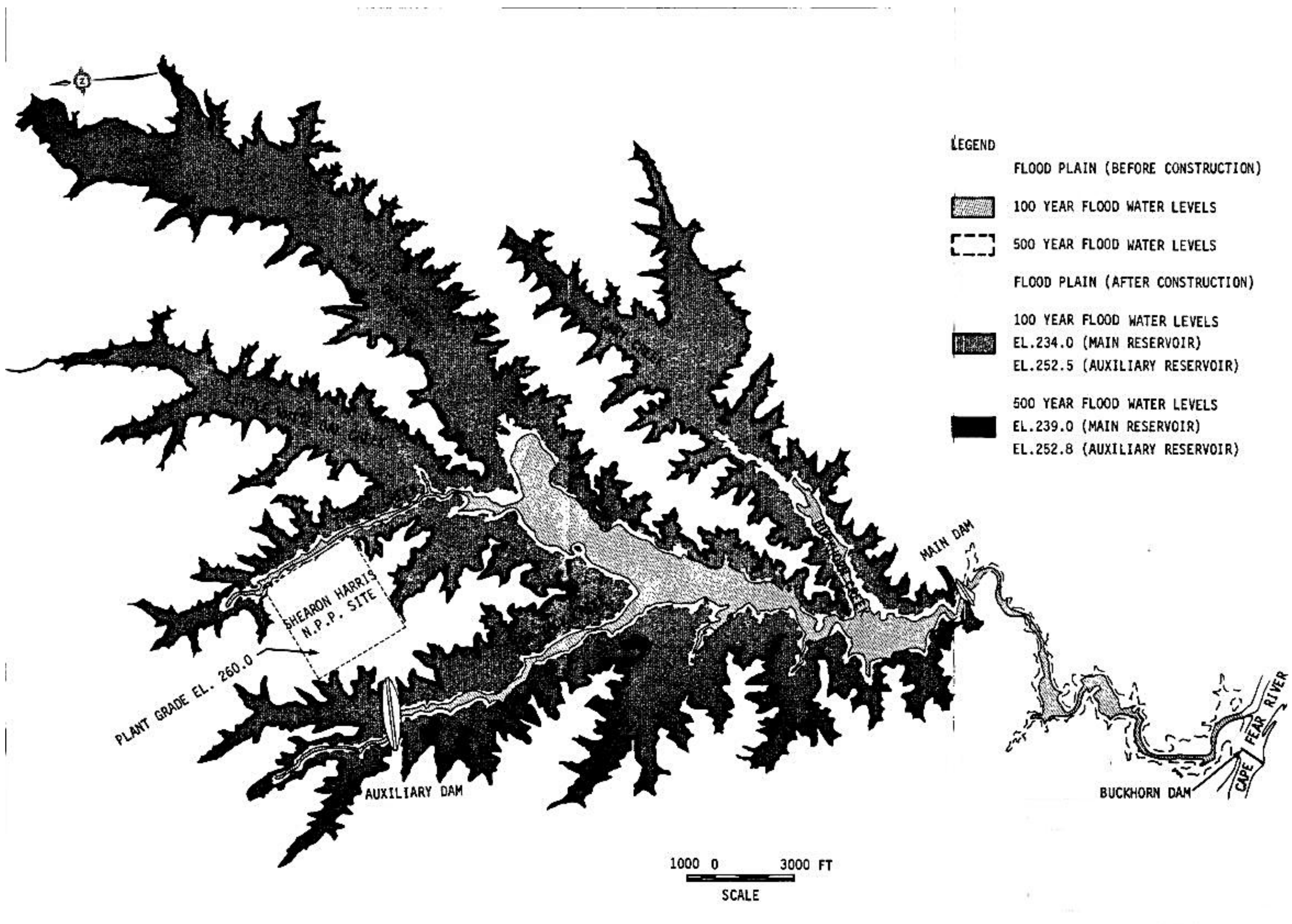


FIGURE 2.4.2-5

WATER LEVELS ON ROOFS OF BUILDINGS FOR PMP WITH ROOF DRAINS CLOGGED

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.4.3-1

OBSERVED AND RECONSTITUTED HYDROGRAPHS FOR THE FEB. 2-4, 1973 FLOOD ON BUCKHORN CREEK NEAR CORINTH, N.C.

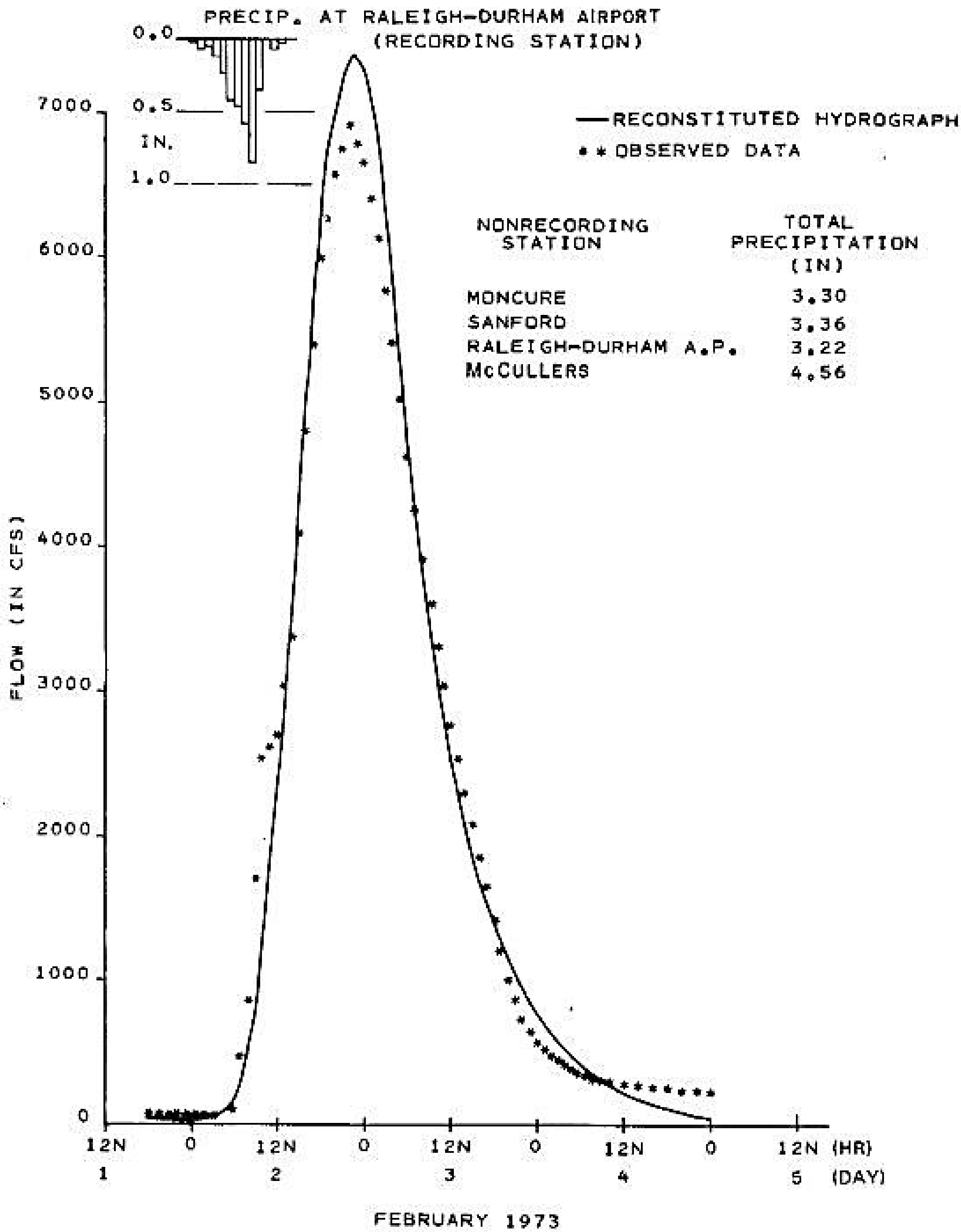


FIGURE 2.4.3-2

BUCKHORN CREEK DRAINAGE AREA AND SUBBASINS I-IX

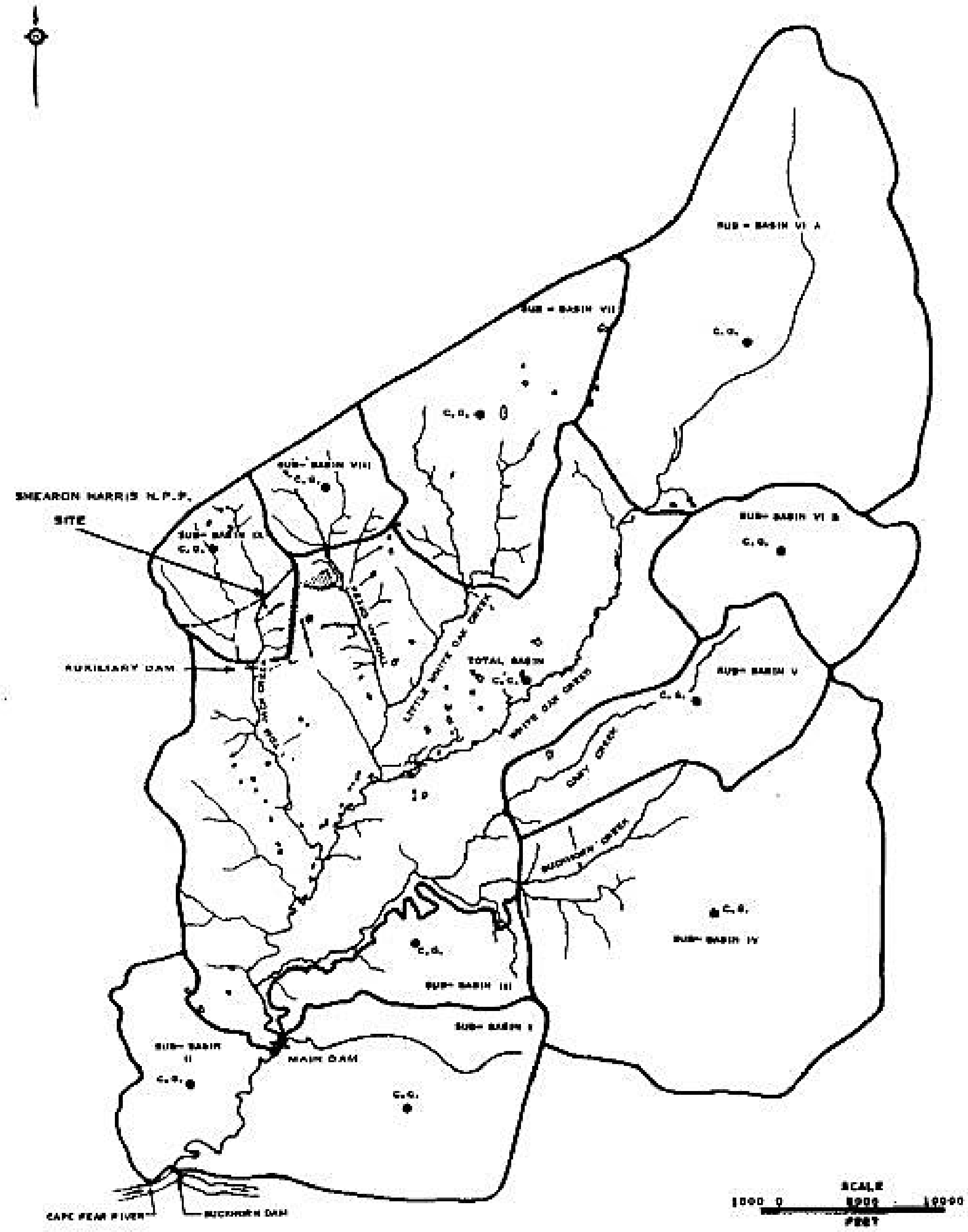


FIGURE 2.4.3-3

MAIN DAM SPILLWAY RATING CURVE

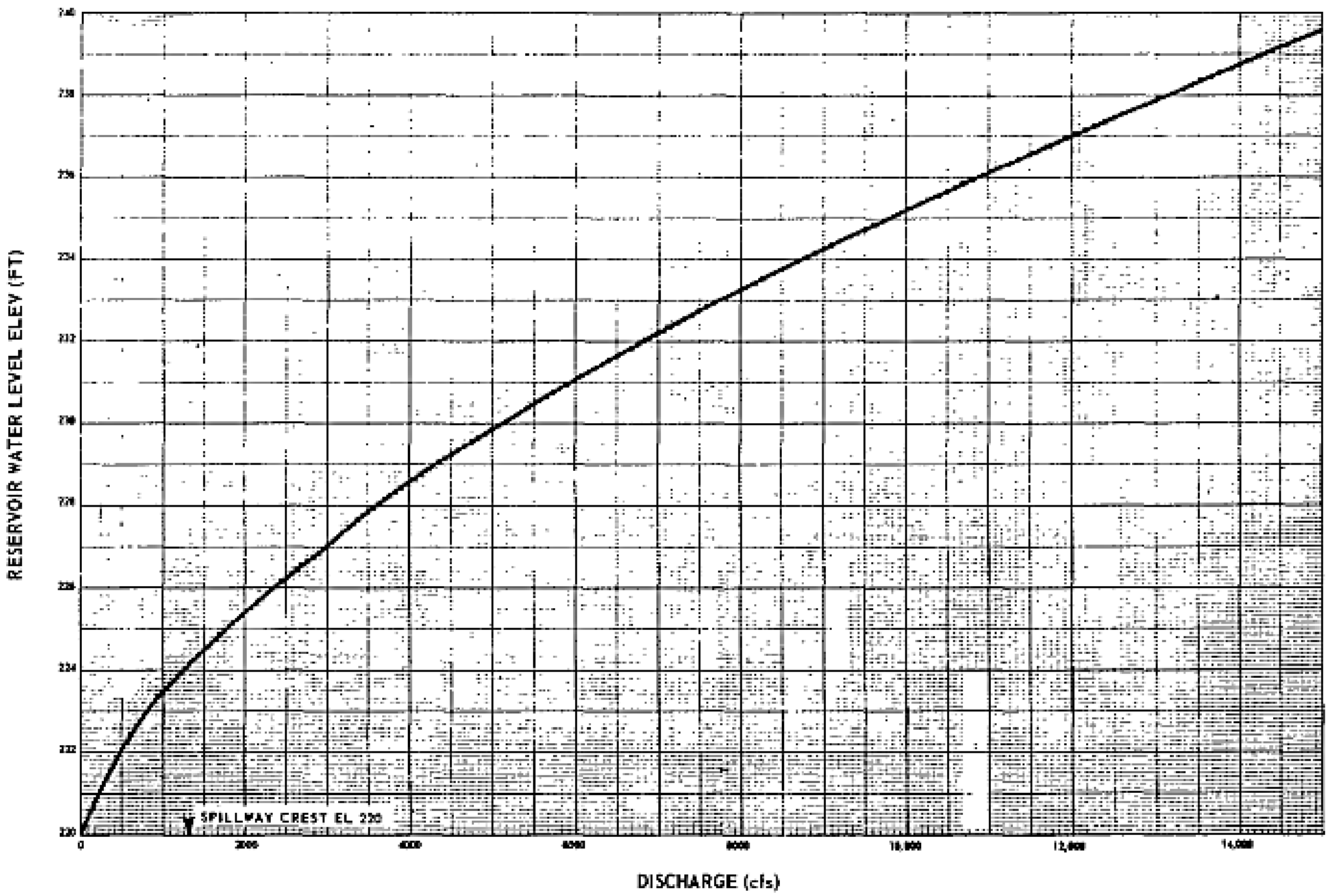


FIGURE 2.4.3-4

AUXILIARY DAM SPILLWAY RATING CURVE

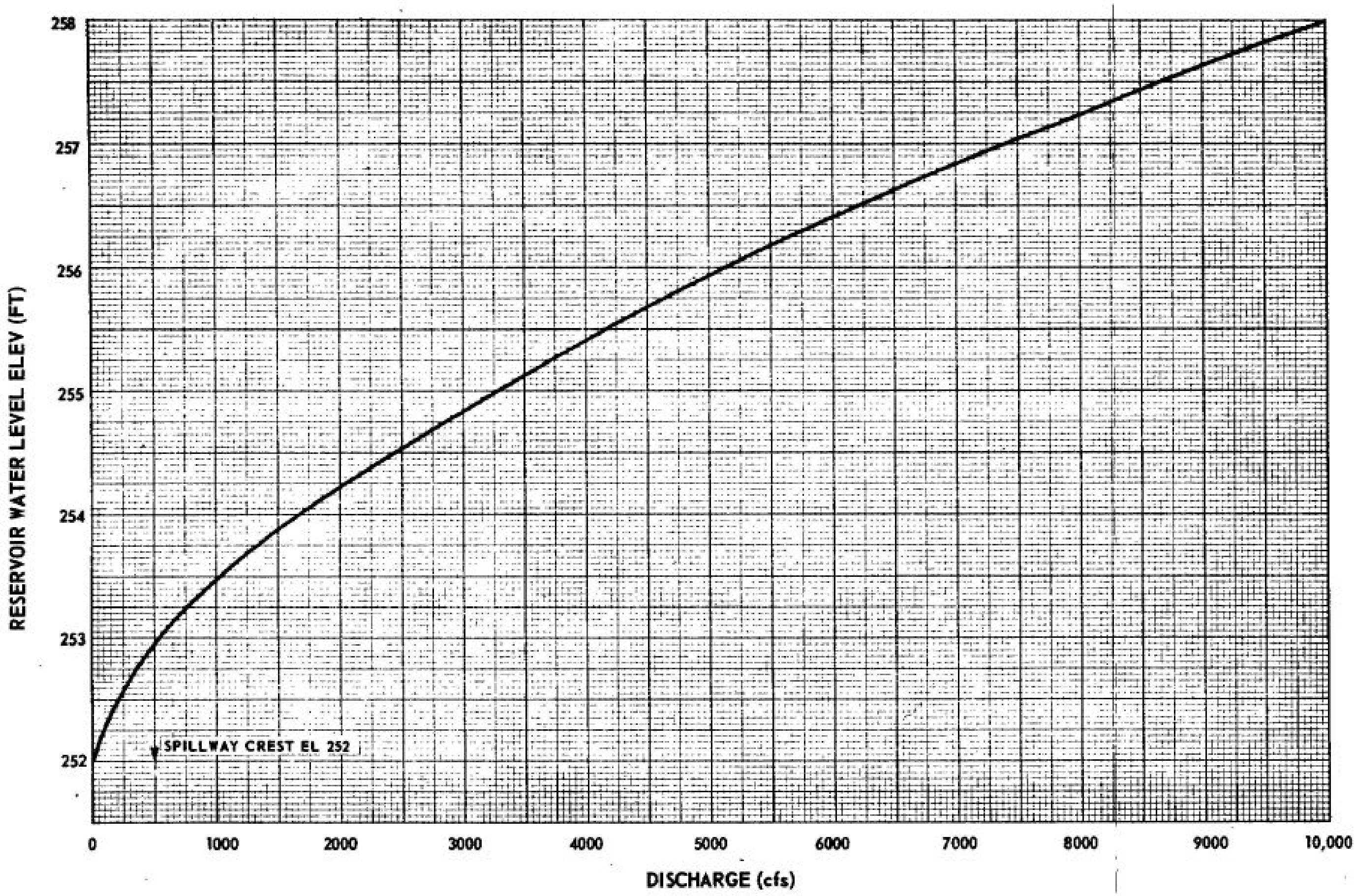


FIGURE 2.4.3-5

MAIN RESERVOIR AREA AND CAPACITY CURVES

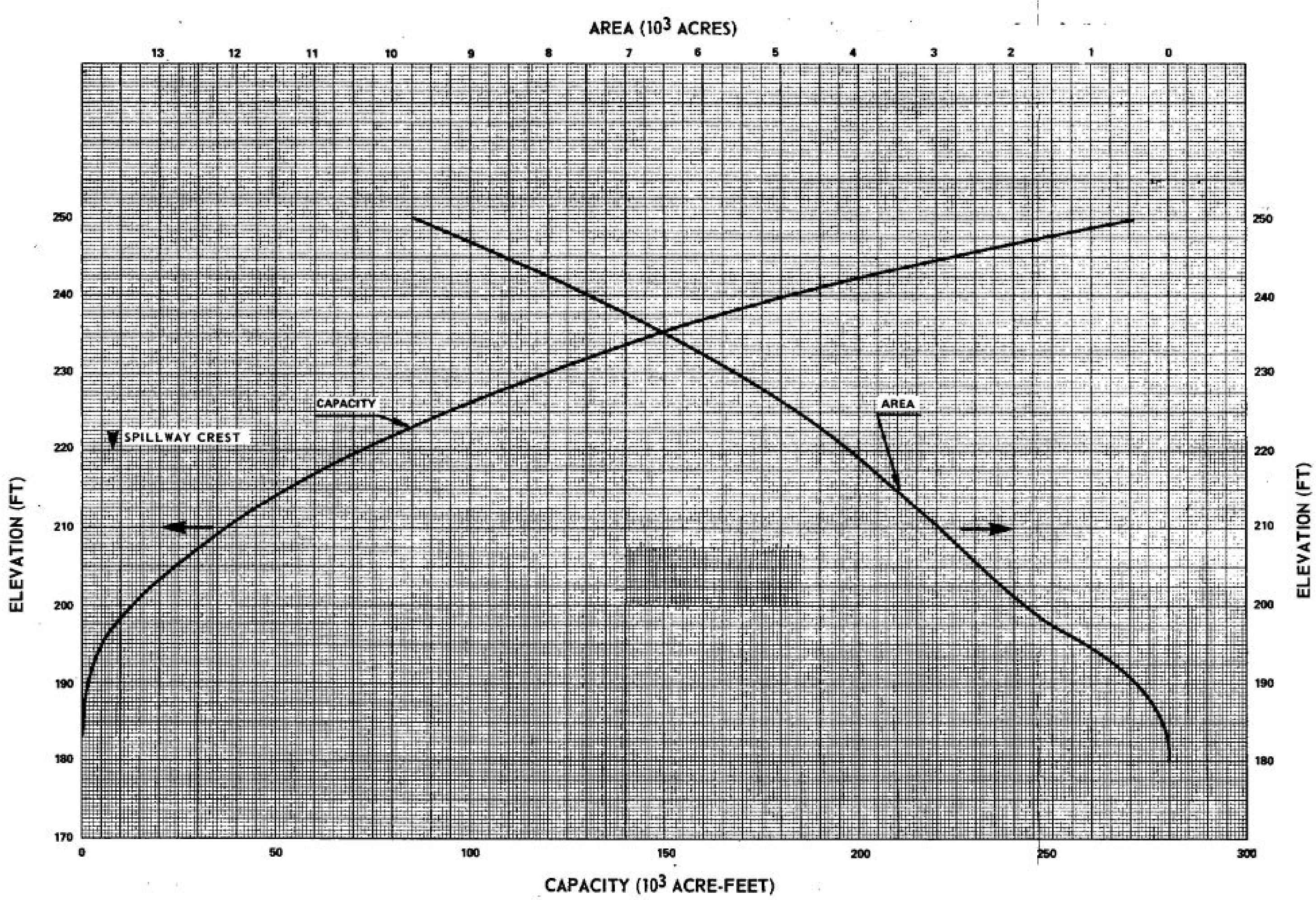


FIGURE 2.4.3-6

AUXILIARY RESERVOIR AREA AND CAPACITY CURVES

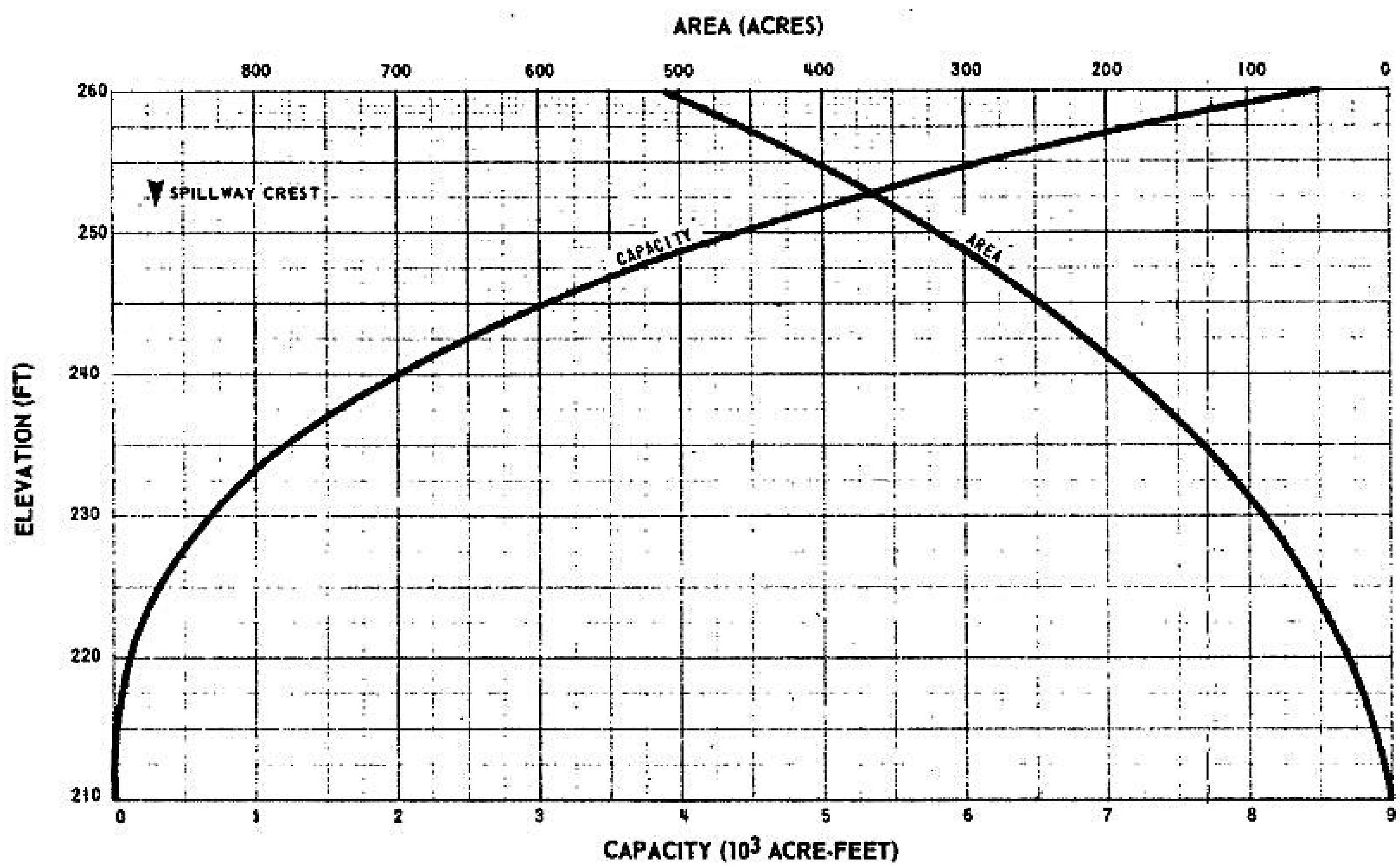


FIGURE 2.4.3-7

PROBABLE MAXIMUM FLOOD HYDROGRAPH UNDER NATURAL CONDITIONS FOR ENTIRE BUCKHORN CREEK BASIN

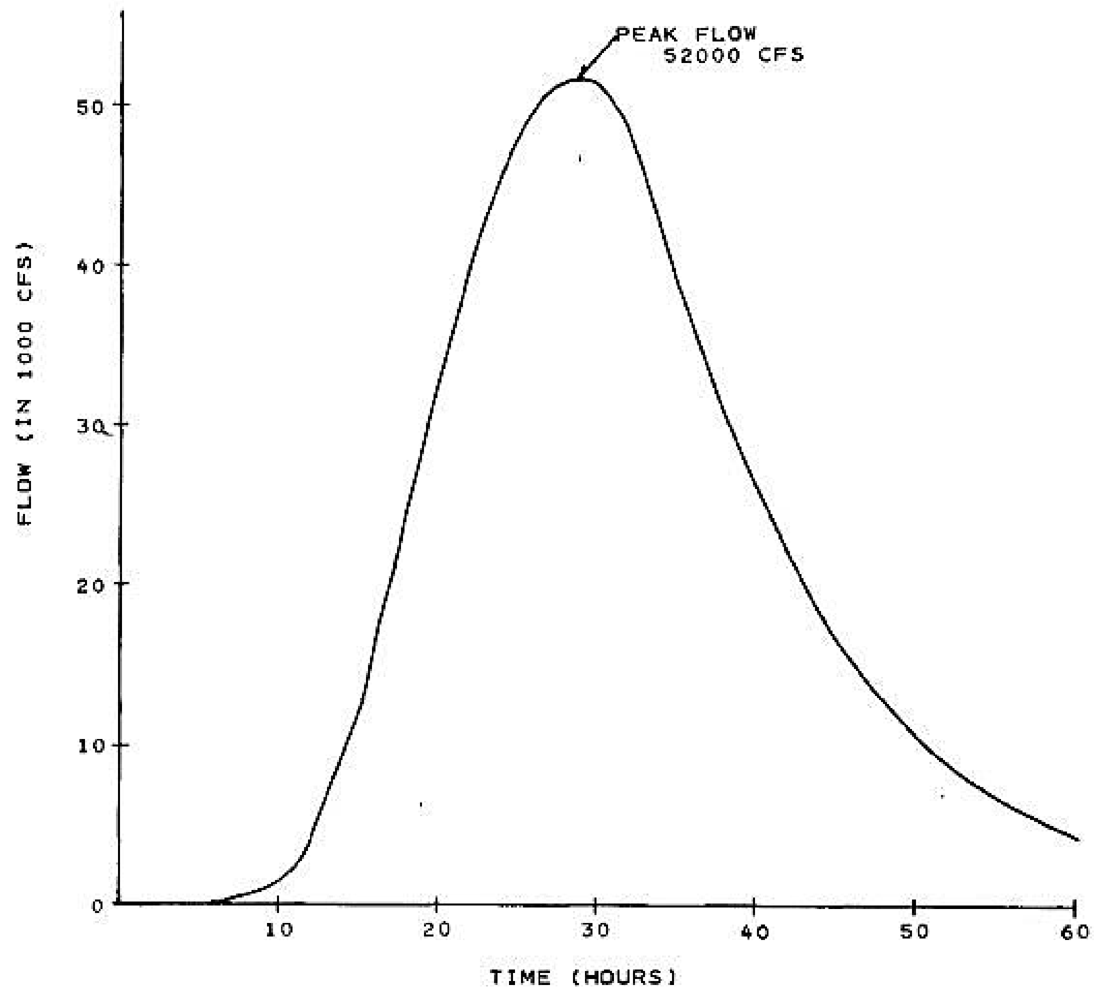


FIGURE 2.4.3-8

INFLOW, OUTFLOW, WATER LEVEL HYDROGRAPHS (PMP = 26.7 IN.) AUXILIARY RESERVOIR

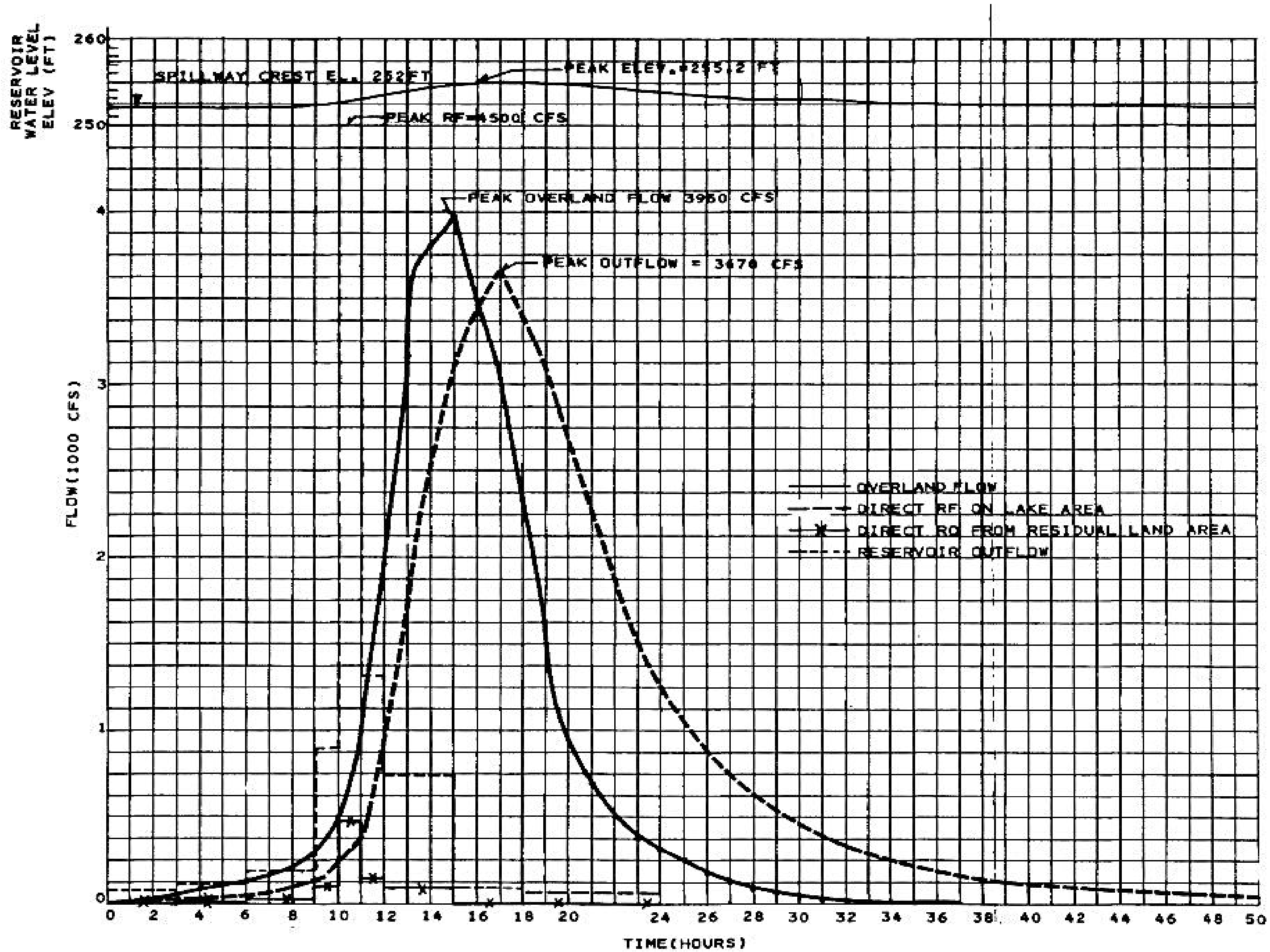


FIGURE 2.4.3-9

INFLOW, OUTFLOW, WATER LEVEL HYDROGRAPHS (PMP = 36.32 IN.) AUXILIARY RESERVOIR

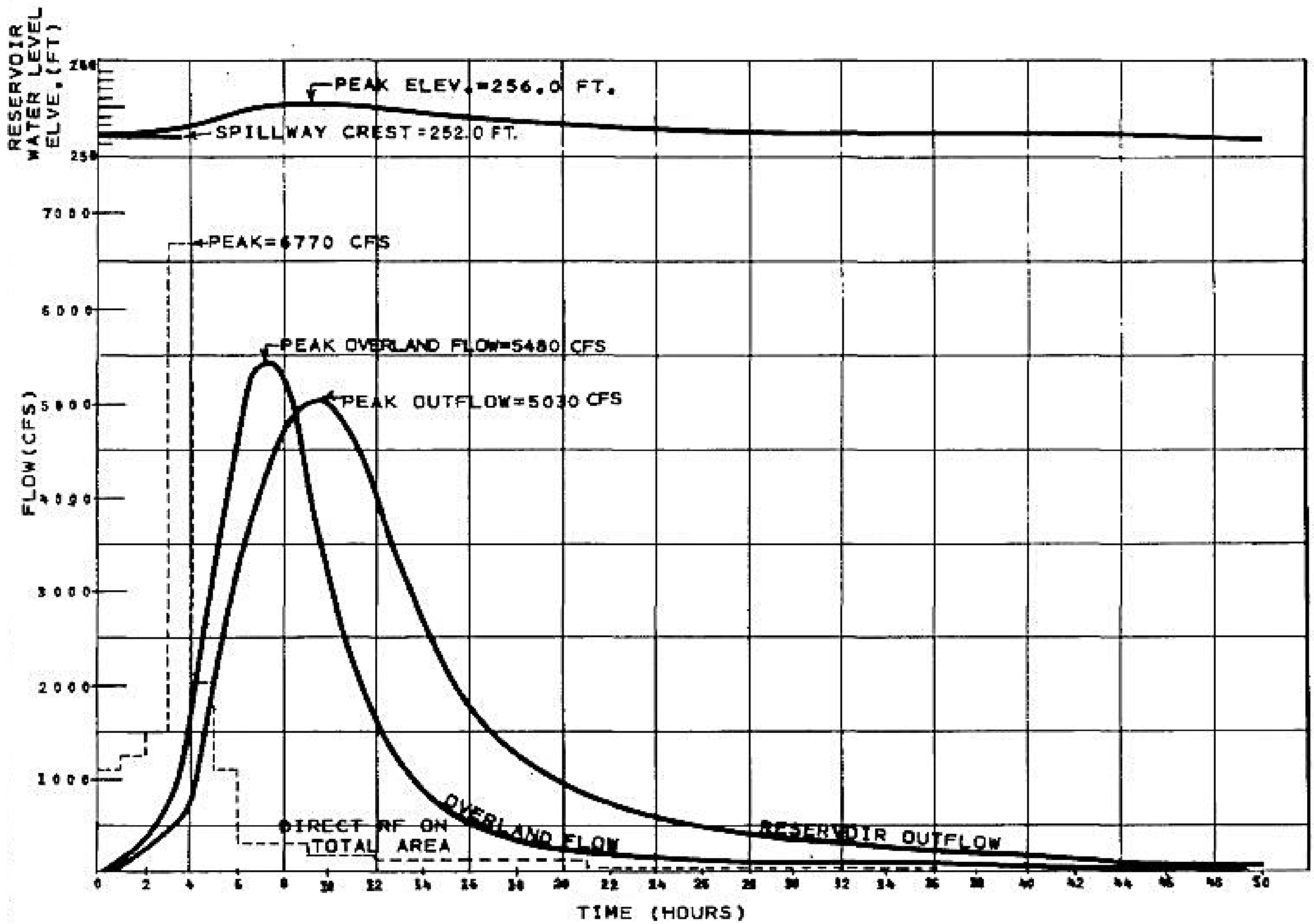


FIGURE 2.4.3-10

MAIN RESERVOIR INFLOW, OUTFLOW, WATER LEVEL HYDROGRAPHS WITHOUT ANTECEDENT FLOOD

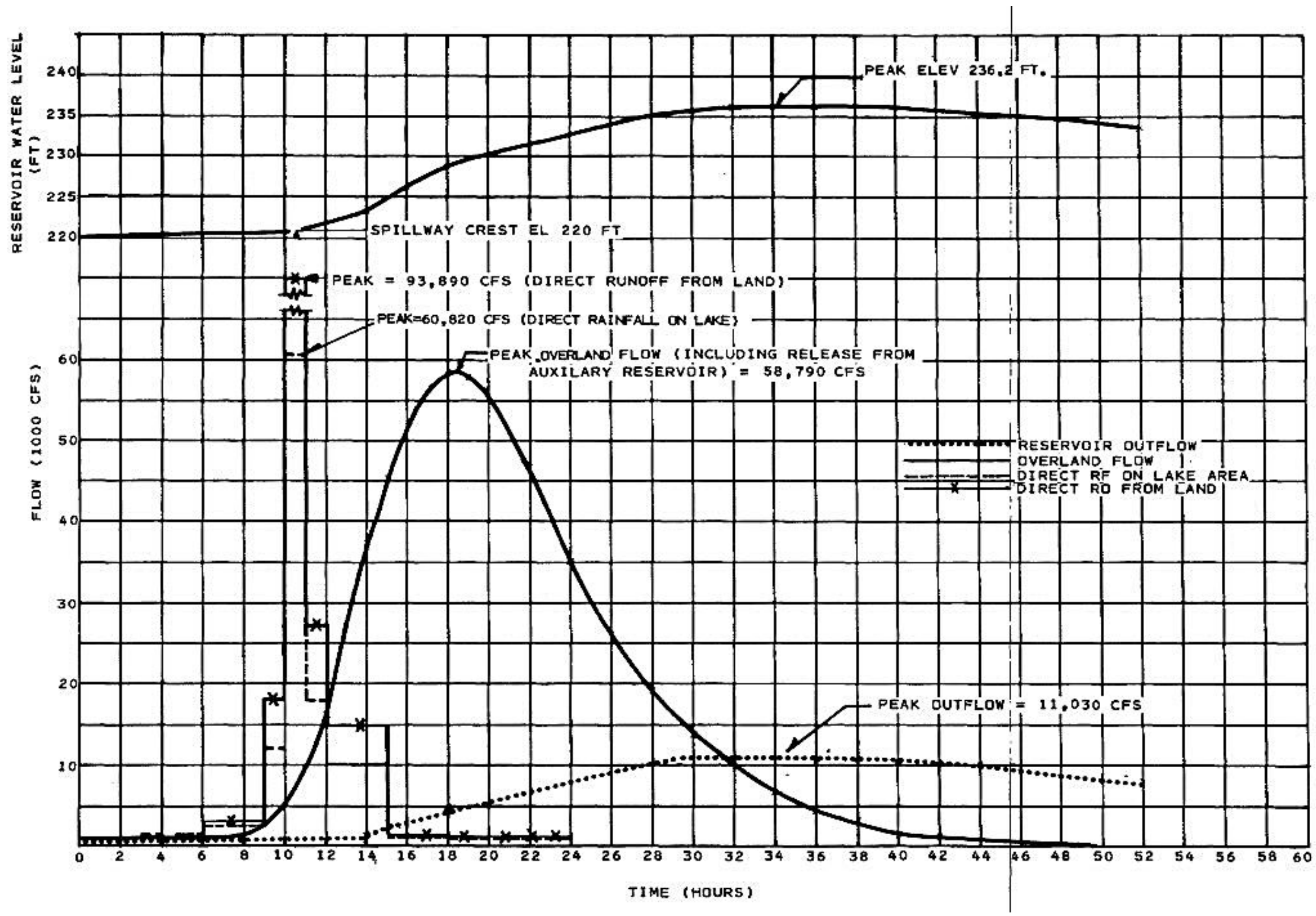


FIGURE 2.4.3-11

MAIN RESERVOIR INFLOW, OUTFLOW, WATER LEVEL HYDROGRAPHS WITH STANDARD PROJECT FLOOD FIVE DAYS PRIOR TO PMP FLOOD

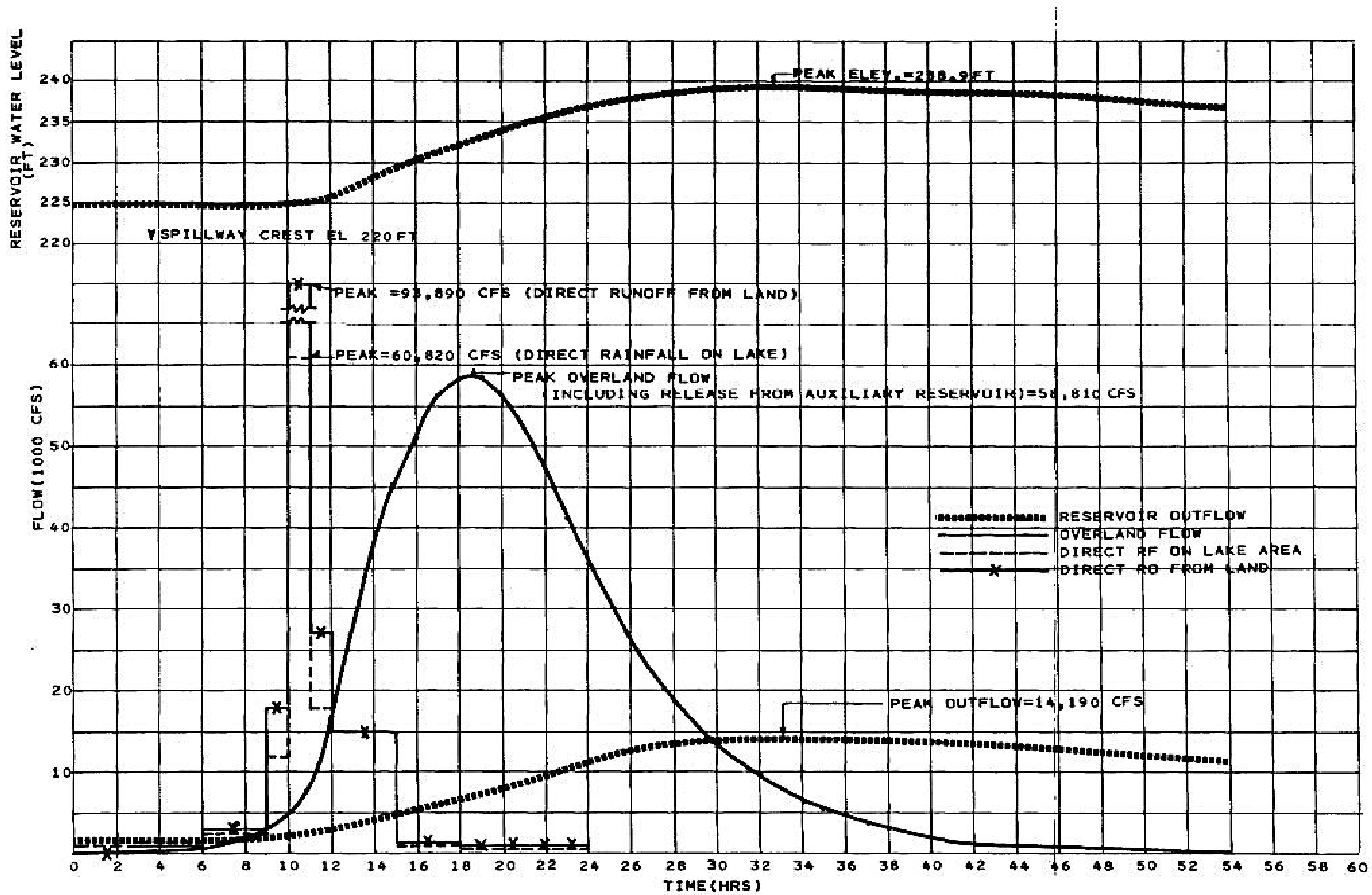


FIGURE 2.4.5-1

WIND FETCH

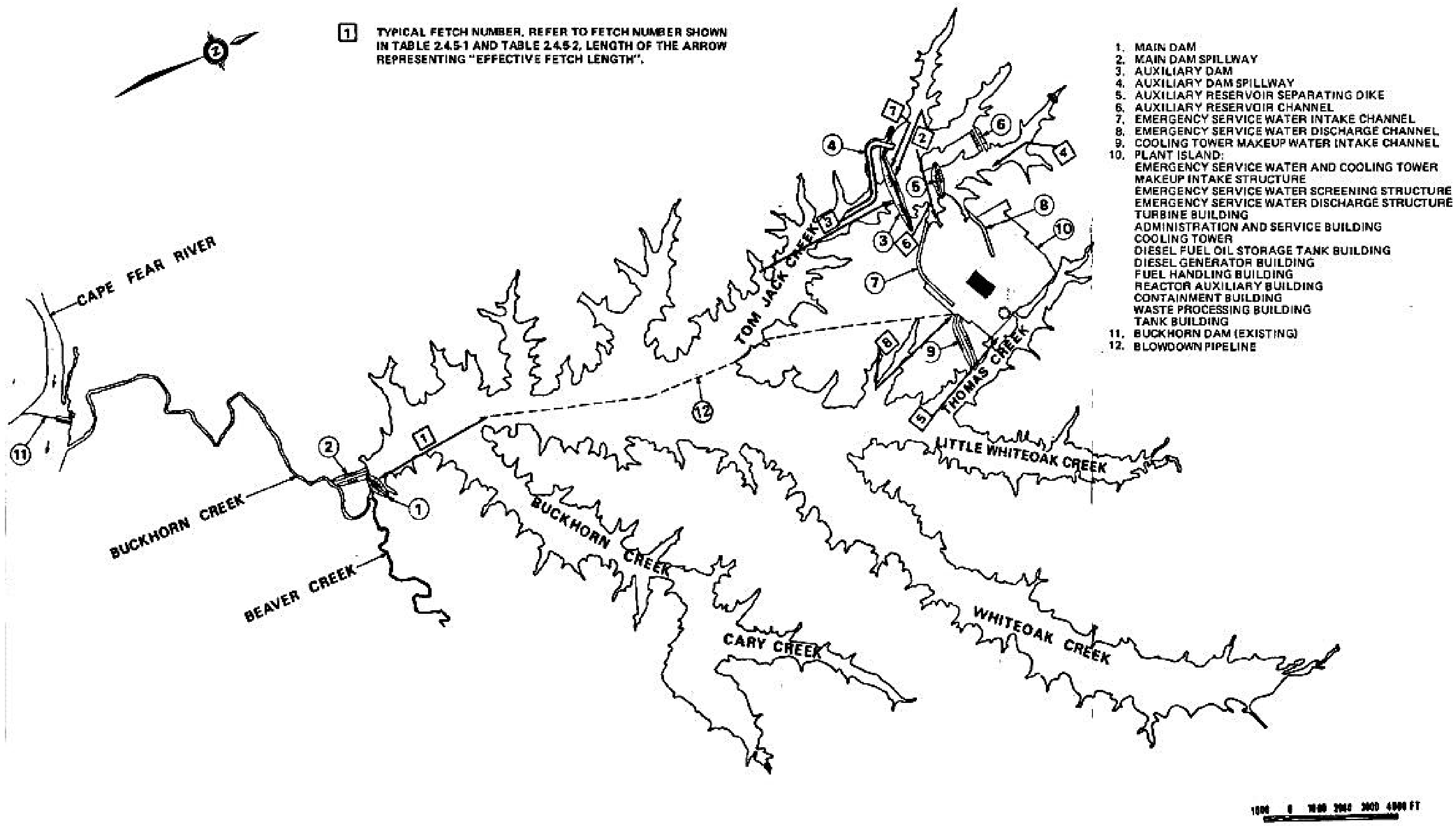
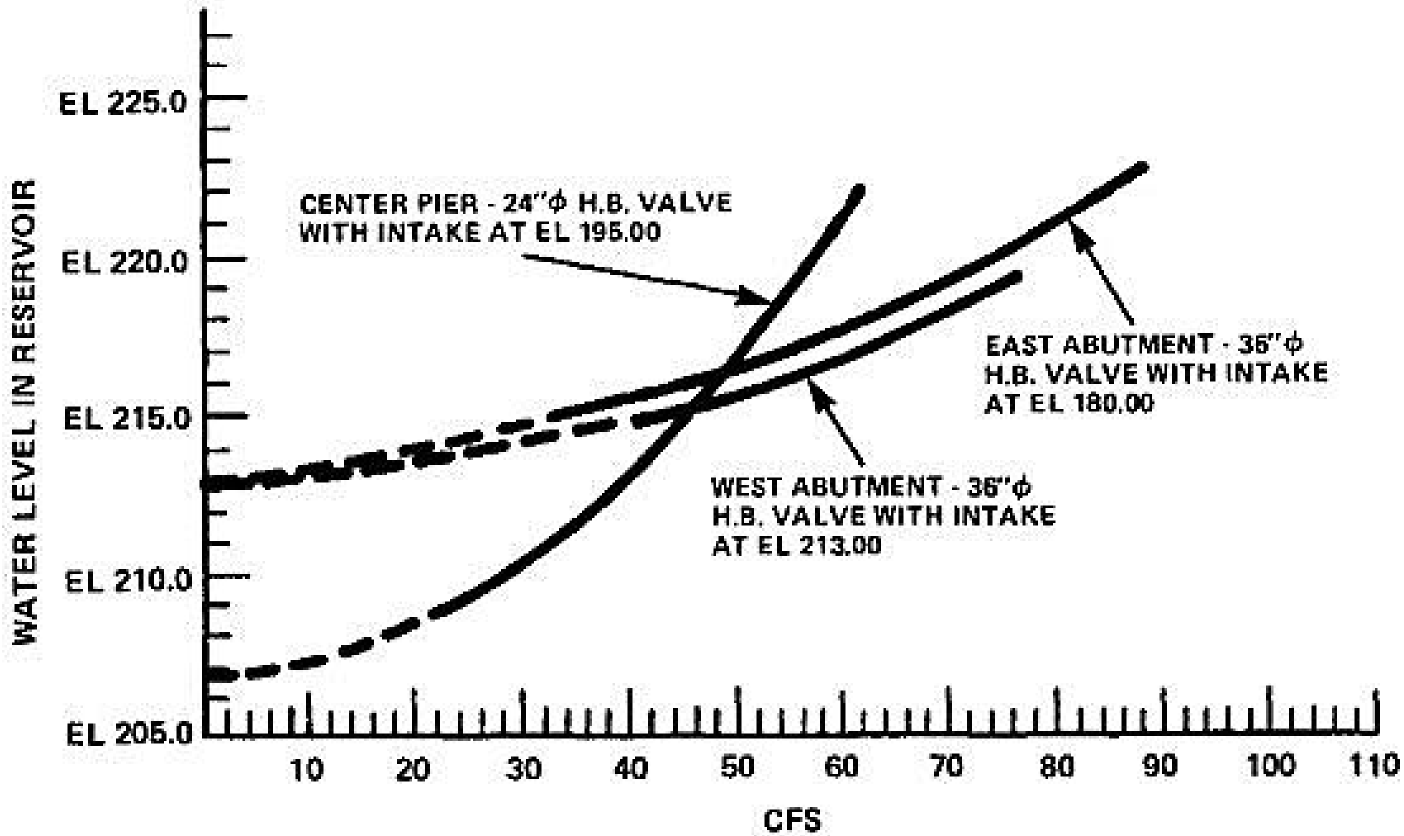


FIGURE 2.4.8-1

MAIN DAM DISCHARGE THRU HOWELL BUNGER VALVES



NOTE: VALVES ARE NOT TO BE OPERATED
WHEN SPILLWAY IS IN OPERATION.

FIGURE 2.4.11-2

BUCKHORN CREEK 1-DAY LOW FLOW FREQUENCY ANALYSIS
(LOG PEARSON TYPE III DISTRIBUTION) 1941-1978

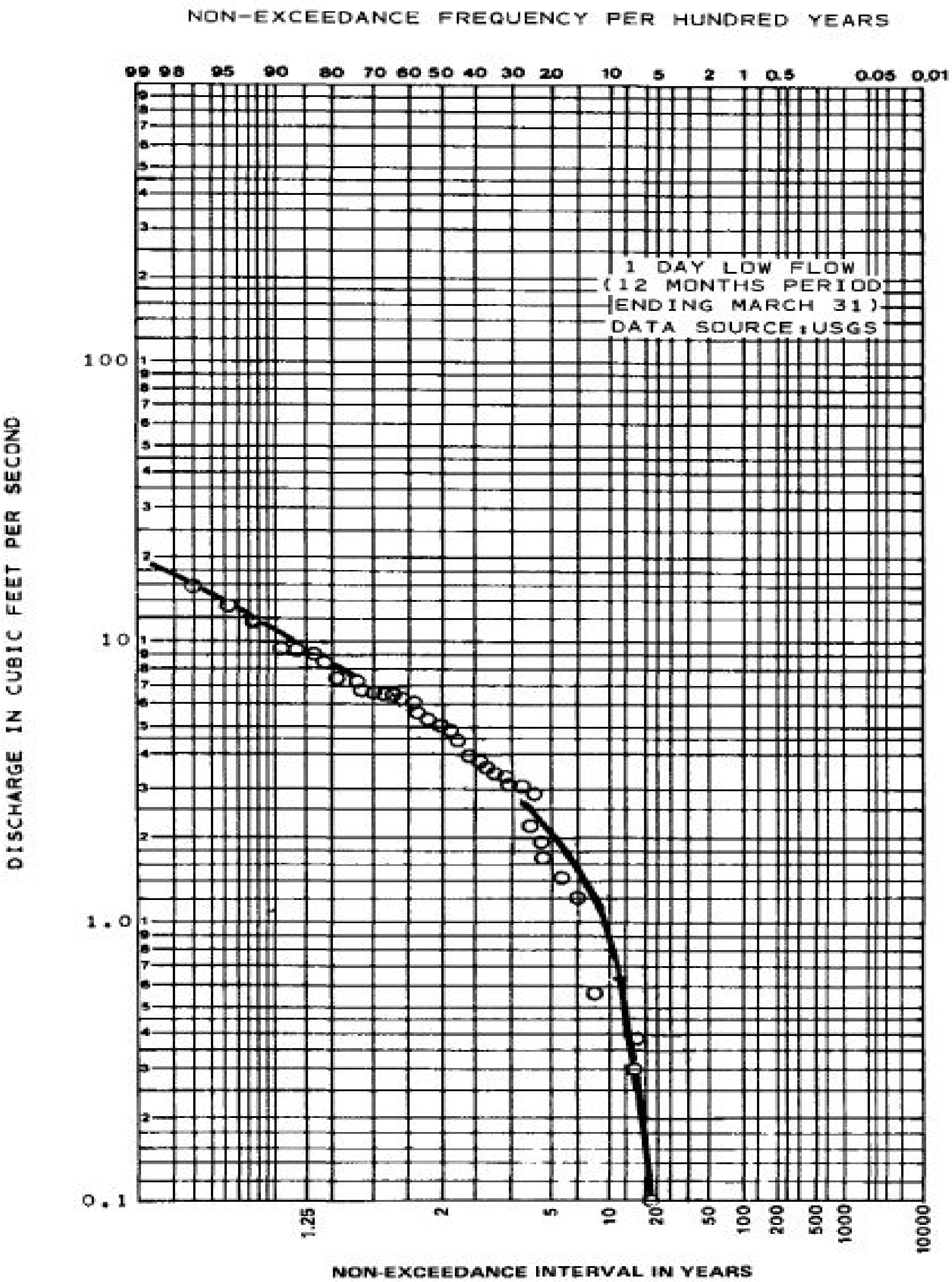


FIGURE 2.4.11-3

BUCKHORN CREEK 7 CONSECUTIVE DAYS LOW FLOW FREQUENCY ANALYSIS (LOG PEARSON TYPE III DISTRIBUTION) 1941-1978

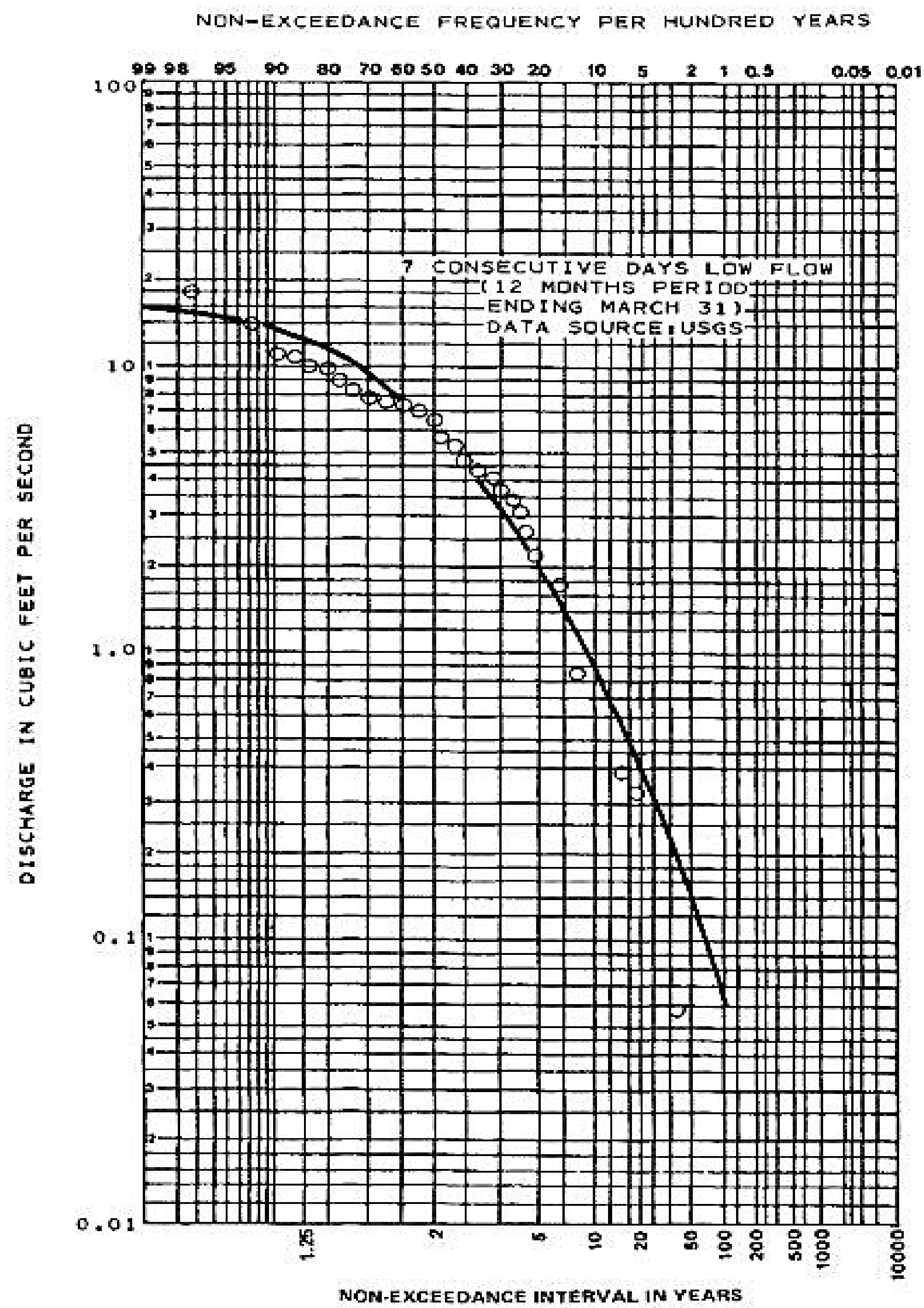


FIGURE 2.4.11-4

BUCKHORN CREEK 30 CONSECUTIVE DAYS LOW FLOW FREQUENCY ANALYSIS (LOG PEARSON TYPE III DISTRIBUTION) 1941-1978

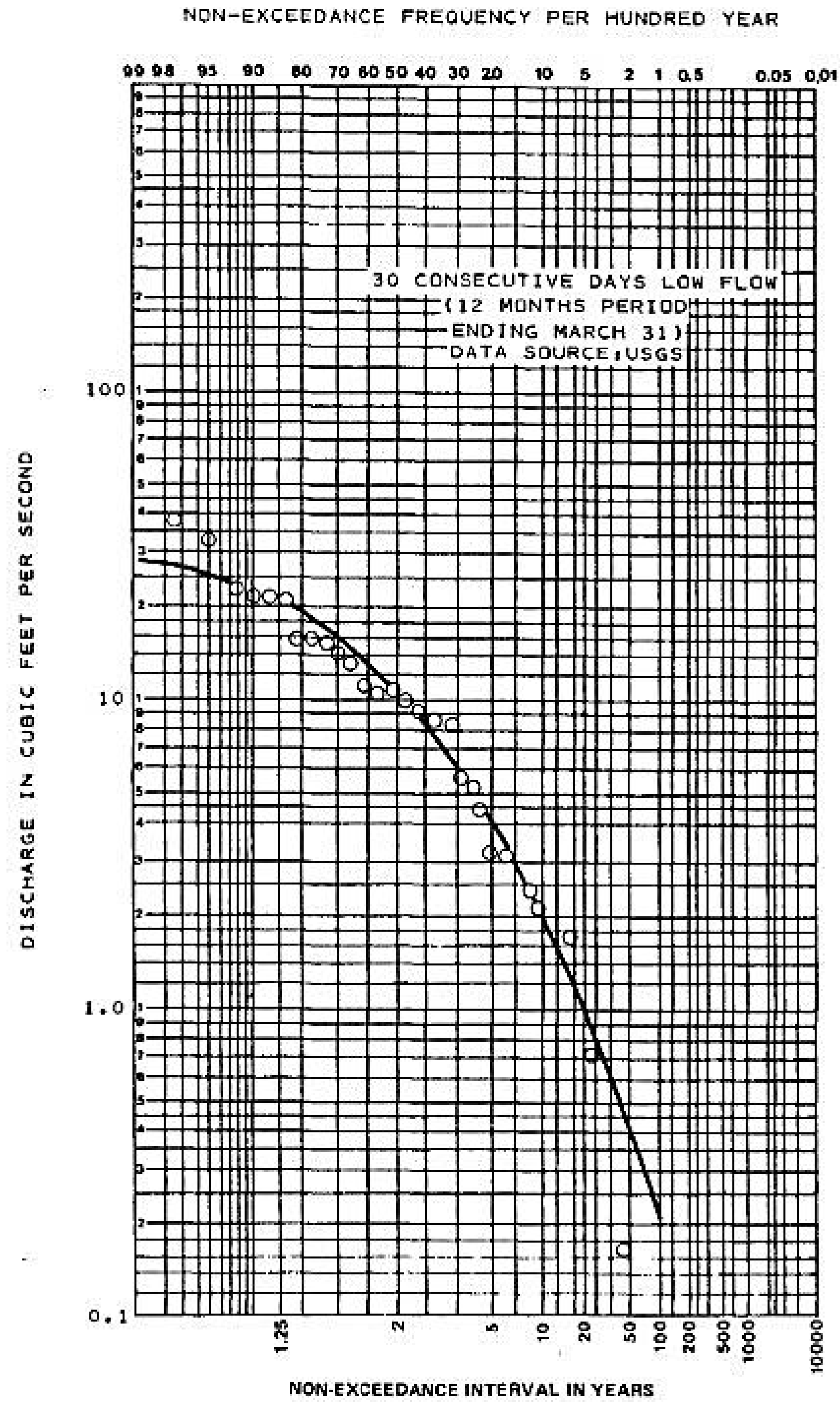


FIGURE 2.4.11-5

BUCKHORN CREEK 60 CONSECUTIVE DAYS LOW FLOW FREQUENCY ANALYSIS (LOG PEARSON TYPE III DISTRIBUTION) 1941-1978

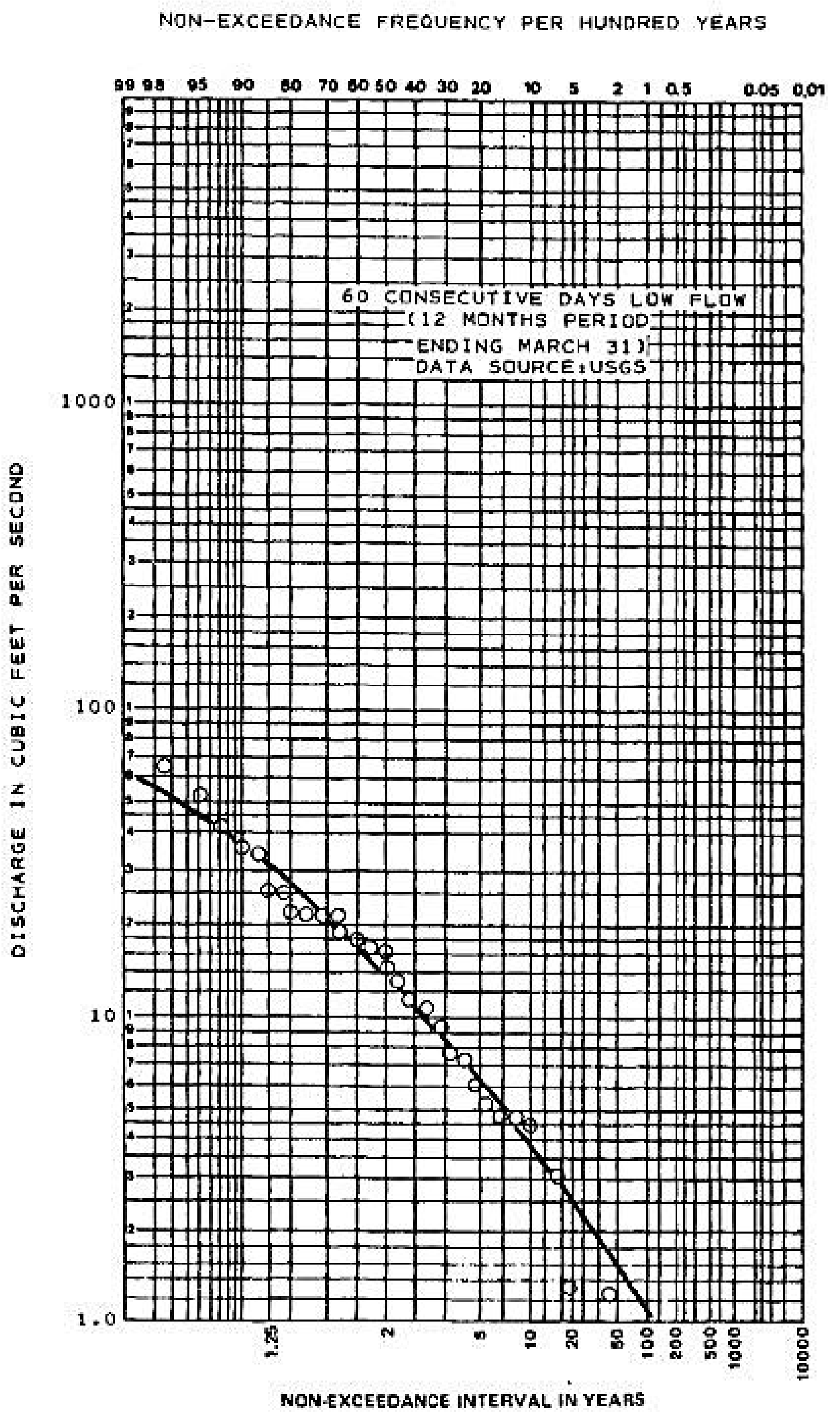
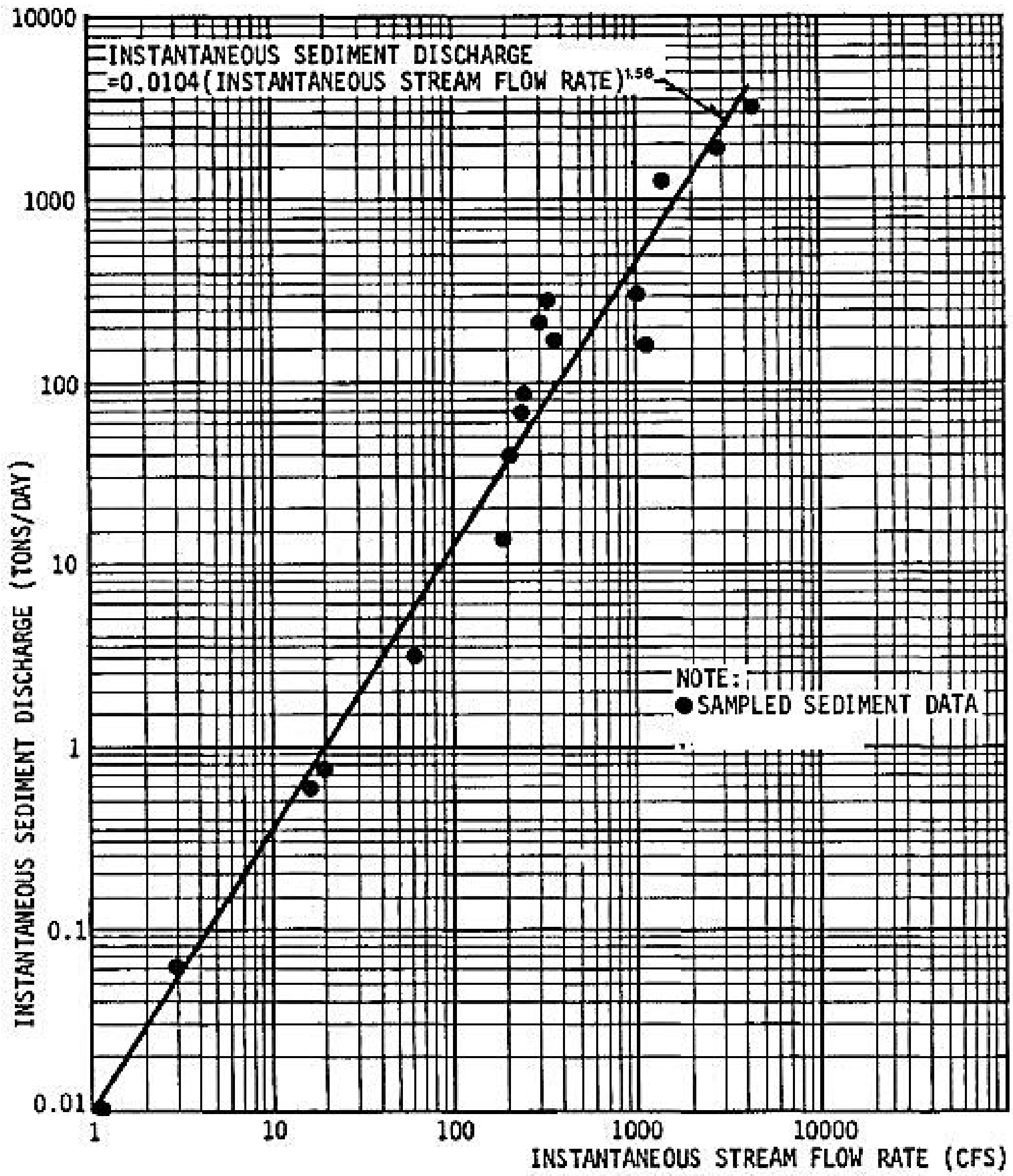


FIGURE 2.4.11-9

SEDIMENT DISCHARGE RATING CURVE BUCKHORN CREEK NEAR CORINTH, N.C.



DATA SOURCE:
"WATER RESOURCES DATA OF
NORTH CAROLINA" USGS, 1973-1977

FIGURE 2.4.13-1

PRE-CONSTRUCTION PIEZOMETRIC LEVELS

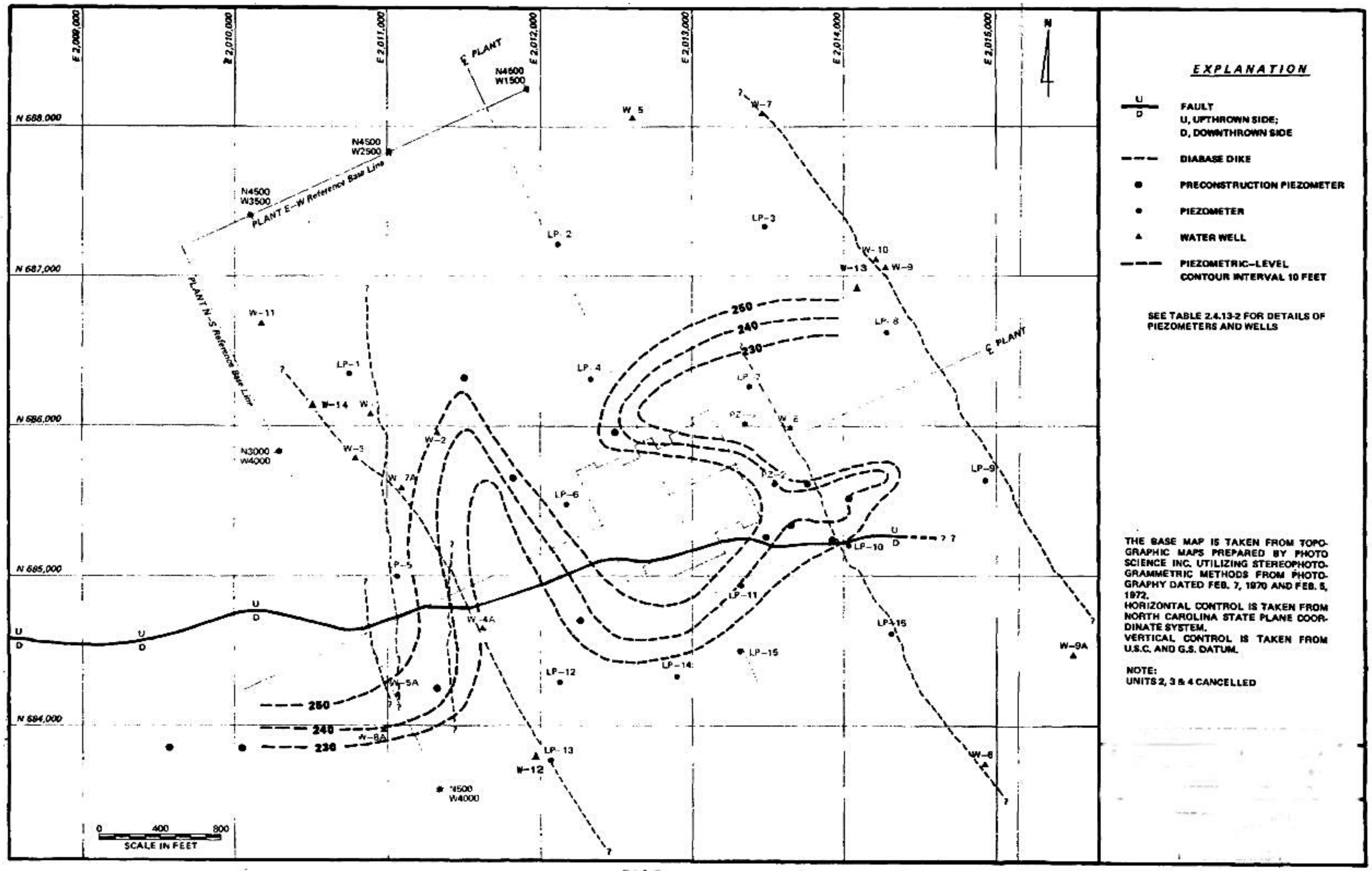


FIGURE 2.4.13-2

PIEZOMETRIC LEVELS, WINTER 1979-1980

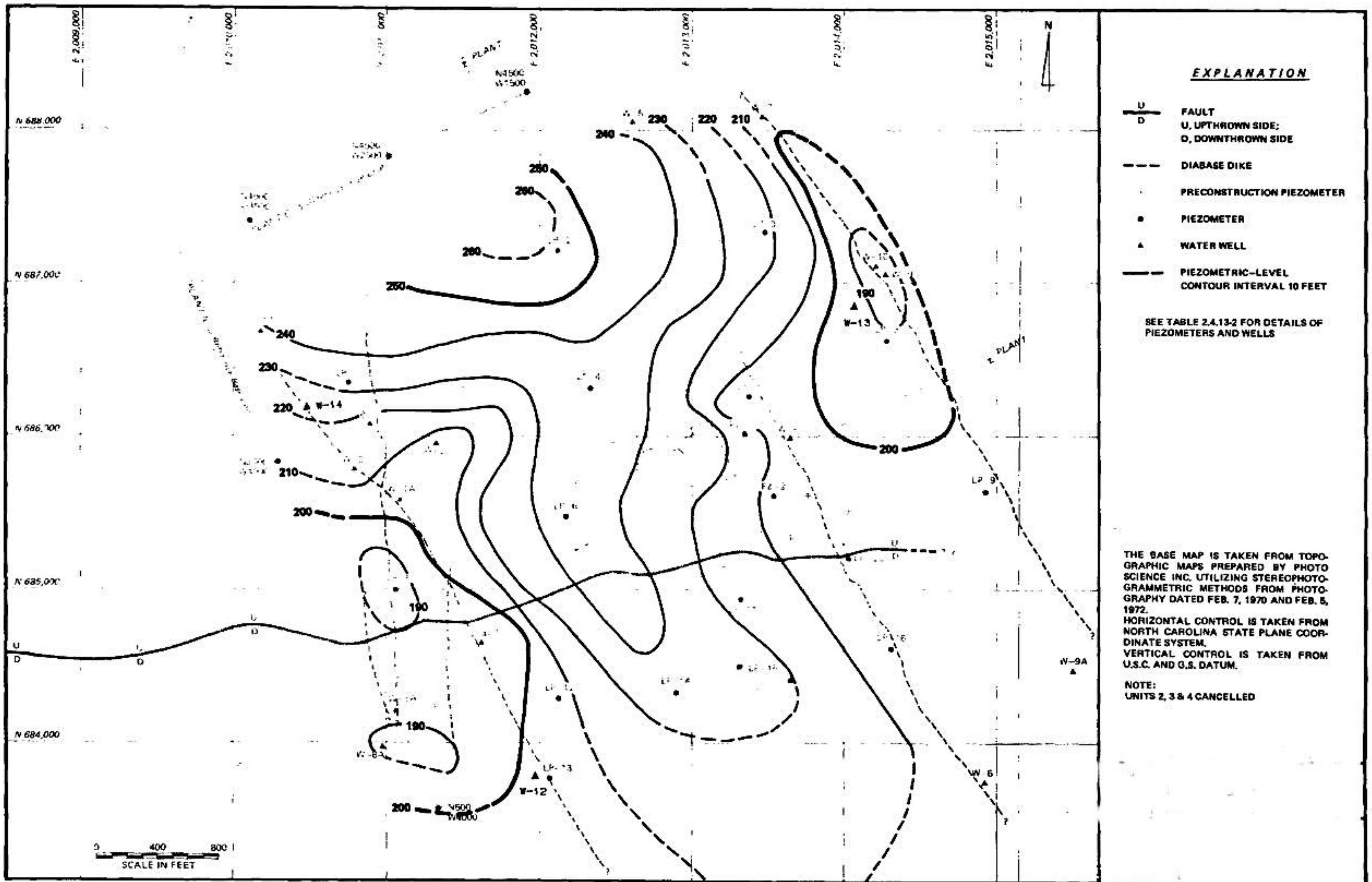
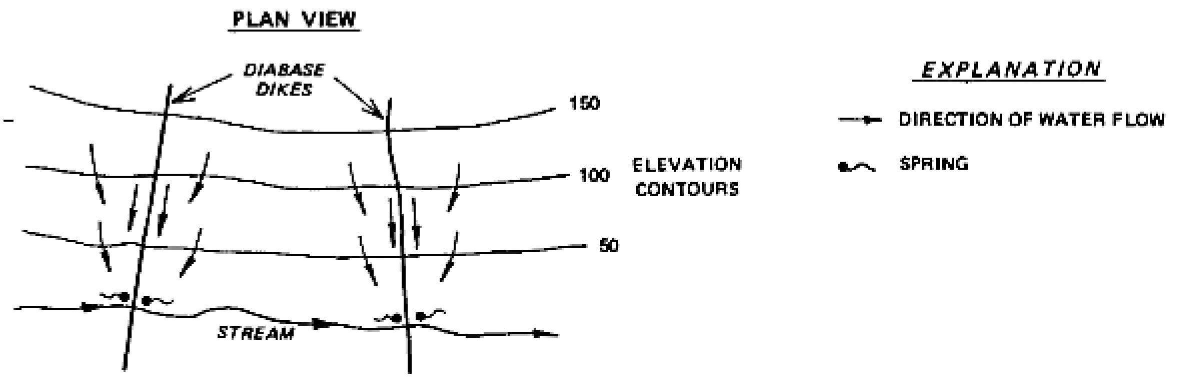


FIGURE 2.4.13-3

DIAGRAMMATIC RELATIONSHIP OF DIKES AND FRACTURES TO GROUND WATER



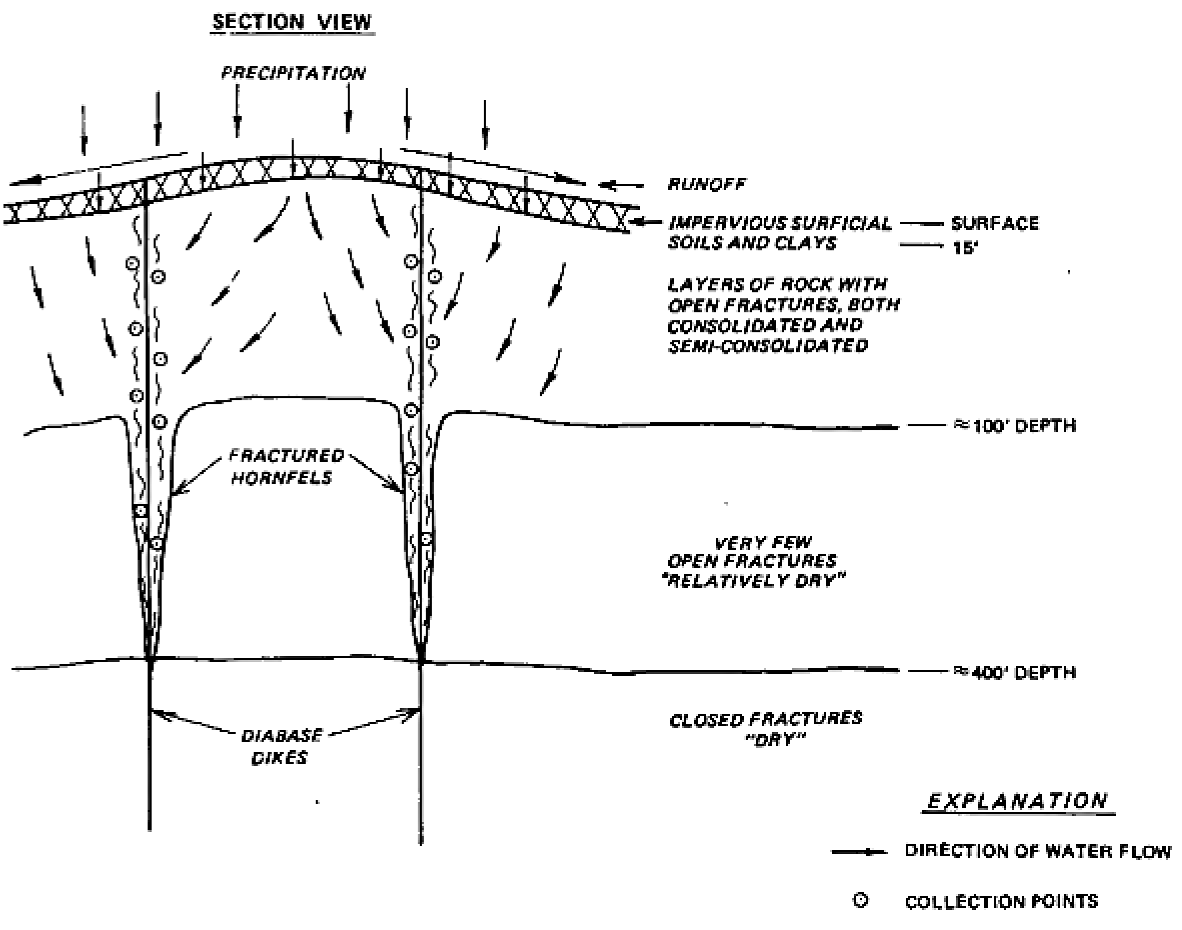


FIGURE 2.4.13-4

PUBLIC WELLS WITHIN 10 MILES OF THE PLANT

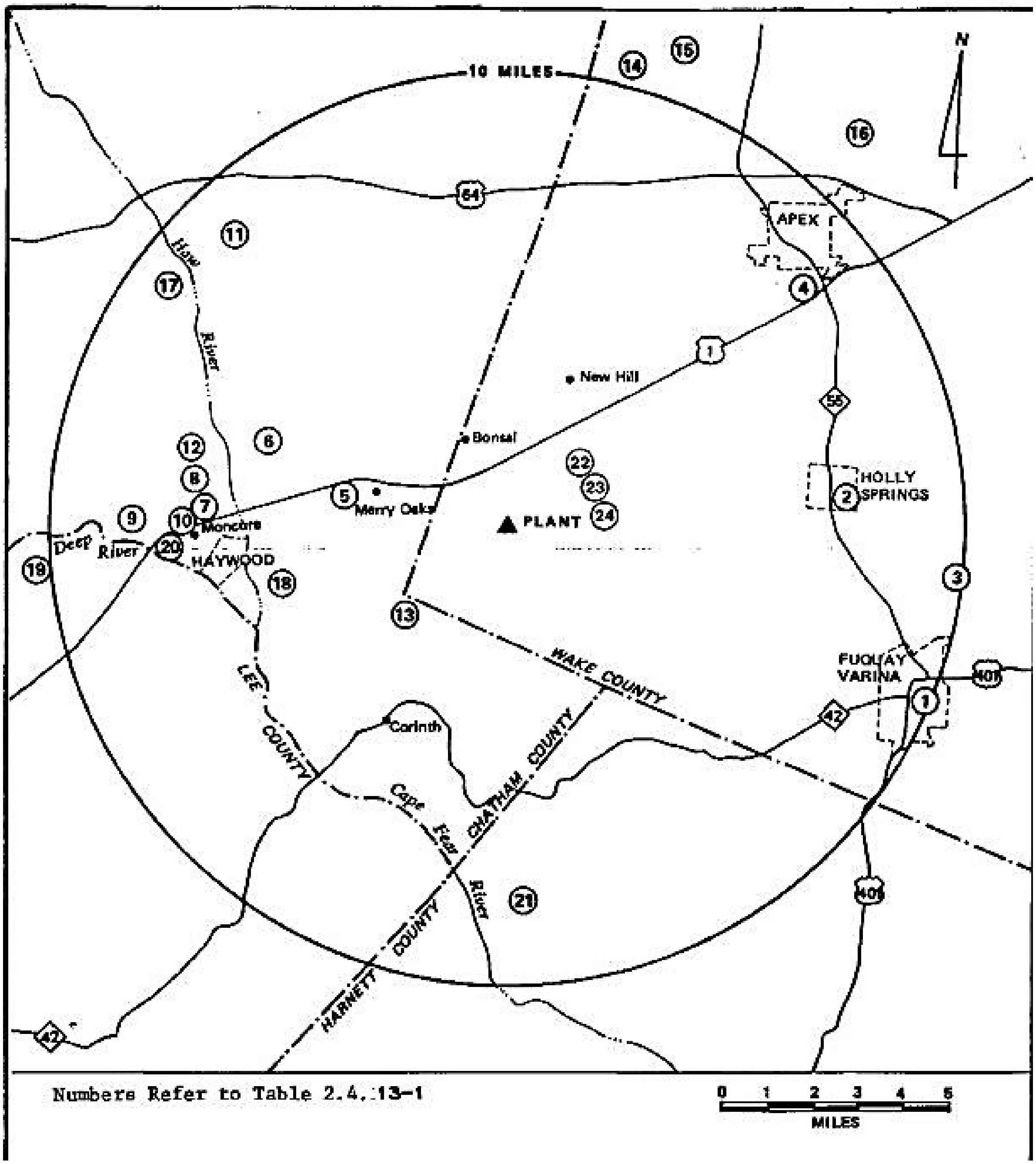


FIGURE 2.5.1-1

REGIONAL PHYSIOGRAPHIC MAP

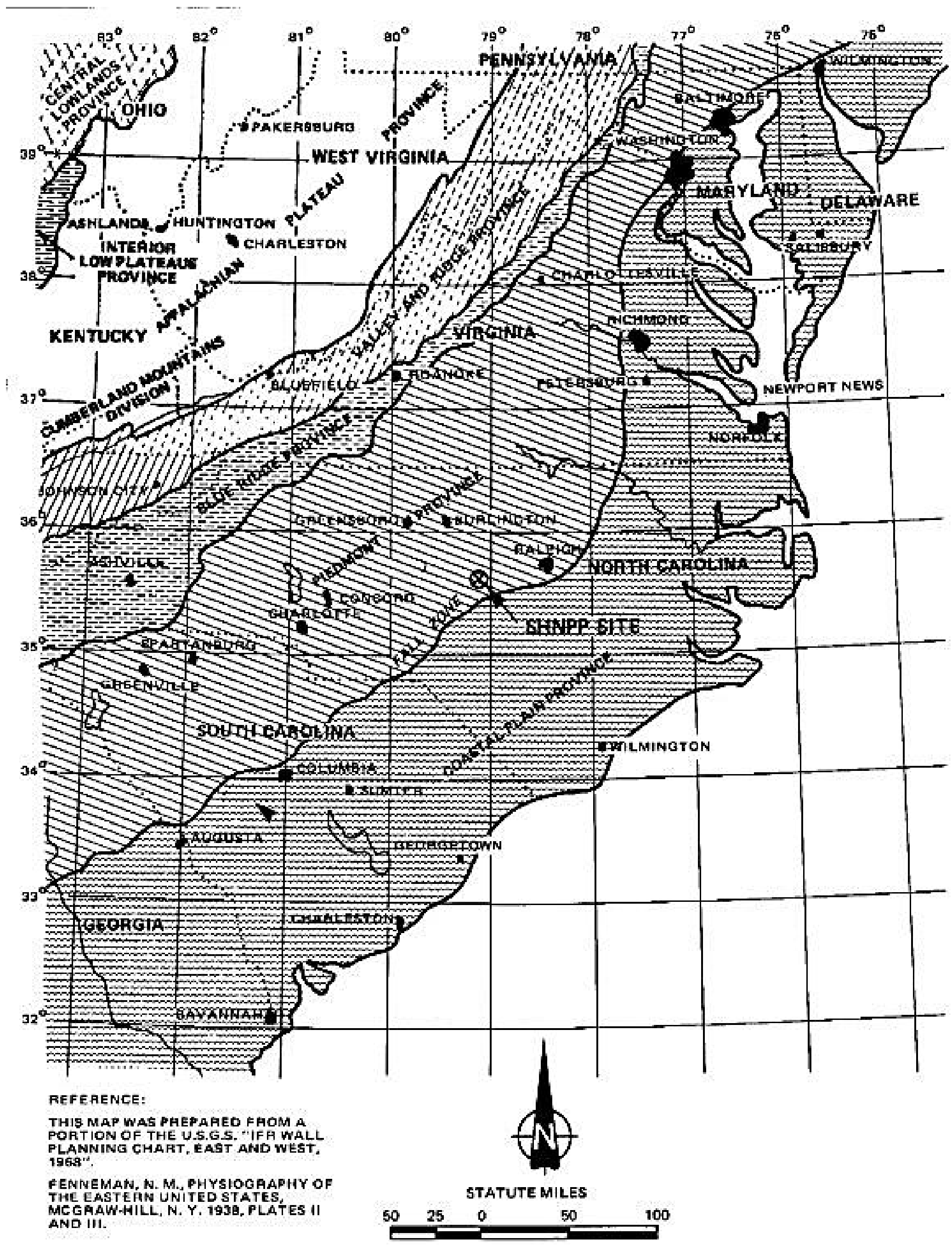
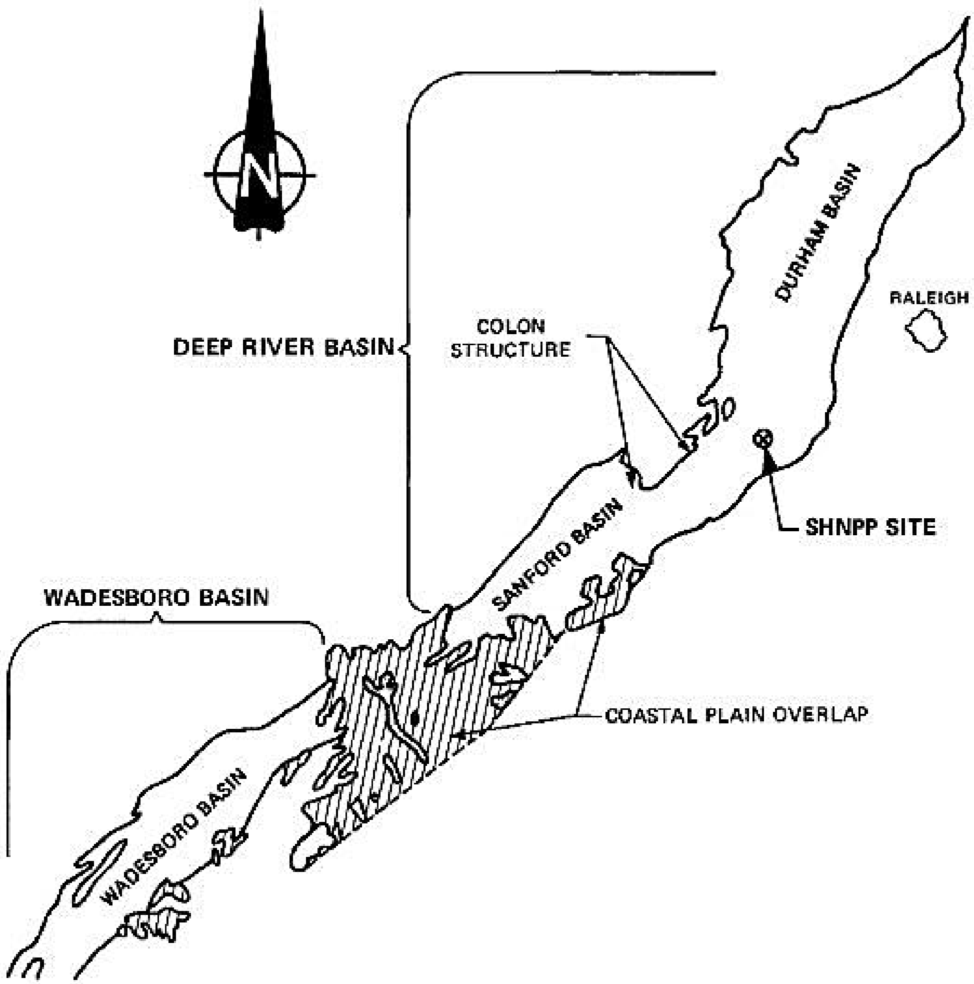


FIGURE 2.5.1-2

DEEP RIVER-WADESBORO TRIASSIC BASIN



REFERENCES:

1. GEOLOGIC MAP OF NORTH CAROLINA, COMPILED BY DEPARTMENT OF CONSERVATION AND DEVELOPMENT; WILLIAM D. SAUNDERS, DIRECTOR, DIVISION OF MINERAL RESOURCES, JASPER L. STUCKEY, STATE GEOLOGIST. SCALE 1: 500,000, DATED 1958.

2. MANN, VIRGIL AND ZABLOCKI, F.S., 1961, "GRAVITY FEATURES OF THE DEEP RIVER – WADESBORO TRIASSIC BASIN", SOUTHEASTERN GEOLOGY, VOL. 2, NO. 4.

NOTE:

MAP OF THE DEEP RIVER – WADESBORO BASIN SHOWING THE LOCATION OF THE FOUR STRUCTURE UNITS: DURHAM BASIN, COLON CROSS STRUCTURE, SANFORD BASIN, AND THE WADESBORO BASIN.

FIGURE 2.5.1-3

REGIONAL GEOLOGIC MAP

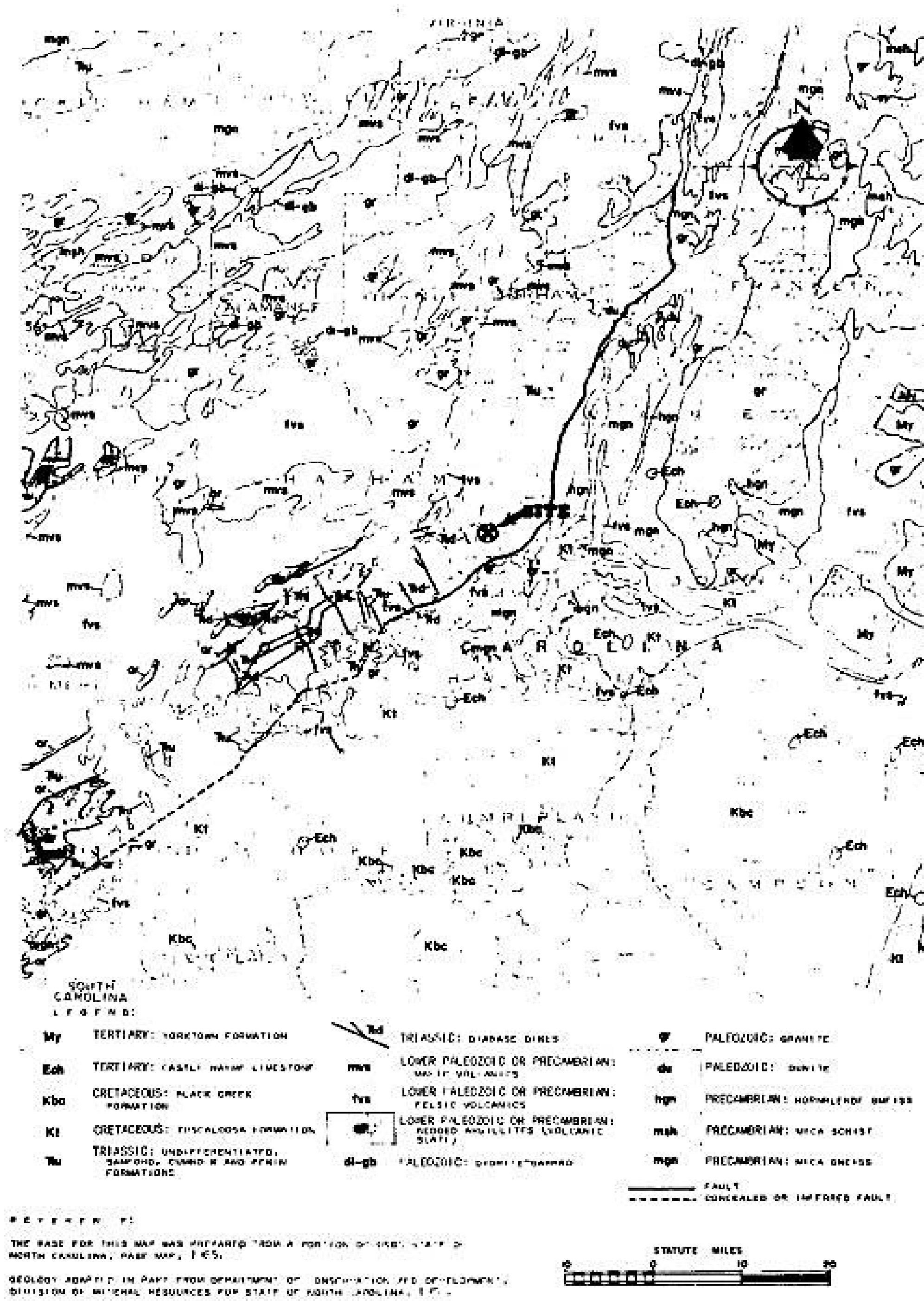
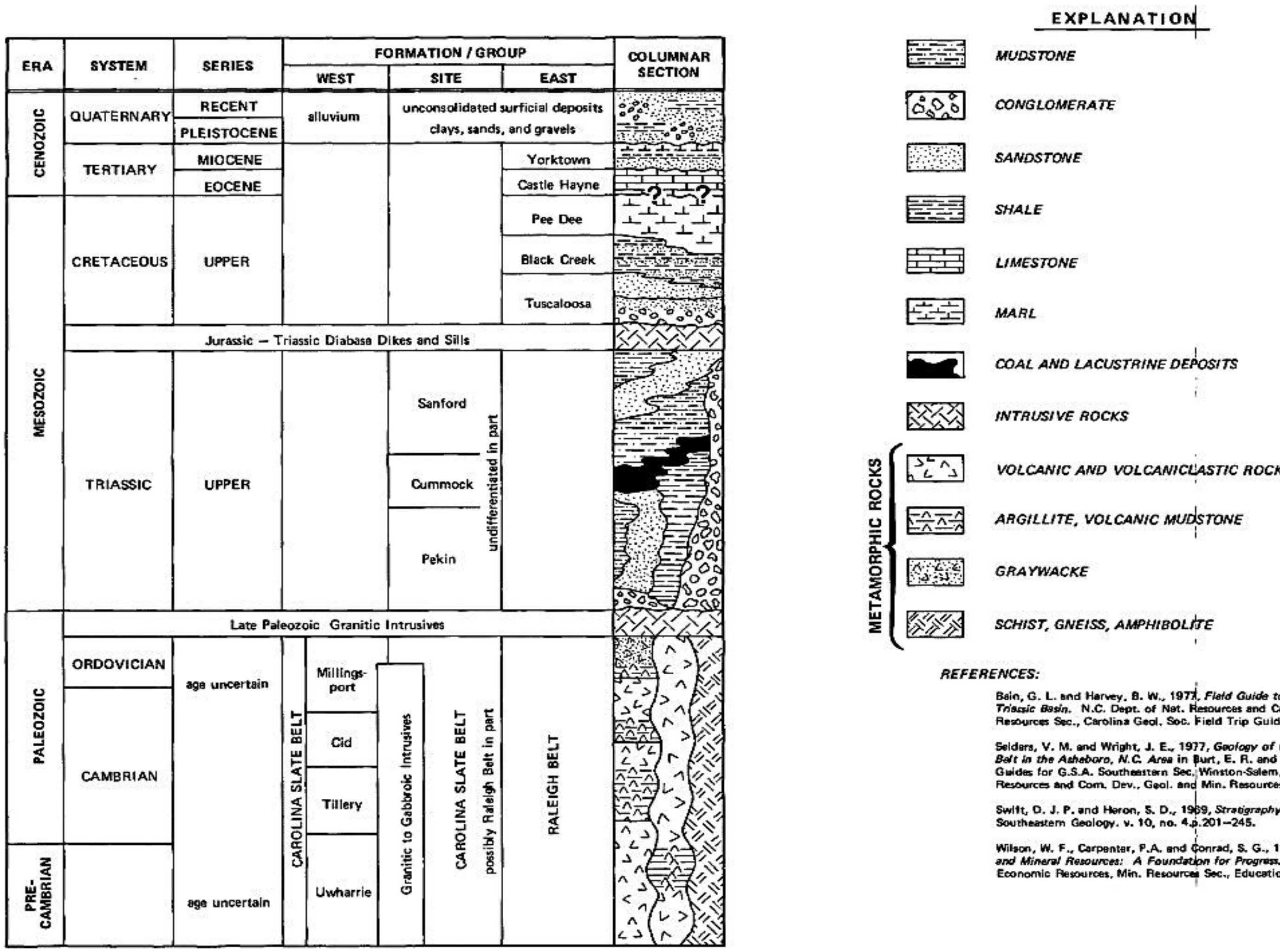


FIGURE 2.5.1-3a

REGIONAL STRATAGRAPHIC COLUMN



REFERENCES:

Bain, G. L. and Harvey, B. W., 1977, *Field Guide to the Geology of the Durham Triassic Basin.* N.C. Dept. of Nat. Resources and Com. Dev., Geol. and Min. Resources Sec., Carolina Geol. Soc. Field Trip Guidebook, 83p.

Seiders, V. M. and Wright, J. E., 1977, *Geology of the Carolina Volcanic State Belt in the Asheboro, N.C. Area* in Burt, E. R. and McKenzie, B. J. (eds.) Field Guides for G.S.A. Southeastern Sec. Winston-Salem, N.C., N.C. Dept. of Nat. Resources and Com. Dev., Geol. and Min. Resources Sec. p. 4–26.

Swift, D. J. P. and Heron, S. D., 1969, *Stratigraphy of the Carolina Cretaceous,* Southeastern Geology. v. 10, no. 4.p.201–245.

Wilson, W. F., Carpenter, P.A. and Conrad, S. G., 1976, *North Carolina Geology and Mineral Resources: A Foundation for Progress.* N.C. Dept. of Natural and Economic Resources, Min. Resources Sec., Educational Series no. 4, 78p.

FIGURE 2.5.1-4

GENERALIZED SITE AREA GEOLOGY

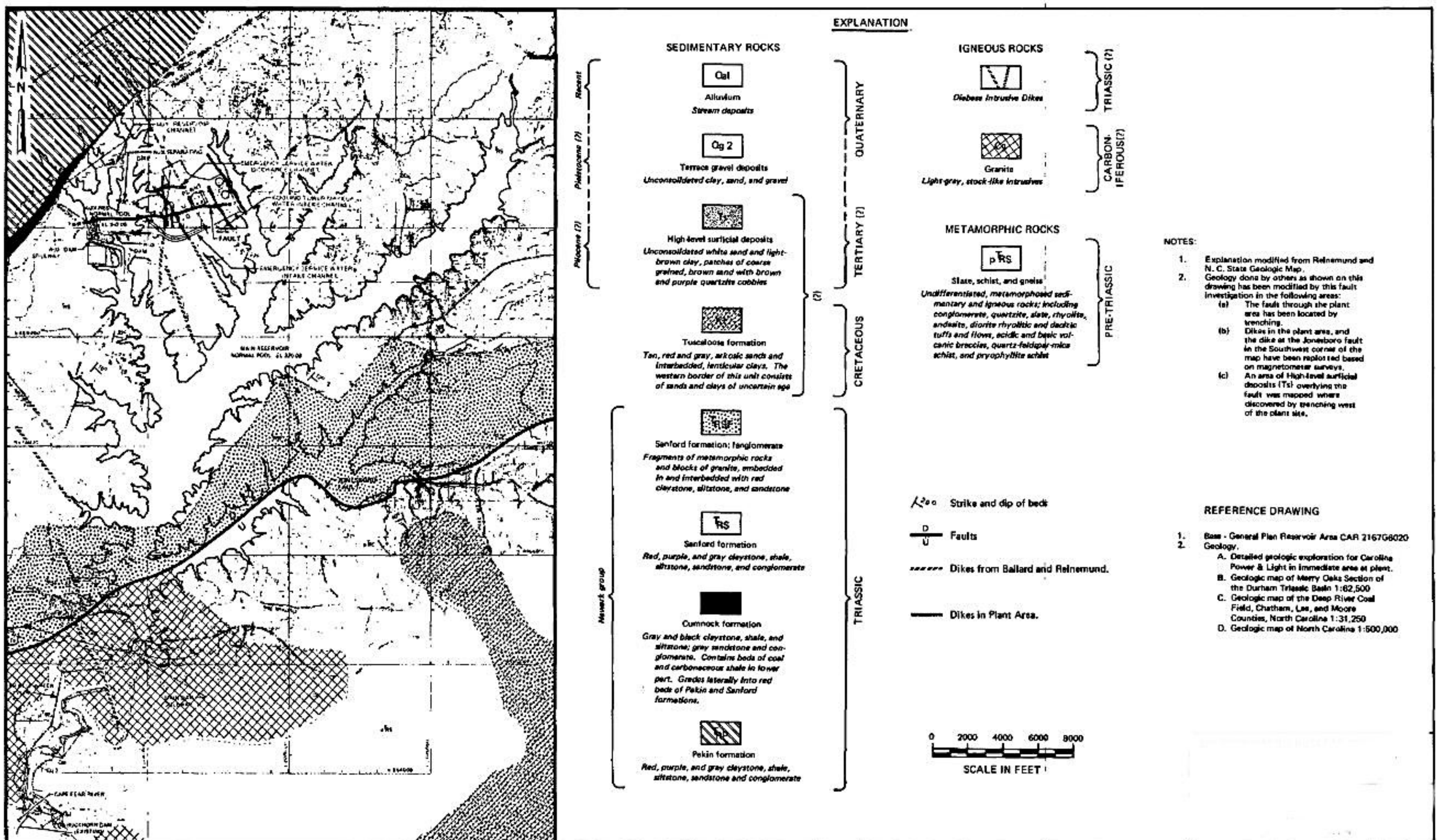


FIGURE 2.5.1-5

BOUGER ANOMALY MAP OF THE DEEP RIVER WADESBORO BASIN

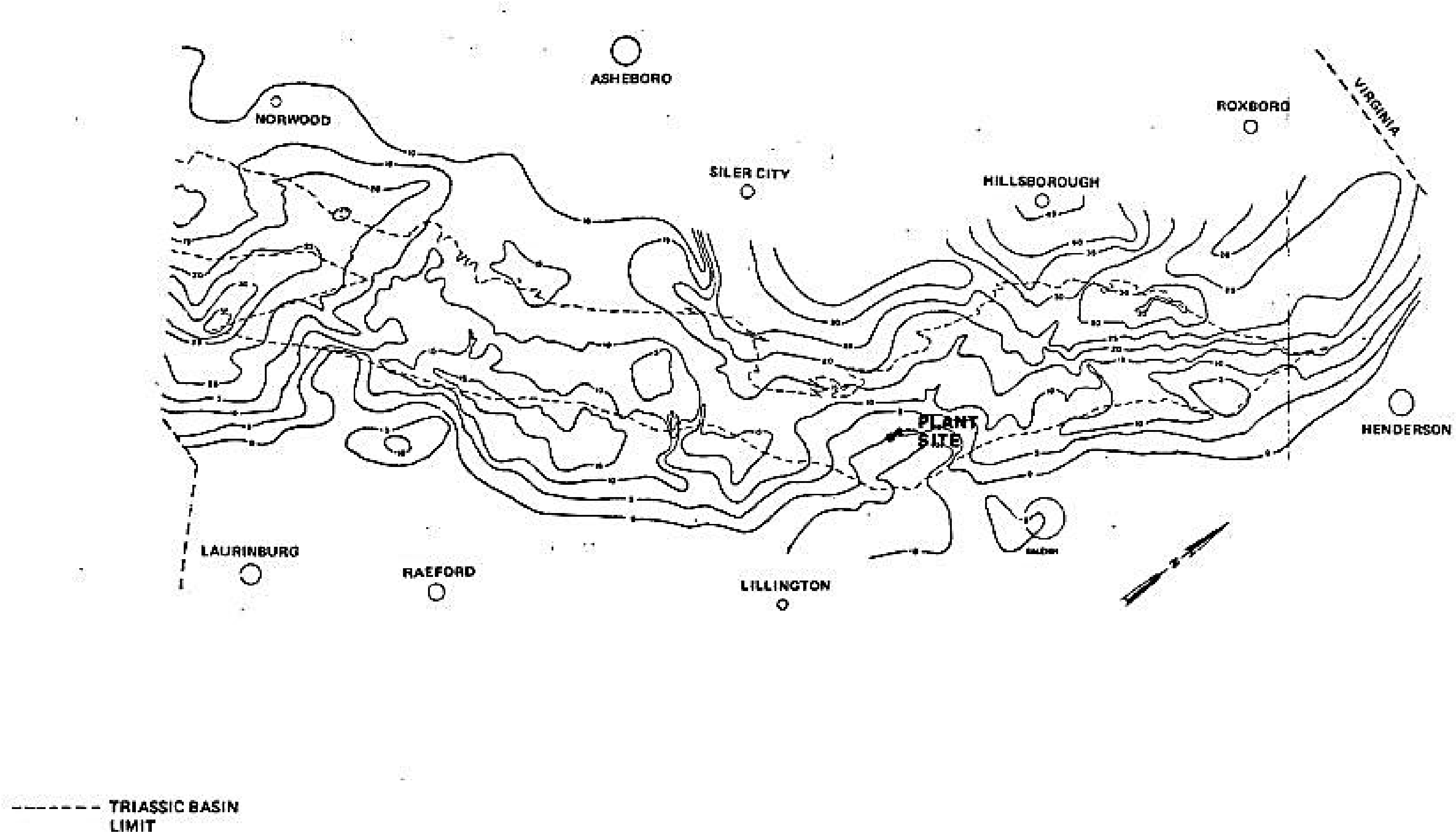
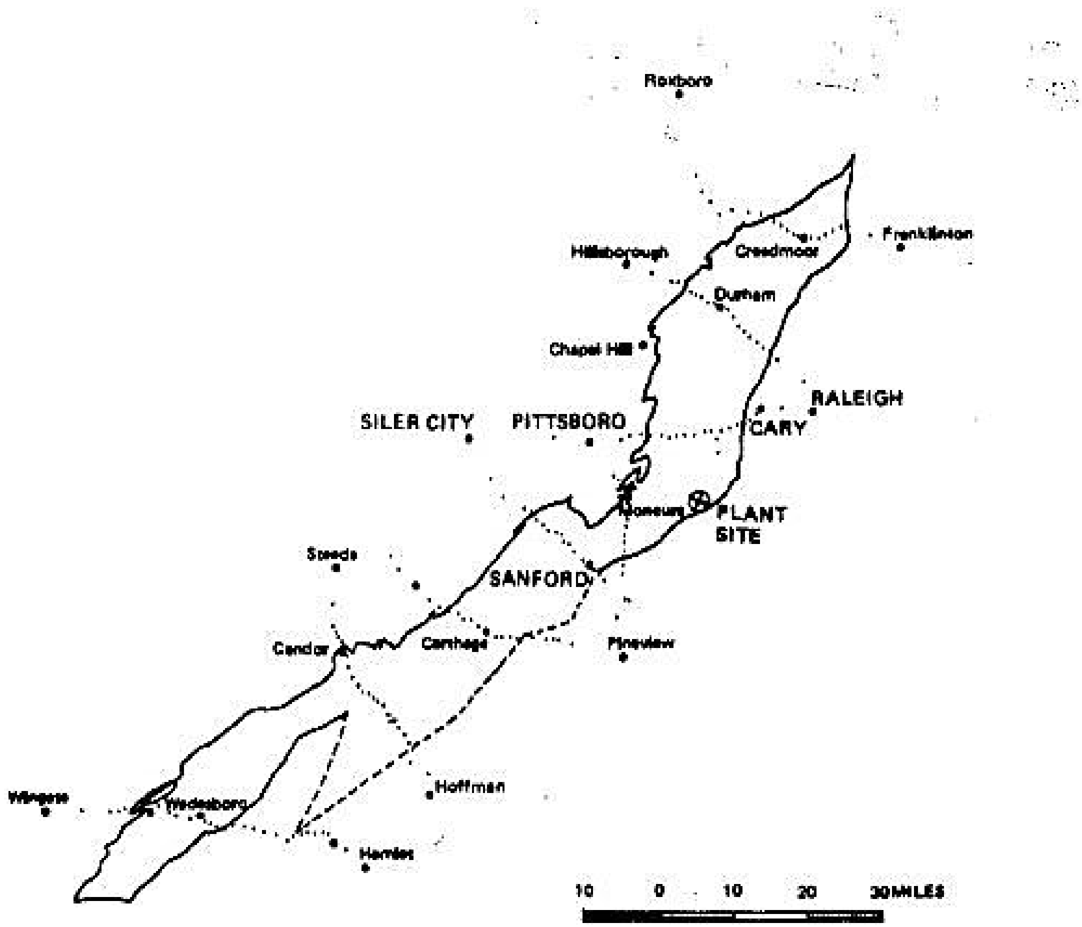


FIGURE 2.5.1-6

OUTLINE OF TRIASSIC BASIN AND GRAVITY PROFILES

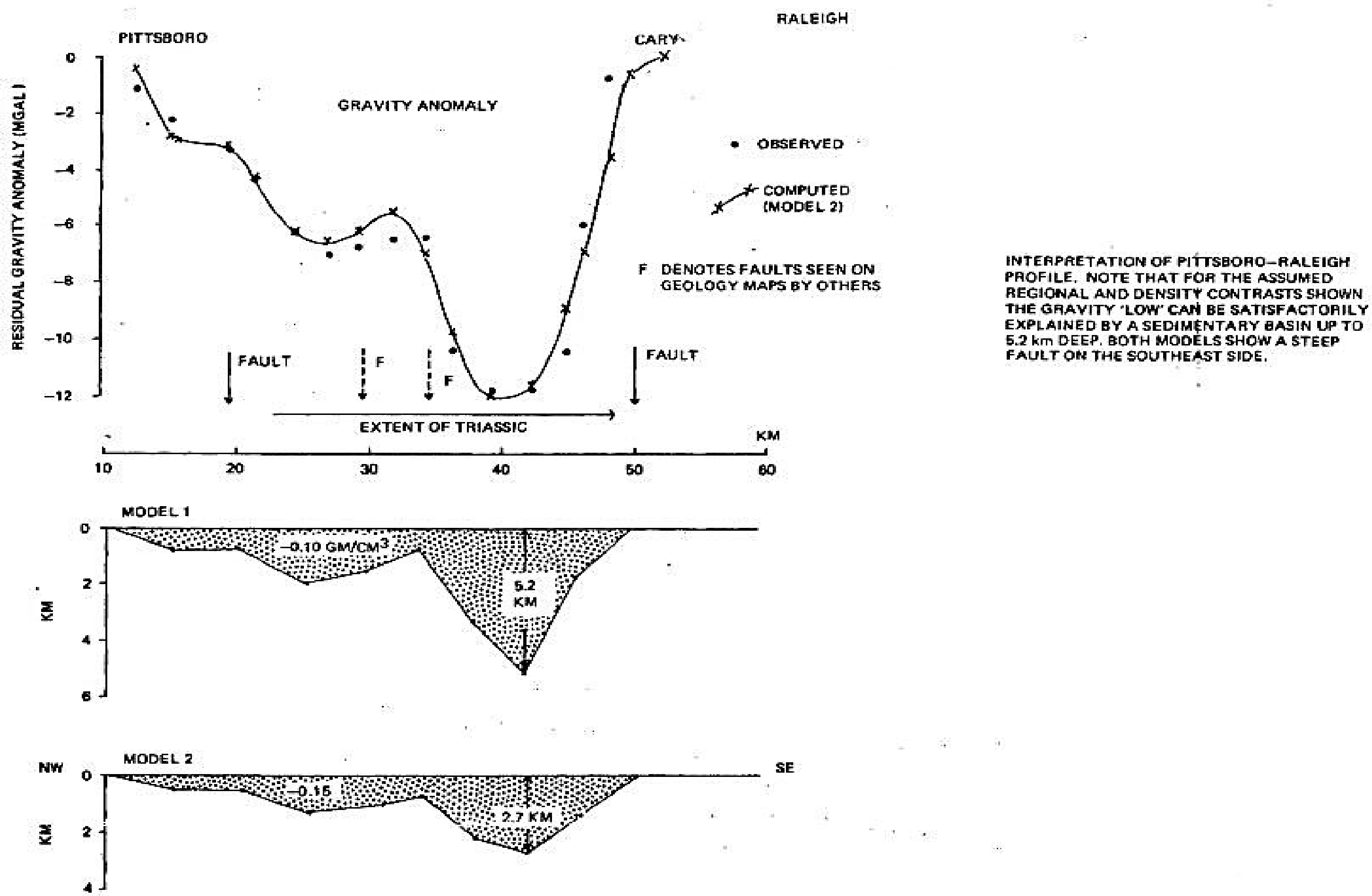


MAP OF THE DEEP RIVER–WADESBORO BASIN SHOWING THE LOCATION OF GRAVITY TRAVERSE LINES. ALL DOTS REPRESENT GRAVITY STATIONS. LARGER DOTS REPRESENT GRAVITY STATIONS IN THE DESIGNATED TOWNS.

FROM MANN AND ZABLOCKI (1961) (REFERENCE 2.5.1-30)

FIGURE 2.5.1-7

GRAVITY PROFILES FROM PITTSBORO TO RALEIGH



MODIFIED AFTER MANN AND ZABLOCKI (1961) (REFERENCE 2.5.1-30)

FIGURE 2.5.1-8

GRAVITY PROFILES FROM SILER CITY TO SANFORD

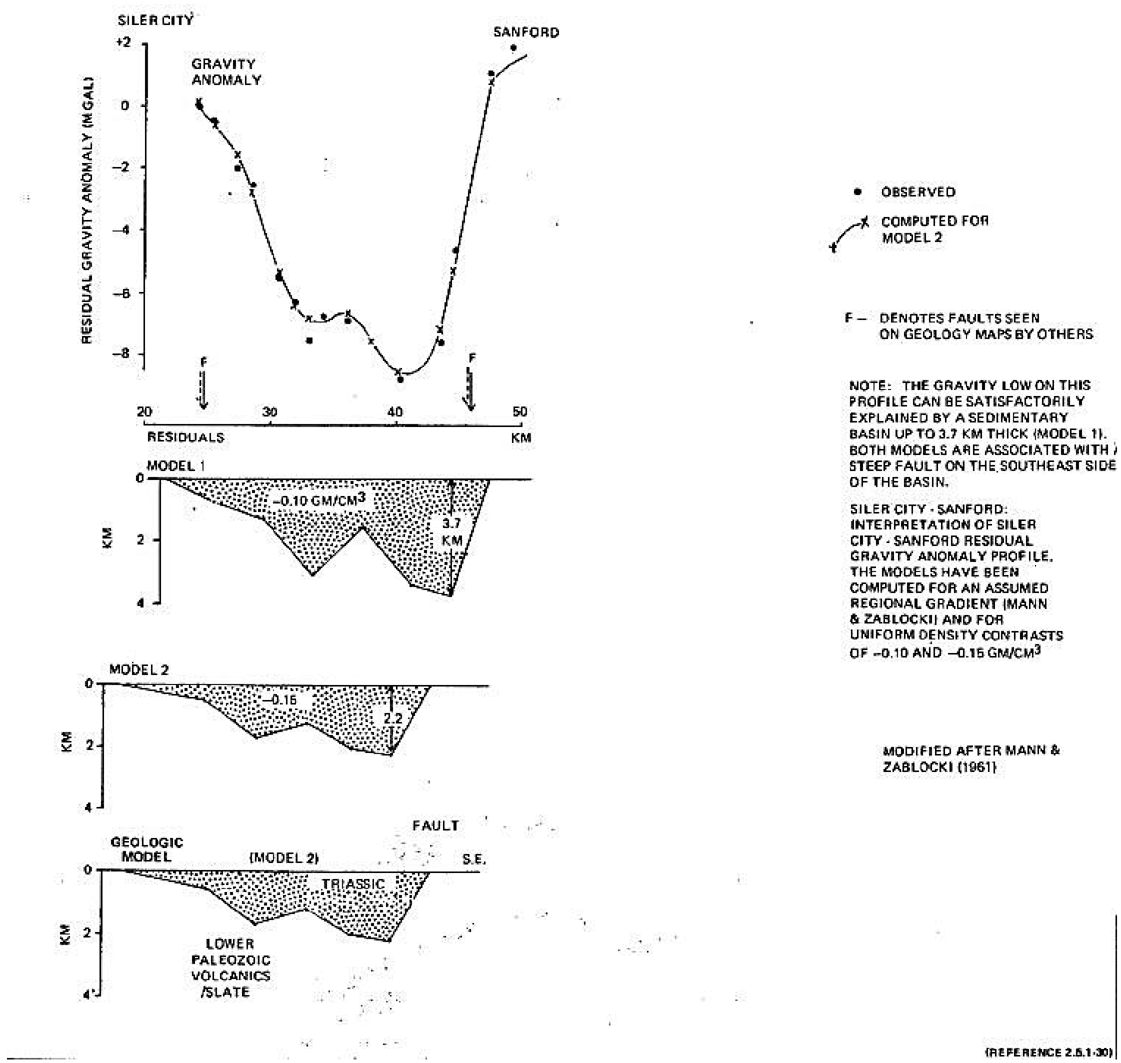


FIGURE 2.5.1-9

AEROMAGNETIC CONTOURS

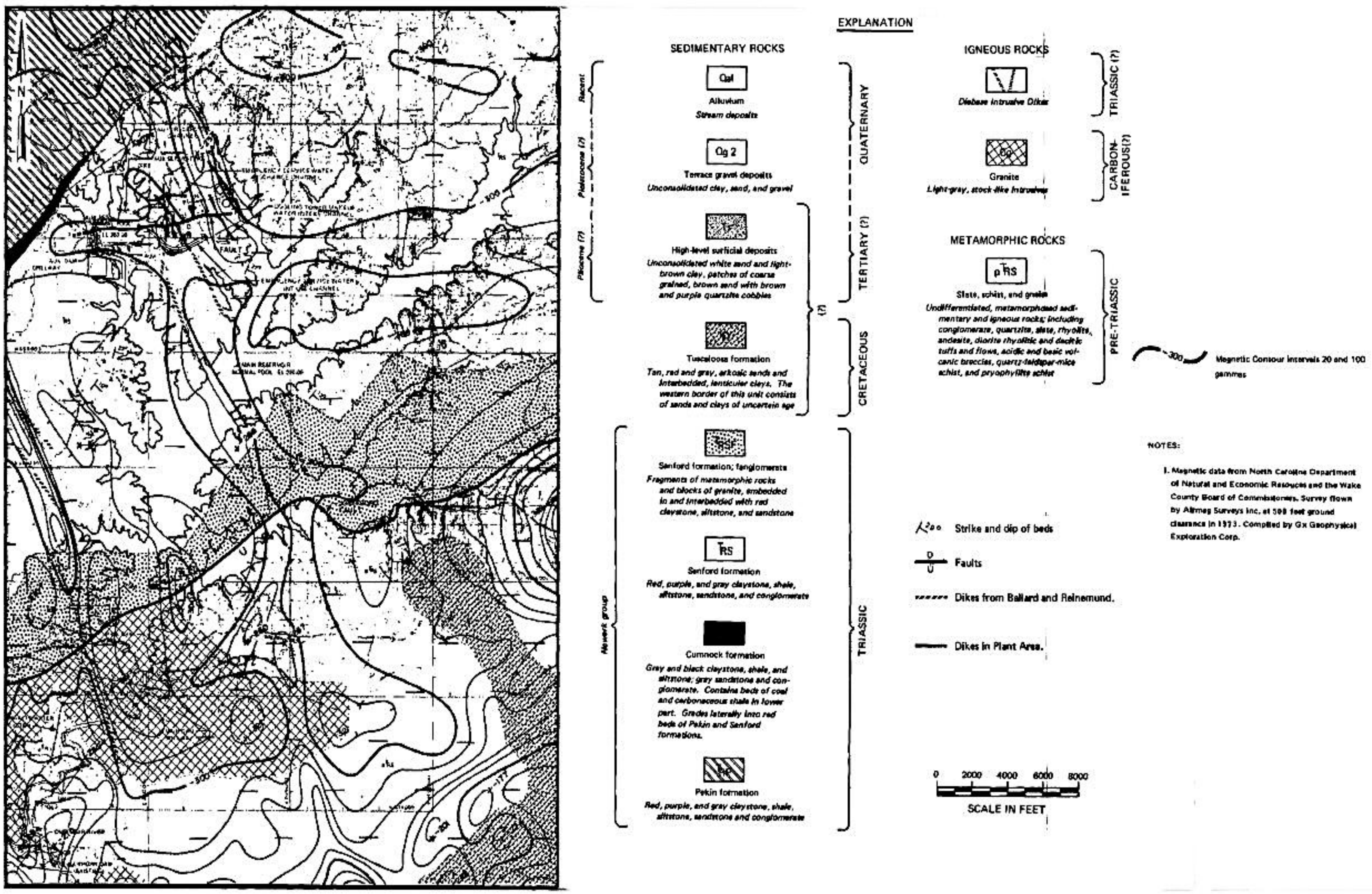


FIGURE 2.5.1-10

PROJECT INDEX MAP

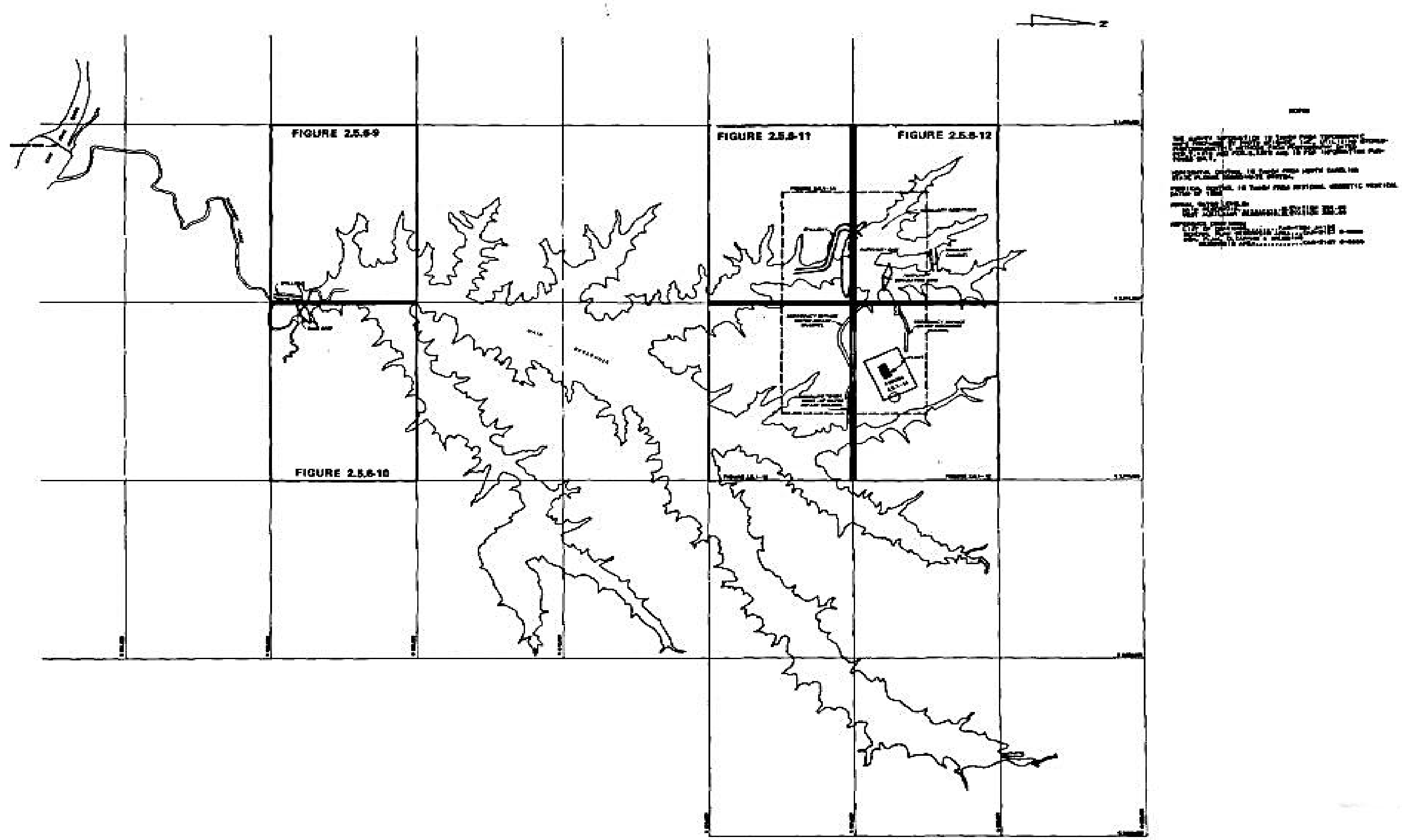
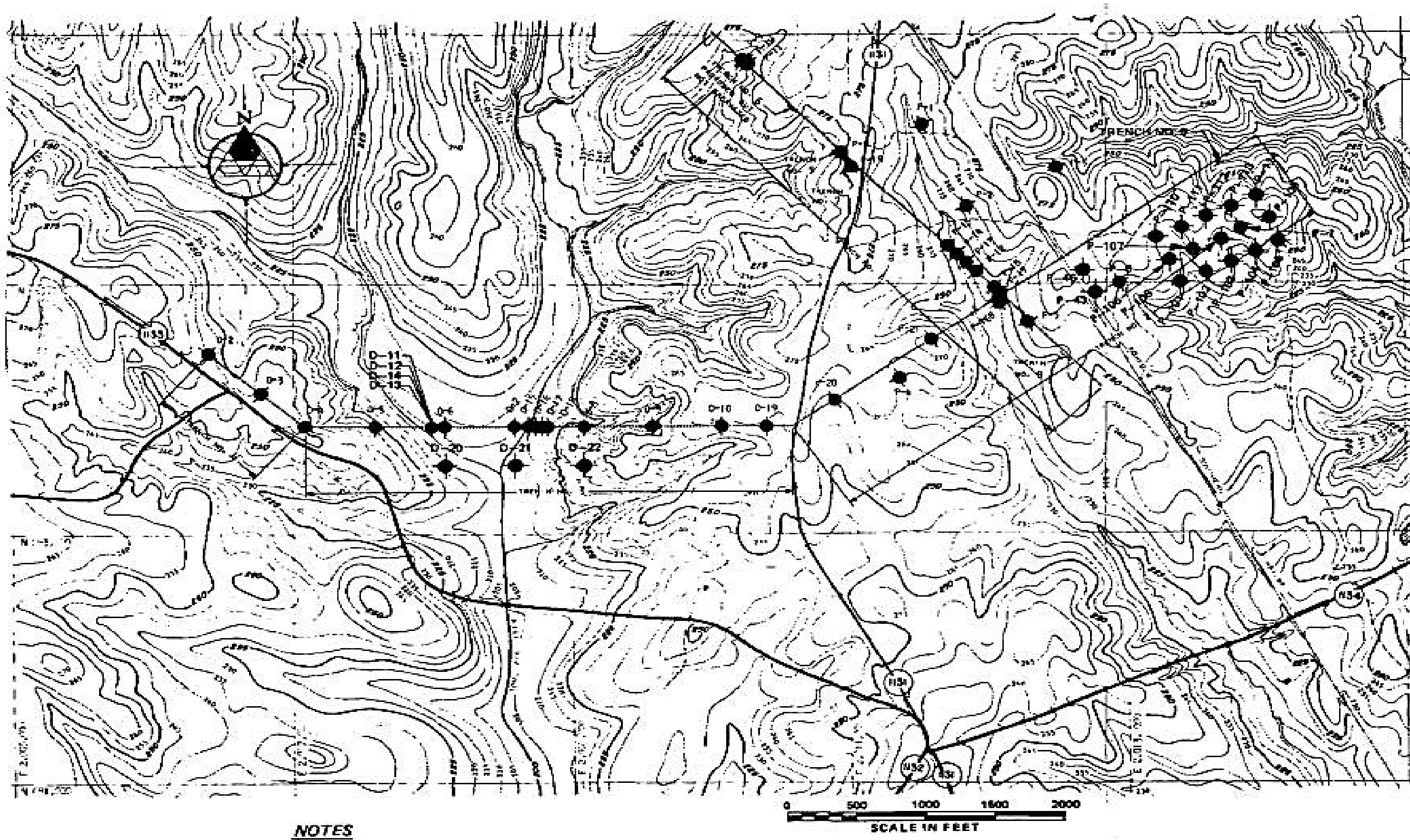


FIGURE 2.5.1-11

PRELIMINARY SUBSURFACE INVESTIGATION: PLANT VICINITY



NOTES

This map was prepared from portions of the following maps:
- *White Oaks Area Site 1*, by Photo Science Inc., Gaithersburg, Md., dated March 1970.
- *Wake County*, by the North Carolina State Highway Commission, revised June 1968.
- *Carolina Power & Light Co., Whiteoak Site*, by Ebasco Services Inc., revised Oct. 13, 1970.

Horizontal control taken from the North Carolina State Plane Coordinate System.

VERTICAL CONTROL TAKEN FROM NATIONAL GEODETIC VERTICAL DATUM OF 1929

EXPLANATION

borings drilled for geologic investigation

trenches excavated for geologic investigation

FIGURE 2.5.1-12

DESIGN SUBSURFACE INVESTIGATION: PLANT VICINITY, SHEET 1

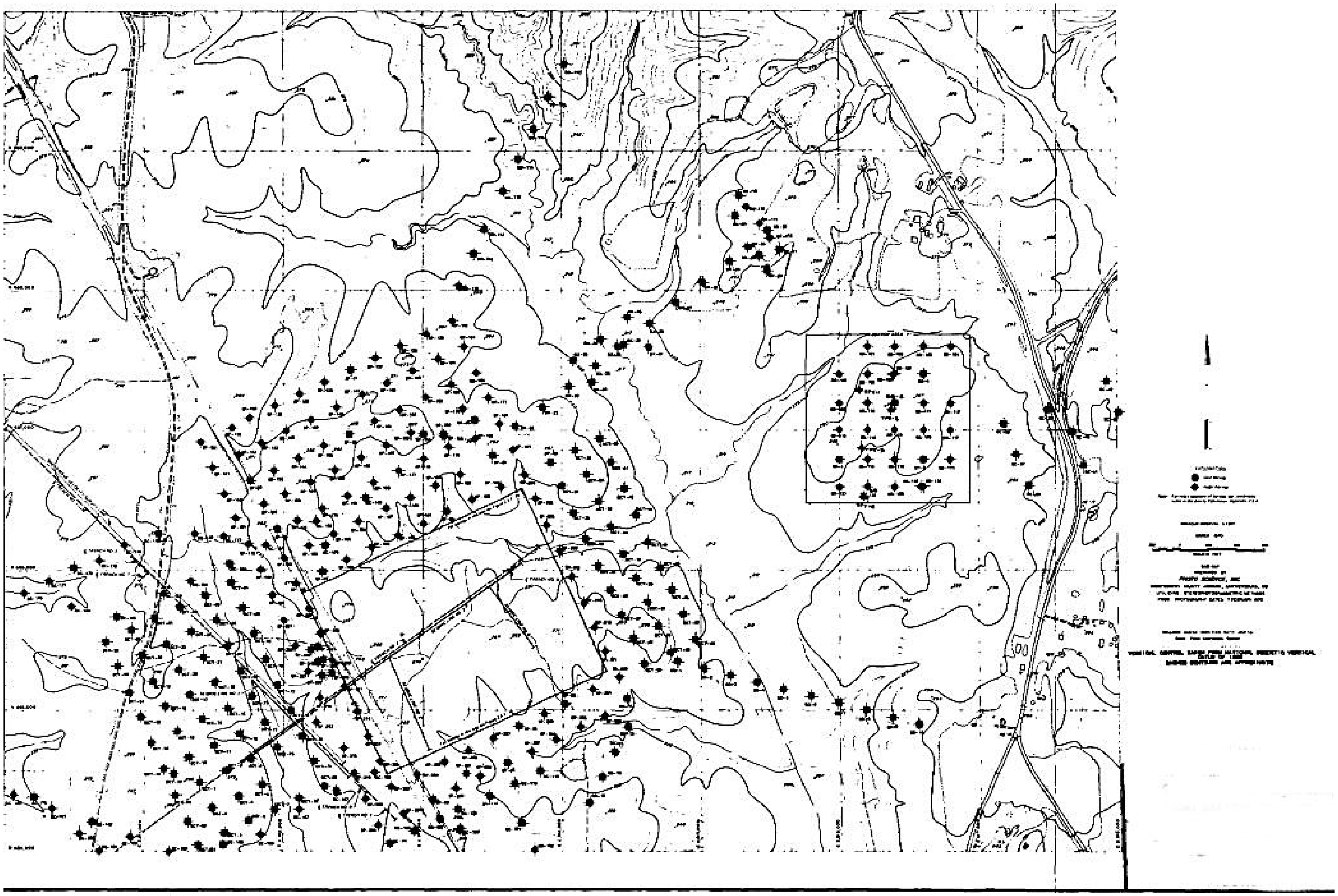


FIGURE 2.5.1-13

DESIGN SUBSURFACE INVESTIGATION: PLANT VICINITY, SHEET 2

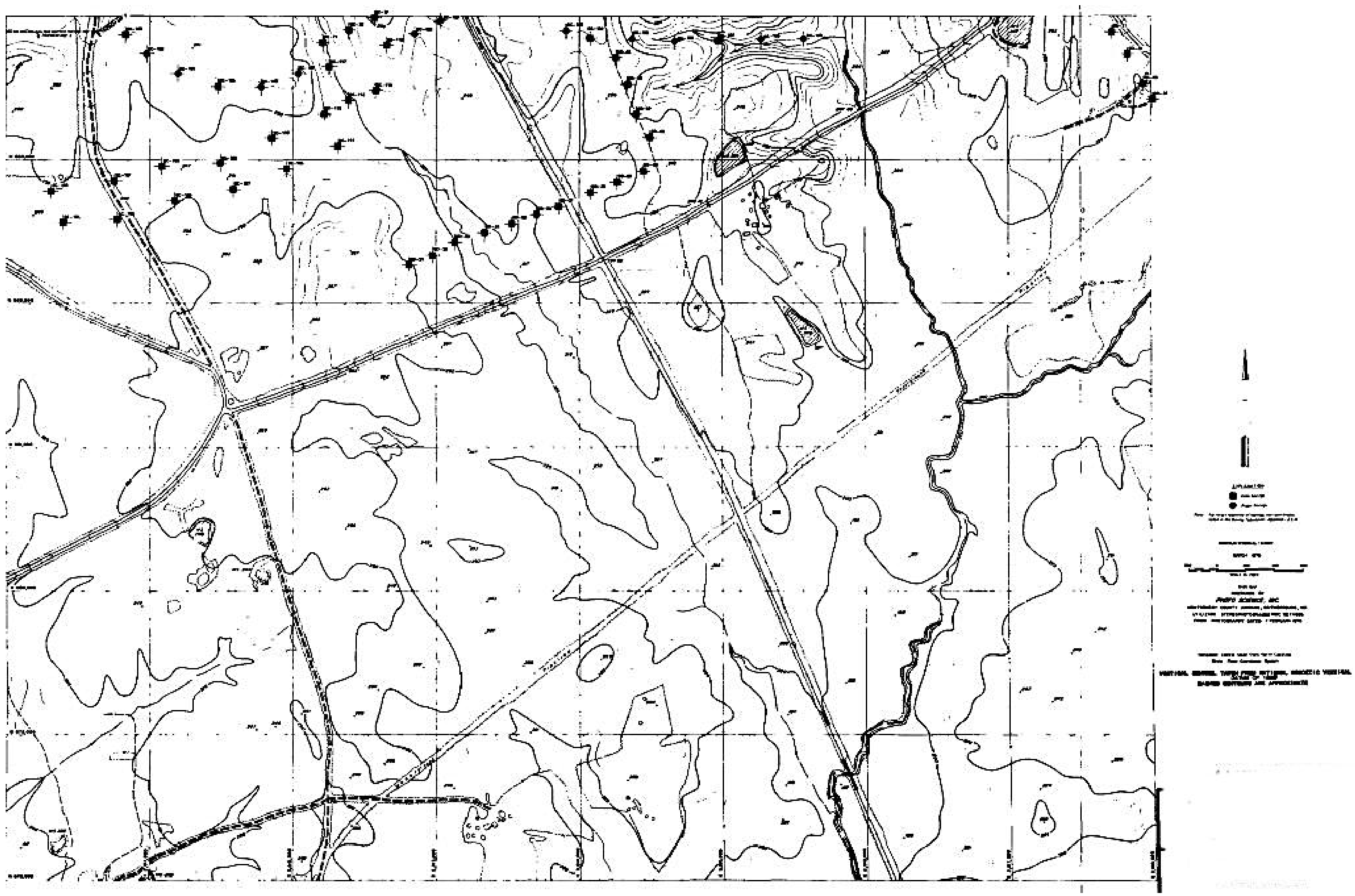


FIGURE 2.5.1-14

DESIGN SUBSURFACE INVESTIGATION: PLANT ISLAND

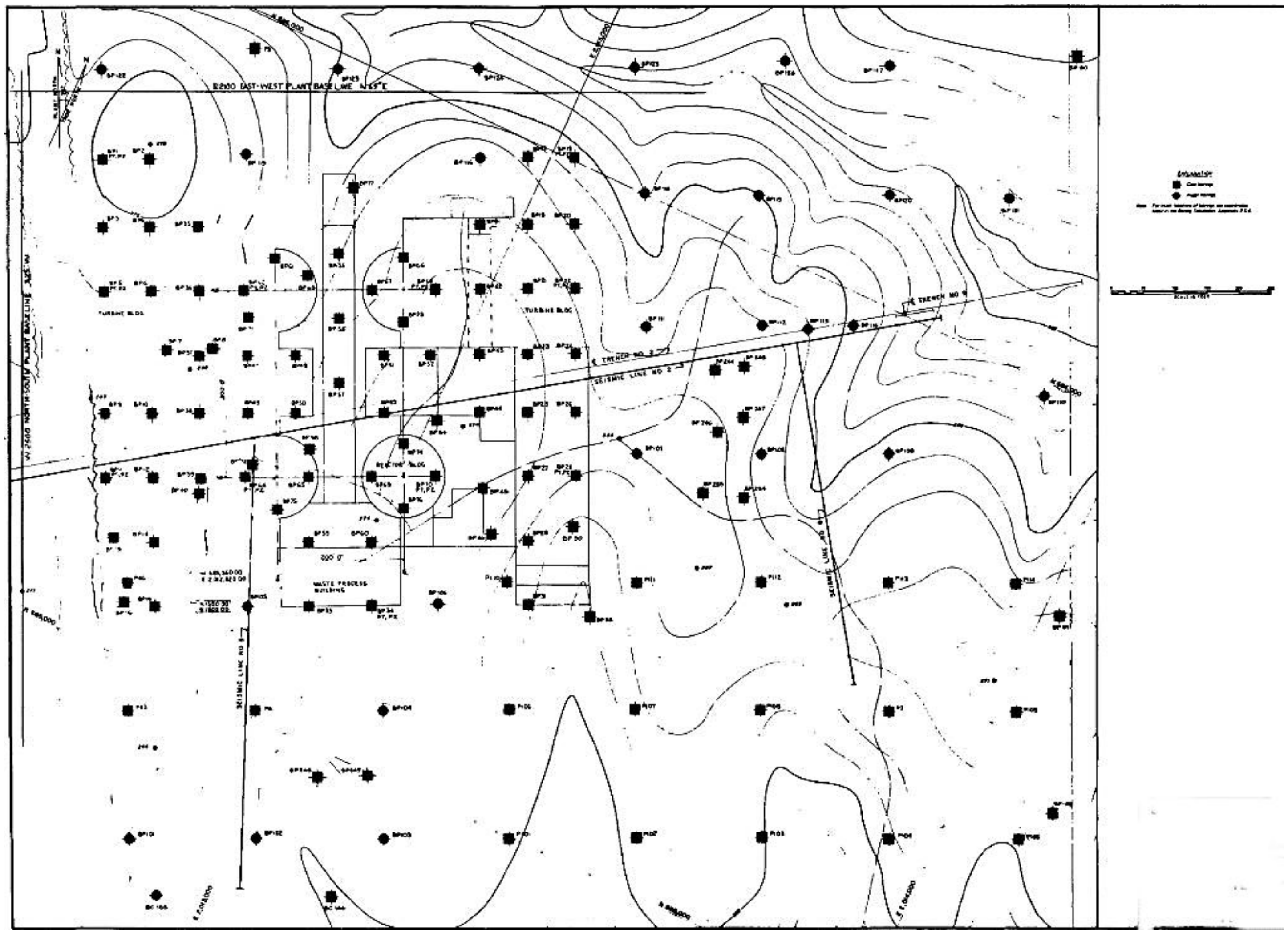


FIGURE 2.5.1-15

SITE FAULT INVESTIGATION SHEET 1

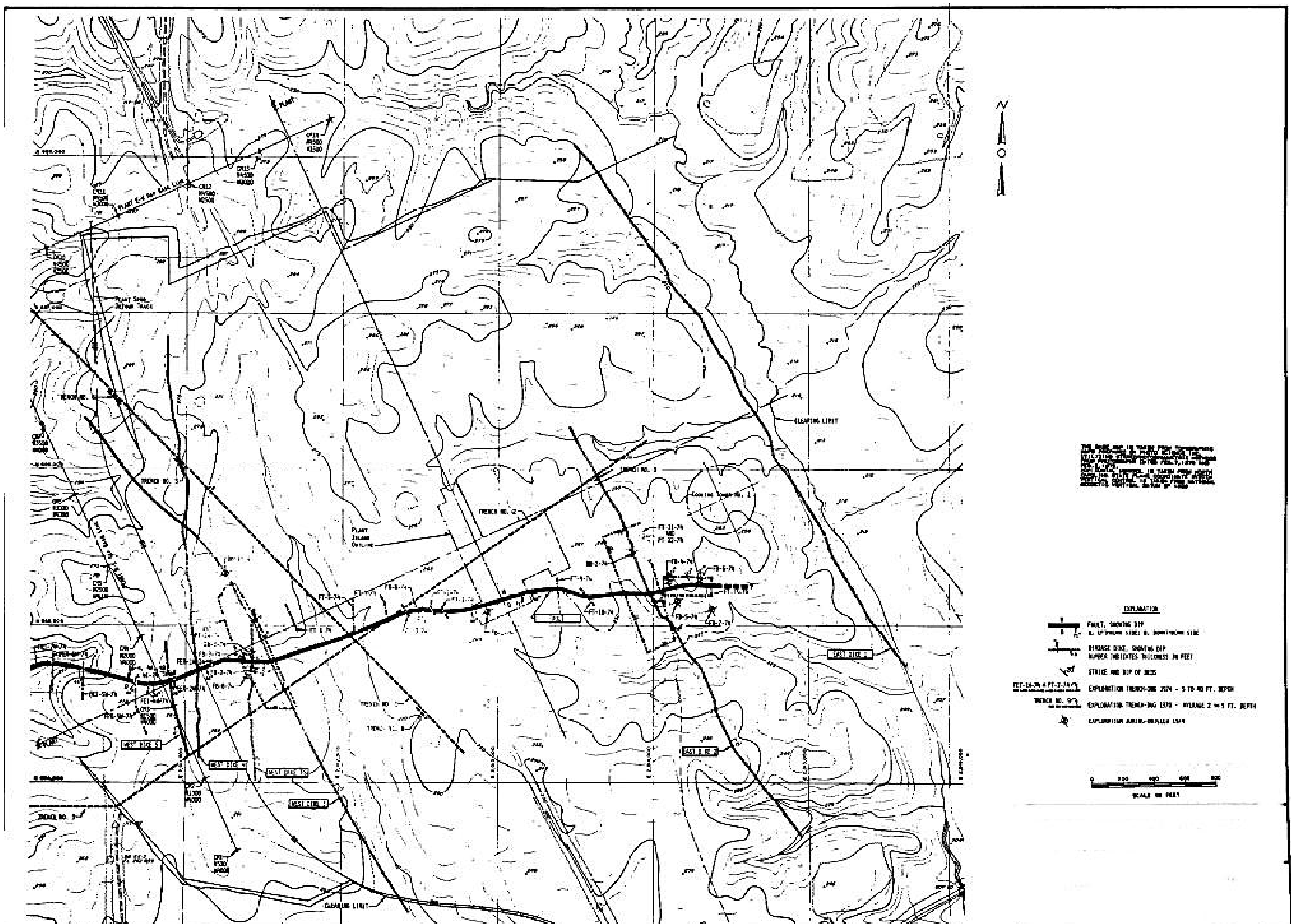


FIGURE 2.5.1-16

SITE FAULT INVESTIGATION SHEET 2

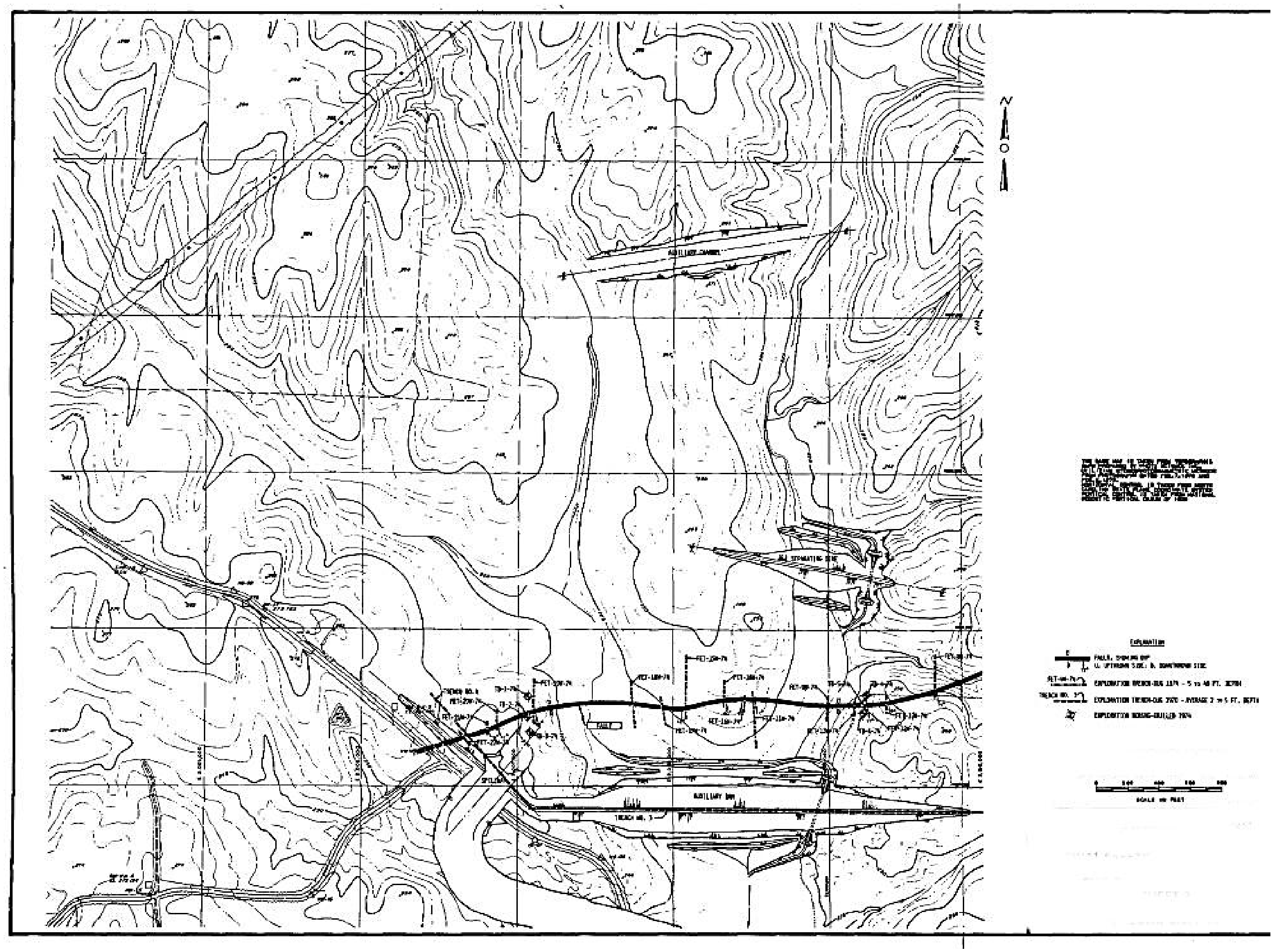


FIGURE 2.5.2-1

SOUTHERN APPALACHIAN SEISMICITY – 1754-1971

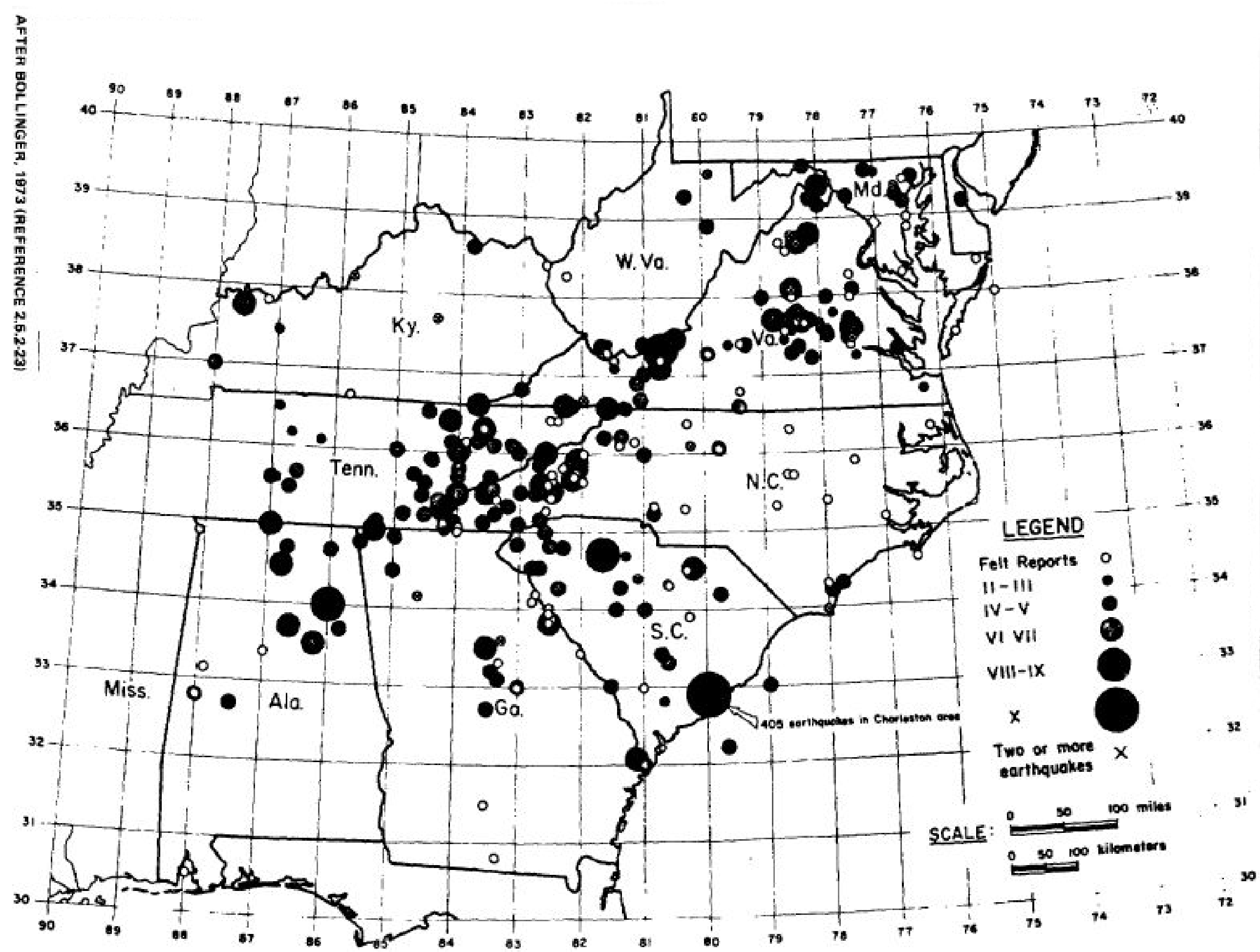


FIGURE 2.5.2-1a

EPICENTER MAP – 1700-1981

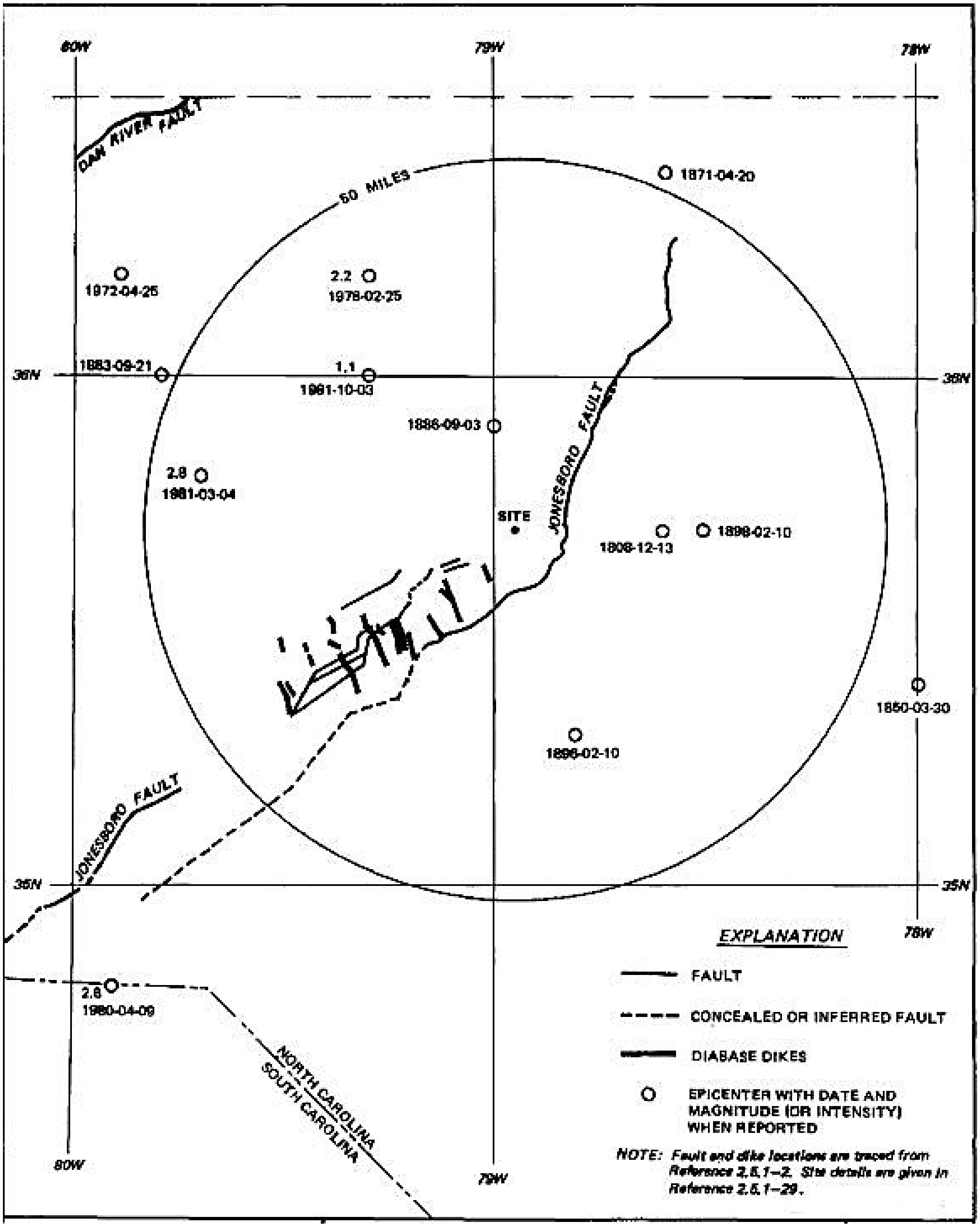
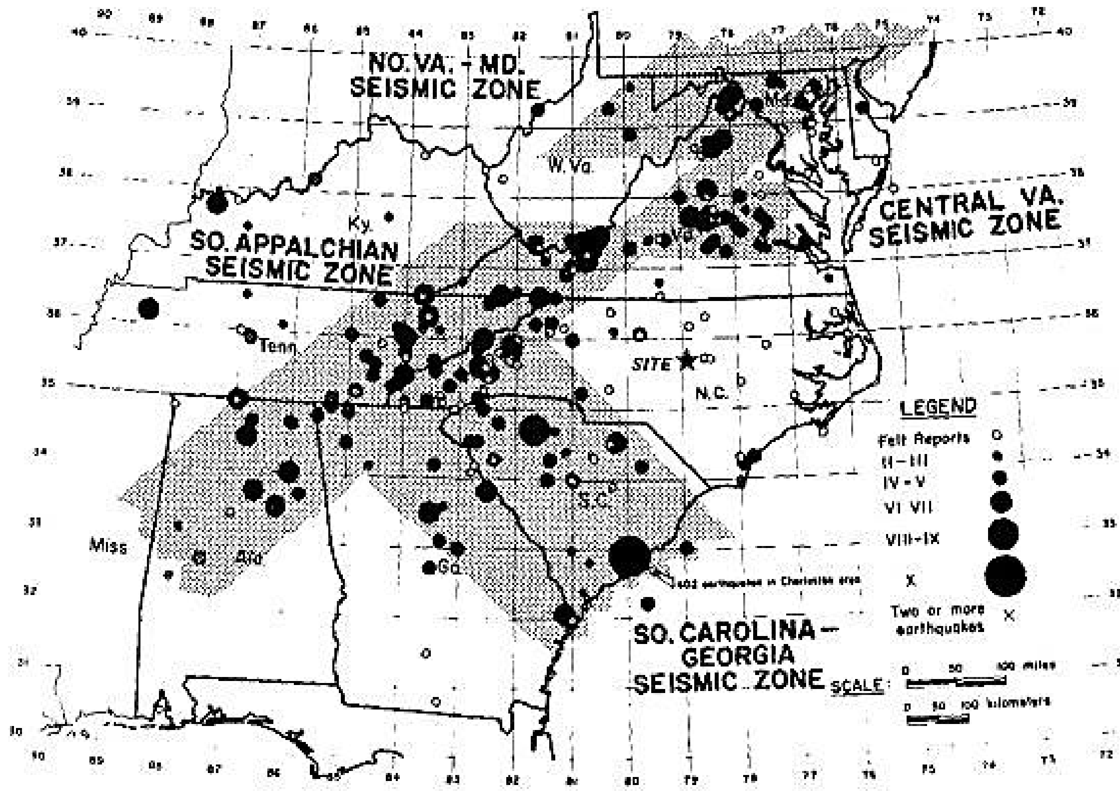


FIGURE 2.5.2-2

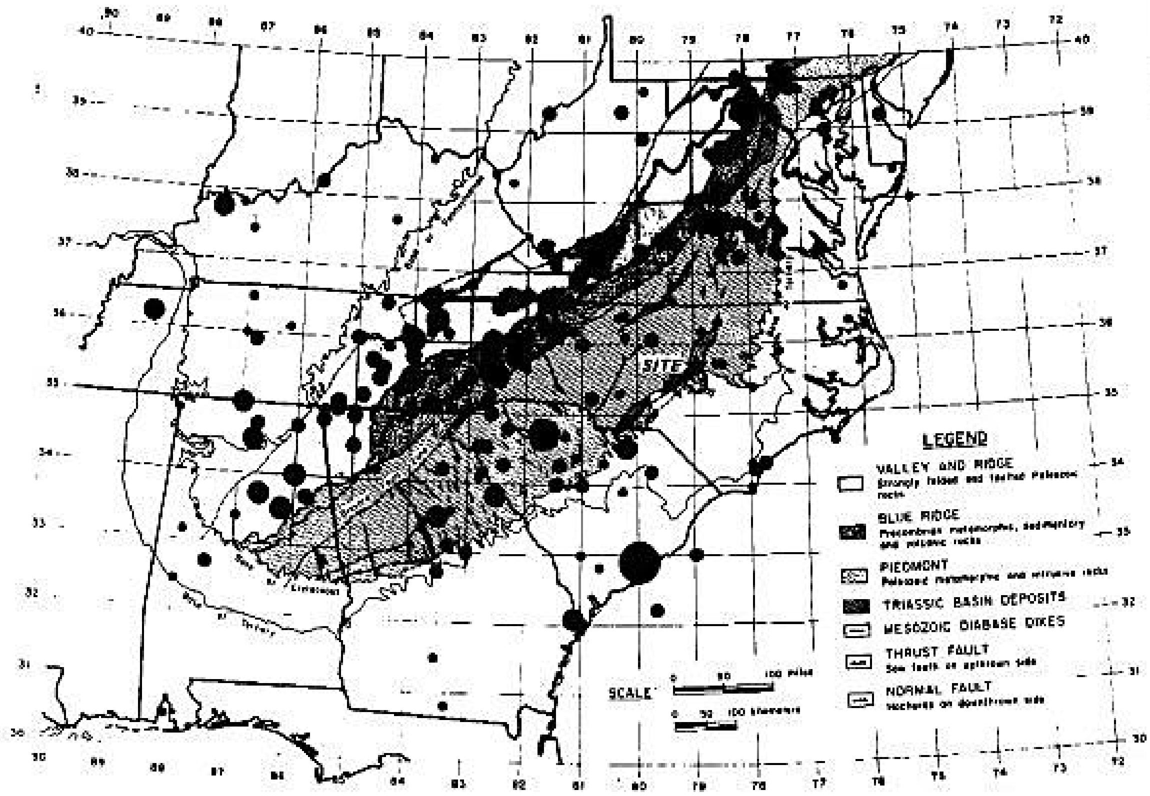
SEISMIC ZONES IN THE SOUTHEASTERN U.S. SHOWING EARTHQUAKE EPICENTERS FROM 1754 TO 1970



After Bollinger, 1973. REFERENCE 2.5.2-23

FIGURE 2.5.2-3

SEISMICITY (FROM 1754 TO 1970) AND GEOLOGICAL PROVINCES IN THE SOUTHEASTERN U.S.

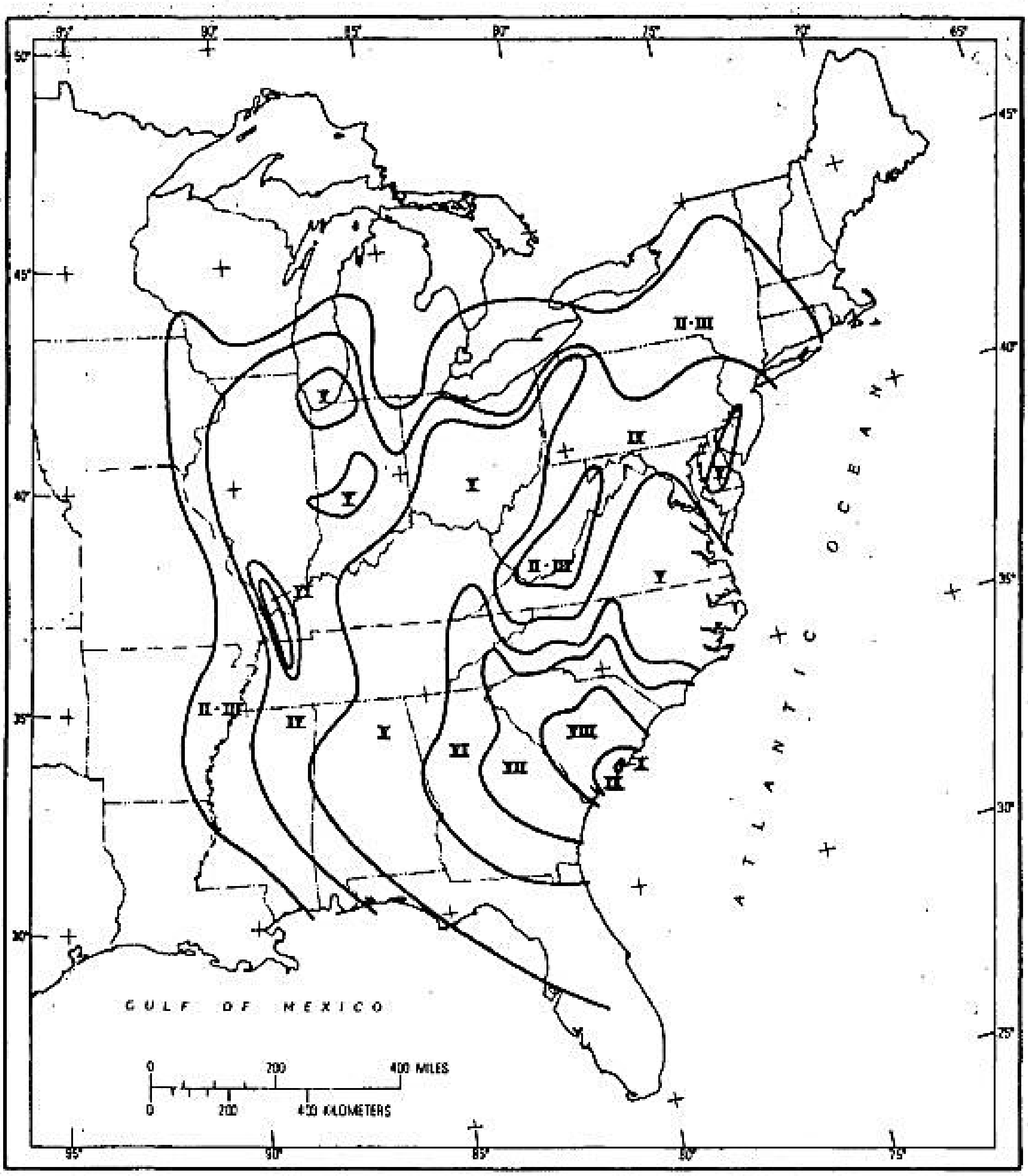


After Bollinger, 1973. REFERENCE 2.5.2-23

FIGURE 2.5.2-4

ISOSEISMAL MAP OF 1886 CHARLESTON EARTHQUAKE

25

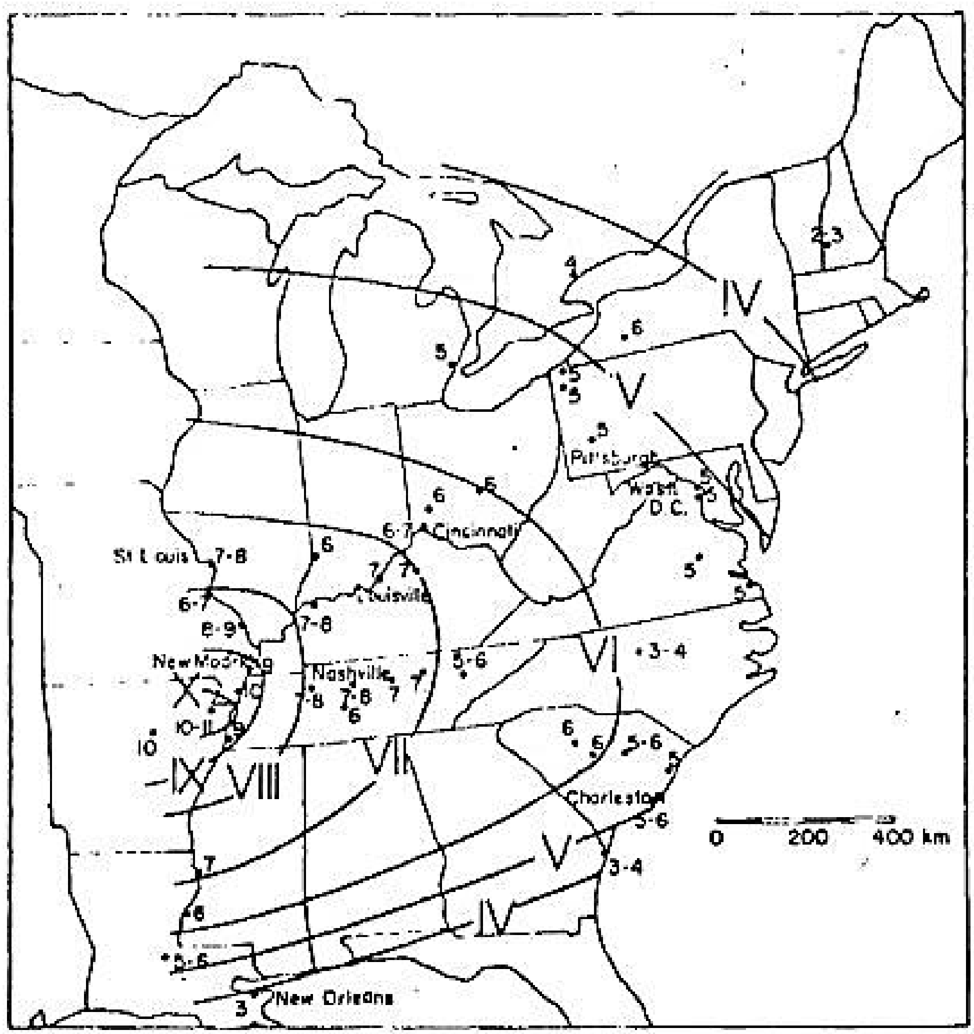


Isoseismal map of the Eastern United States contoured to show the broad regional patterns of the reported intensities for the 1886 Charleston earthquake. Contoured intensity levels are shown in Roman numerals.

AFTER BOLLINGER, 1977 (REF. 2.5-2-25)

FIGURE 2.5.2-5

ISOSEISMAL MAP OF DEC 16, 1811 NEW MADRID EARTHQUAKE



Generalized isoseismal map of the earthquake of December 16, 1811 at 08^{h}15^m GMT, MM intensity values at individual points are given in Arabic numerals. The isoseisms labeled with Roman numerals, indicate the outer bound of the region of specified intensity.

(AFTER NUTTLI, 1973 REFERENCE 2.5.2-34)

FIGURE 2.5.2-6

LOCATIONS OF GEOPHYSICAL EXPLORATIONS

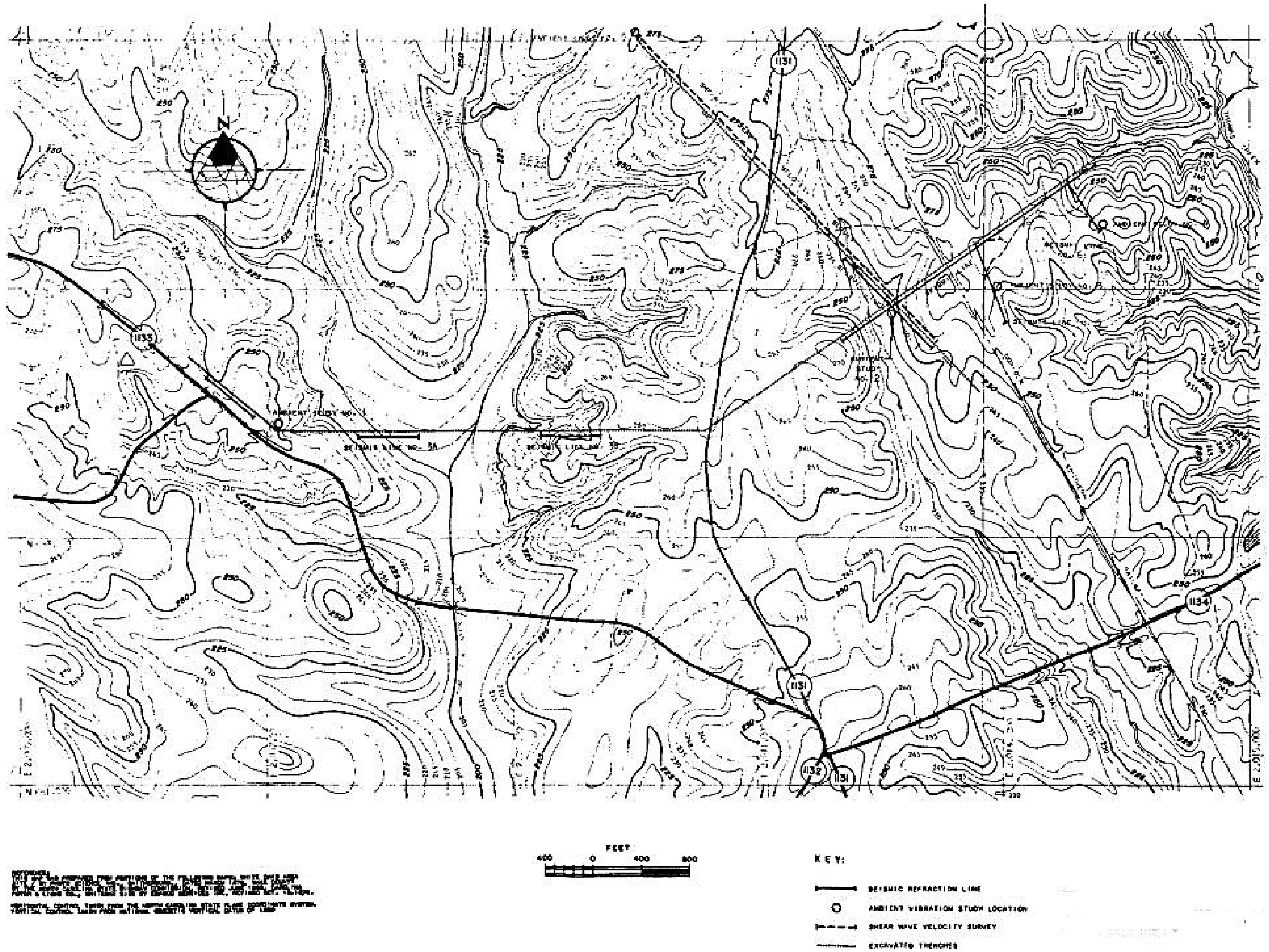
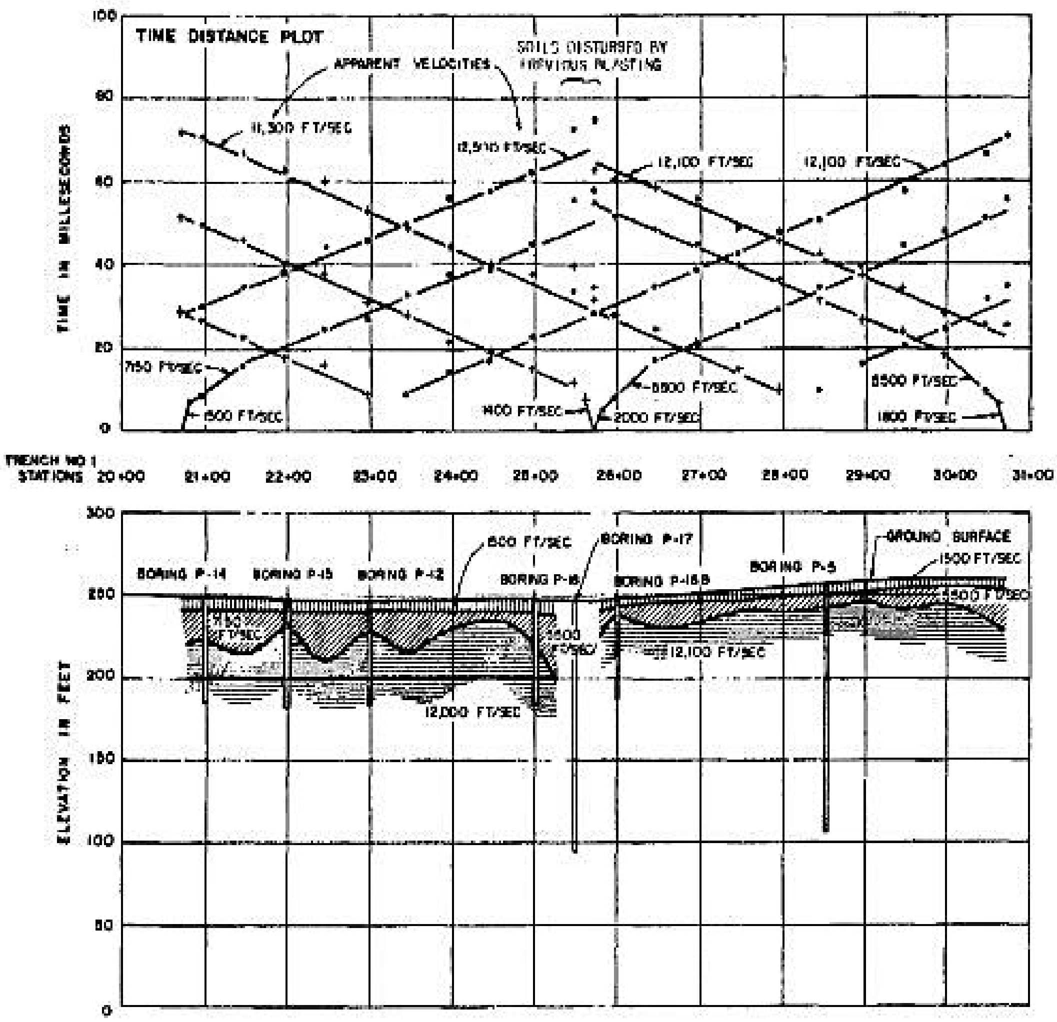


FIGURE 2.5.2-7

SEISMIC REFRACTION SURVEY LINE 1



SEISMIC LINE I - TRENCH I

NOTES:

TIME DISTANCE PLOTS REFLECT INFORMATION COLLECTED FROM SHOT POINTS MADE AT SEVERAL LOCATIONS ALONG A SEISMIC LINE. FOR CLARIFICATION, TWO PLOT SYMBOLS HAVE BEEN USED TO INDICATE THE ORIGIN OF THE SHOCK: FROM THE LEFT (•) FROM THE RIGHT (+)

THE SUBSURFACE SECTIONS SHOWN REPRESENT OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED UPON INTERPRETATION OF PRESENTLY AVAILABLE DATA. SOME VARIATIONS FROM THESE CONDITIONS MUST BE EXPECTED.

KEY:

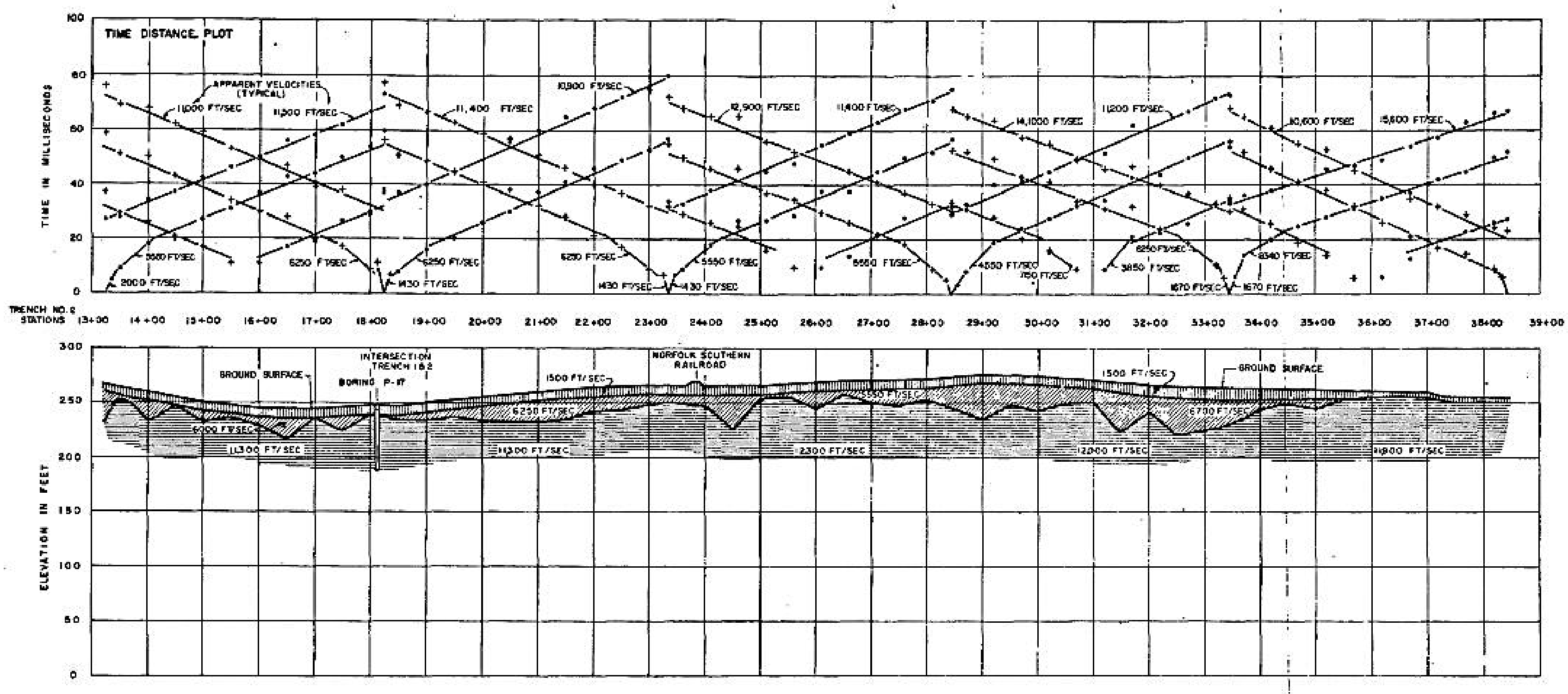
LOW VELOCITY 1250 TO 2000 FT./SEC.

MEDIUM VELOCITY 5000 TO 7150 FT./SEC.

HIGH VELOCITY 10,000 TO 15,650 FT./SEC.

FIGURE 2.5.2-8

SEISMIC REFRACTION SURVEY LINE 2



SEISMIC LINE 2 - TRENCH 2

NOTES:

TIME DISTANCE PLOTS REFLECT INFORMATION COLLECTED FROM SHOT POINTS MADE AT SEVERAL LOCATIONS ALONG A SEISMIC LINE. FOR CLARIFICATION, TWO PLOT SYMBOLS HAVE BEEN USED TO INDICATE THE ORIGIN OF THE SHOCK: FROM THE LEFT (•) FROM THE RIGHT (+)

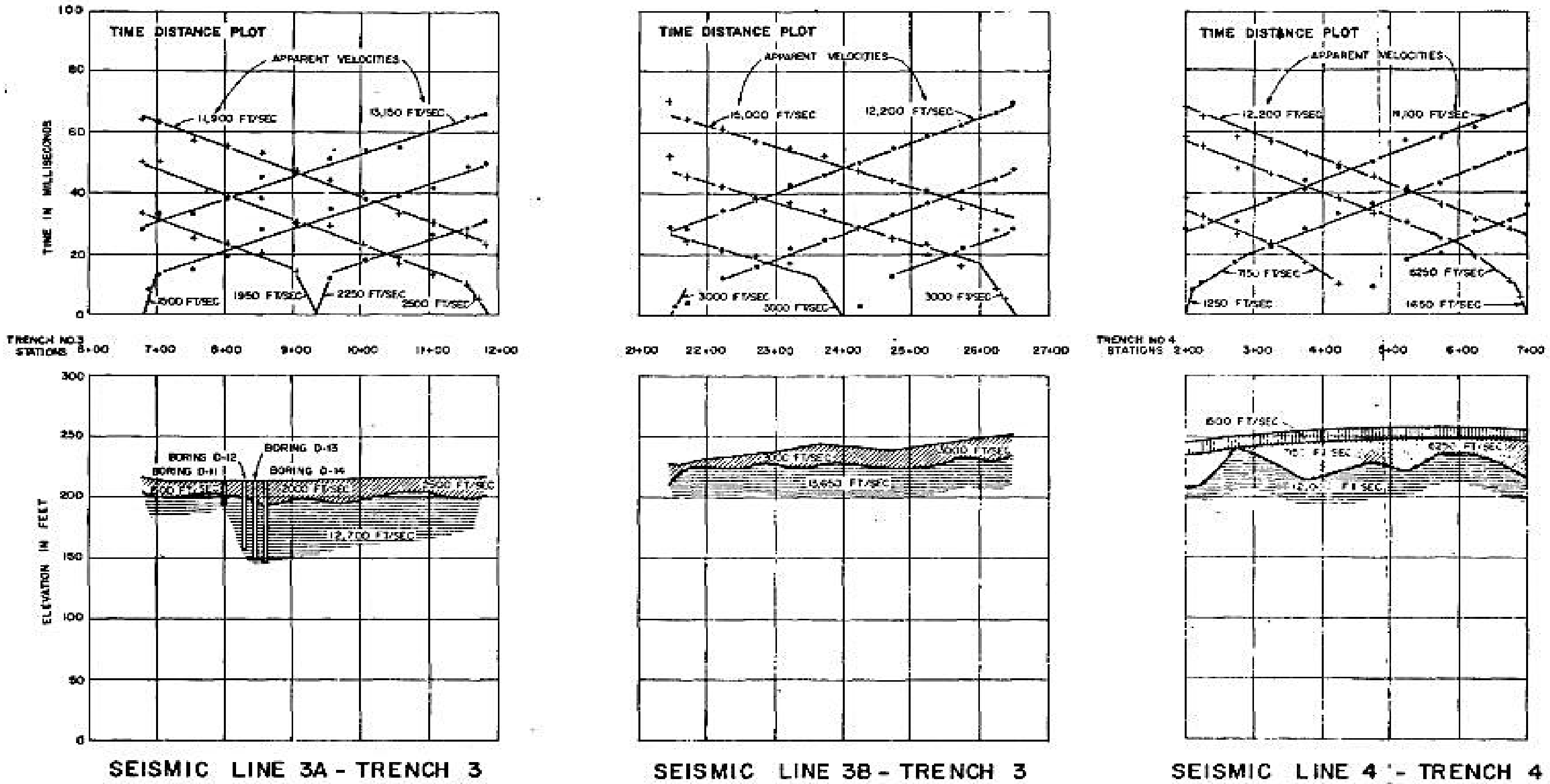
THE SUBSURFACE SECTIONS SHOWN REPRESENT OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED UPON INTERPRETATION OF PRESENTLY AVAILABLE DATA. SOME VARIATIONS FROM THESE CONDITIONS MUST BE EXPECTED.

KEY:

- LOW VELOCITY 1250 TO 2000 FT./SEC.
- MEDIUM VELOCITY 5000 TO 7150 FT./SEC.
- HIGH VELOCITY 10,900 TO 13,650 FT./SEC.

FIGURE 2.5.2-9

SEISMIC REFRACTION SURVEY LINES 3A, 3B, AND 4



NOTES:

TIME DISTANCE PLOTS REFLECT INFORMATION COLLECTED FROM SHOT POINTS MADE AT SEVERAL LOCATIONS ALONG A SEISMIC LINE. FOR CLASSIFICATION, TWO PLOT SYMBOLS HAVE BEEN USED TO INDICATE THE ORIGIN OF THE SHOCK: FROM THE LEFT (•) FROM THE RIGHT (+)

THE SUBSURFACE SECTIONS SHOWN REPRESENT OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED UPON INTERPRETATION OF PRESENTLY AVAILABLE DATA. SOME VARIATIONS FROM THESE CONDITIONS MUST BE EXPECTED.

KEY:

LOW VELOCITY 1250 TO 2000 FT./SEC.

MEDIUM VELOCITY 5000 TO 7150 FT./SEC.

HIGH VELOCITY 10,400 TO 13,650 FT./SEC.

FIGURE 2.5.2-10

SEISMIC REFRACTION SURVEY LINES 5 AND 6

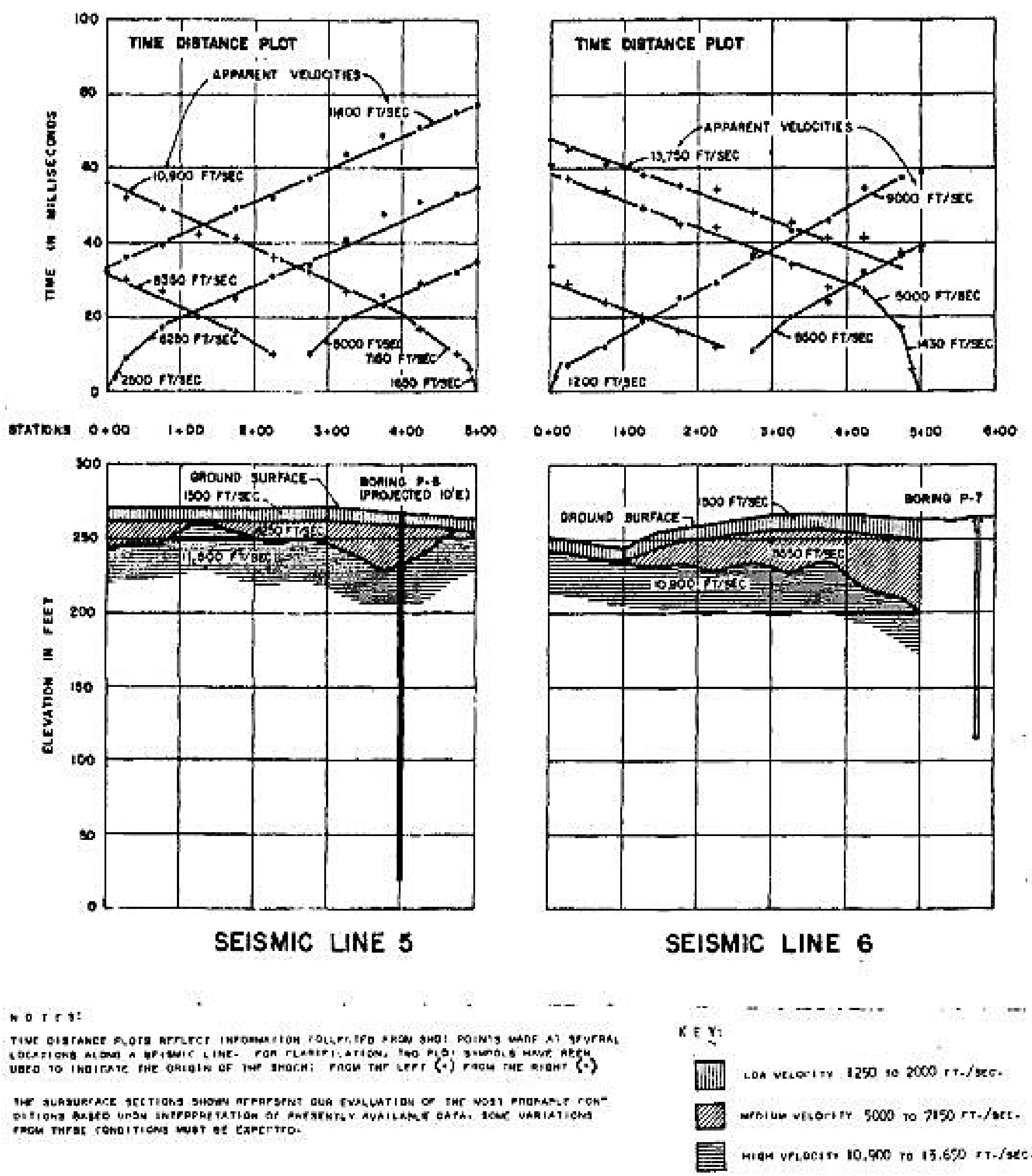


FIGURE 2.5.2-11

DOWN HOLE COMPRESSIONAL WAVE VELOCITY SURVEY

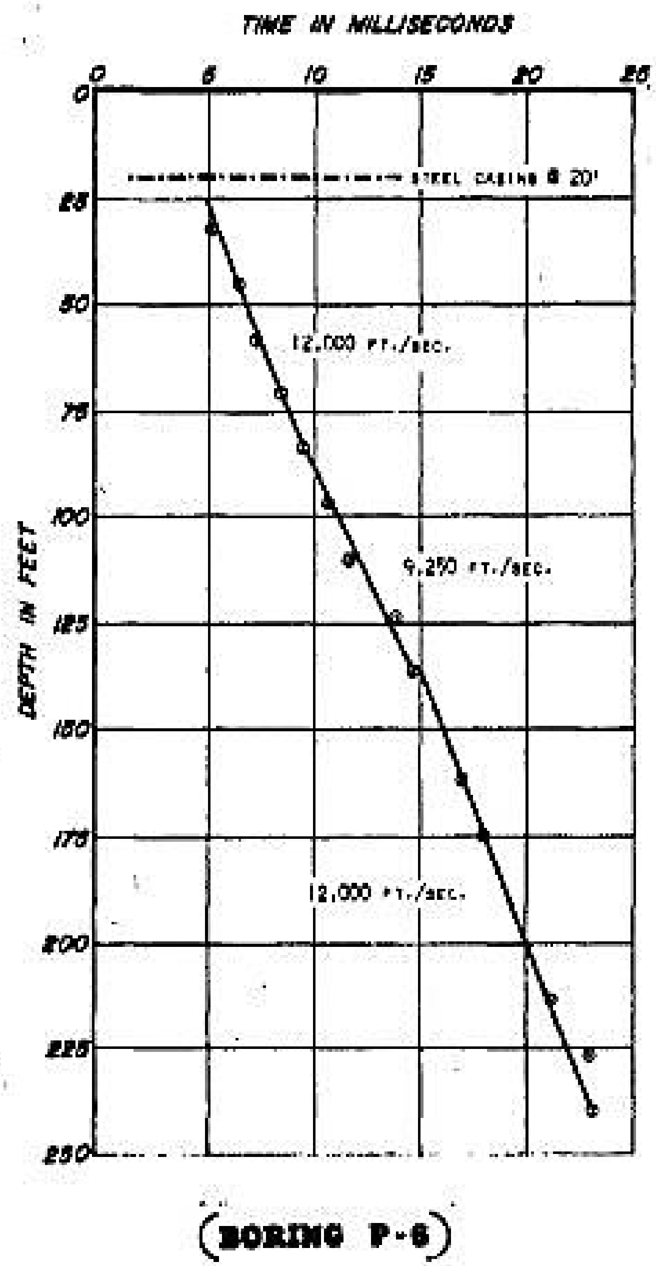


FIGURE 2.5.2-12

HORIZONTAL DESIGN RESPONSE SPECTRA SCALED TO 0.15G HORIZONTAL GROUND ACCELERATION

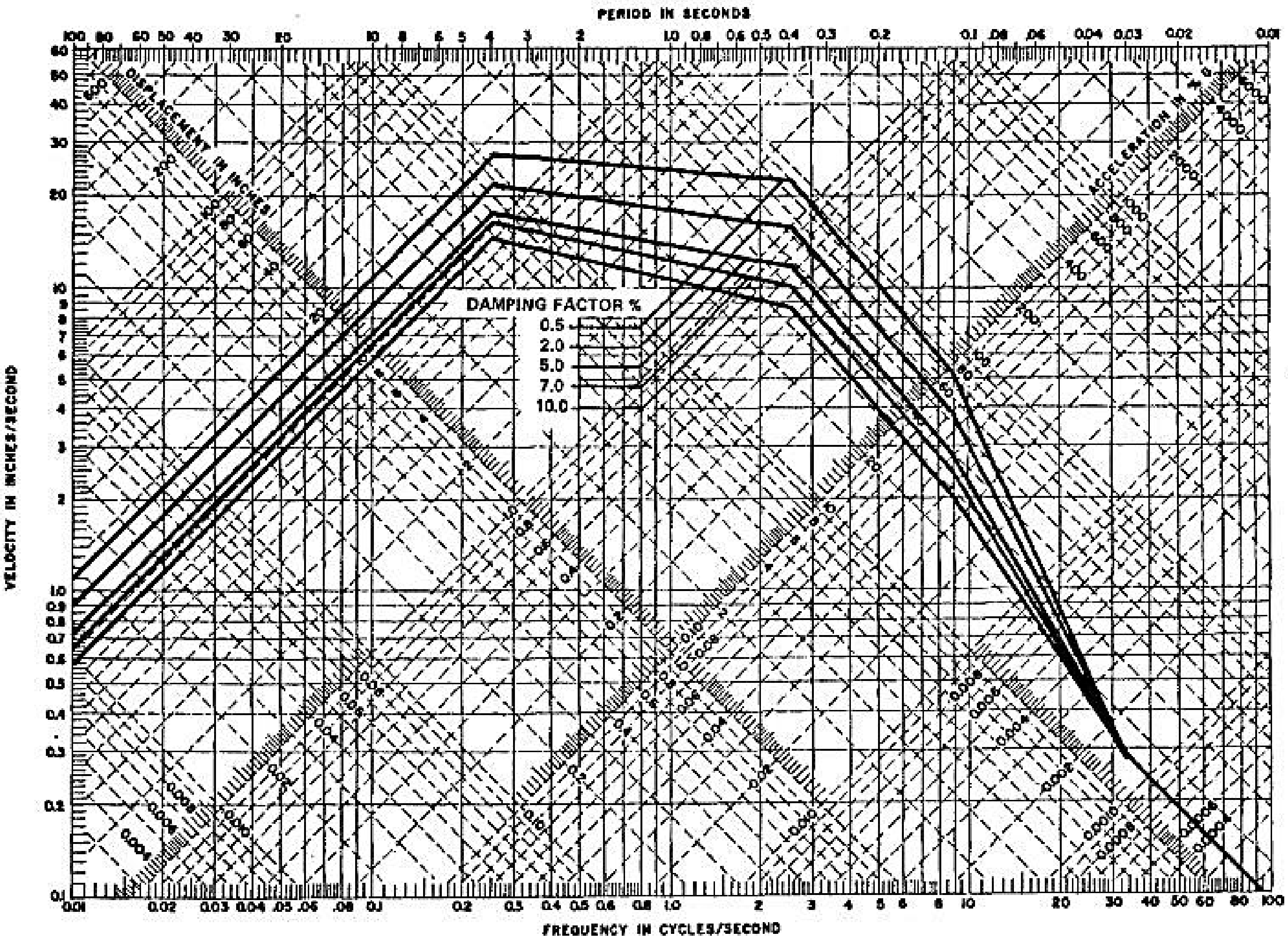


FIGURE 2.5.2-13

VERTICAL DESIGN RESPONSE SPECTRA SCALED TO 0.15G VERTICAL GROUND ACCELERATION

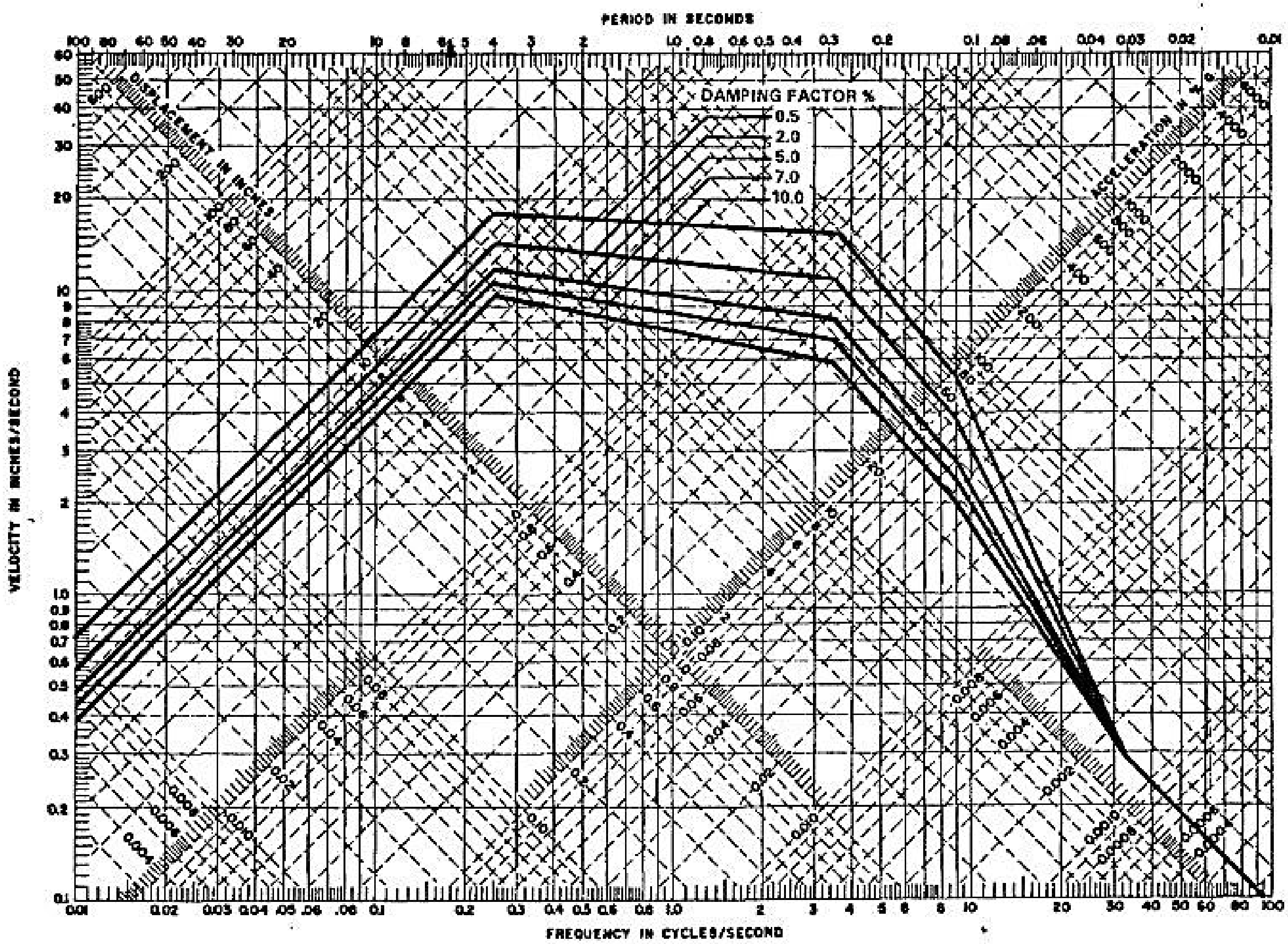


FIGURE 2.5.2-14

HORIZONTAL DESIGN RESPONSE SPECTRA SCALED TO 0.075G HORIZONTAL GROUND ACCELERATION

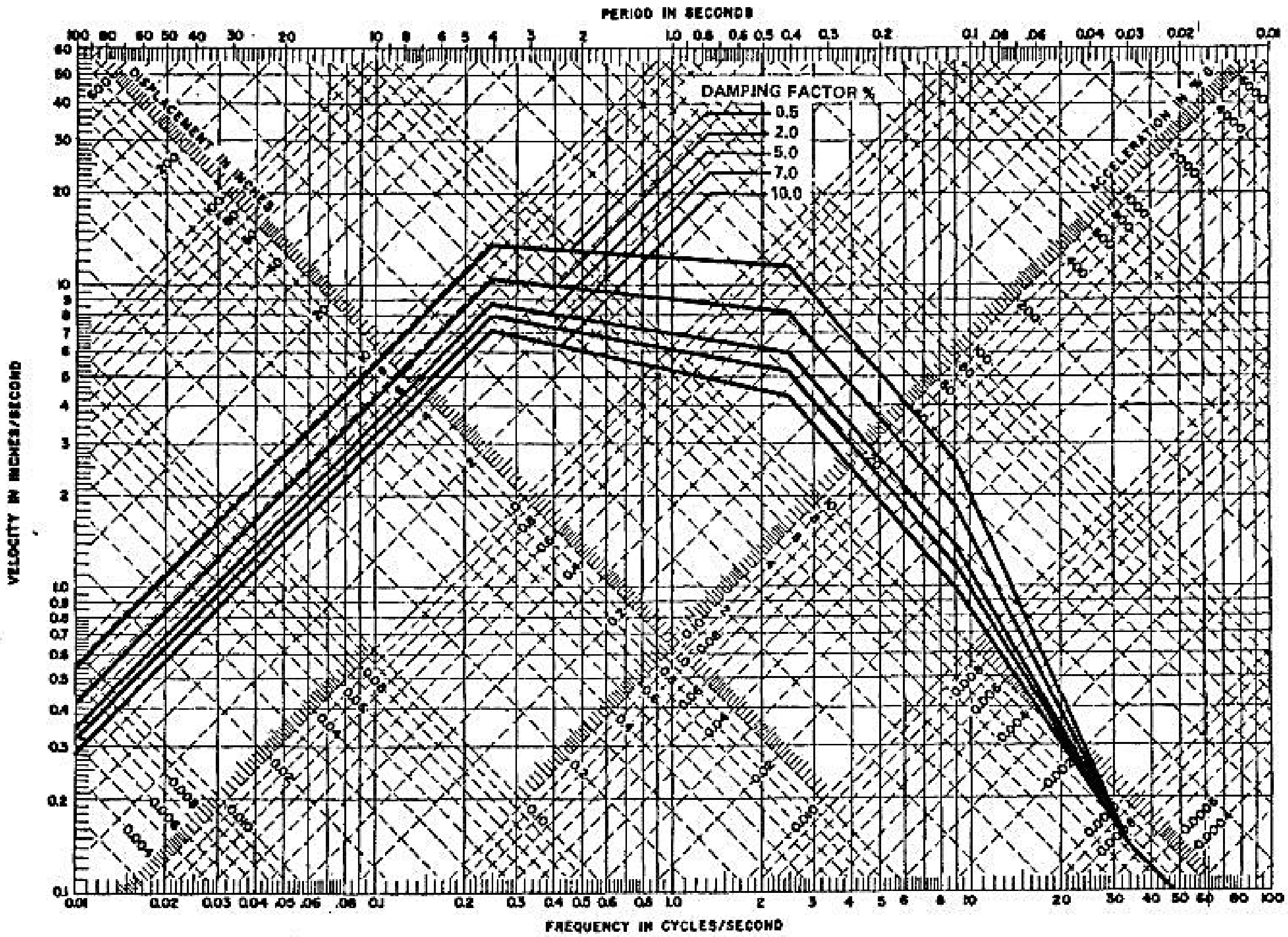


FIGURE 2.5.2-15

VERTICAL DESIGN RESPONSE SPECTRA SCALED TO 0.075G VERTICAL GROUND ACCELERATION

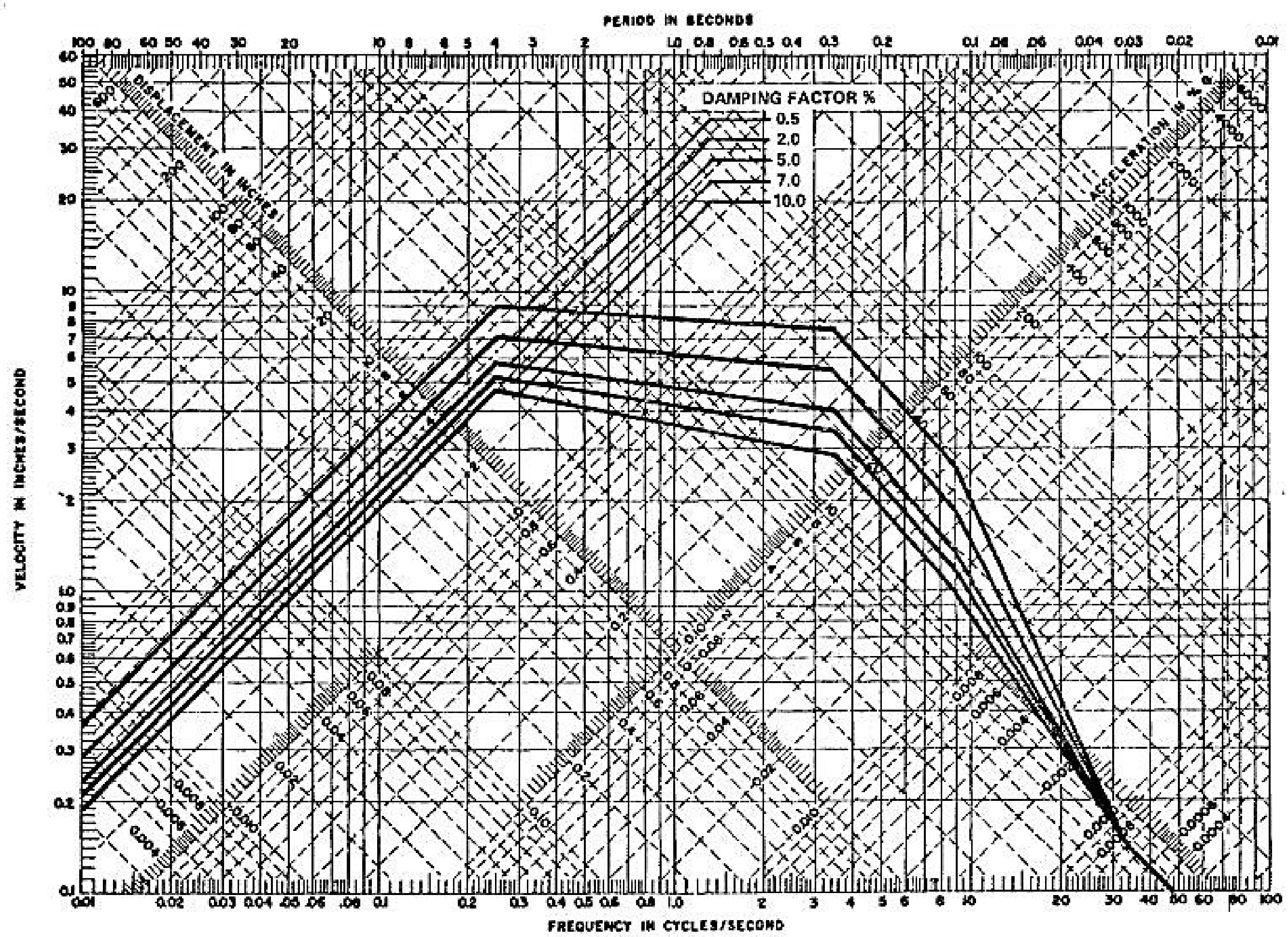


FIGURE 2.5.2-16

EARTHQUAKE OCCURRENCE PROBABILITY

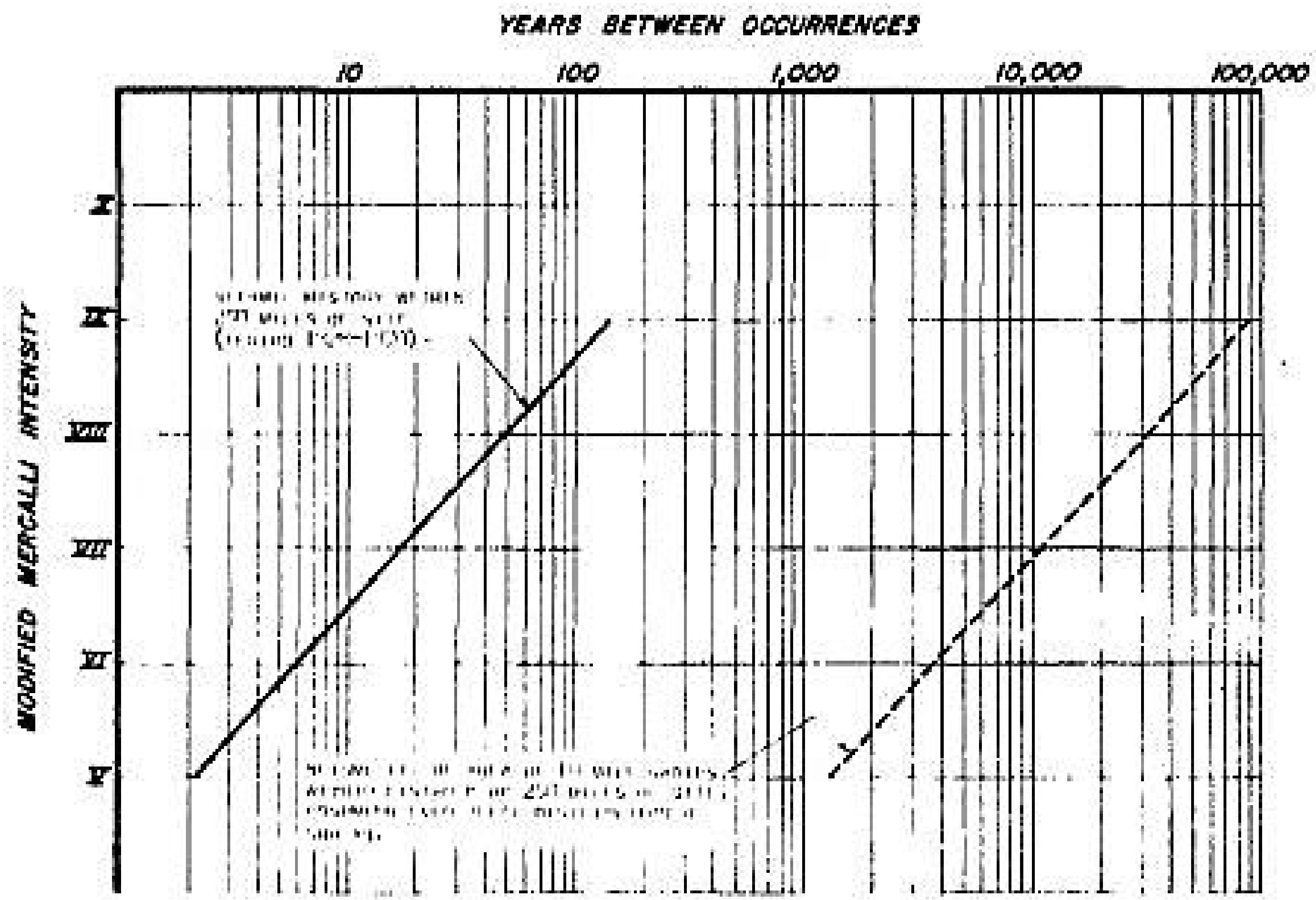
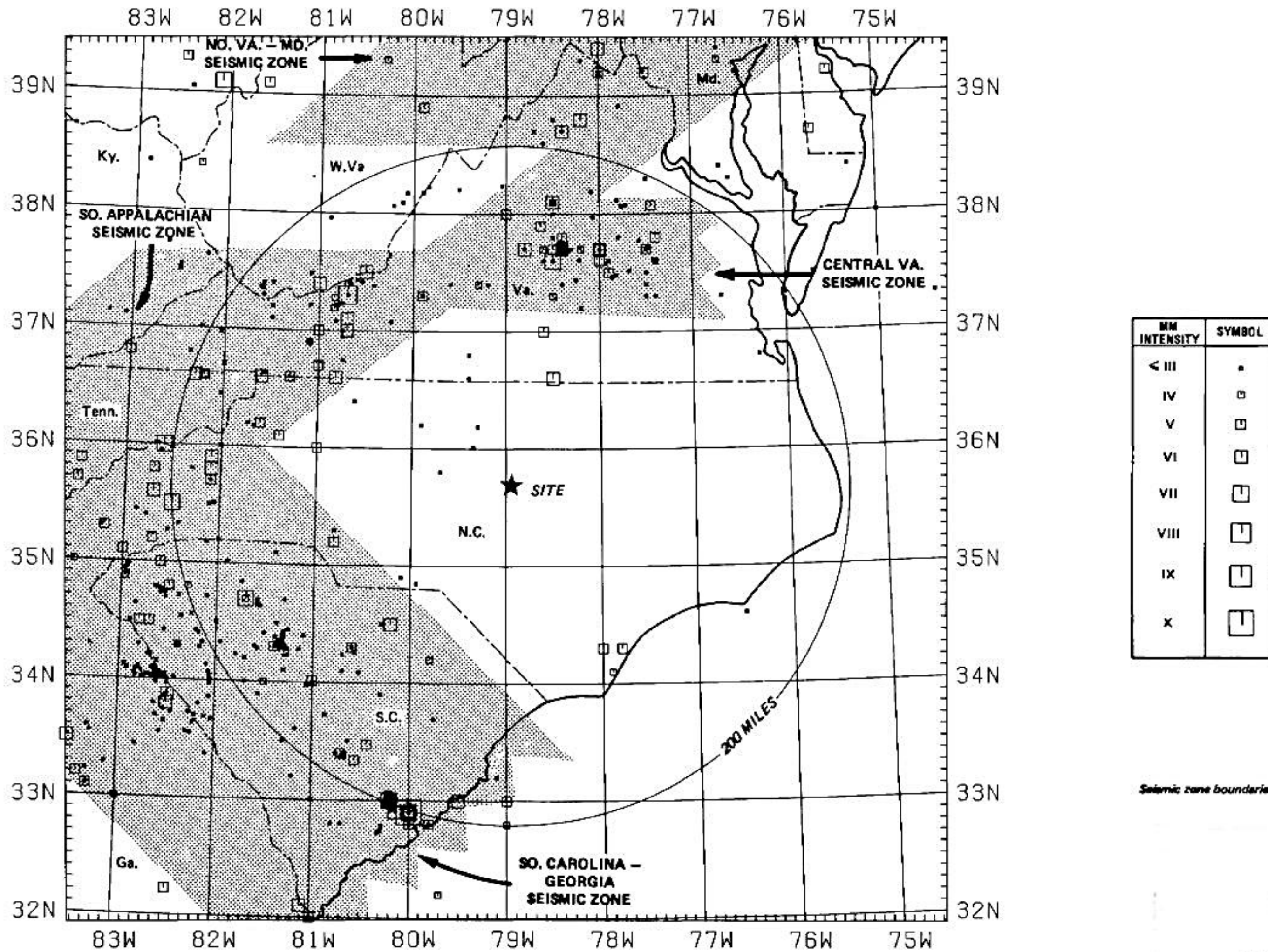


FIGURE 2.5.2-17

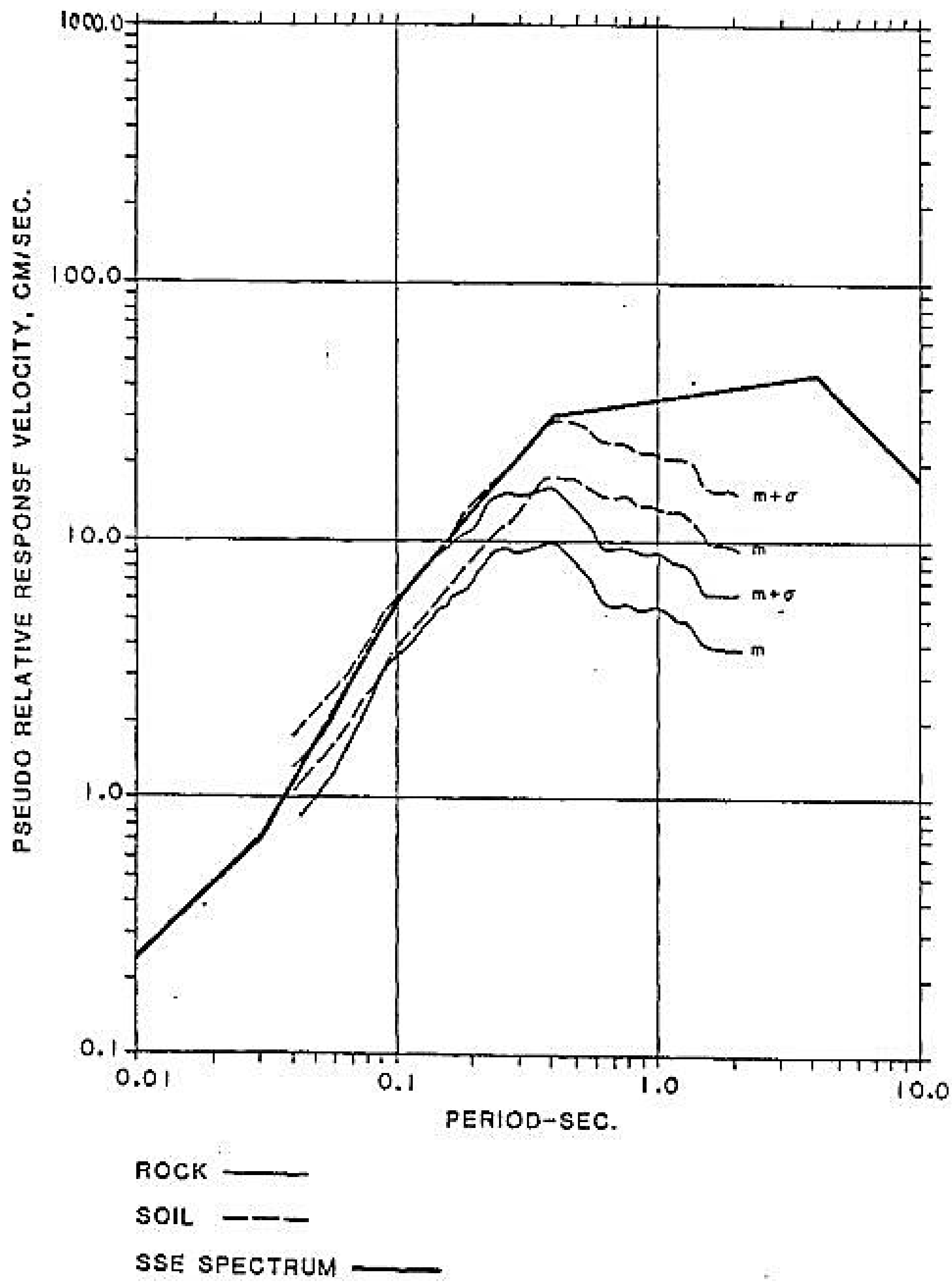
EPICENTER MAP (1696-1961), WITH SEISMIC ZONES, SHOWING EARTHQUAKES WITHIN ABOUT 200 MILES FROM THE SITE



Seismic zone boundaries adapted from Bollinger, 1973.
REFERENCE 2.5.2-23

FIGURE 2.5.2-18

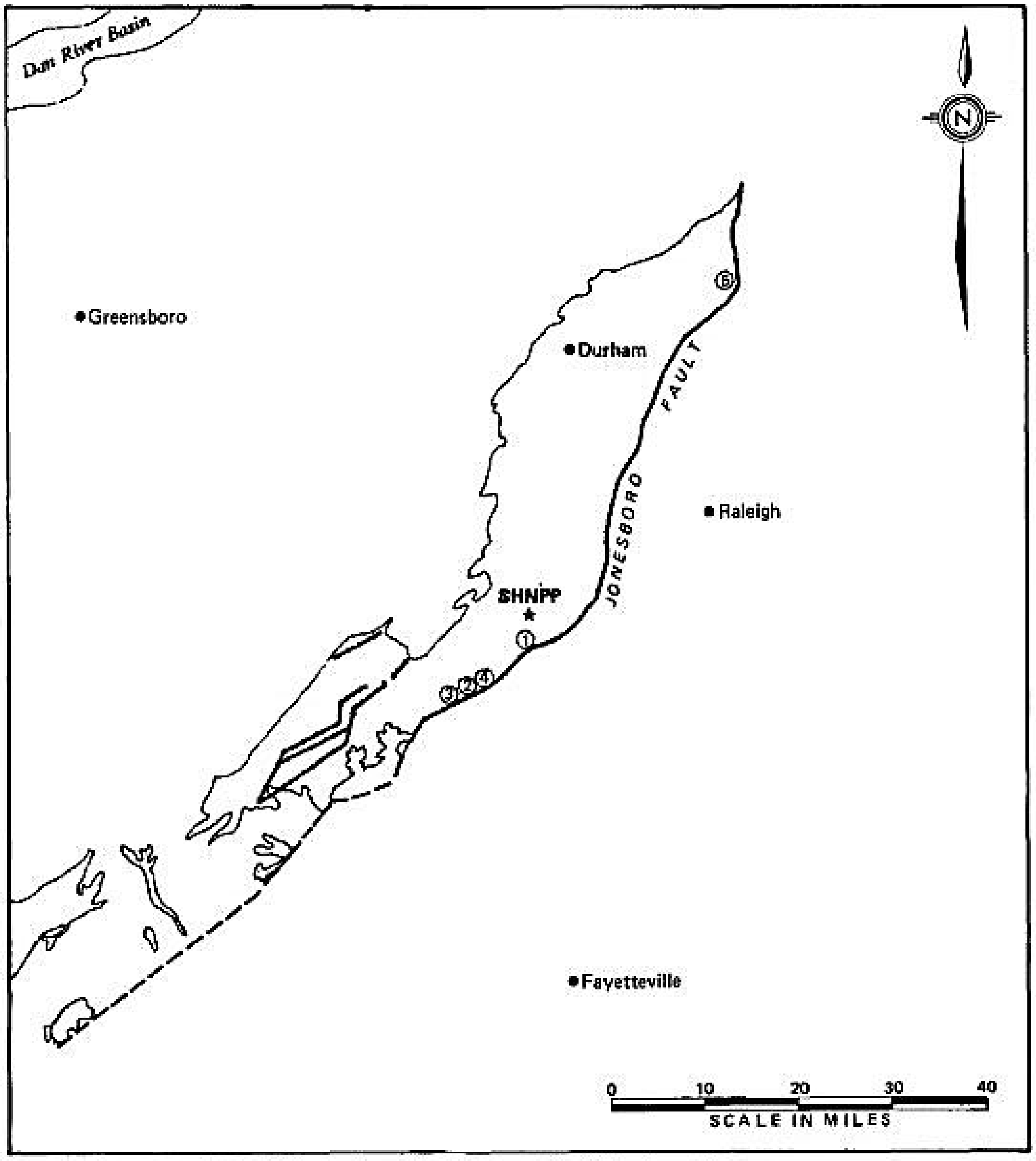
COMPARISON OF SE AND LLNL SPECTRA



Amendment No. 5

FIGURE 2.5.3-1

LOCATION OF MAGNETOMETER SURVEY TRAVERSES



Source: Base map – USGS, 1955. Geology – from Conservation & Development, Division of Mineral Resources. Taken from Oak Creek, CP&L 1971.

EXPLANATION

Triassic Sedimentary Rocks

① *Magnetometer Survey Location*

FIGURE 2.5.3-2

GEOLOGIC PLAN AND SECTIONS ALONG FAULT TRACE AT PLANT EXCAVATION, SHEET 1

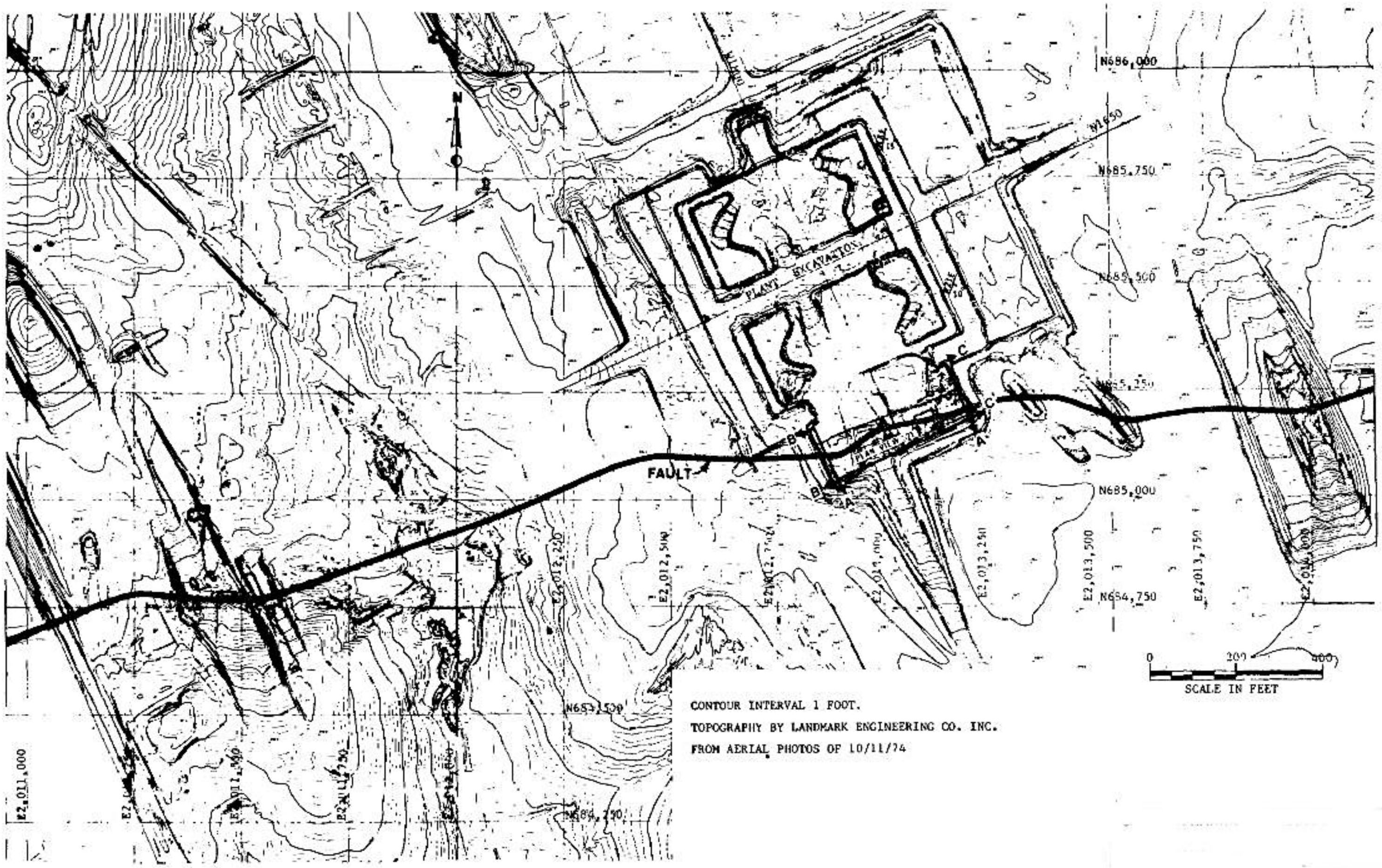


FIGURE 2.5.3-3

GEOLOGIC PLAN AND SECTIONS ALONG FAULT TRACE AT PLANT EXCAVATION, SHEET 2

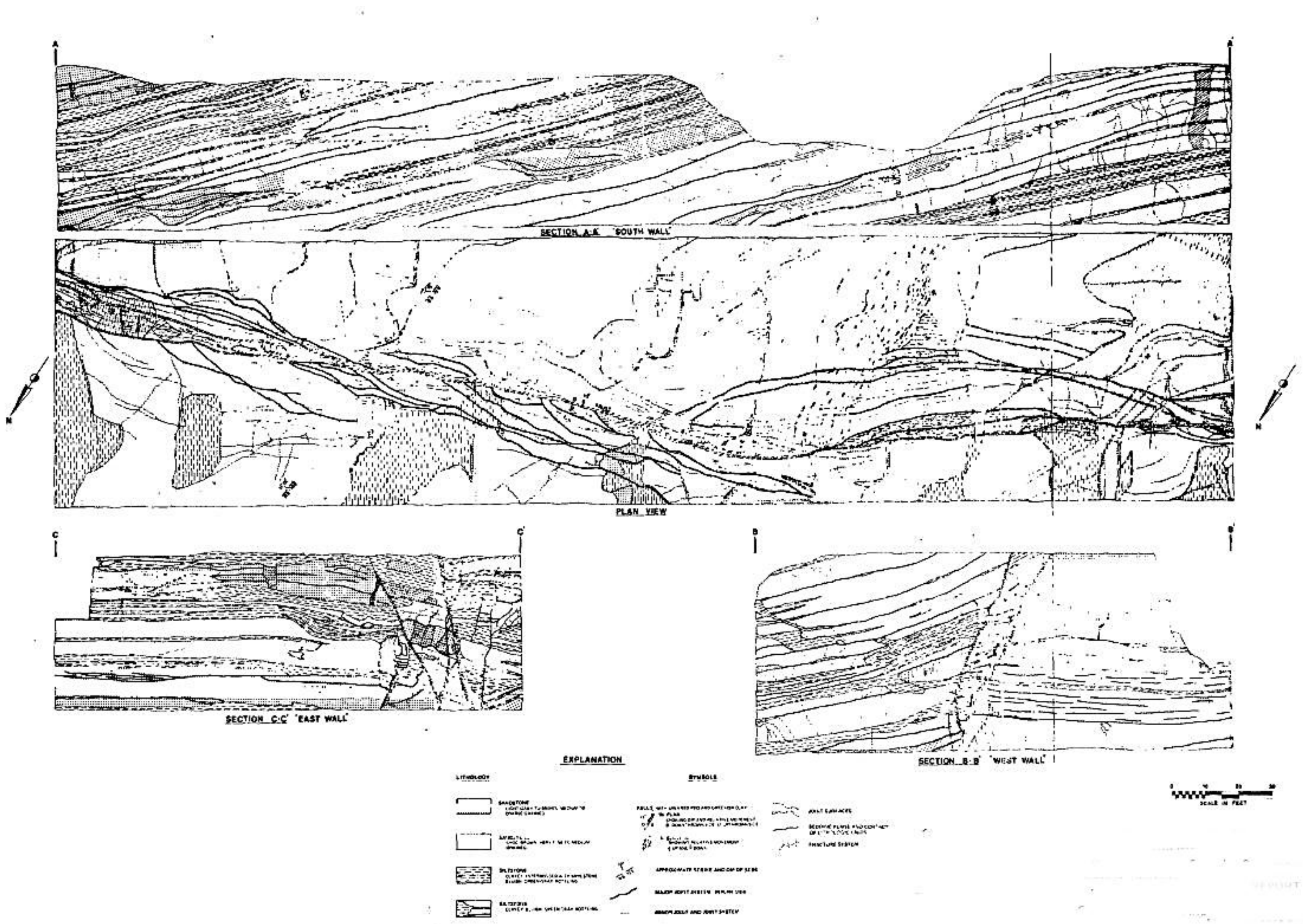


FIGURE 2.5.3-4

WESTWARD FAULT EXTENSION, TRENCH LOCATION PLAN AND SECTIONS

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.3-5

FAULT TRENCH SECTIONS EAST AND WEST SIDE OF PLANT EXCAVATION

FIGURE 2.5.3-6

GEOLOGIC SECTION FROM BORING AT PLANT EXCAVATION

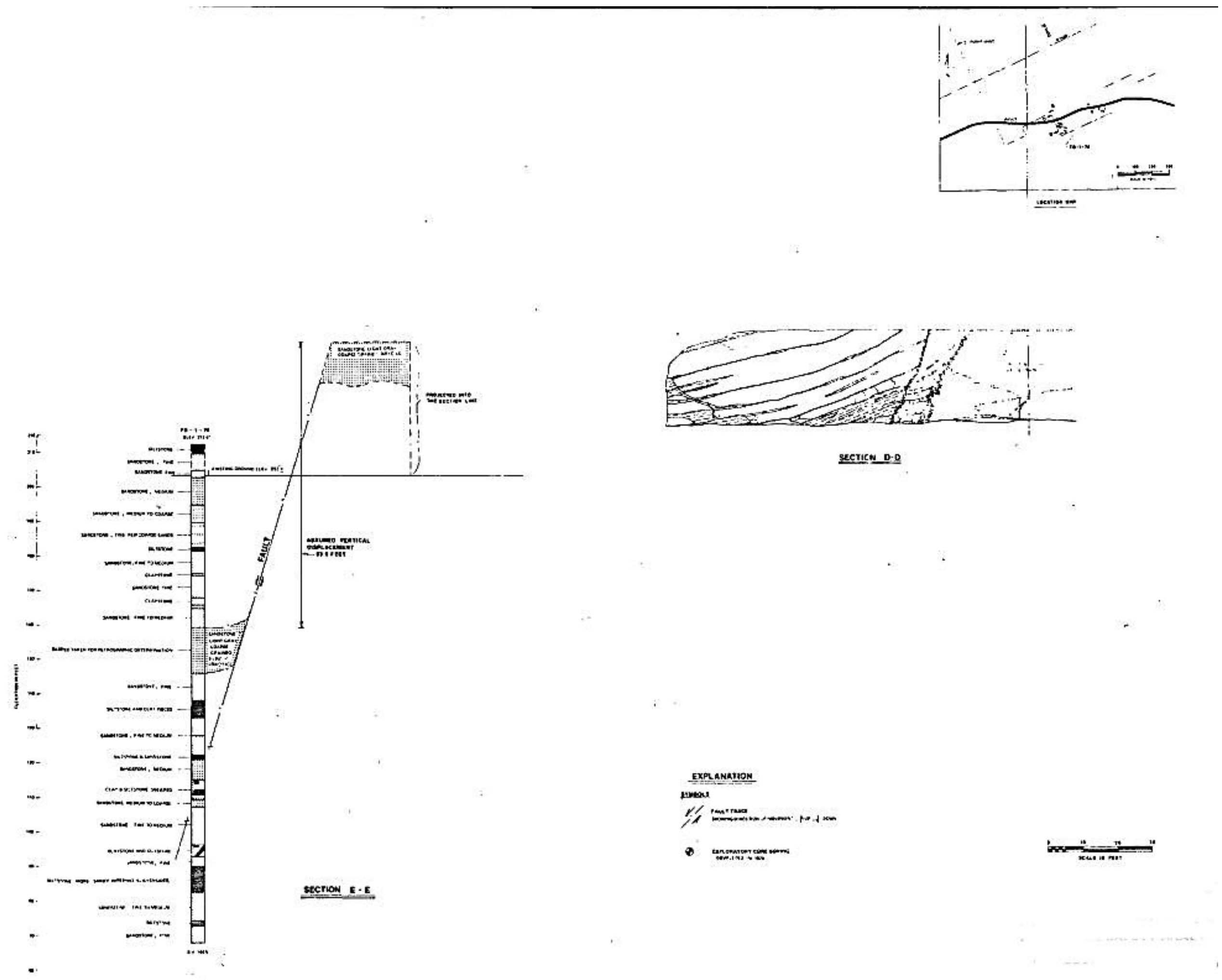


FIGURE 2.5.3-7

GEOLOGIC SECTIONS FROM BORINGS, SHEET 1

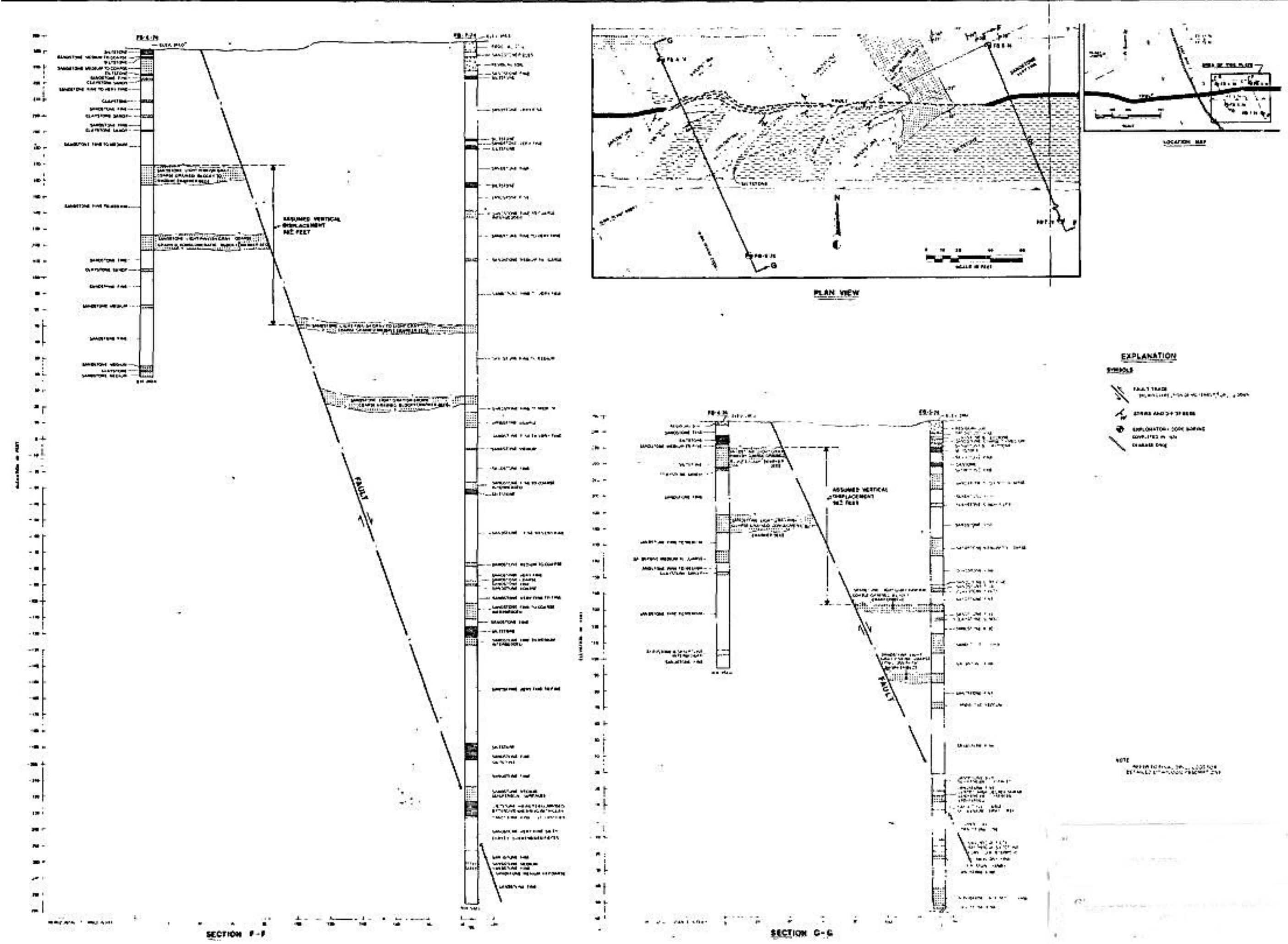


FIGURE 2.5.3-8

GEOLOGIC SECTIONS FROM BORINGS, SHEET 2

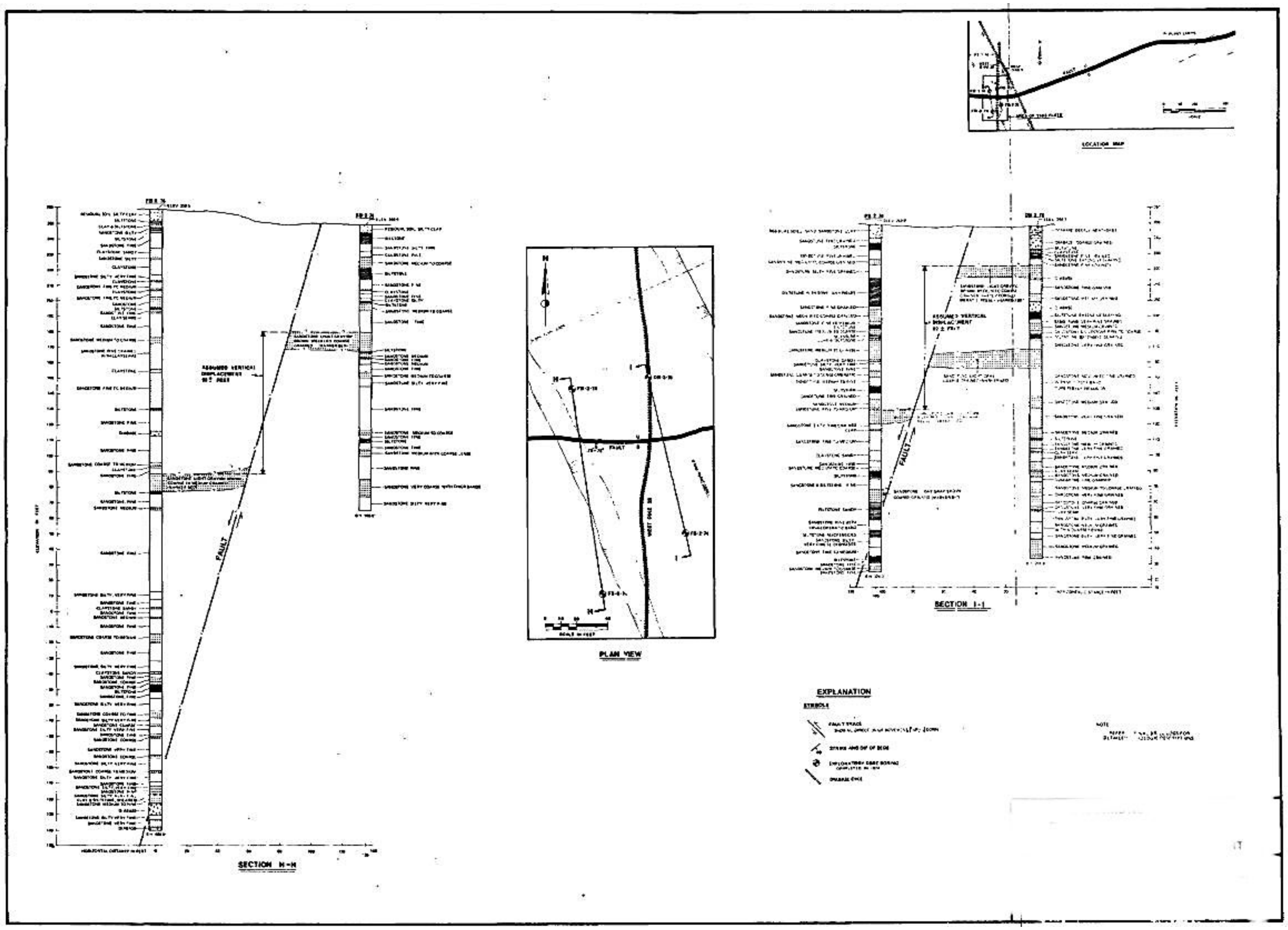


FIGURE 2.5.3-9

GEOLOGIC PLAN VIEWS AT VARIOUS ELEVATIONS ON EAST DIKE 2 AT FAULT

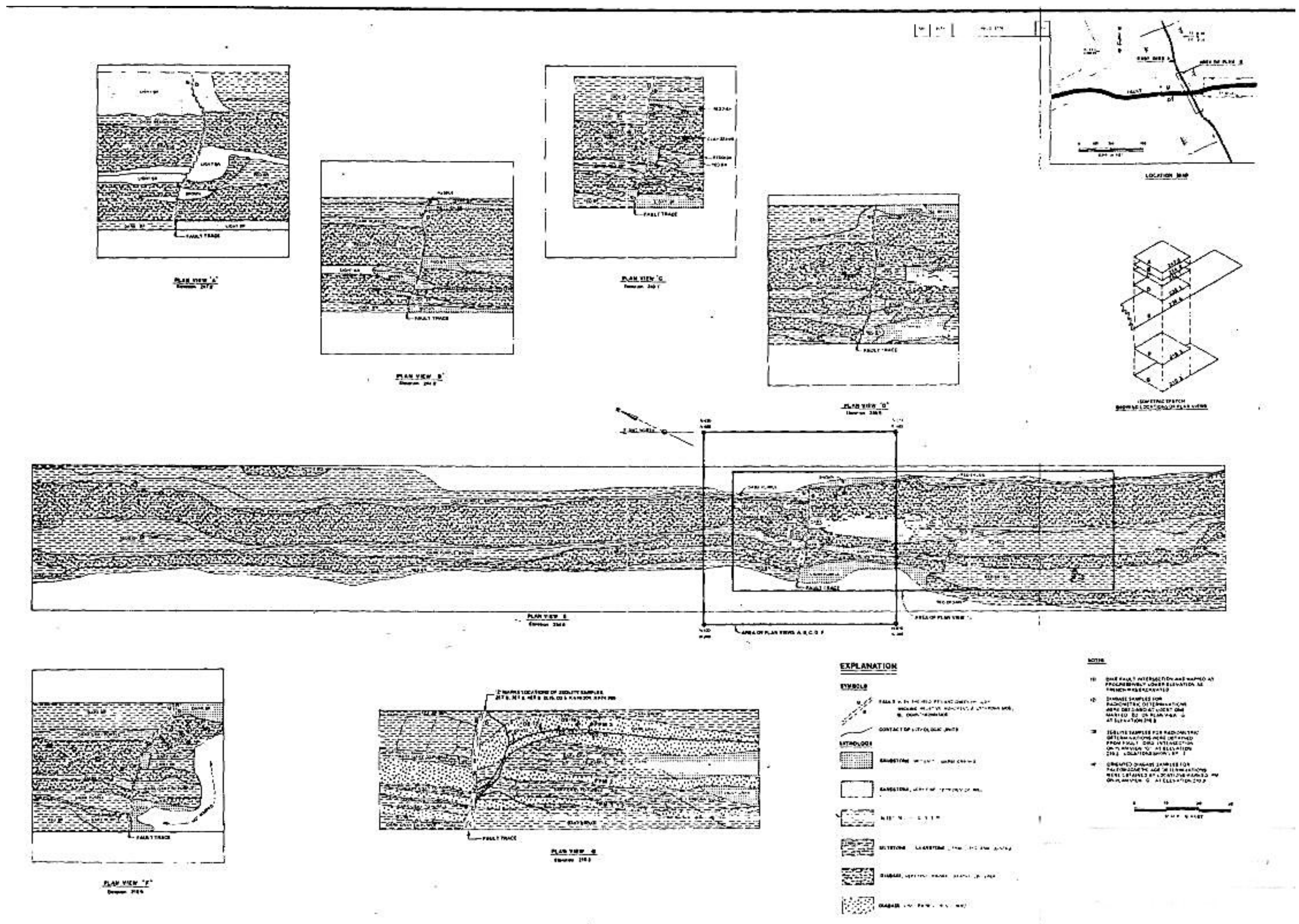


FIGURE 2.5.3-10

EASTWARD VIEW OF FAULT AT EAST DIKE 2

LEGEND
B – LIMITS OF THE THREE DIABASE SEGMENTS WHICH COMPRISE THE COMPOSITE DIKE.
C – HORNFELS AREAS.
D – SECONDARY FRACTURES ASSOCIATED WITH THE COMMONLY OBSERVED DRAG FOLDING IN MATERIAL IMMEDIATELY SOUTH OF THE FAULT.
Z – LOCATIONS FROM WHICH ZEOLITE MINERAL SAMPLES WERE TAKEN FROM THE FAULT GOUGE.

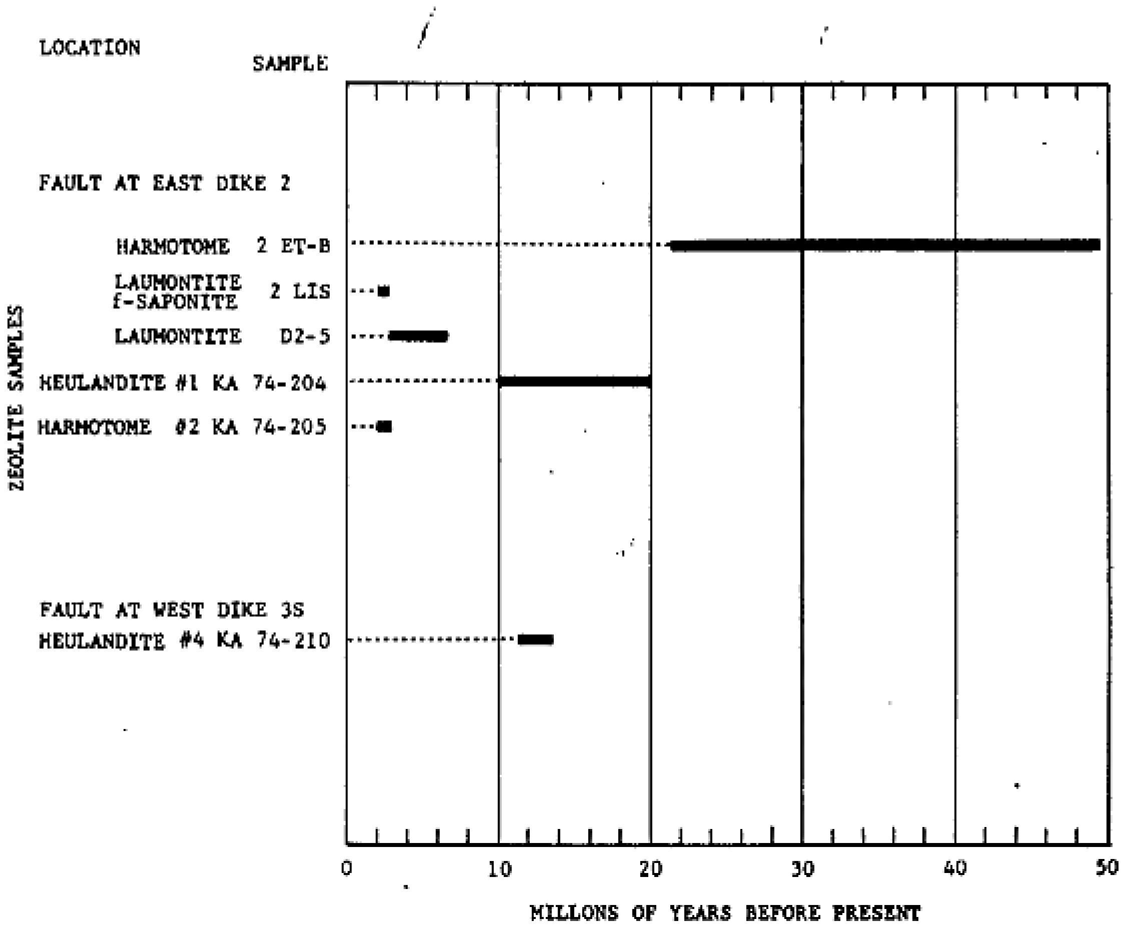
FIGURE 2.5.3-11

FAULT INTERSECTION WITH WEST DIKE 3S

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.3-12

MINIMUM AGE DETERMINATIONS OF ZEOLITE MINERALS IN FAULT



NOTES:

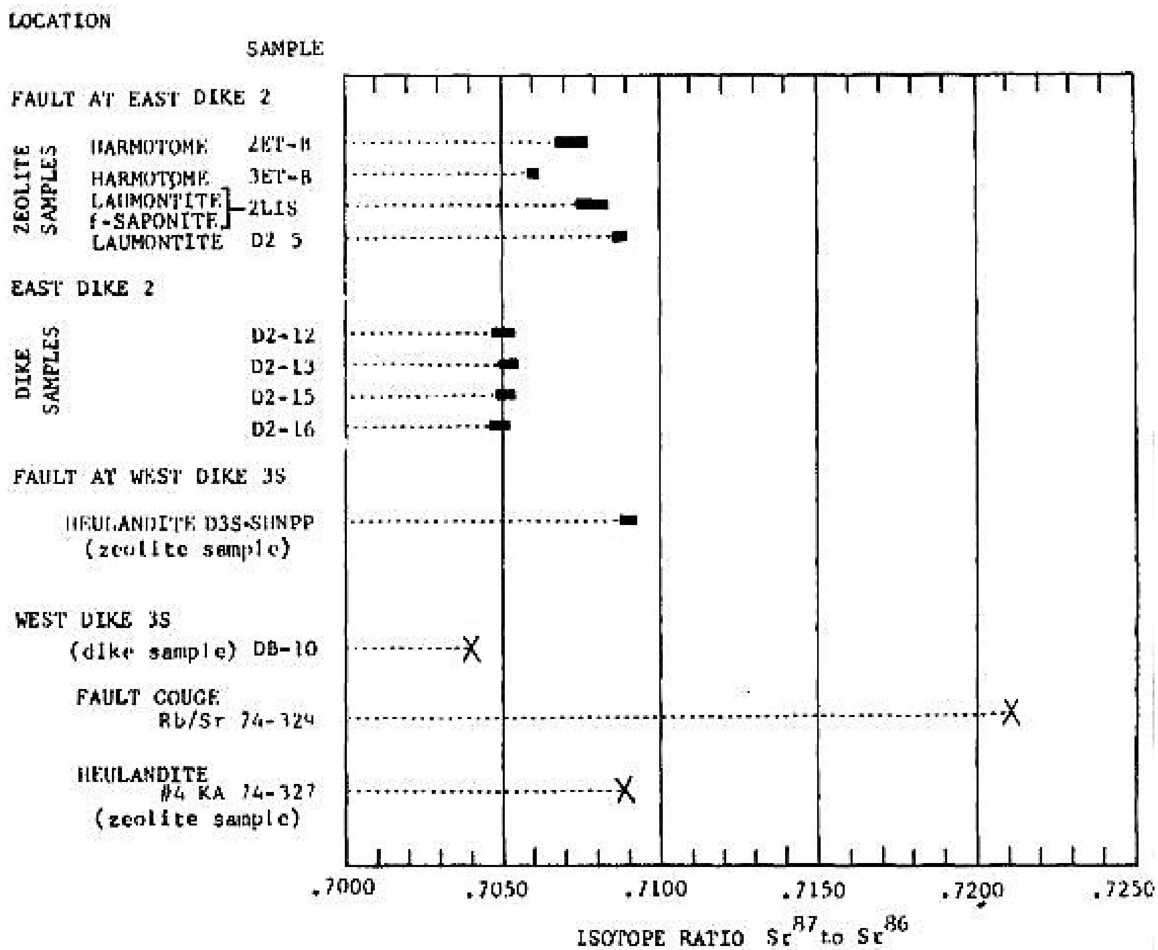
1. FOR ZEOLITE SAMPLE LOCATIONS SEE FIGURES 2.5.3-8, 2.5.3-15 AND 2.5.3-16.
2. FOR ZEOLITE MINERAL IDENTIFICATION SEE APPENDIX H OF FAULT INVESTIGATION REPORT (REFERENCE 2.5.1-29)
3. FOR RADIOMETRIC DATING OF SPECIFIC SAMPLES SEE APPENDIXES OF FAULT INVESTIGATION REPORT LISTED BELOW:

   SAMPLES 2 ET-B, 2 LIS, D2-5; APPENDIX F

   SAMPLES #1 KA 74-204, #2 KA 74-205, #4 KA 74-210: APPENDIX E

FIGURE 2.5.3-13

STRONTIUM ISOTOPE RATIOS OF ZEOLITES AND DIKES



ISOTOPE RATIO DETERMINED BY CENTER FOR INTERCONTINENTAL STUDIES ALLOWING FOR ANALYTICAL ERROR (See Appendix F of FAULT INVESTIGATION REPORT, REFERENCE 2.5.1-29)

X ISOTOPE RATIO DETERMINED BY TELEDYNE ISOTOPES NOT ALLOWING FOR ANALYTICAL EFFOR (See Appendix E OF FAULT INVESTIGATION REPORT, REFERENCE 2.5.1-29)

NOTES:
1. FOR LOCATIONS OF ZEOLITE AND DIKE SAMPLES SEE FIGURES 2.5.3-9, 2.5.3-15 AND 2.5.3-16.
2. FOR ZEOLITE MINERAL IDENTIFICATIONS SEE Appendix H OF FAULT INVESTIGATION REPORT, REFERENCE 2.5.1-29.

FIGURE 2.5.3-14

DETAILED LOG OF TRENCH FET – 19W

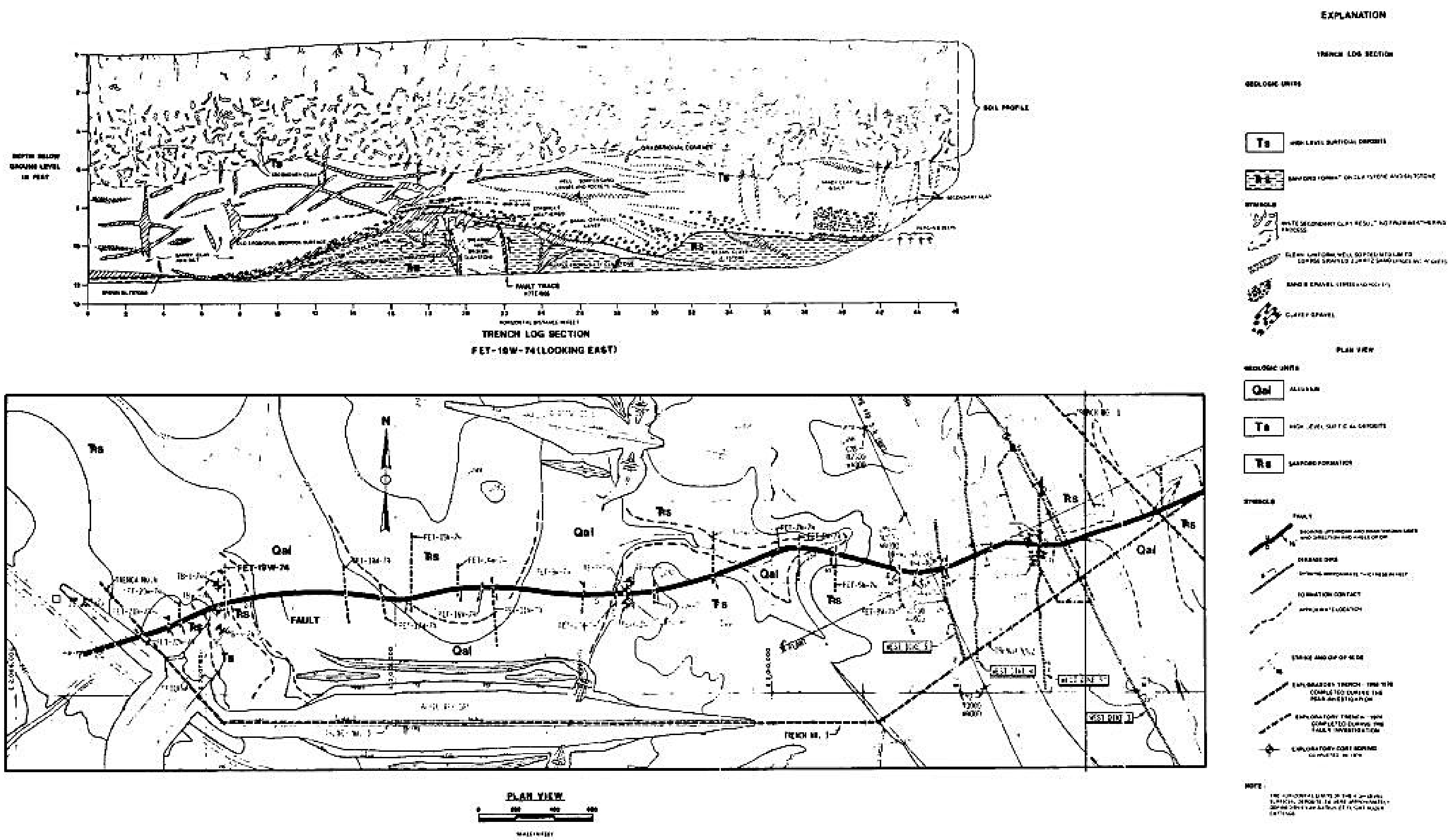


FIGURE 2.5.3-15

GEOLOGIC PLAN AND SECTION FAULT INTERSECTION OF WEST DIKE 3 AND WEST DIKE 3S

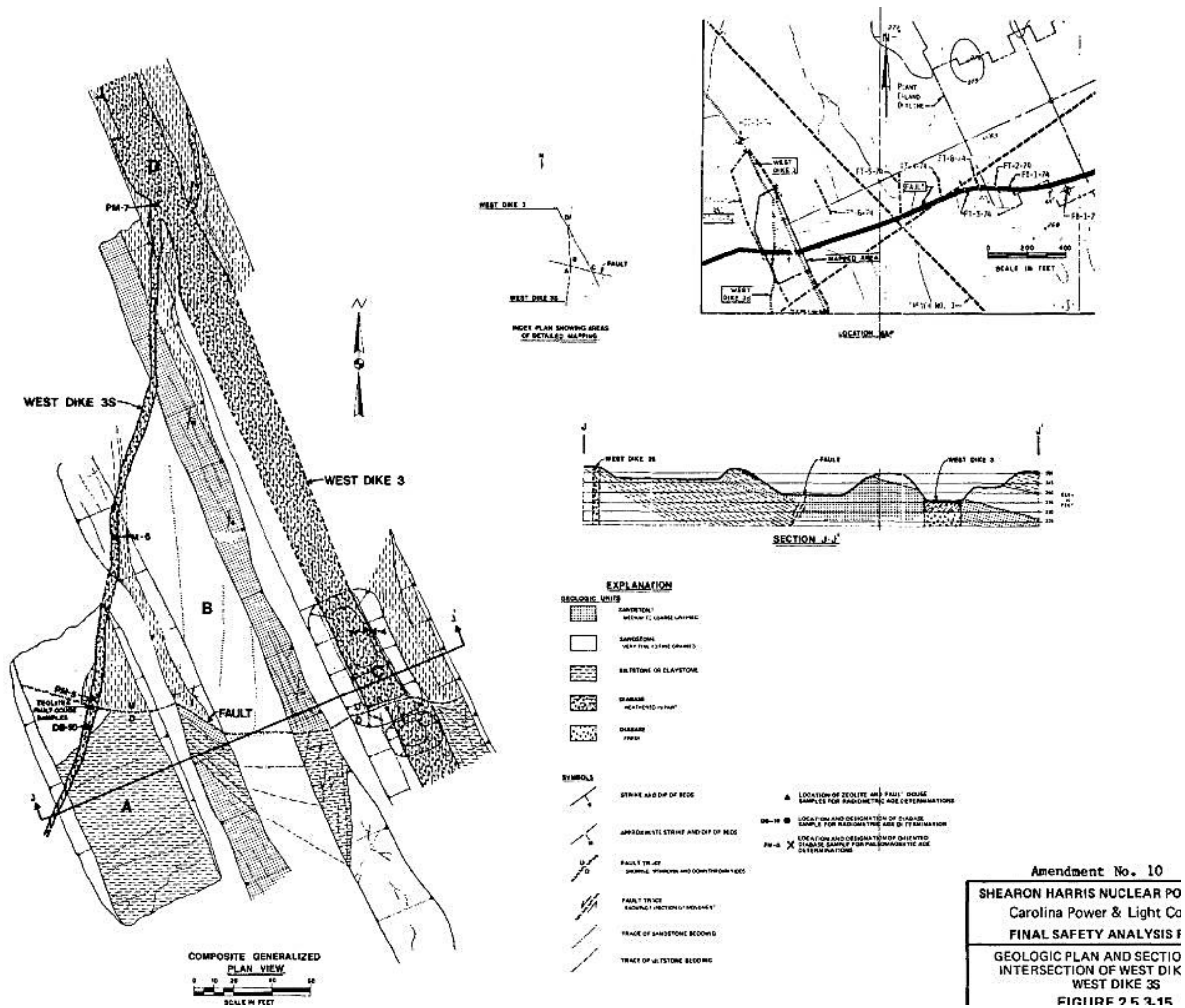


FIGURE 2.5.3-16

DETAILED GEOLOGIC PLAN VIEW OF WEST DIKE 3S AT FAULT, SHEET 1

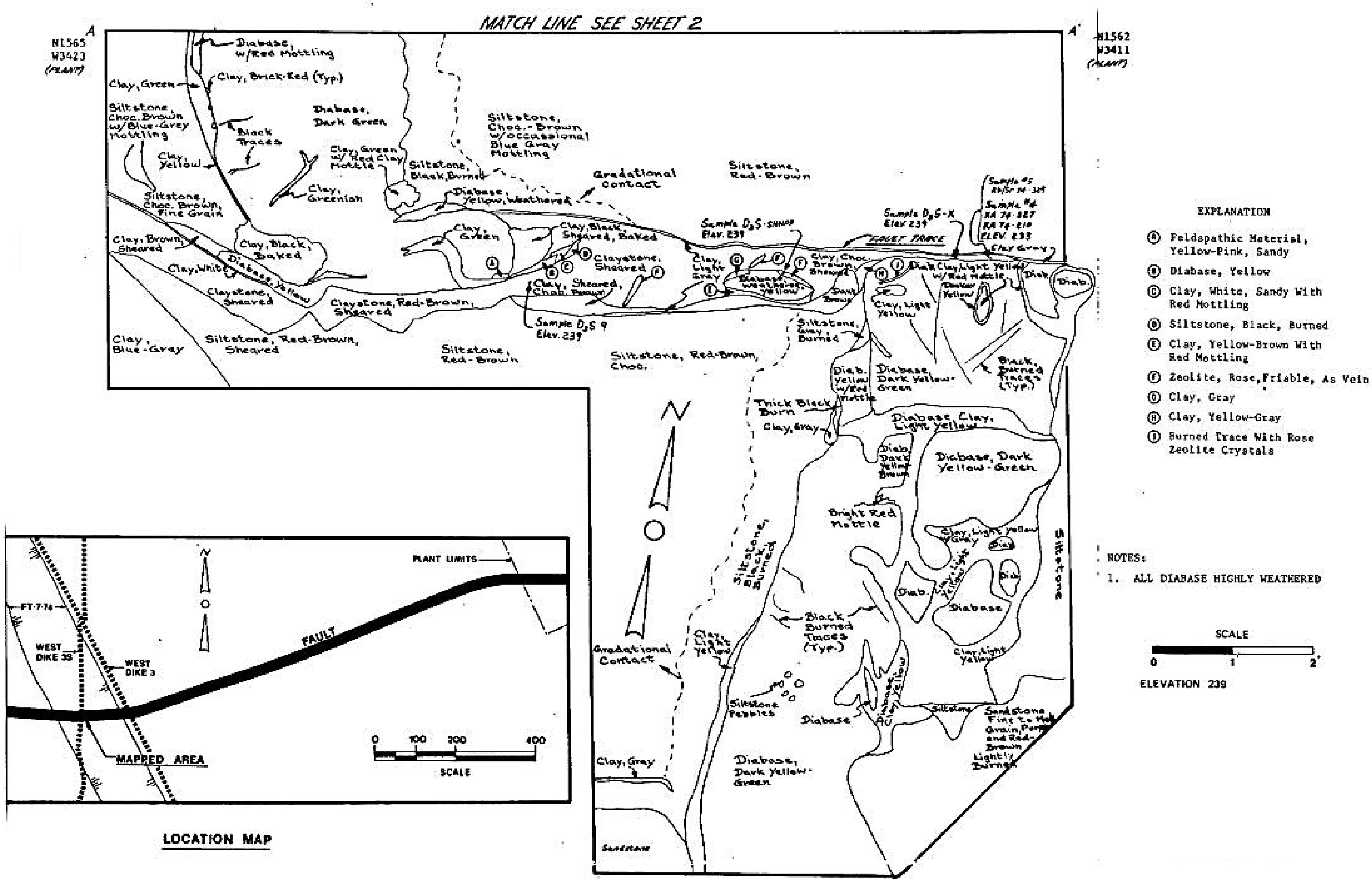


FIGURE 2.5.3-17

DETAILED GEOLOGIC PLAN VIEW OF WEST DIKE 3S AT FAULT, SHEET 2

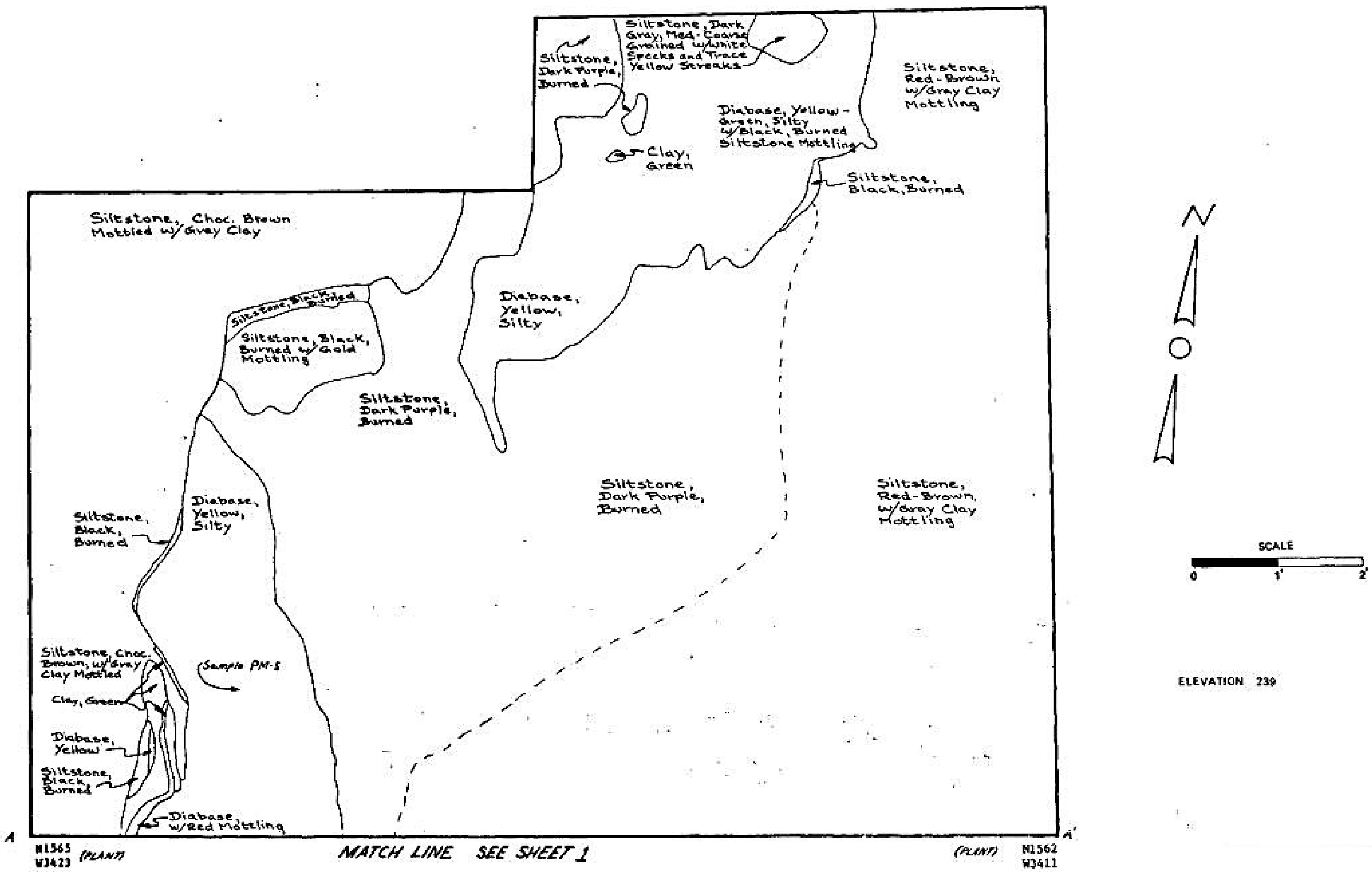


FIGURE 2.5.3-18

IDEALIZED FAULTED STRUCTURE DIAGRAMS AT WEST DIKE 3S – FAULT INTERSECTION

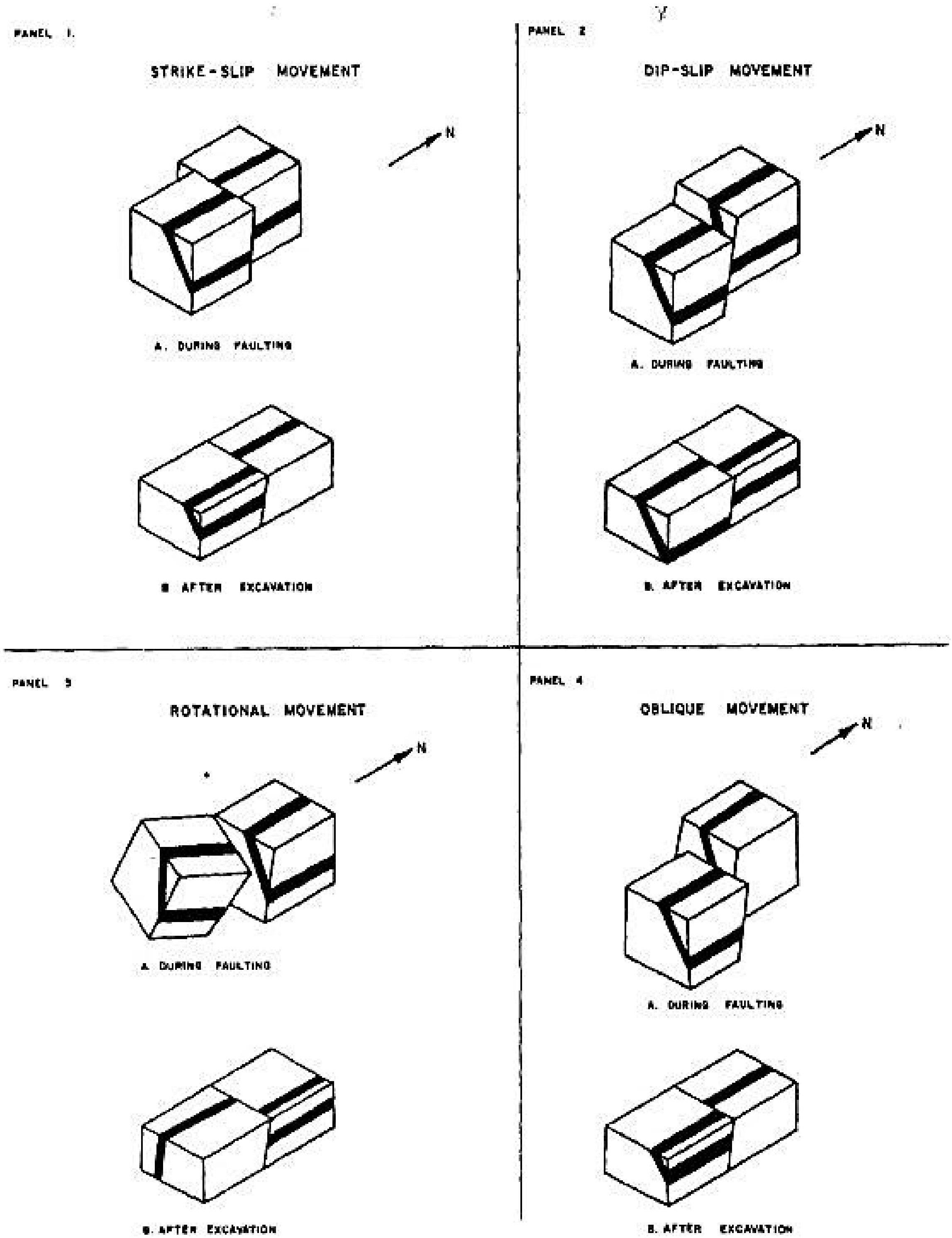
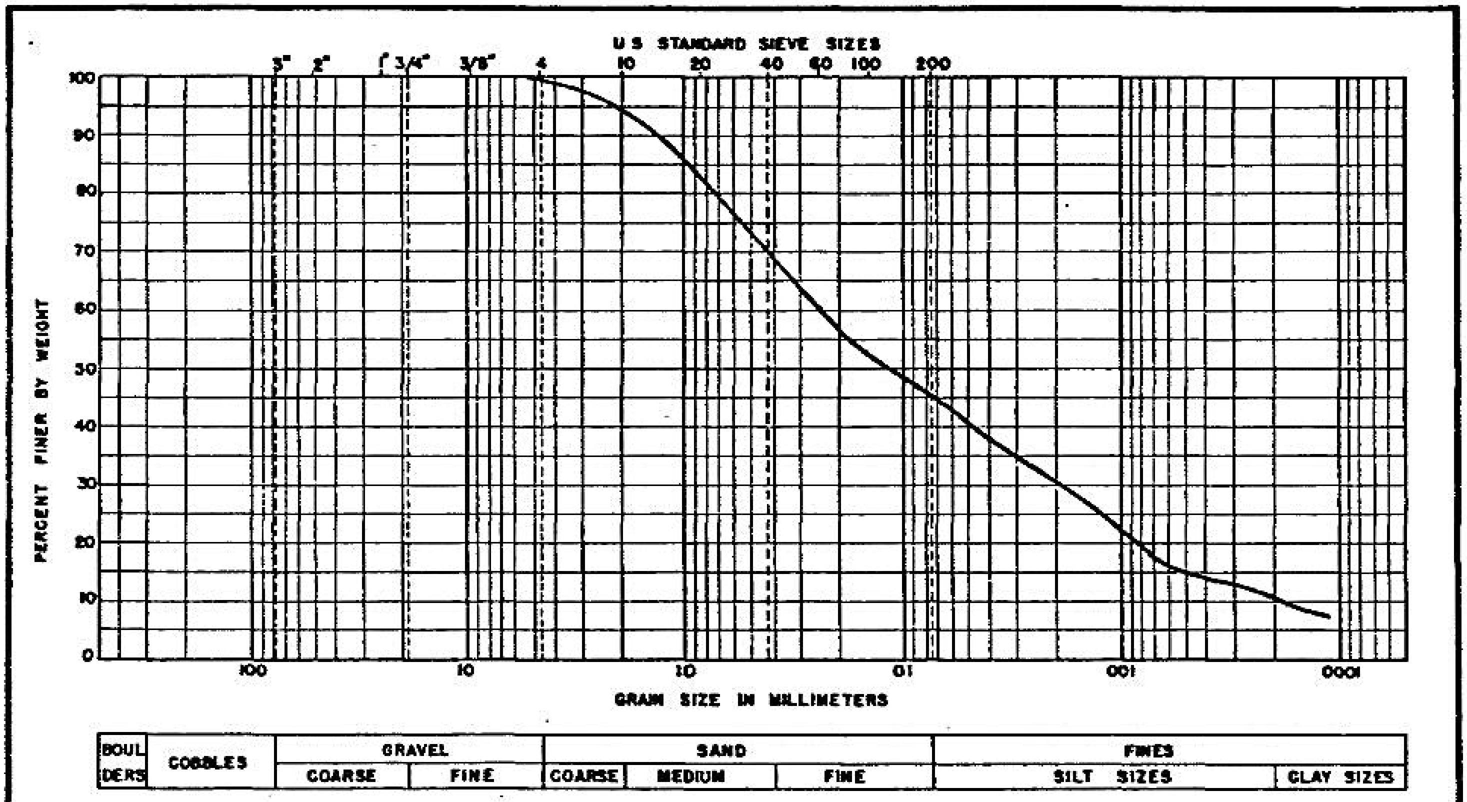


FIGURE 2.5.4-1

GRAIN SIZE DISTRIBUTION, BORING NO. BP-1, S-1



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BP-1	5.0'-6.5' S-2					BROWN AND GRAY SILTY FINE TO COARSE SAND

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-2

GRAIN SIZE DISTRIBUTION, BORING NO. BP-1, S-2

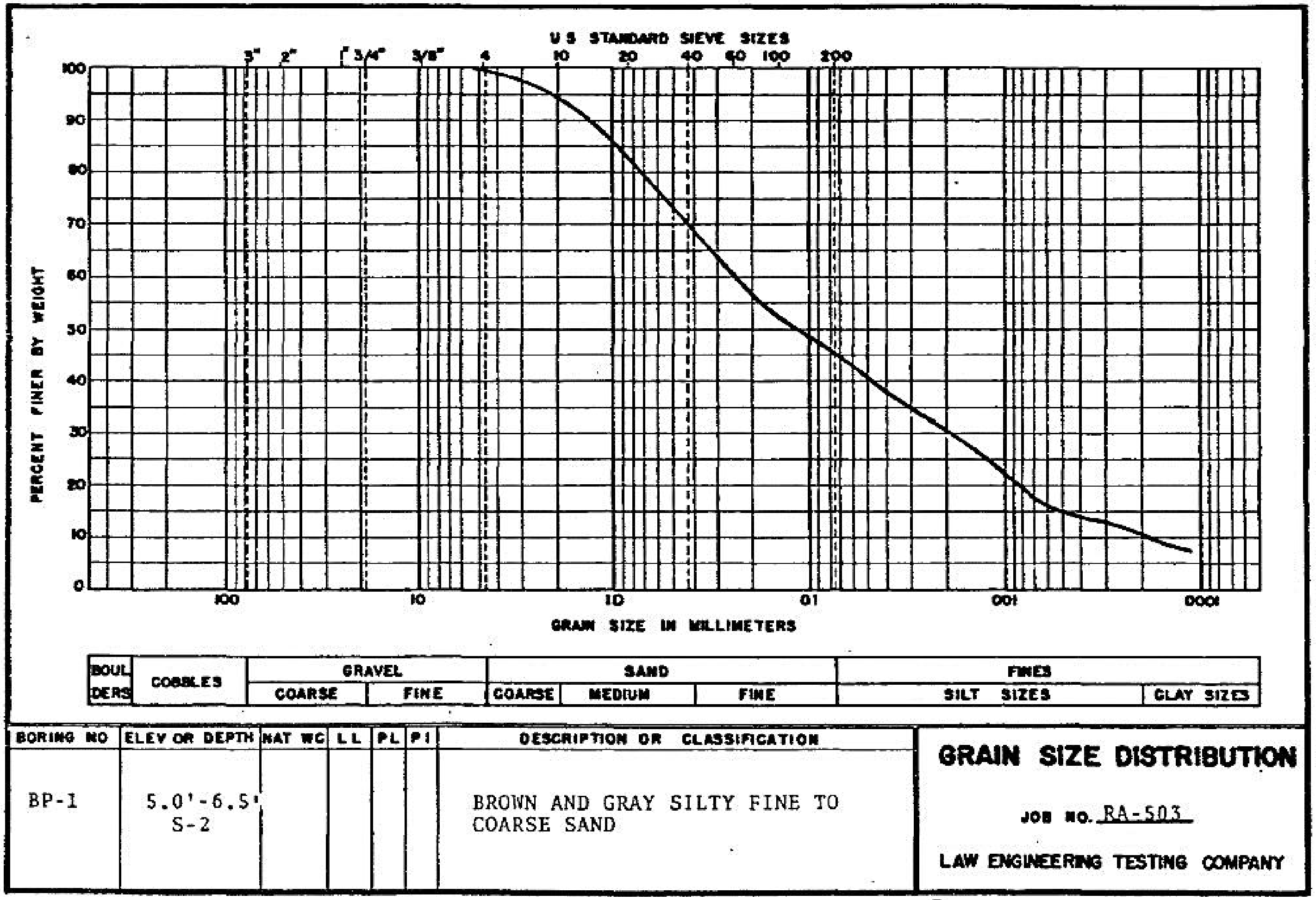
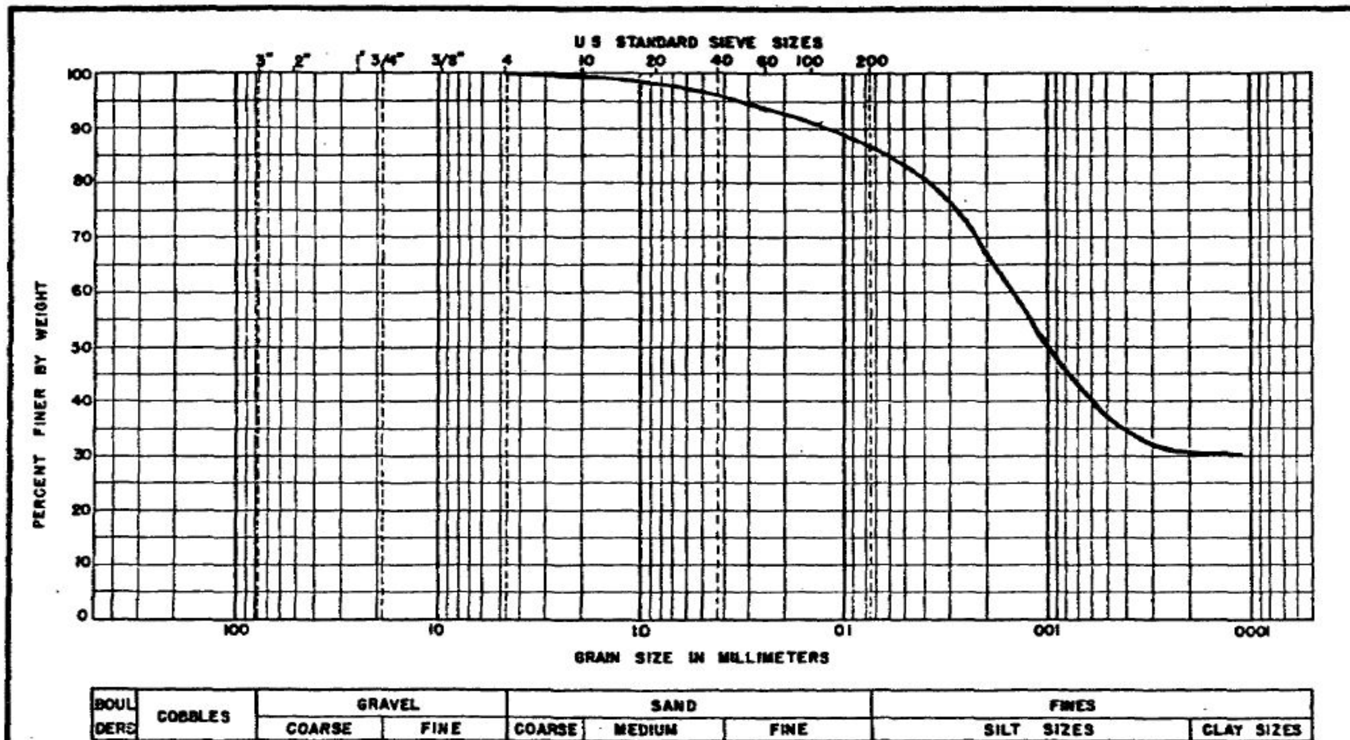


FIGURE 2.5.4-3

GRAIN SIZE DISTRIBUTION, BORING NO. BP-5, S-1



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BP-5	1.0'-2.5' S-1					YELLOW AND BROWN FINE SANDY VERY CLAYEY SILT

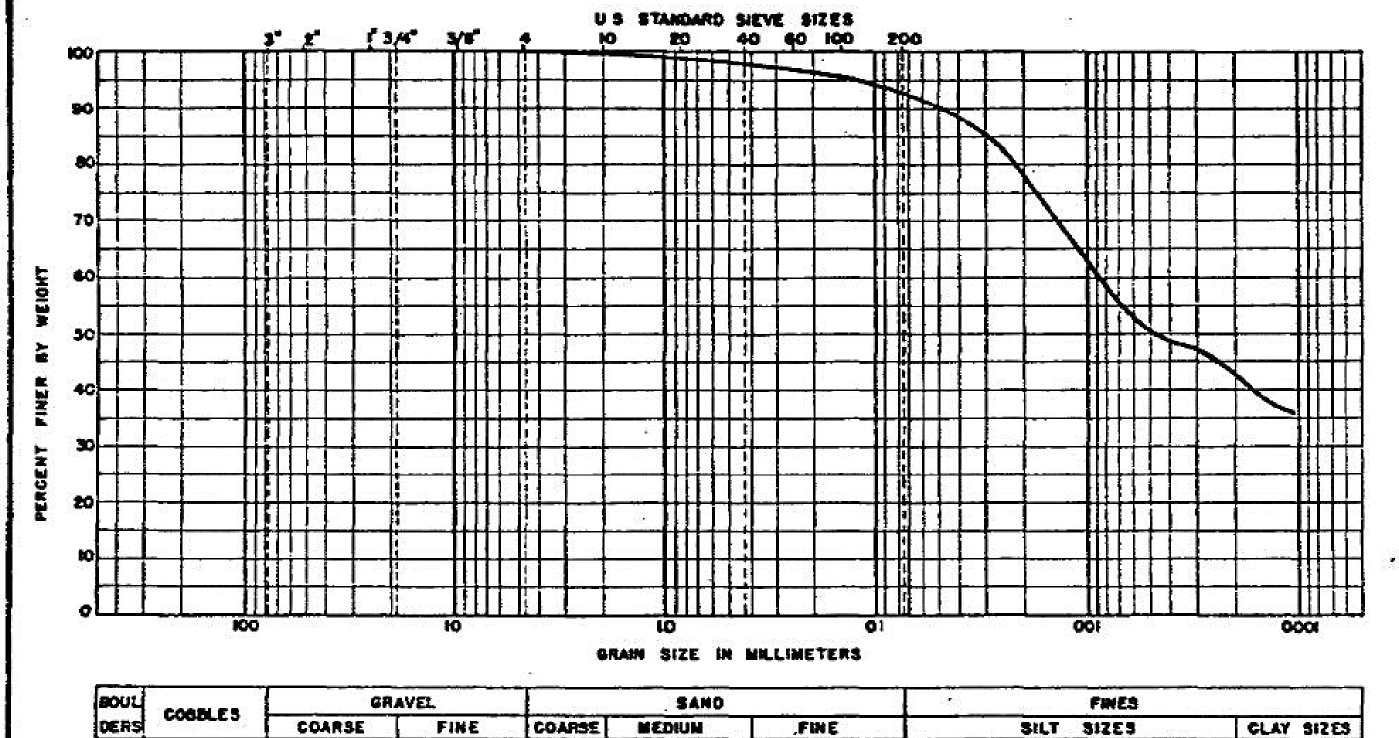
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-4

GRAIN SIZE DISTRIBUTION, BORING NO. BP-5, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BP-5	3.5'-5.0' S-2					YELLOW AND BROWN FINE SANDY CLAYEY SILT

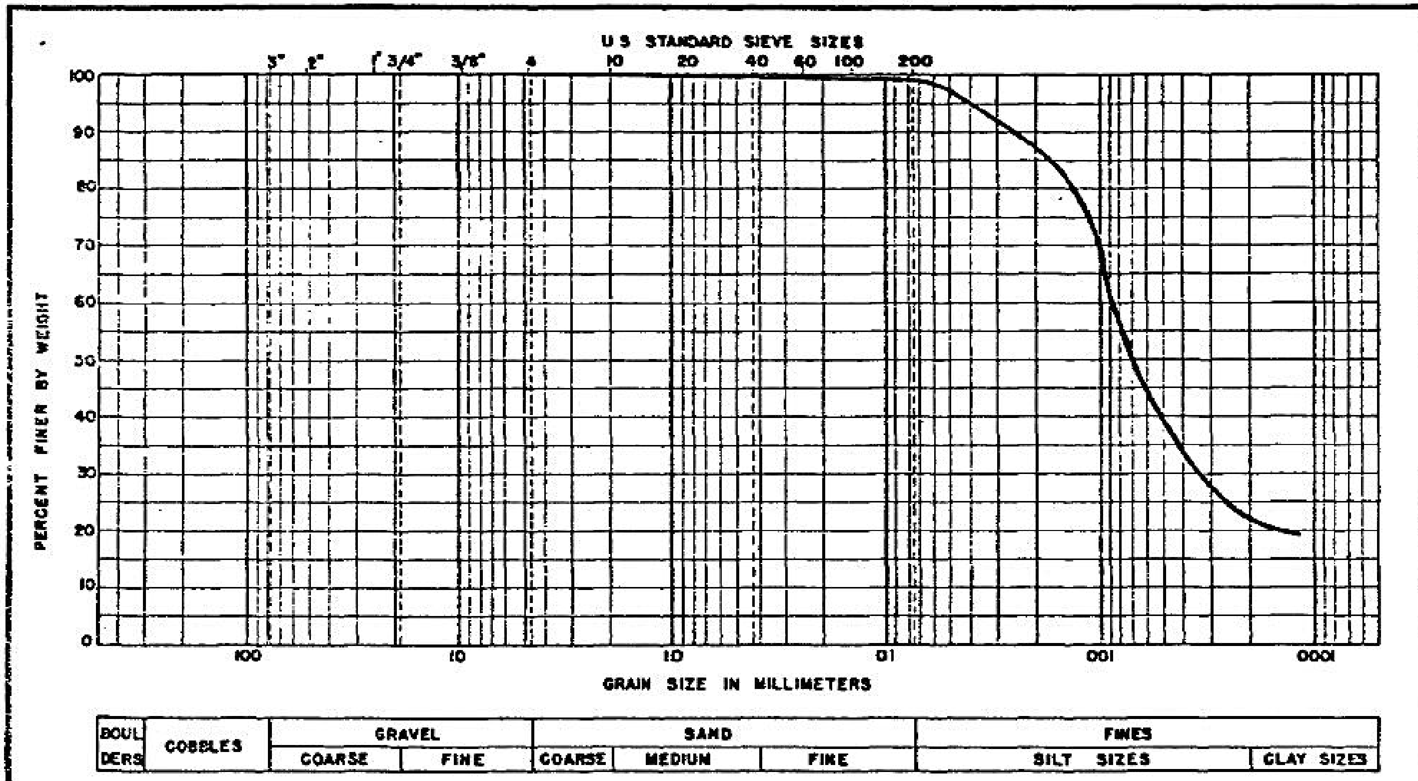
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-5

GRAIN SIZE DISTRIBUTION, BORING NO. BP-9, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BP-9	5.0'-6.5' S-2					RED BROWN FINE SANDY CLAYEY SILT

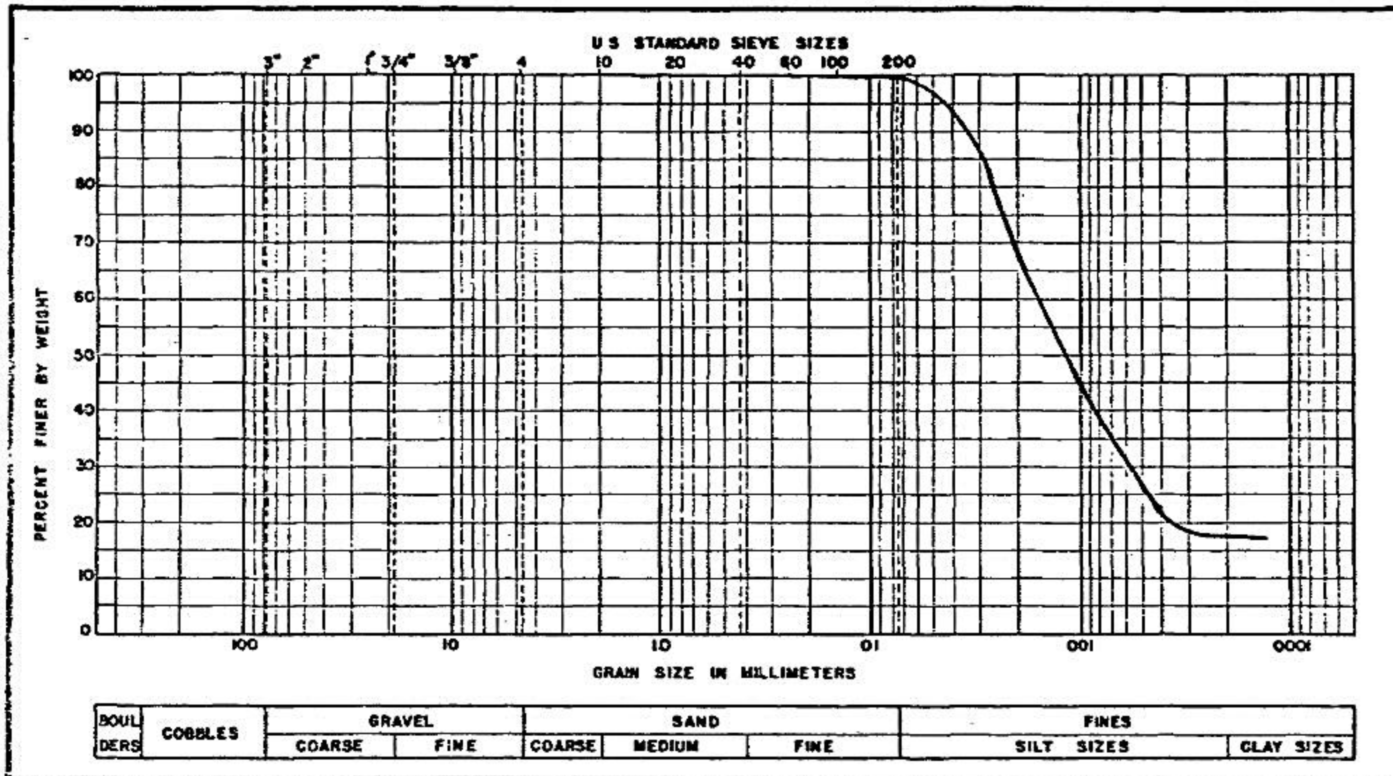
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-6

GRAIN SIZE DISTRIBUTION, BORING NO. BP-9, S-3



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BP-9	10.0'-11.5' S-3					RED BROWN FINE SANDY CLAYEY SILT

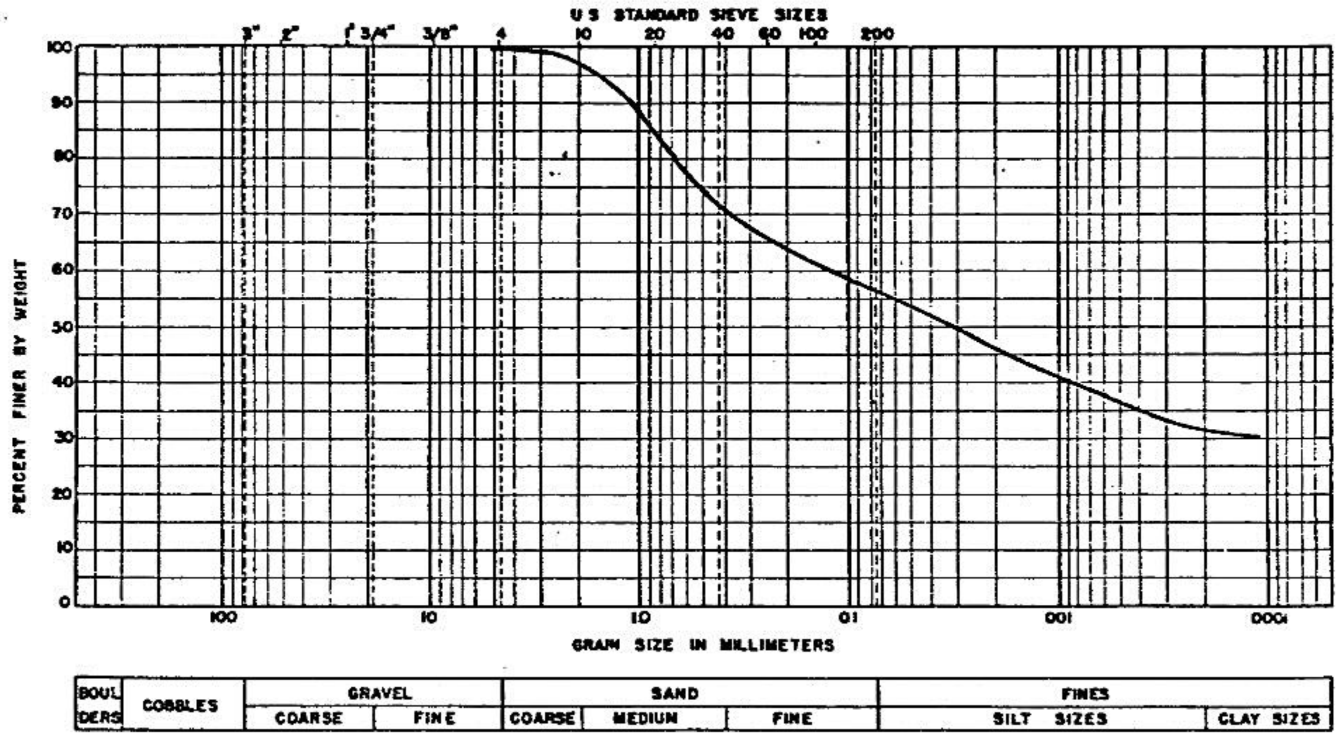
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-7

GRAIN SIZE DISTRIBUTION, BORING NO. BP-16, S-1



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BP-16	1.0'-2.5' S-1					YELLOW AND GRAY SILTY FINE TO MEDIUM SANDY CLAY

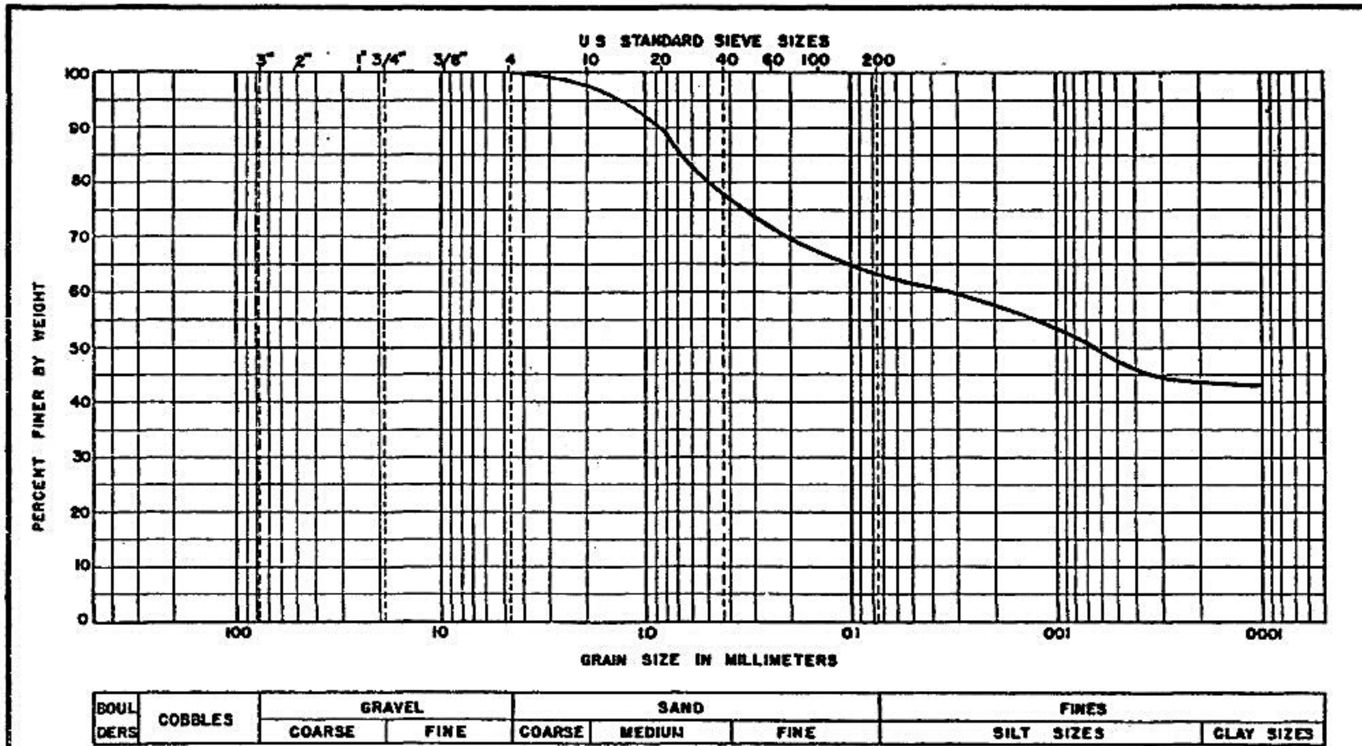
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-8

GRAIN SIZE DISTRIBUTION, BORING NO. BP-16, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BP-16	5.0'-6.5' S-2					YELLOW AND GRAY SILTY FINE TO COARSE SANDY CLAY

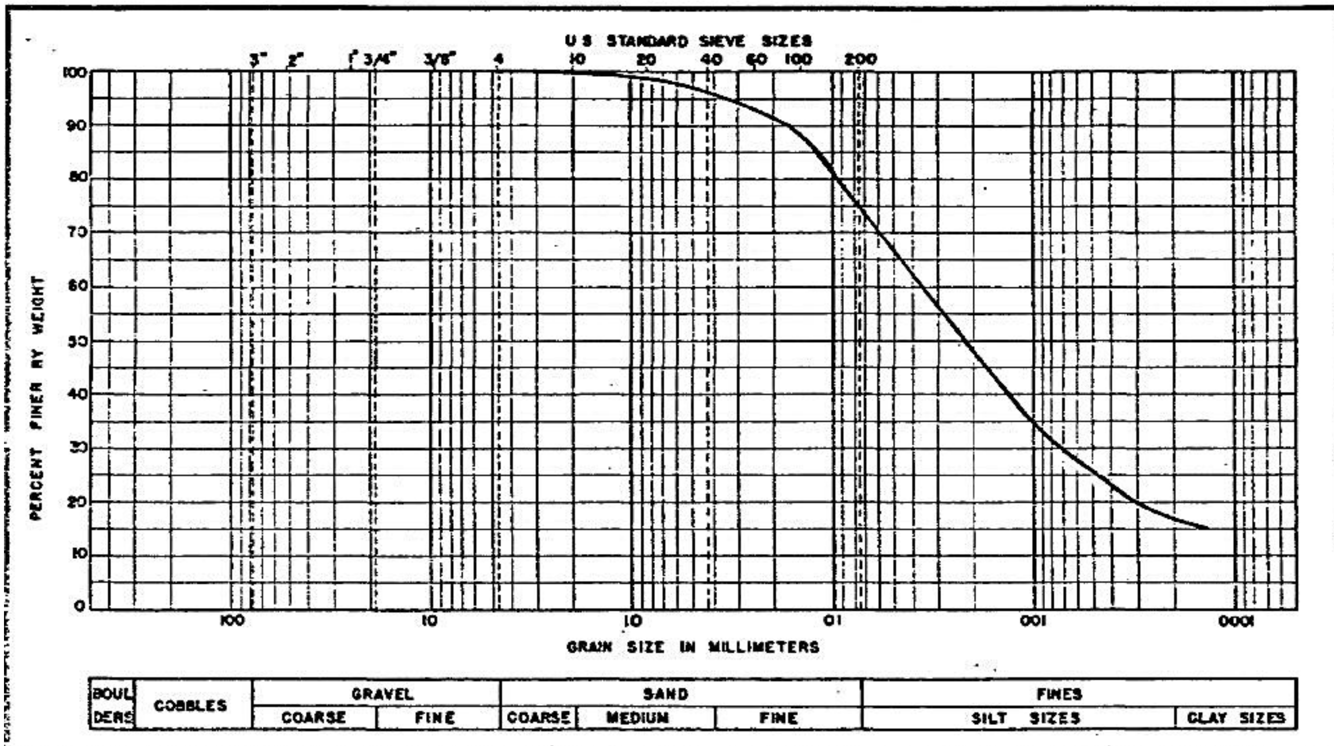
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-9

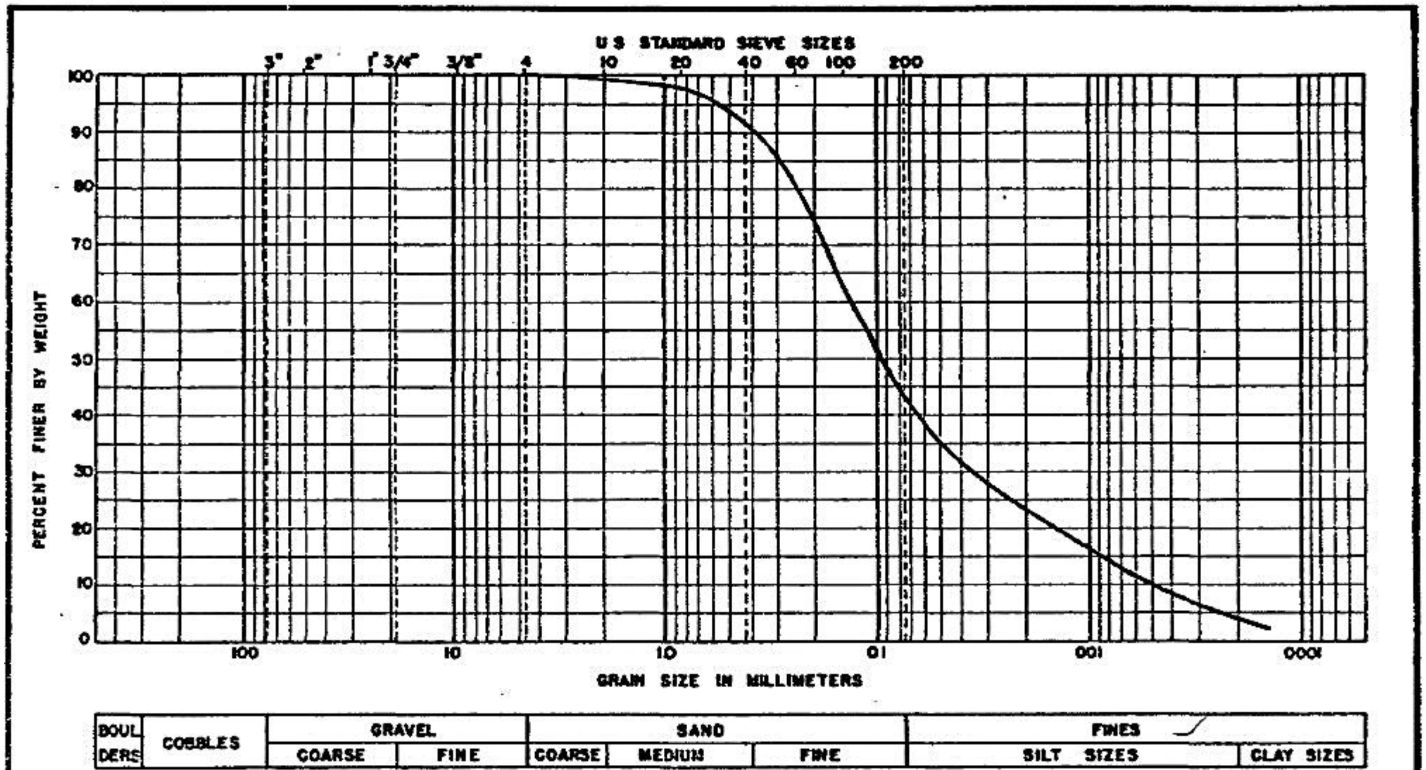
GRAIN SIZE DISTRIBUTION, BORING NO. BP-17, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
BP-17	5.0'-6.5' S-2					RED FINE SANDY CLAYEY SILT	JOB NO. RA-503 LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-10

GRAIN SIZE DISTRIBUTION, BORING NO. BP-24, S-1



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BP-24	3.0'-4.5' S-1					YELLOW AND BROWN VERY SILTY FINE TO MEDIUM SAND

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-11

GRAIN SIZE DISTRIBUTION, BORING NO. BP-27, S-1

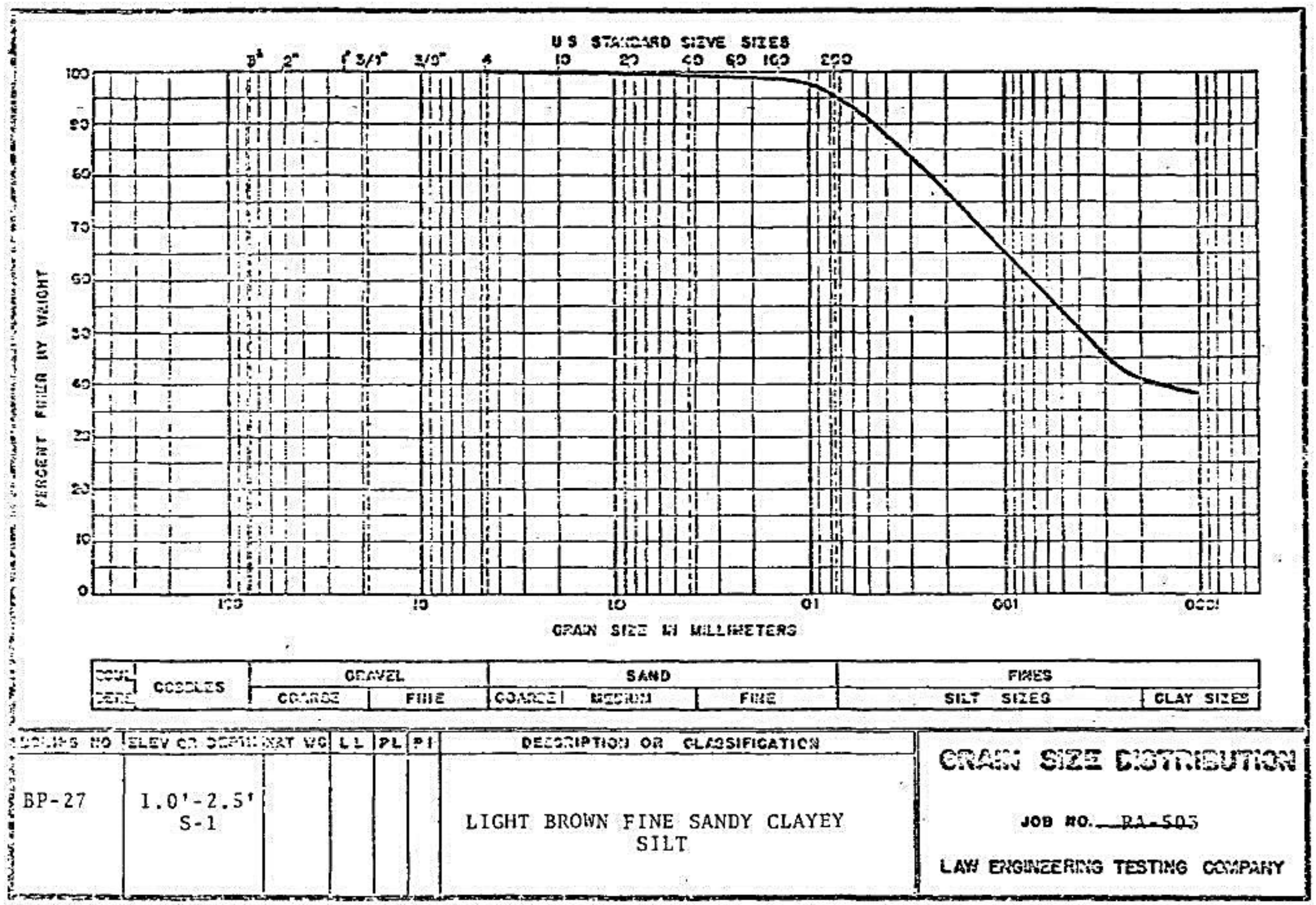
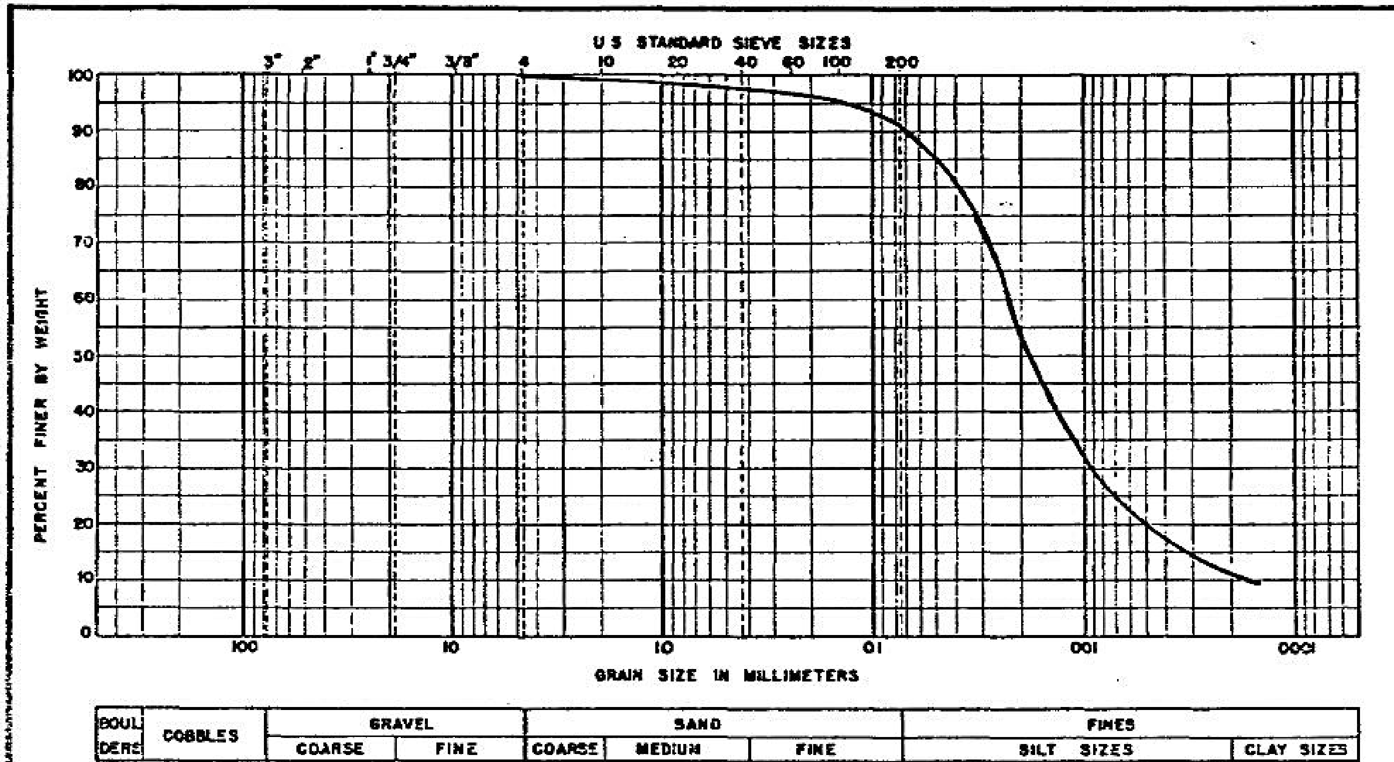


FIGURE 2.5.4-12

GRAIN SIZE DISTRIBUTION, BORING NO. BP-27, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BP-27	10.0'-10.5 S-2					BROWN SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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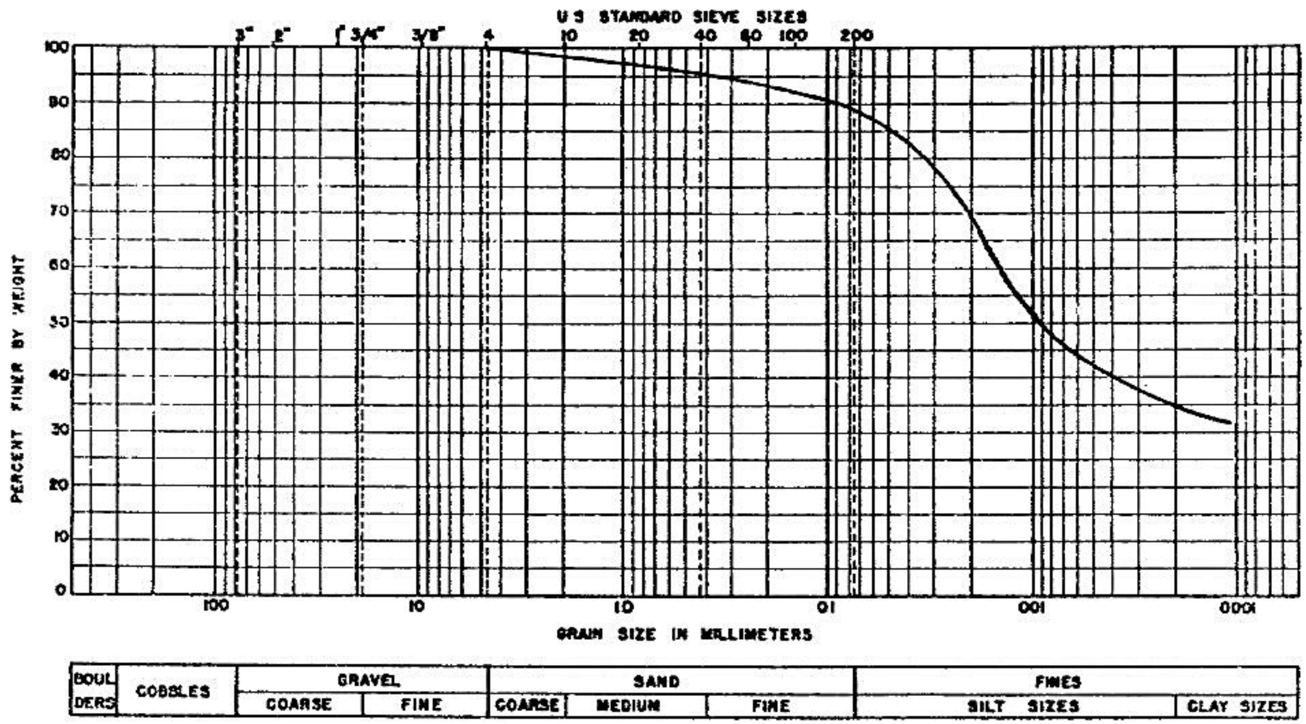
FIGURE 2.5.4-13

GRAIN SIZE DISTRIBUTION, BORING NO. BP-38, S-1



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BP-38	1.5'-3.0' S-1					YELLOW AND BROWN FINE SANDY SILTY CLAY

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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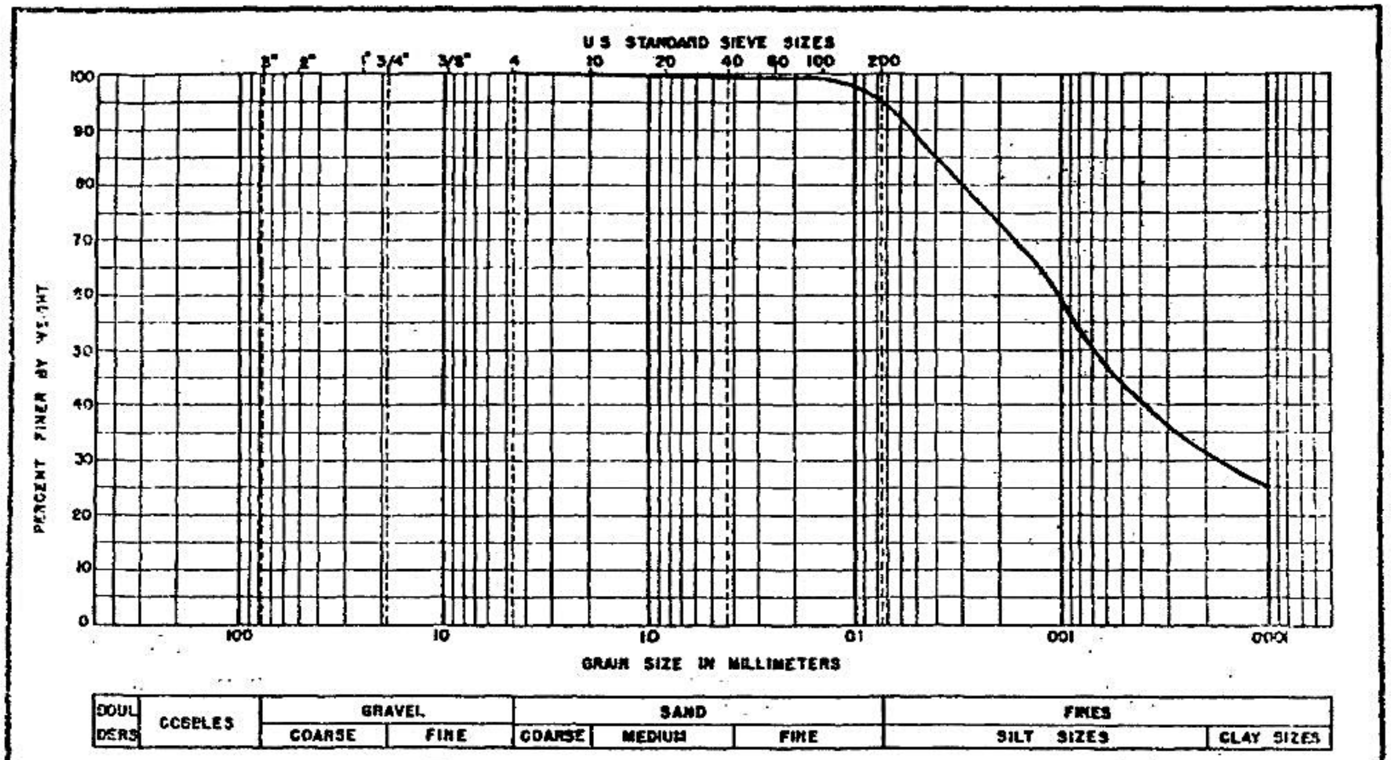
FIGURE 2.5.4-14

GRAIN SIZE DISTRIBUTION, BORING NO. BP-38, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BP-38	5.0'-6.5' S-2					GRAY FINE SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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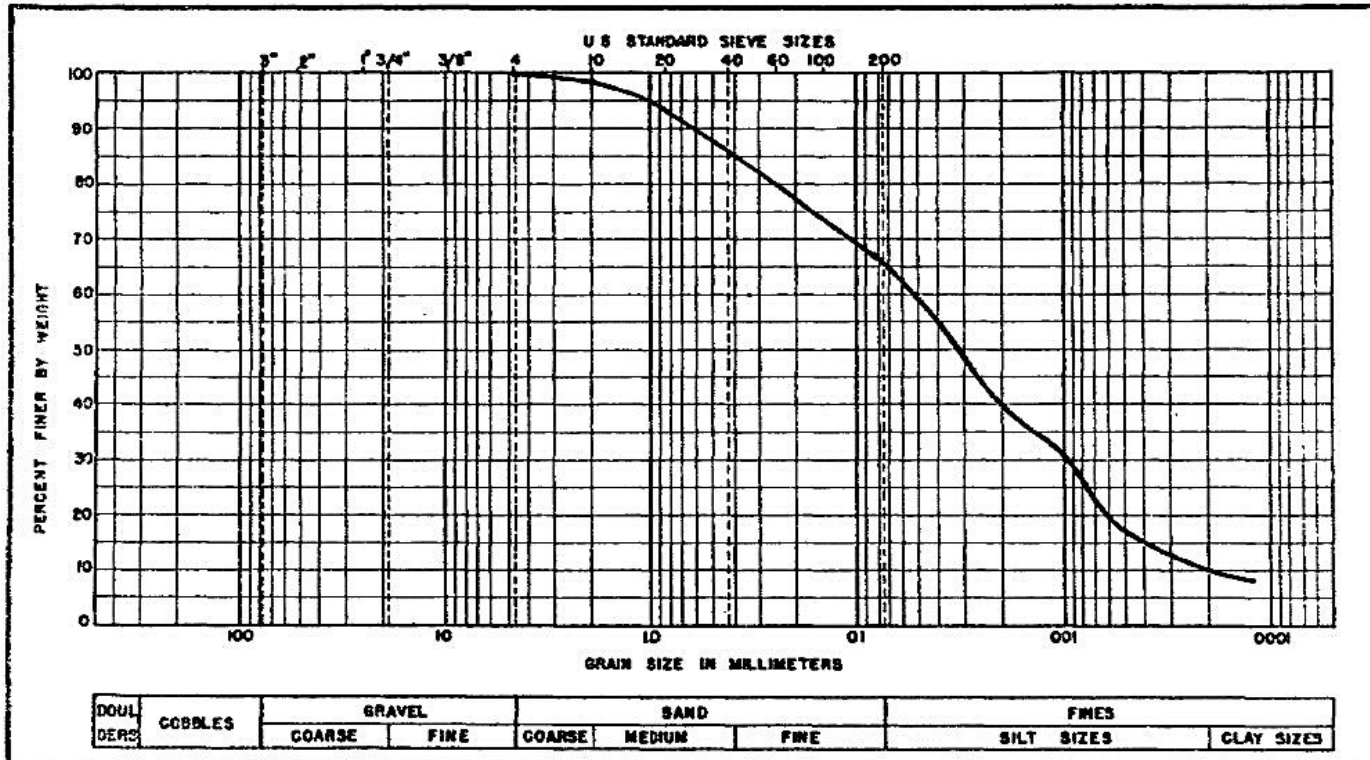
FIGURE 2.5.4-15

GRAIN SIZE DISTRIBUTION, BORING NO. BP-77, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BP-77	5.0'-6.5' S-2					RED CLAYEY FINE SANDY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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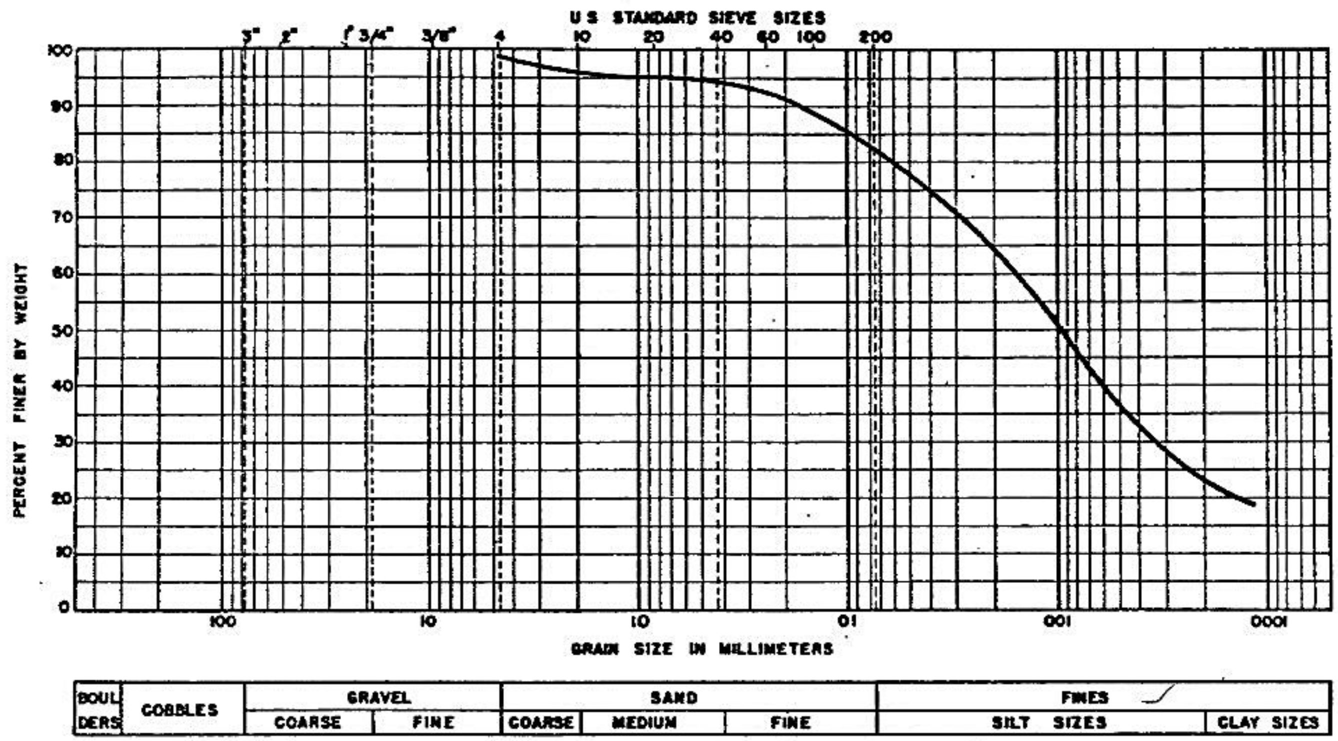
FIGURE 2.5.4-16

GRAIN SIZE DISTRIBUTION, BORING NO. BP-146, S-3



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BP-146	10.0'-14.0' S-3	10.9%				REDDISH BROWN FINE SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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FIGURE 2.5.4-17

GRAIN SIZE DISTRIBUTION, BORING NO. BP-184, S-3

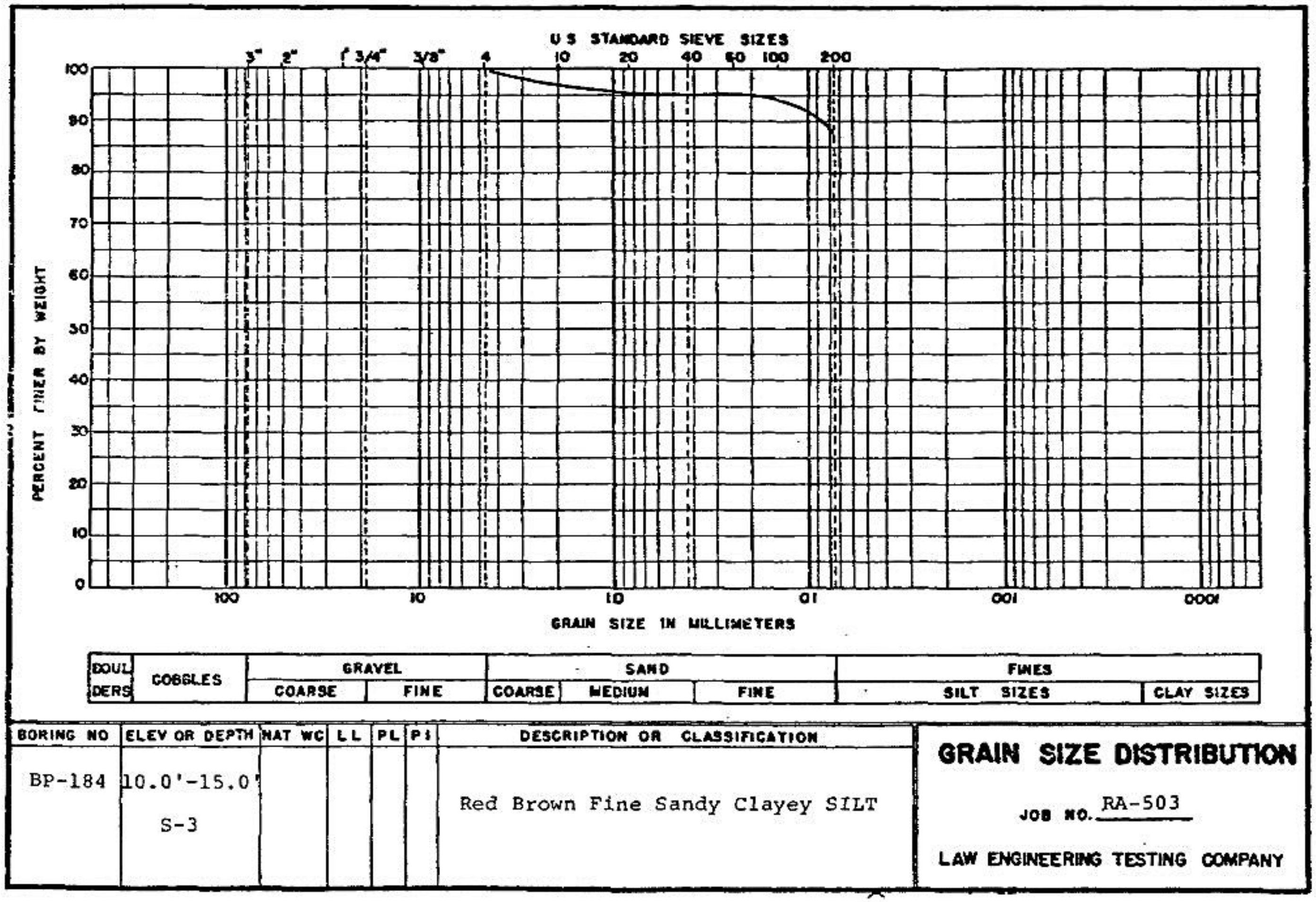
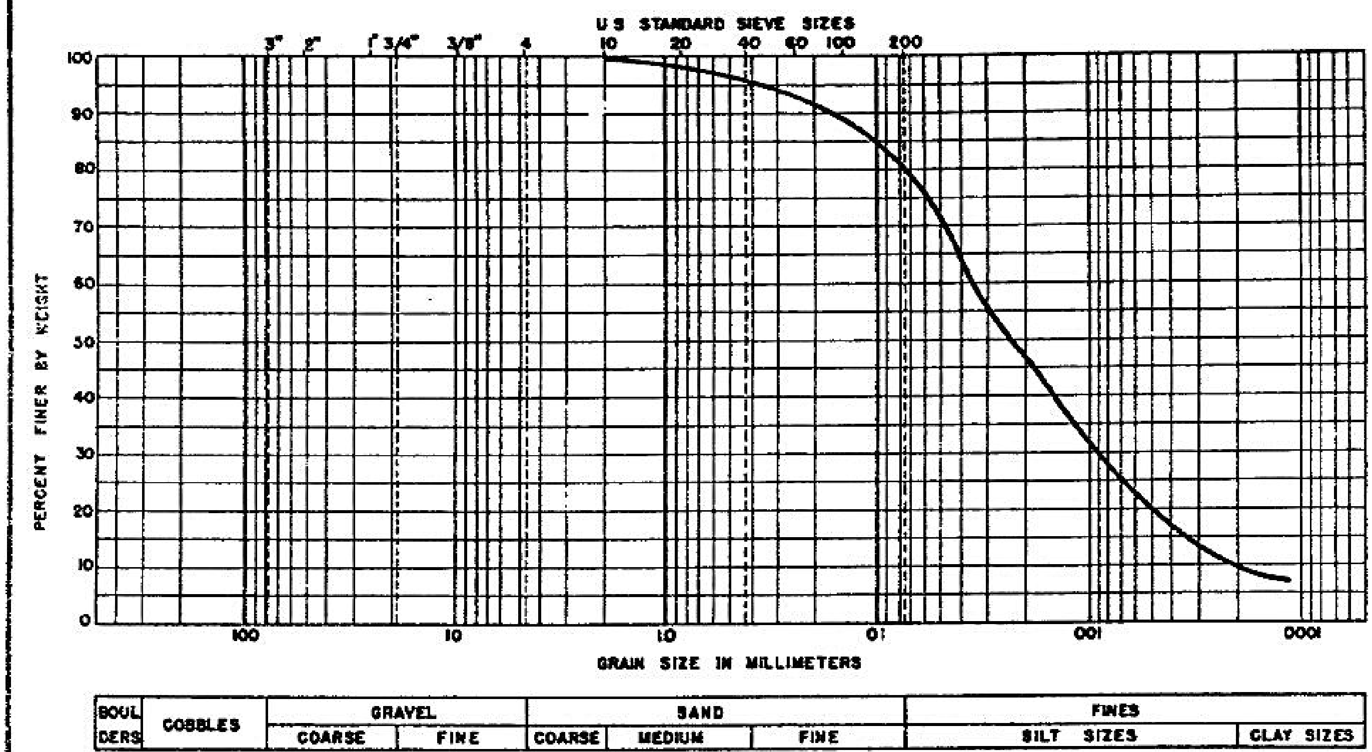


FIGURE 2.5.4-18

GRAIN SIZE DISTRIBUTION, BORING NO. BP-185, S-3



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BP-185	7.5'-12.5' S-3	5.9%	26.40%	21.34%	5.06	REDDISH BROWN FINE SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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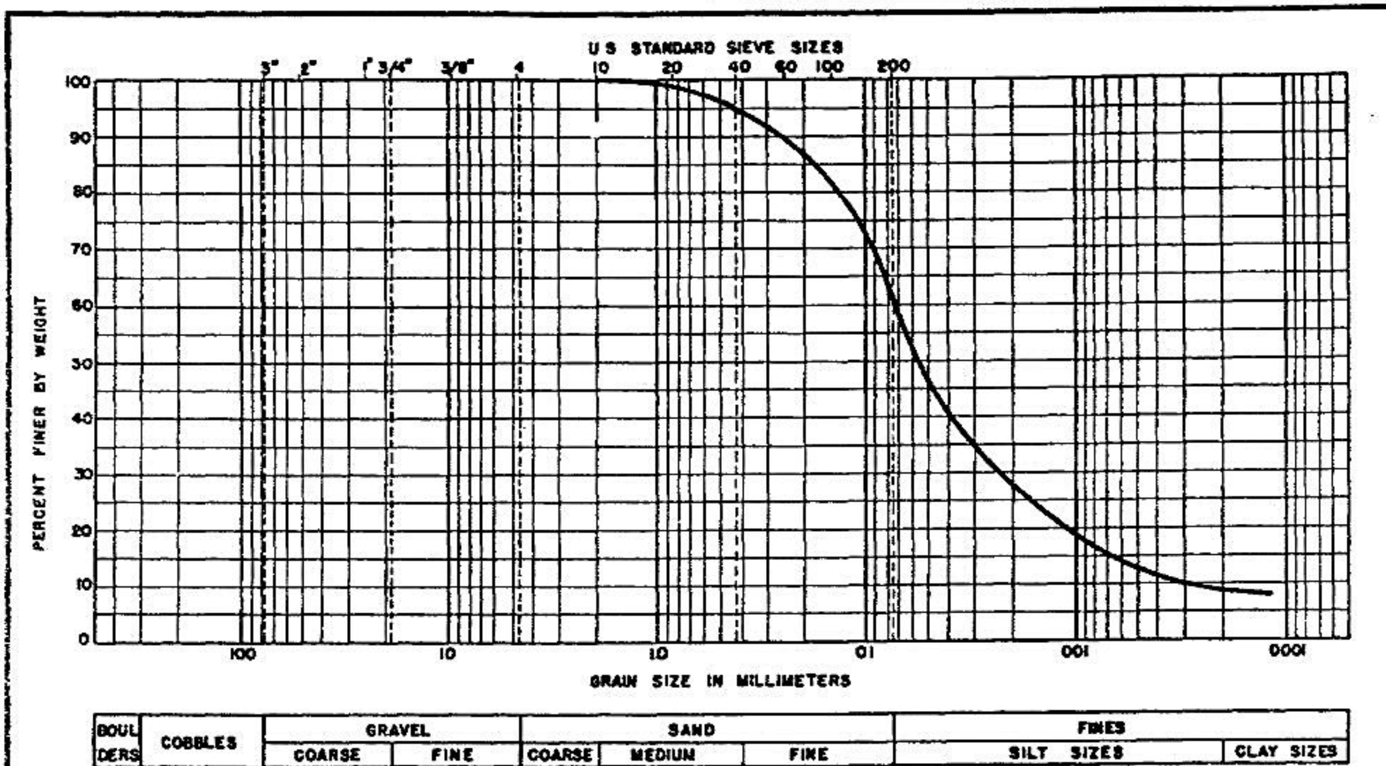
FIGURE 2.5.4-19

GRAIN SIZE DISTRIBUTION, BORING NO. BP-185, S-4



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BP-185	12.5'-17.5' S-4	4.8%	26.7%	20.0%	6.7	REDDISH BROWN FINE SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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FIGURE 2.5.4-20

GRAIN SIZE DISTRIBUTION, BORING NO. BP-189, S-3

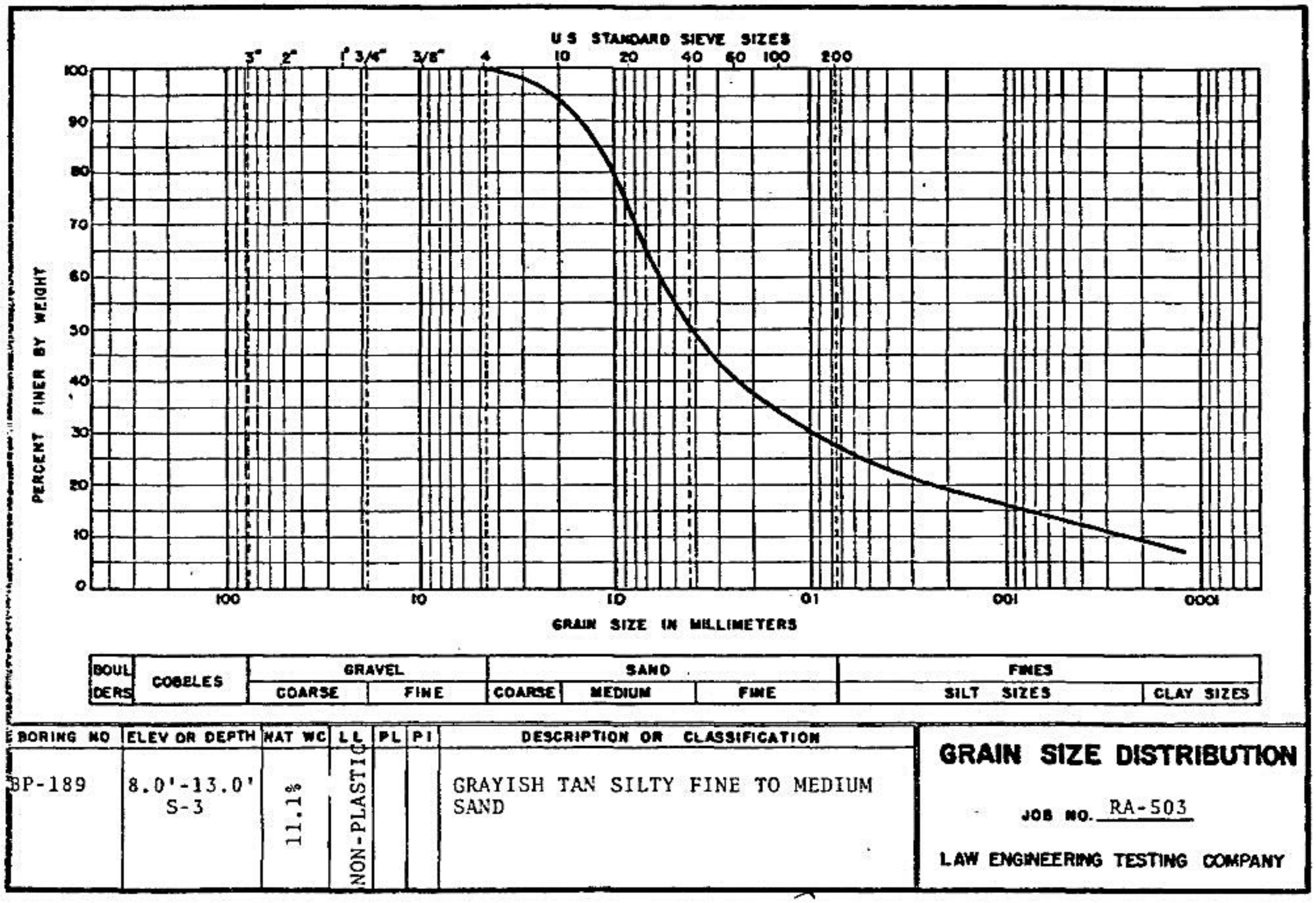
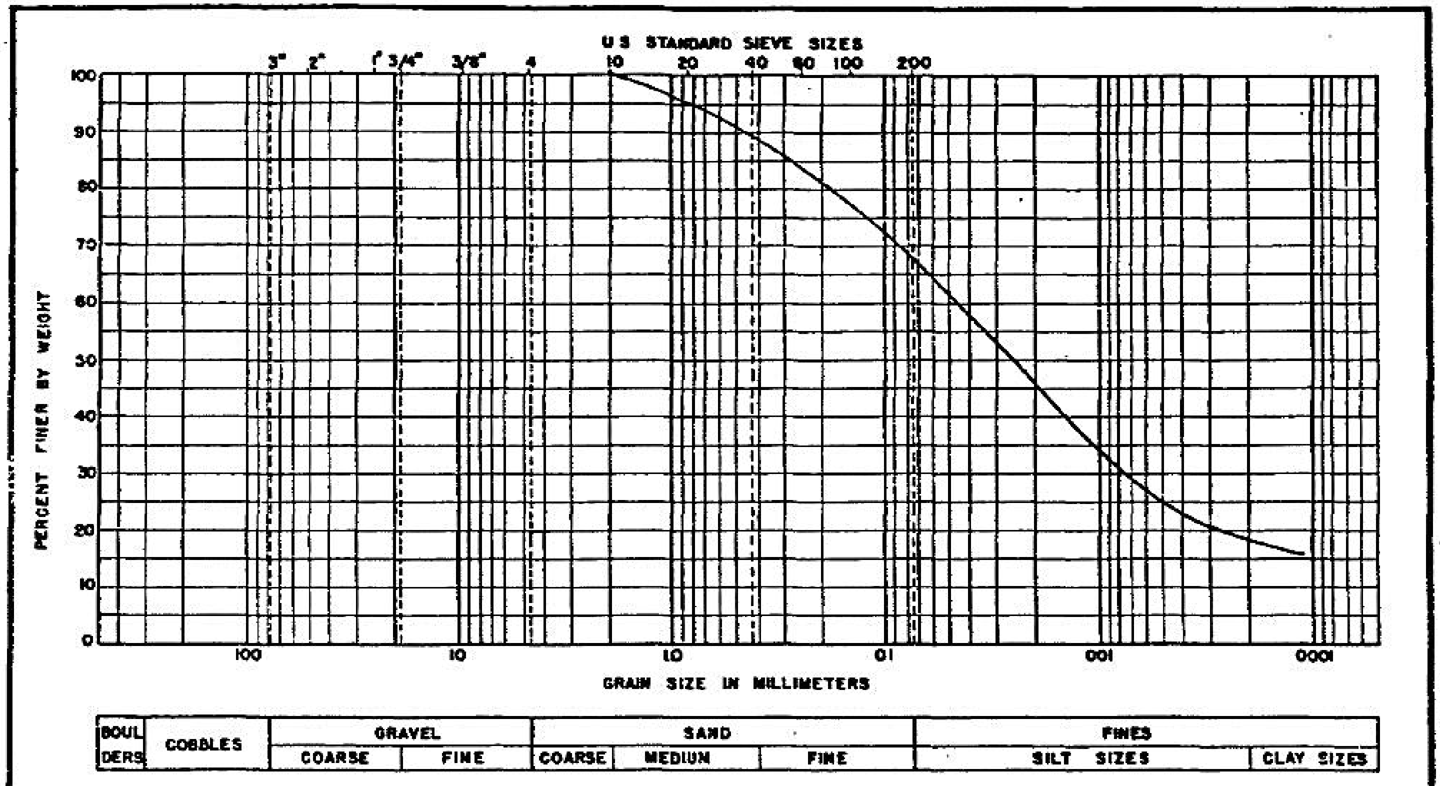


FIGURE 2.5.4-21

GRAIN SIZE DISTRIBUTION, BORING NO. BP-215, S-1



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	PI	DESCRIPTION OR CLASSIFICATION
BP-215 S-1	0.5'-8.0'					Tan Clayey Very Sandy SILT

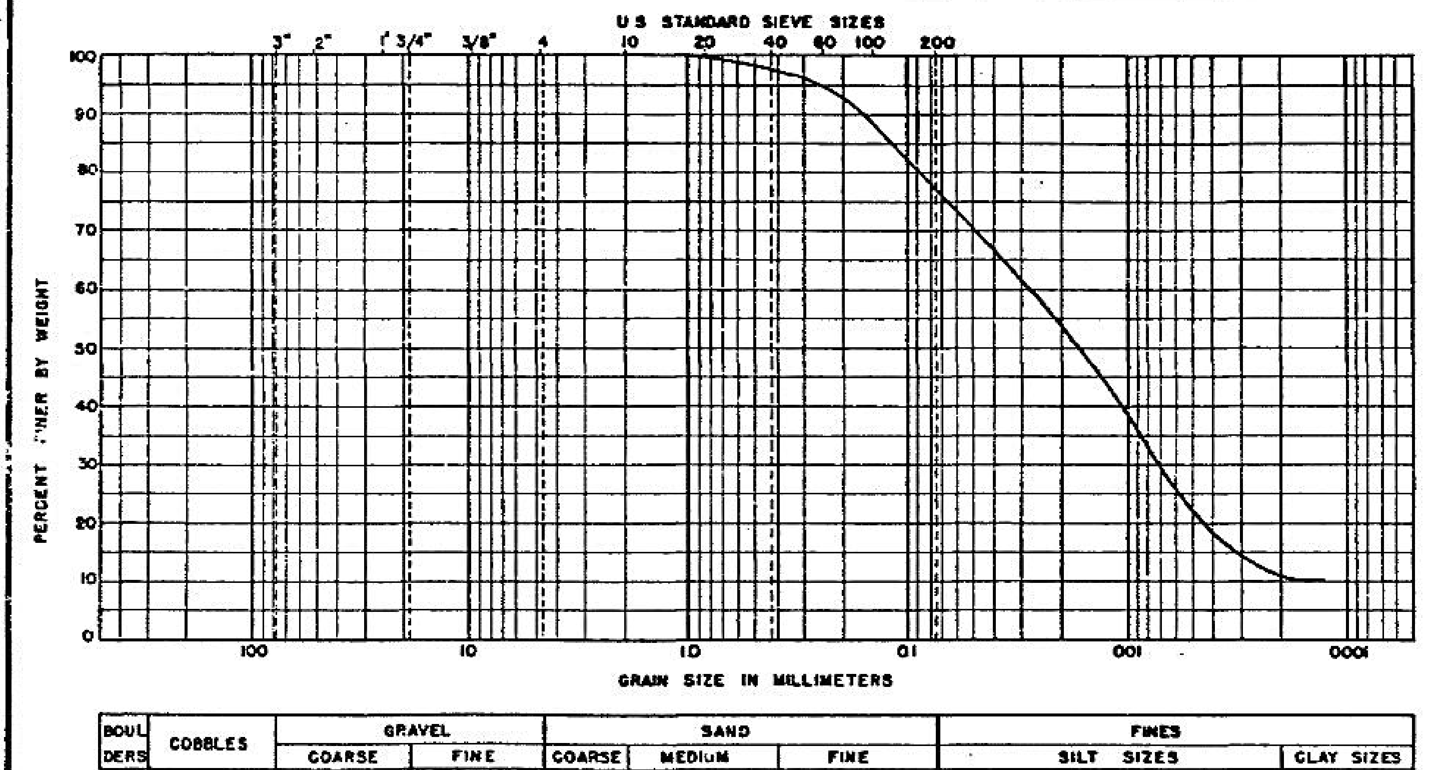
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-22

GRAIN SIZE DISTRIBUTION, BORING NO. BP-215, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BP-215 S-2	8.0'-11.0'					Reddish Brown Clayey Fine to Medium Sandy SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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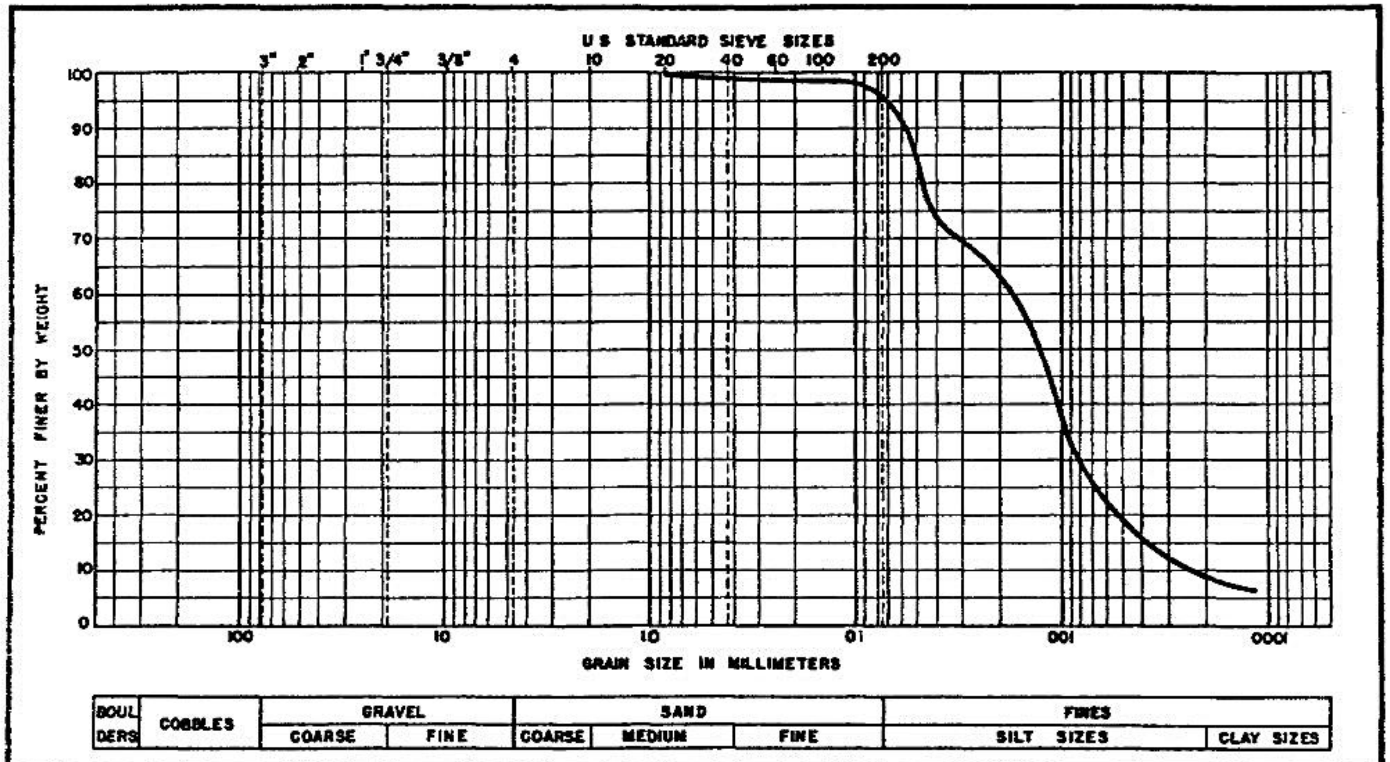
FIGURE 2.5.4-23

GRAIN SIZE DISTRIBUTION, BORING NO. BP-235, S-3



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BP-235	10.0'-15.0' S-3	15.0%	35.5%	27.1%	8.4%	REDDISH BROWN SLIGHTLY SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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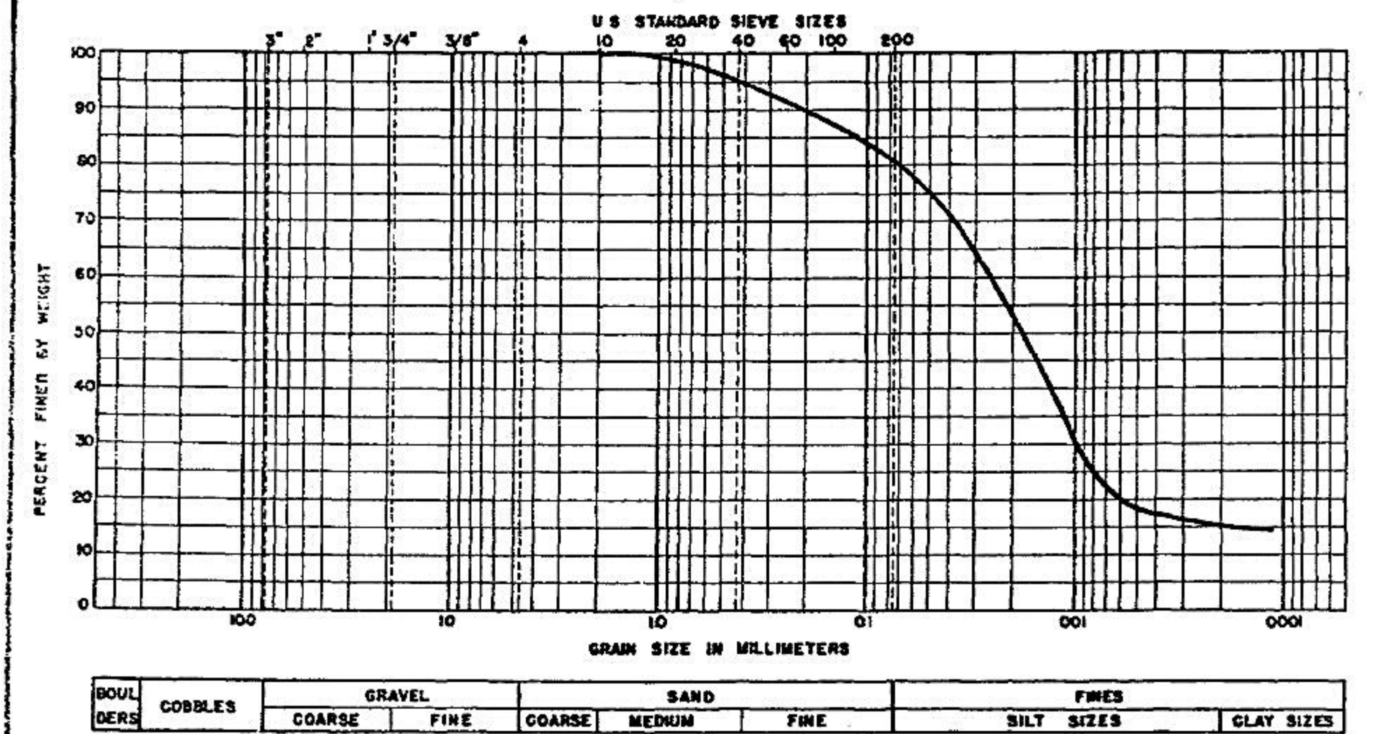
FIGURE 2.5.4-24

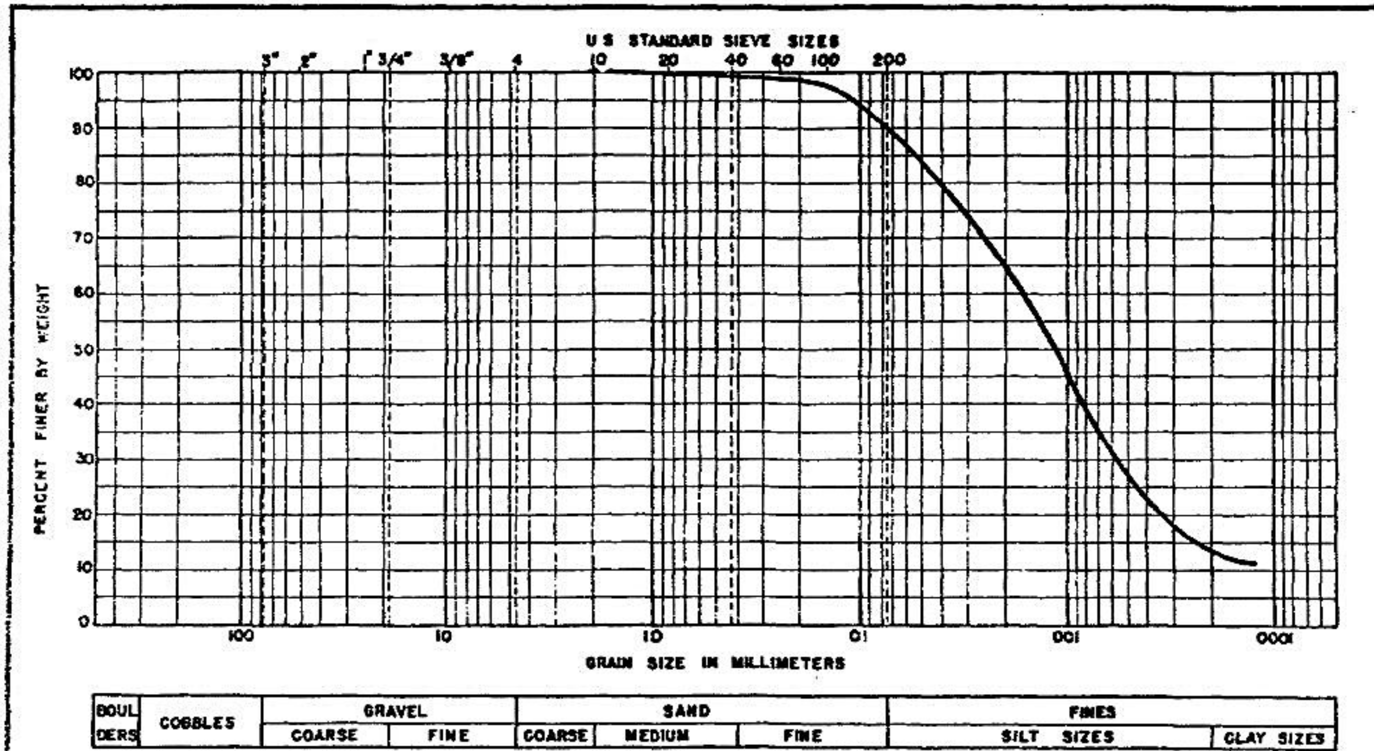
GRAIN SIZE DISTRIBUTION, BORING NO. BP-237, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
BP-237	5.0'-10.0' S-2	27.6%	24.4%	18.1%	6.3%	TAN SLIGHTLY FINE SANDY VERY SILTY CLAY	JOB NO. RA-503 LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-25

GRAIN SIZE DISTRIBUTION, BORING NO. BP-237, S-3



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BP-237	10.0'-15.0' S-3	26.8%	33.7%	23.2%	10.5	TAN SLIGHTLY FINE SANDY CLAYEY SILT

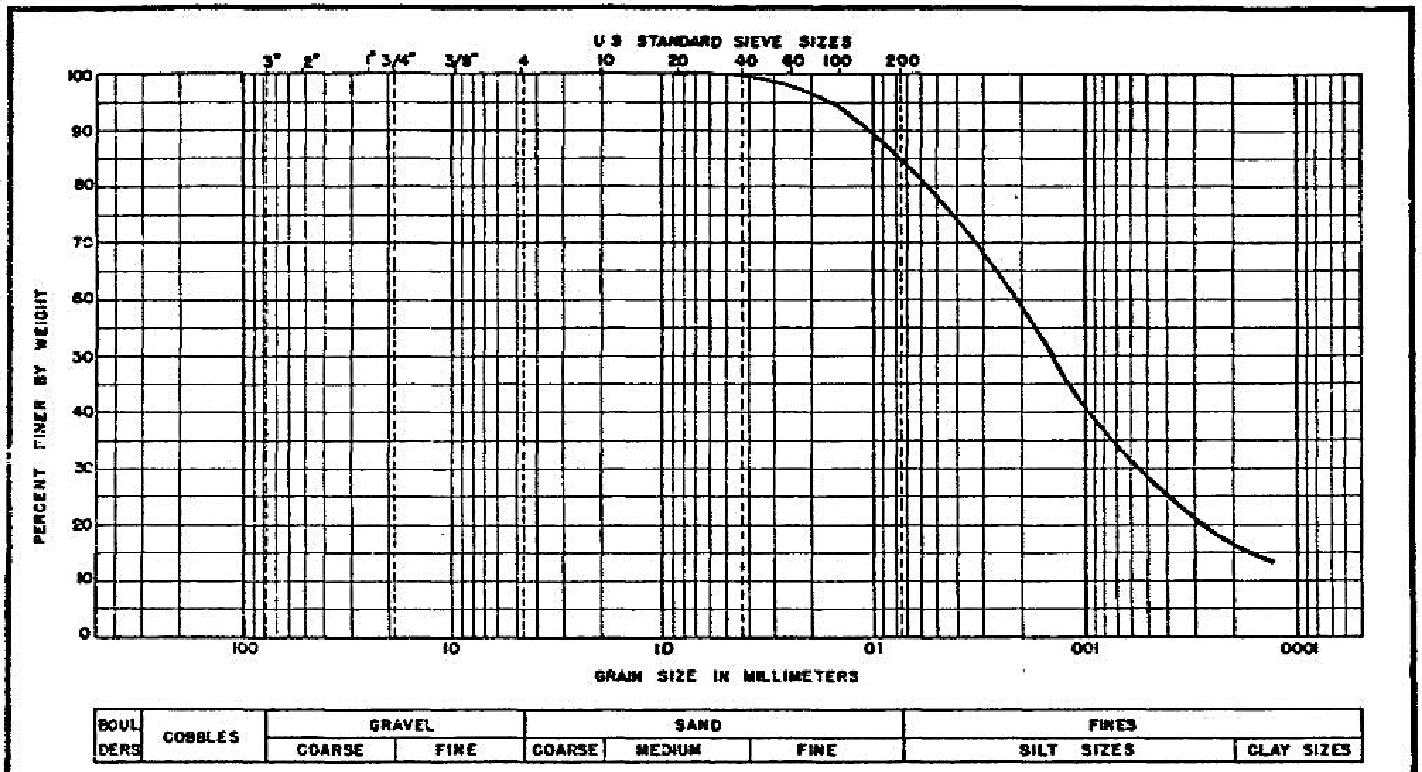
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-26

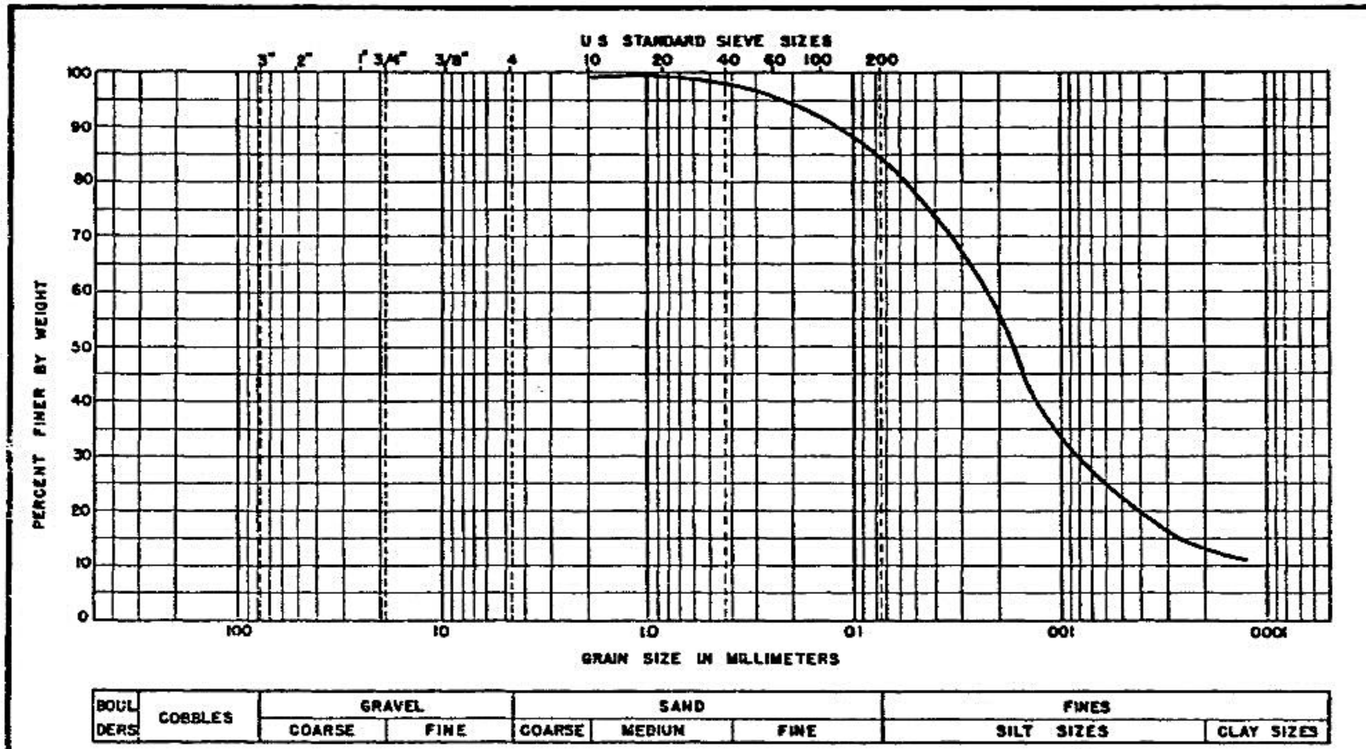
GRAIN SIZE DISTRIBUTION, BORING NO. BC-153, S-3



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
BC-153	8.0'-13.0' S-3					RED BROWN FINE SANDY CLAYEY SILT	JOB NO. RA-503 LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-27

GRAIN SIZE DISTRIBUTION, BORING NO. BC-154, S-3



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BC-154	10.0'-15.0' S-3					RED BROWN FINE SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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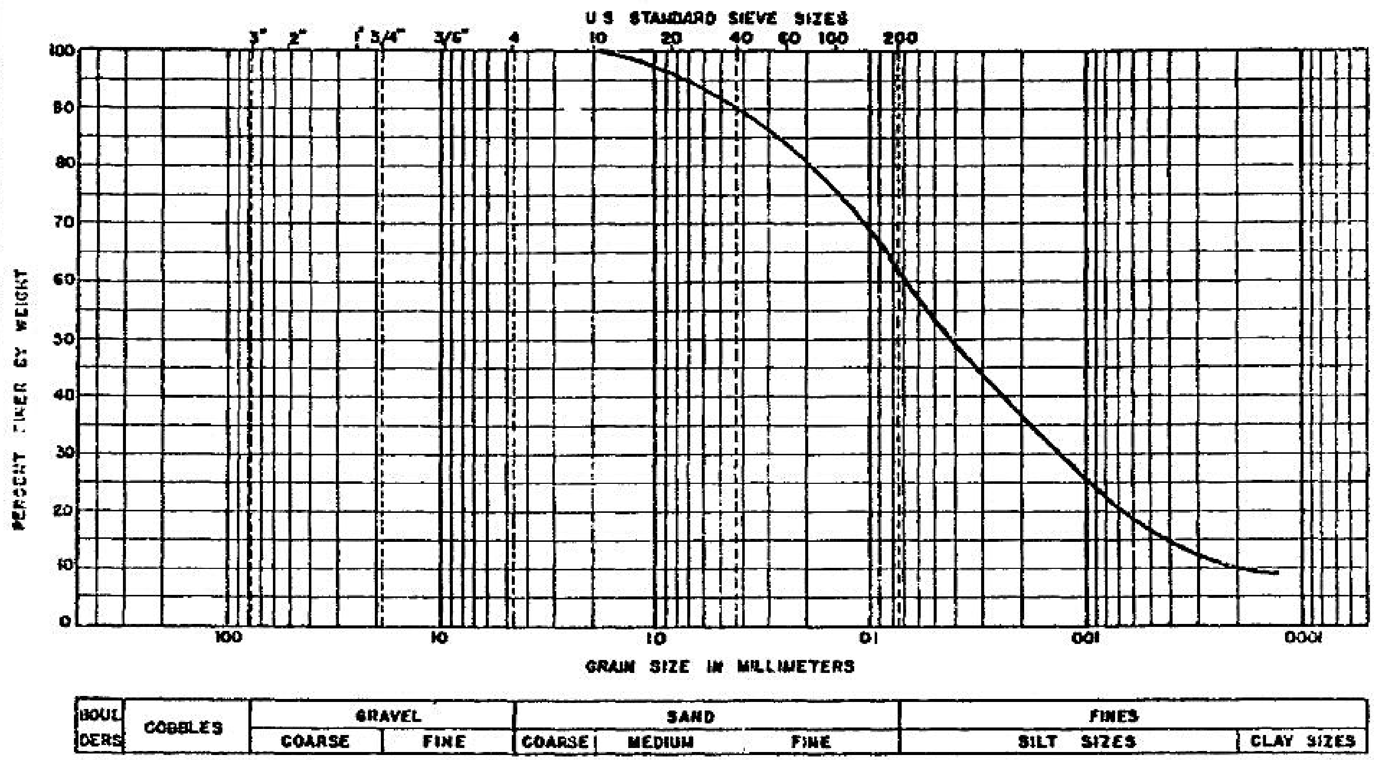
FIGURE 2.5.4-28

GRAIN SIZE DISTRIBUTION, BORING NO. BC-155, S-3



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BC-155	9.0' - 14.0' S-3					RED VERY FINE TO MEDIUM SANDY SLIGHTLY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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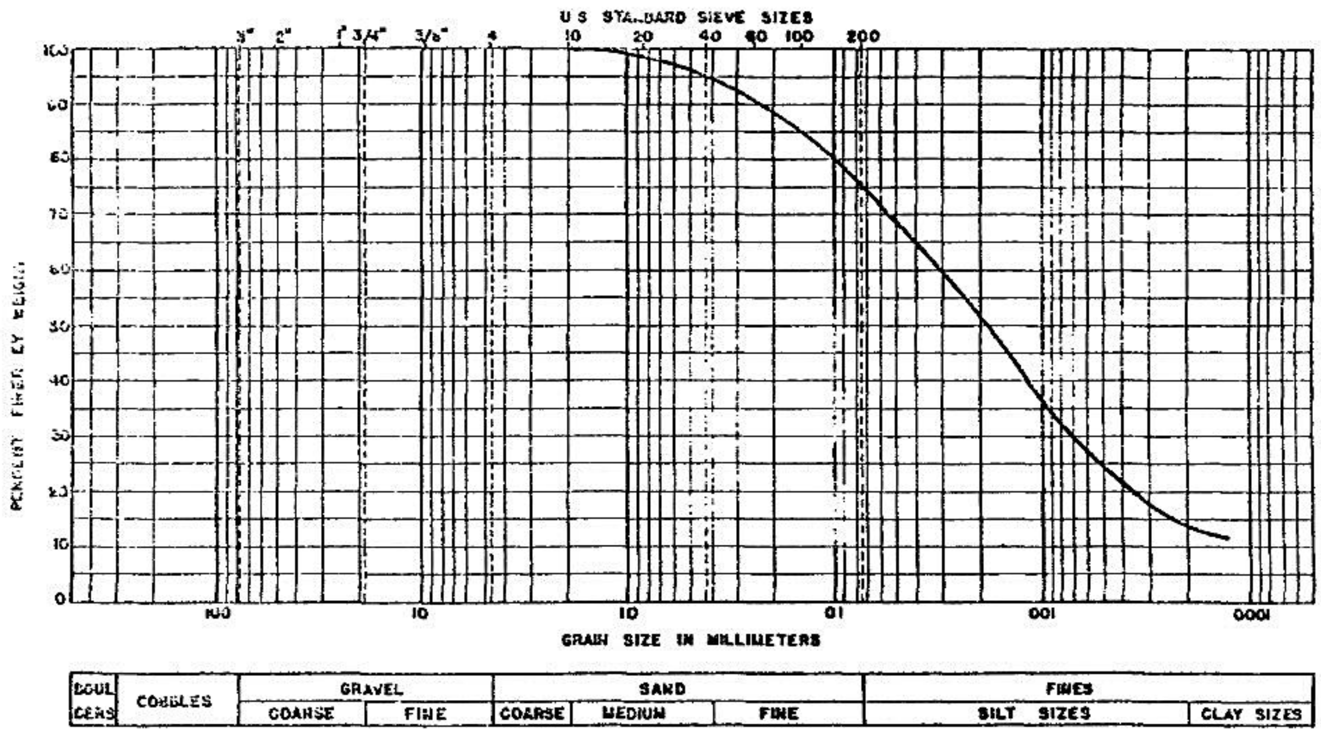
FIGURE 2.5.4-29

GRAIN SIZE DISTRIBUTION, BORING NO. BC-156, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BC-156	4.0' - 9.0' S-2					RED BROWN FINE SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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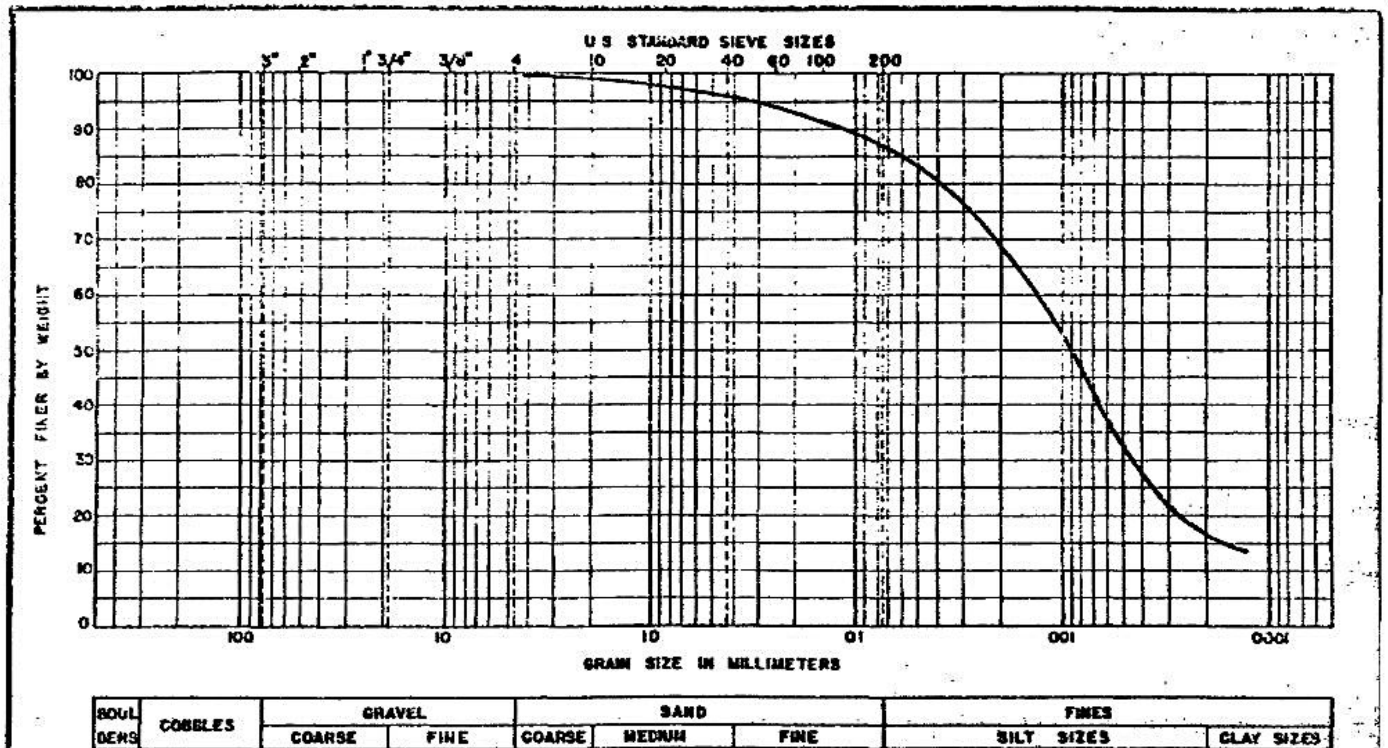
FIGURE 2.5.4-30

GRAIN SIZE DISTRIBUTION, BORING NO. BC-157, S-2



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION	GRAIN SIZE DISTRIBUTION
BC-157	5.0' - 10.0' S-2					RED BROWN SLIGHTLY FINE TO MEDIUM SANDY CLAYEY SILT	JOB NO. RA-503 LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-31

GRAIN SIZE DISTRIBUTION, BORING NO. BC-157, S-3

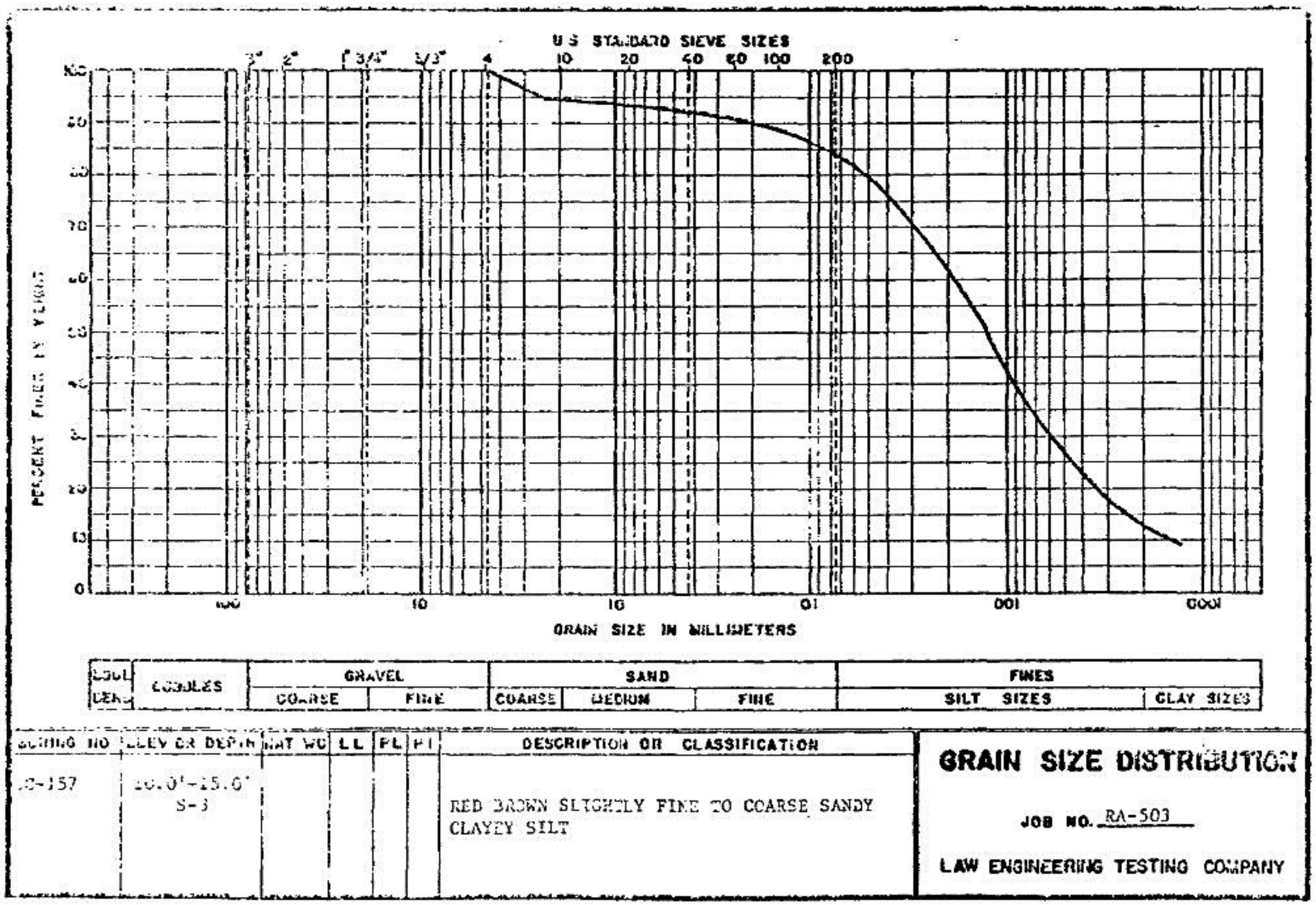


FIGURE 2.5.4-32

GRAIN SIZE DISTRIBUTION, BORING NO. BC-158, S-2

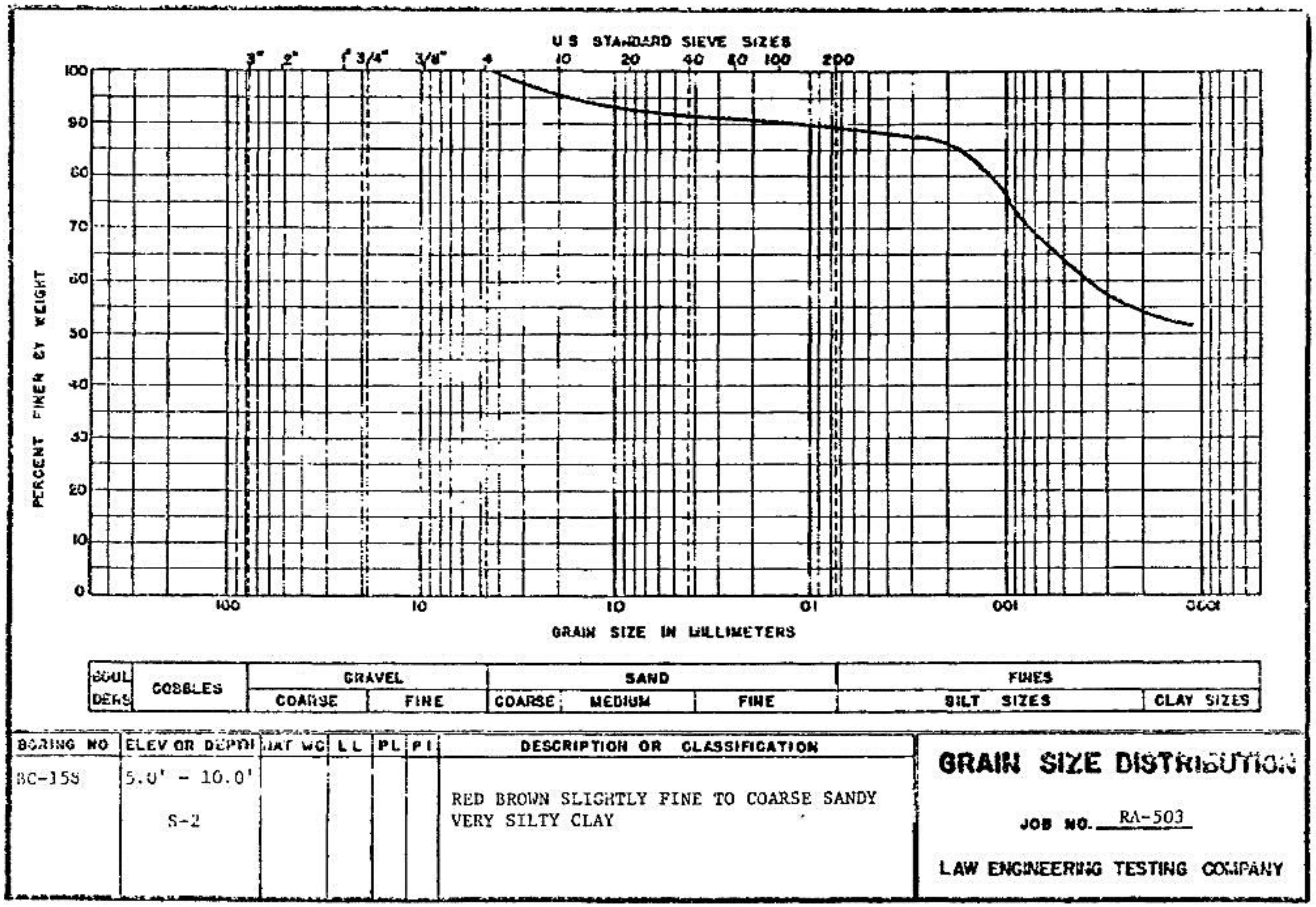
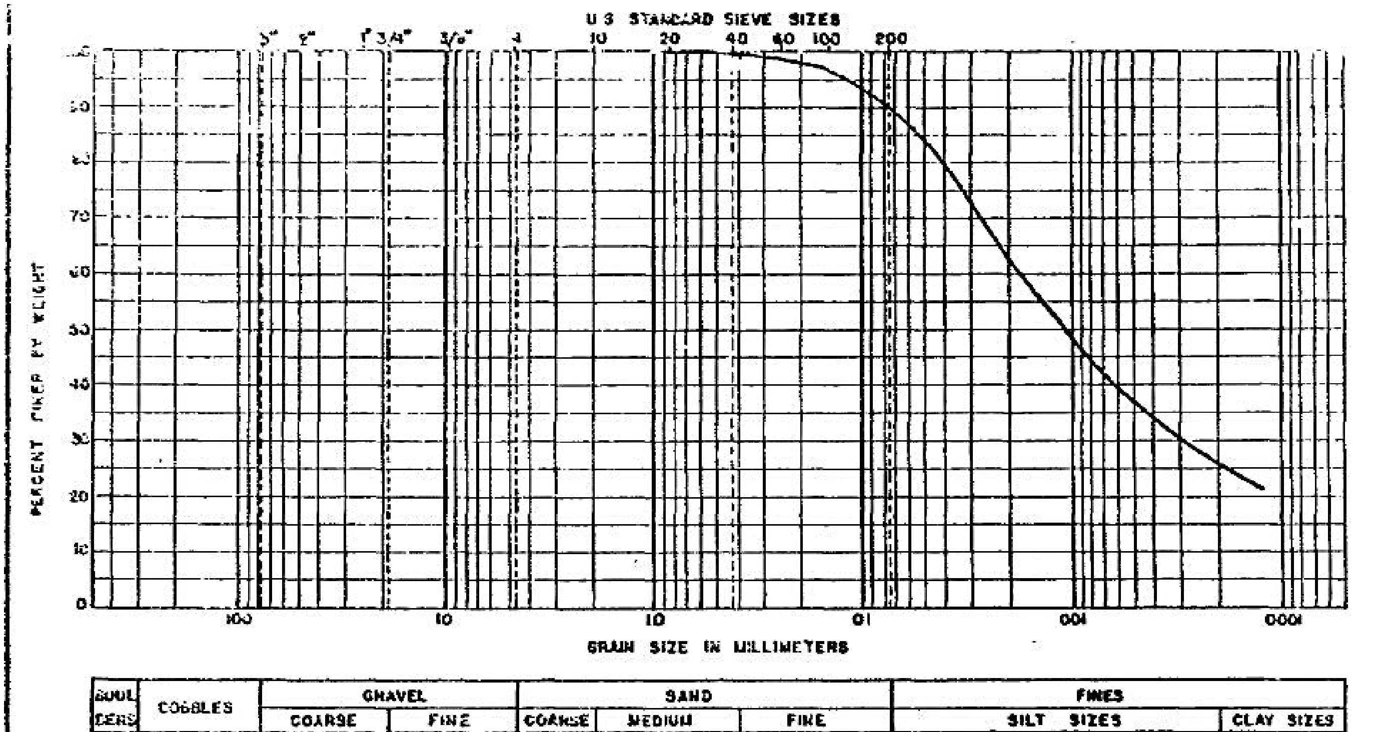


FIGURE 2.5.4-33

GRAIN SIZE DISTRIBUTION, BORING NO. BC-158, S-3



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BC-158	10.0'-15.0' S-3					RED BROWN FINE SANDY VERY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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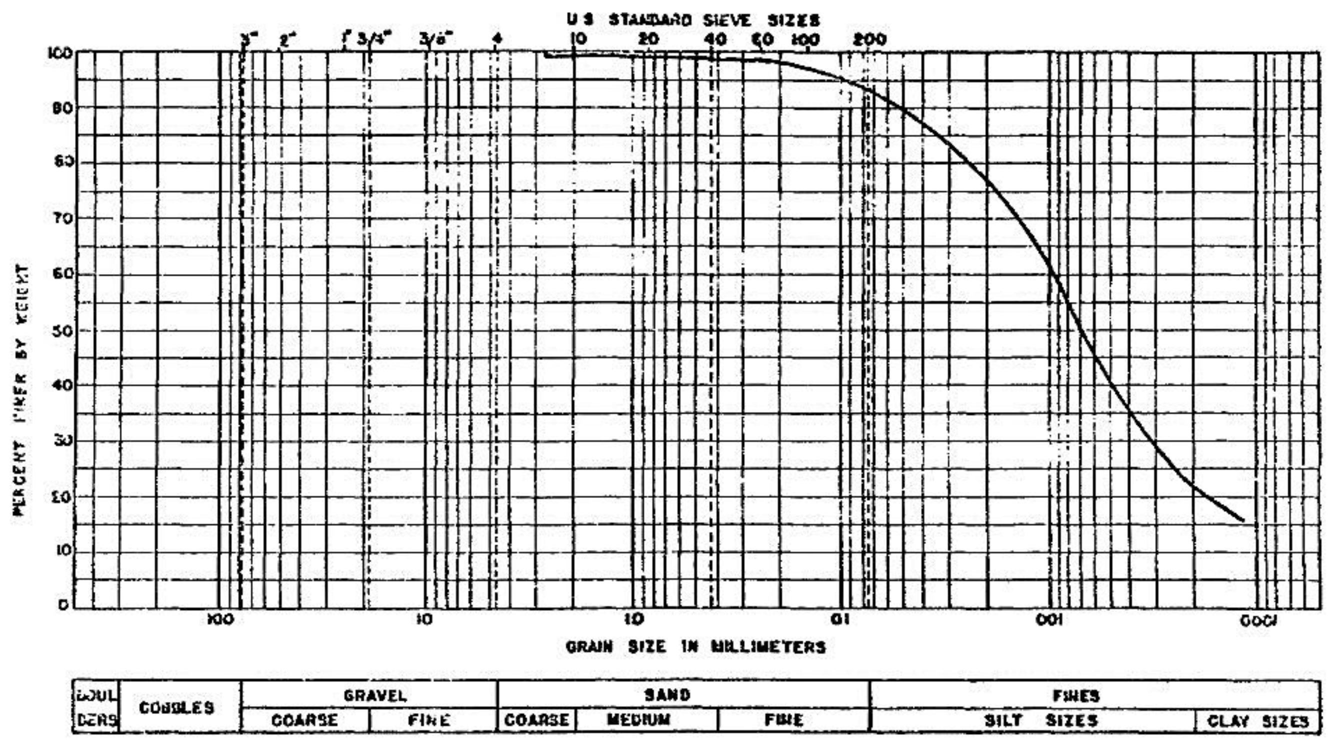
FIGURE 2.5.4-34

GRAIN SIZE DISTRIBUTION, BORING NO. BC-159, S-2



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BC-159	5.0' - 10.0' S-2					BROWN RED SLIGHTLY FINE SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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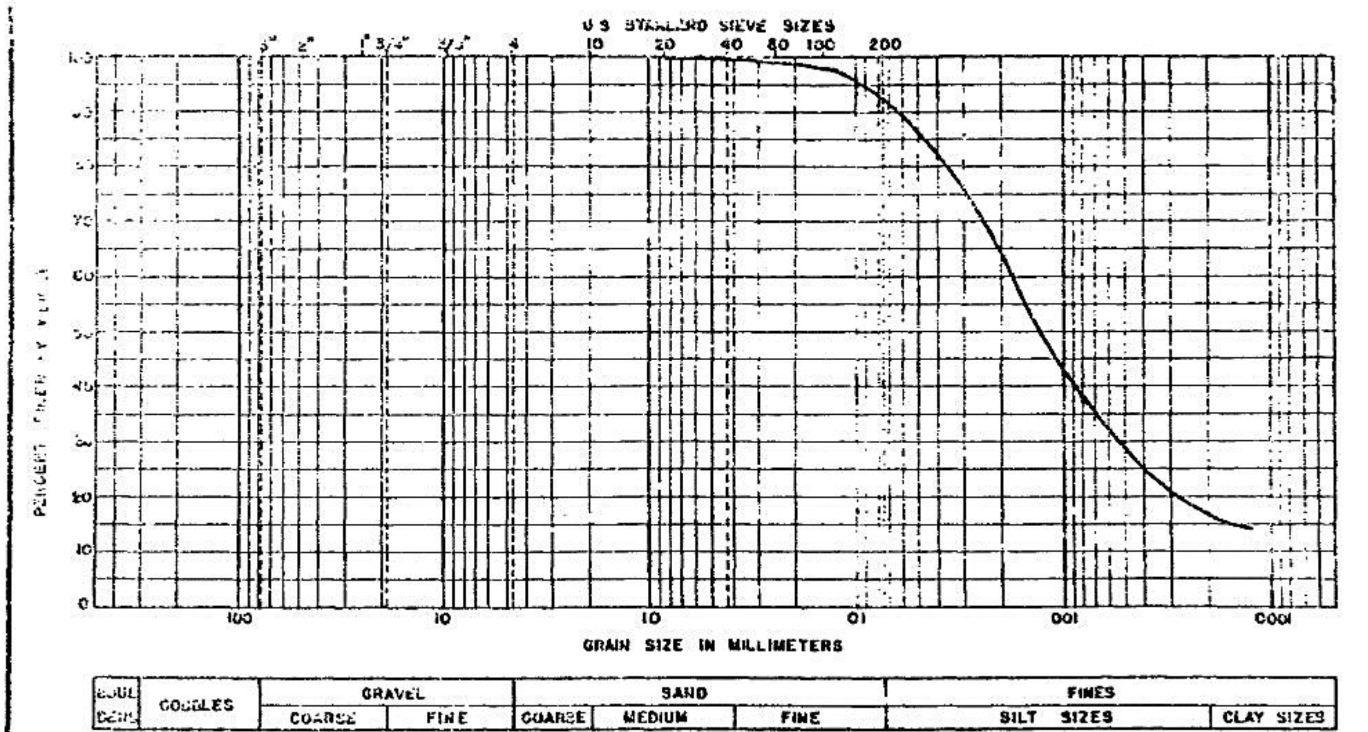
FIGURE 2.5.4-35

GRAIN SIZE DISTRIBUTION, BORING NO. BC-160, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BC-160	5.0' - 10.0' S-2					LIGHT BROWN VERY SLIGHTLY FINE SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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FIGURE 2.5.4-36

GRAIN SIZE DISTRIBUTION, BORING NO. BC-160, S-3

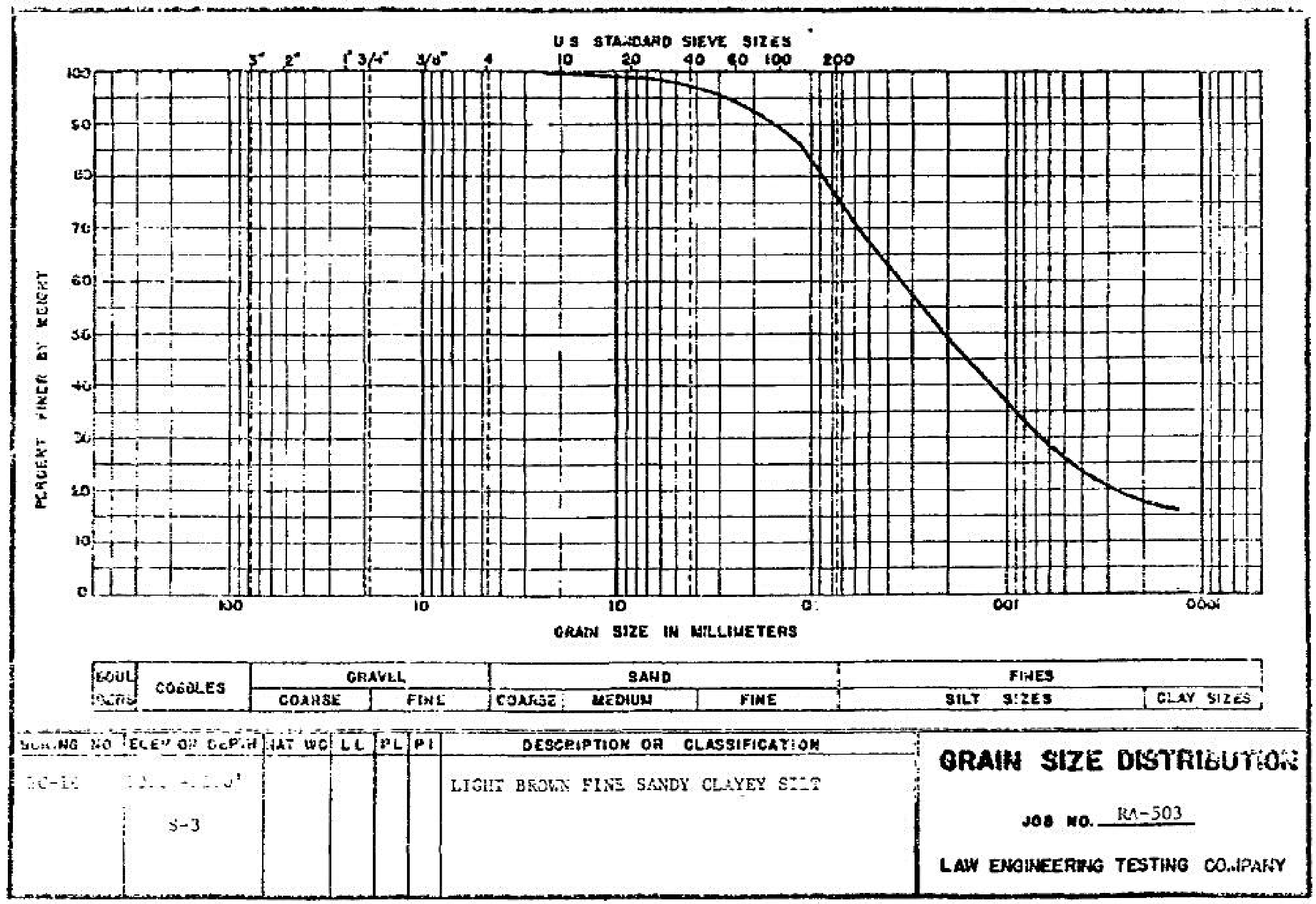
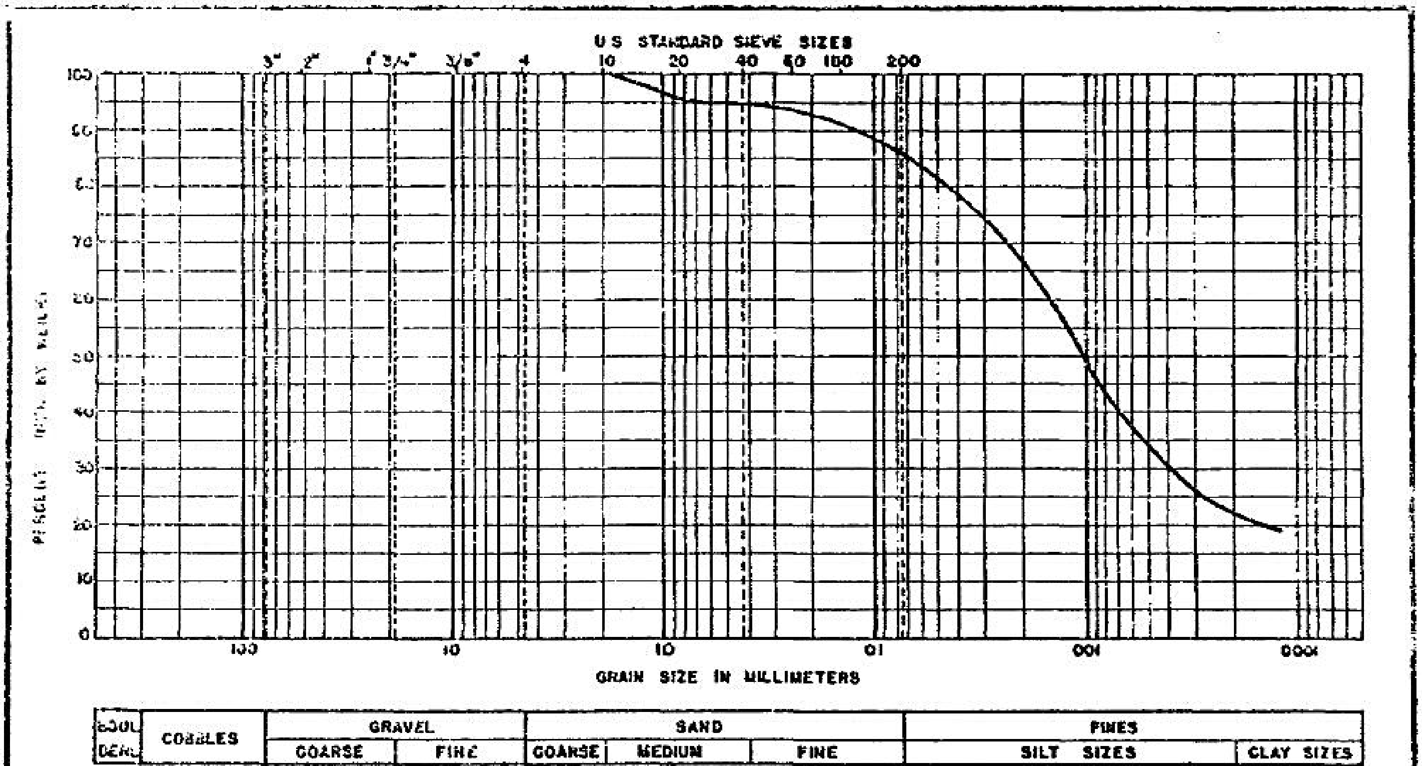


FIGURE 2.5.4-37

GRAIN SIZE DISTRIBUTION, BORING NO. BC-161, S-2



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BC-161	5.0' - 10.0' S-2					BROWN RED SLIGHTLY FINE TO MEDIUM SANDY CLAYEY SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-38

GRAIN SIZE DISTRIBUTION, BORING NO. BC-161, S-3

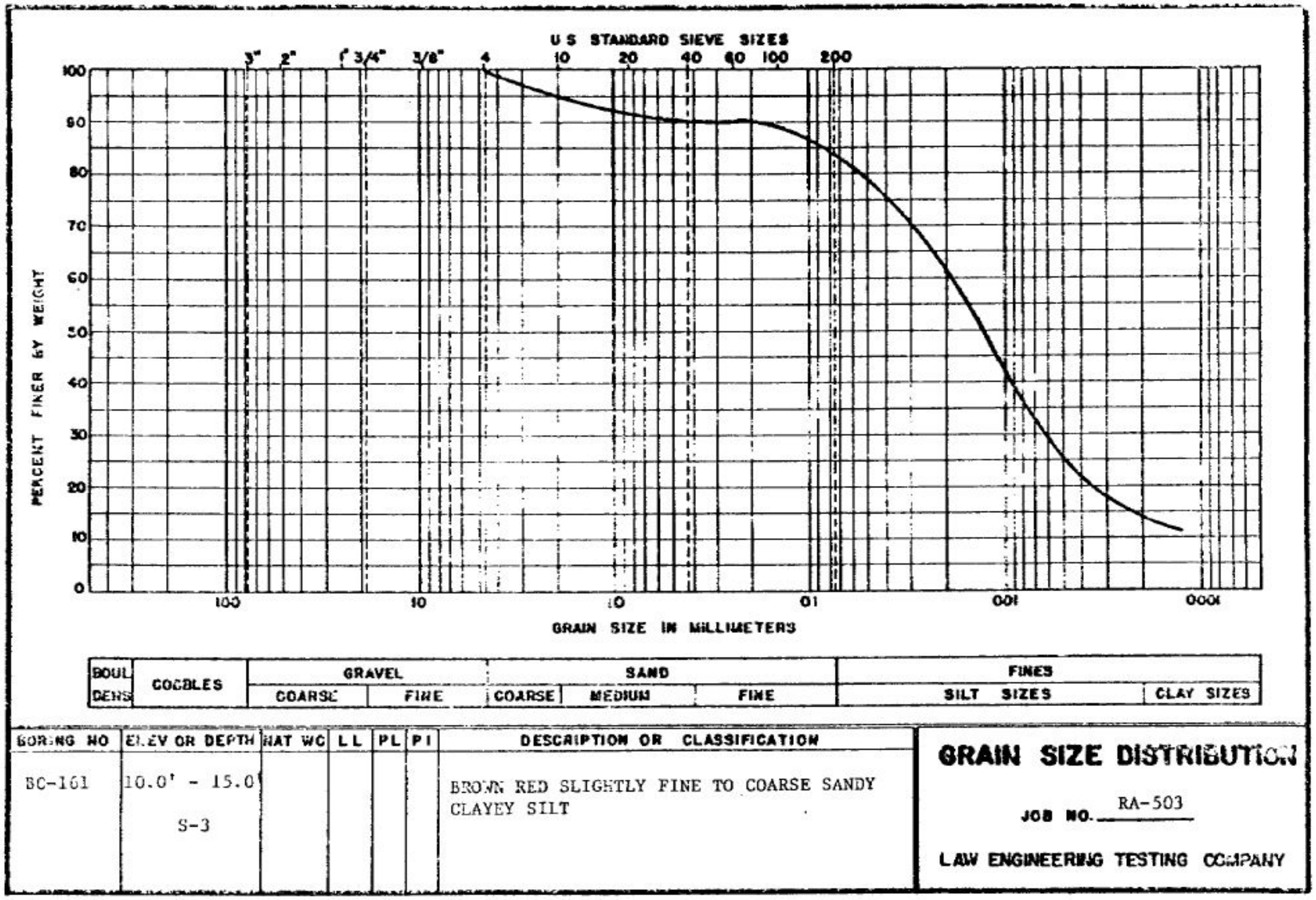


FIGURE 2.5.4-39

GRAIN SIZE DISTRIBUTION, BORING NO. BC-162, S-2

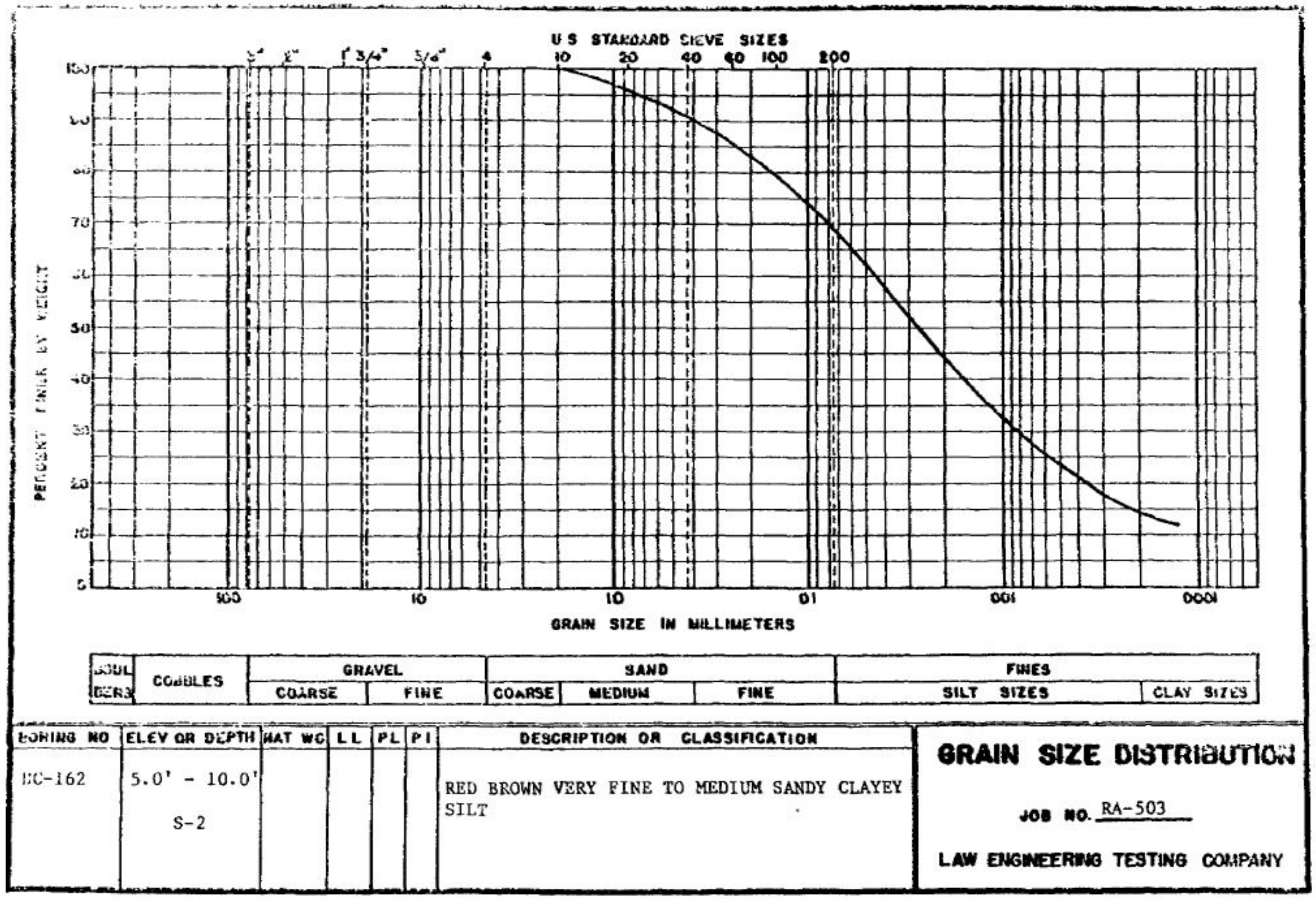
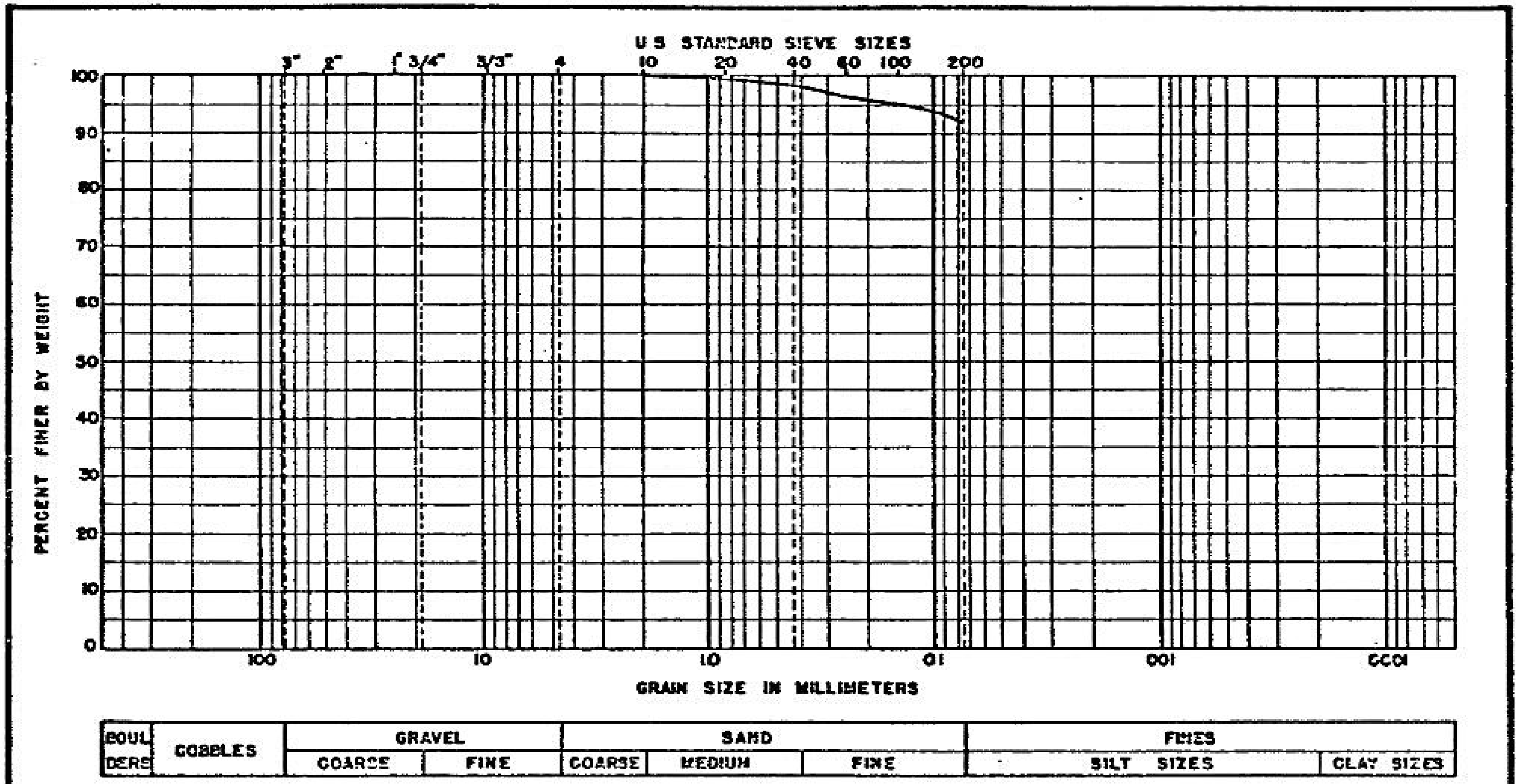


FIGURE 2.5.4-40

GRAIN SIZE DISTRIBUTION, BORING NO. BC-172, S-3



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BC-172 S-3	6.0'-7.5'					Red Slightly Fine Sandy SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

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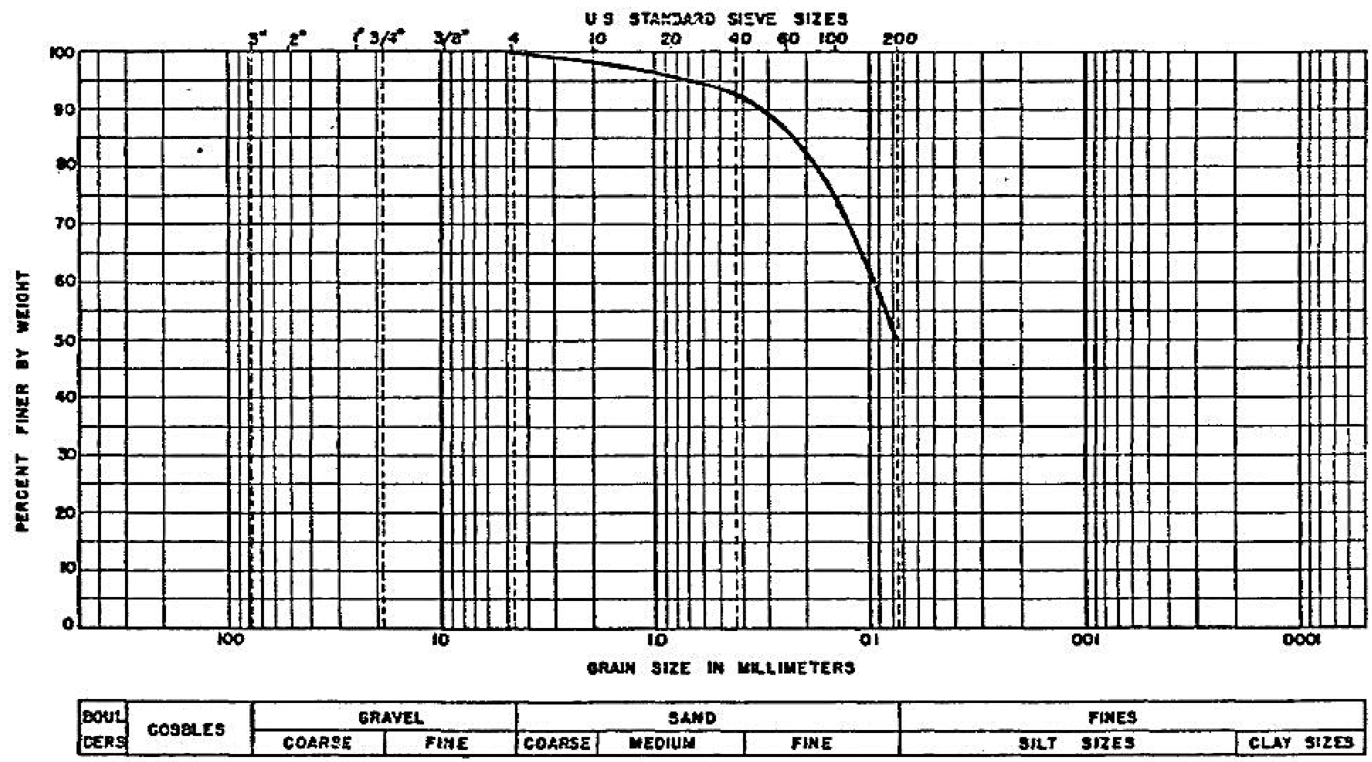
FIGURE 2.5.4-41

GRAIN SIZE DISTRIBUTION, BORING NO. BC-172, S-6



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BC 172	18.5'- 20.0' S-6					WHITE AND GRAY SILTY FINE TO MEDIUM SAND

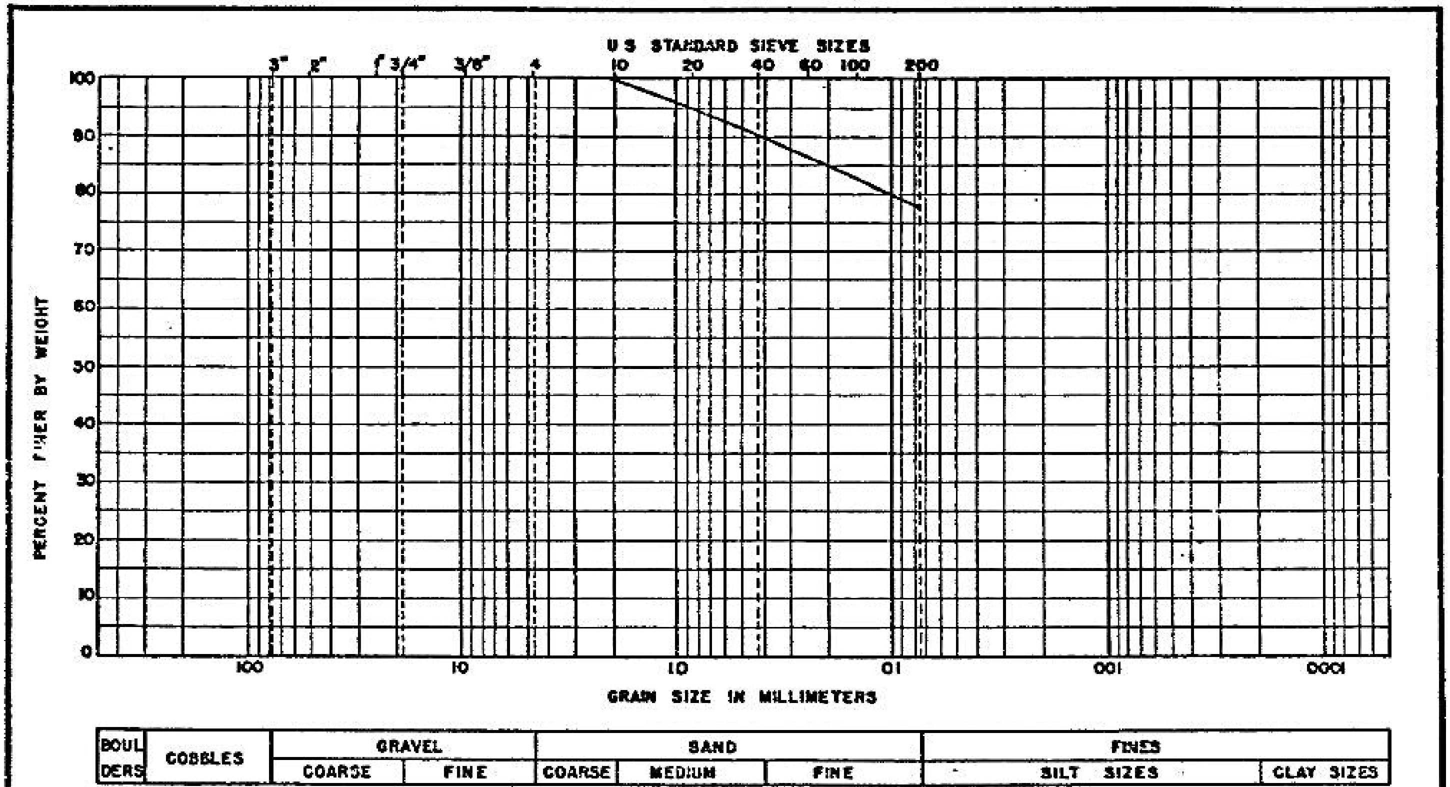
GRAIN SIZE DISTRIBUTION

JOB NO. 503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-42

GRAIN SIZE DISTRIBUTION, BORING NO. BC-173, S-3



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BC-173 S-3	7.0'-12.0'					Red Fine to Medium Sandy SILT

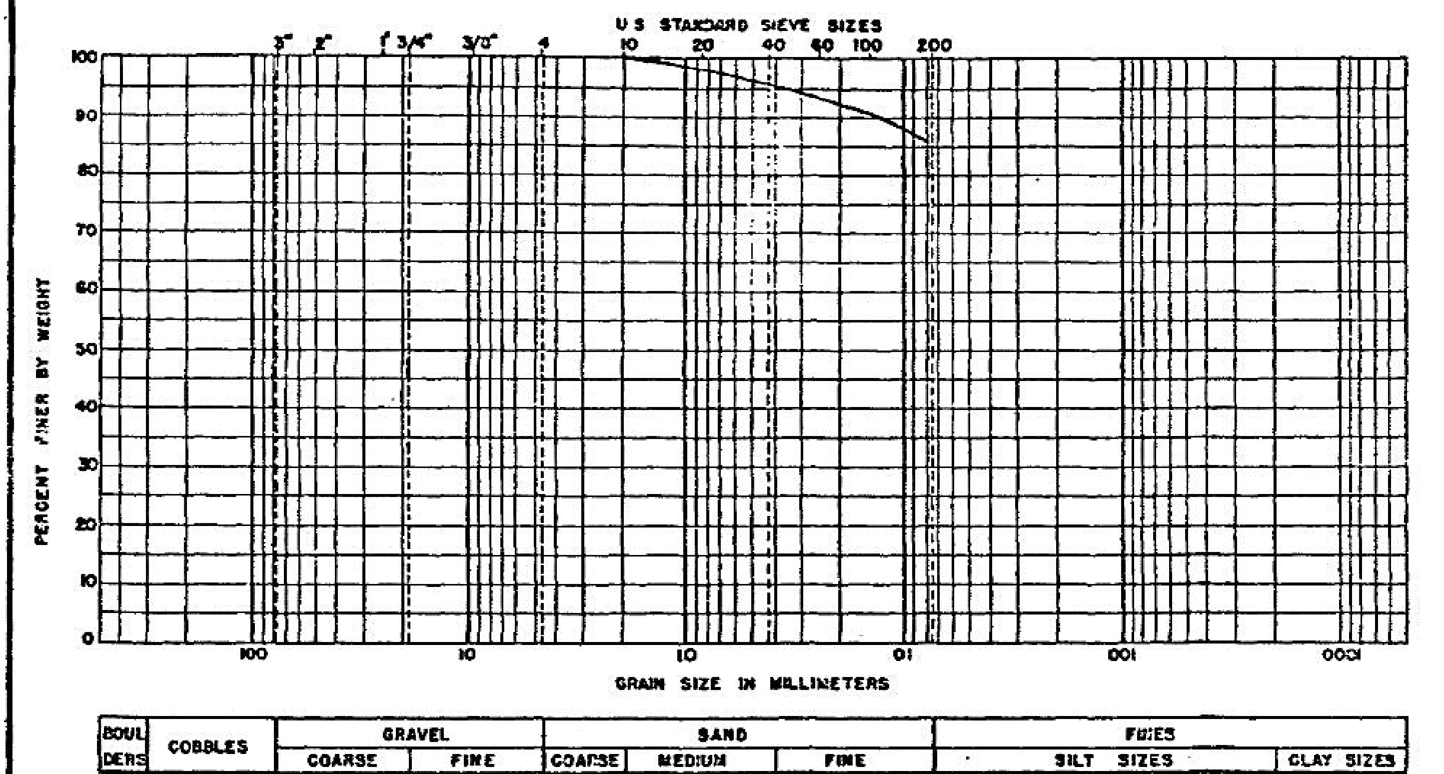
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-43

GRAIN SIZE DISTRIBUTION, BORING NO. BC-176, S-6



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BC-176	19.0'-24.0' S-6					Red Find Sandy Clayey SILT

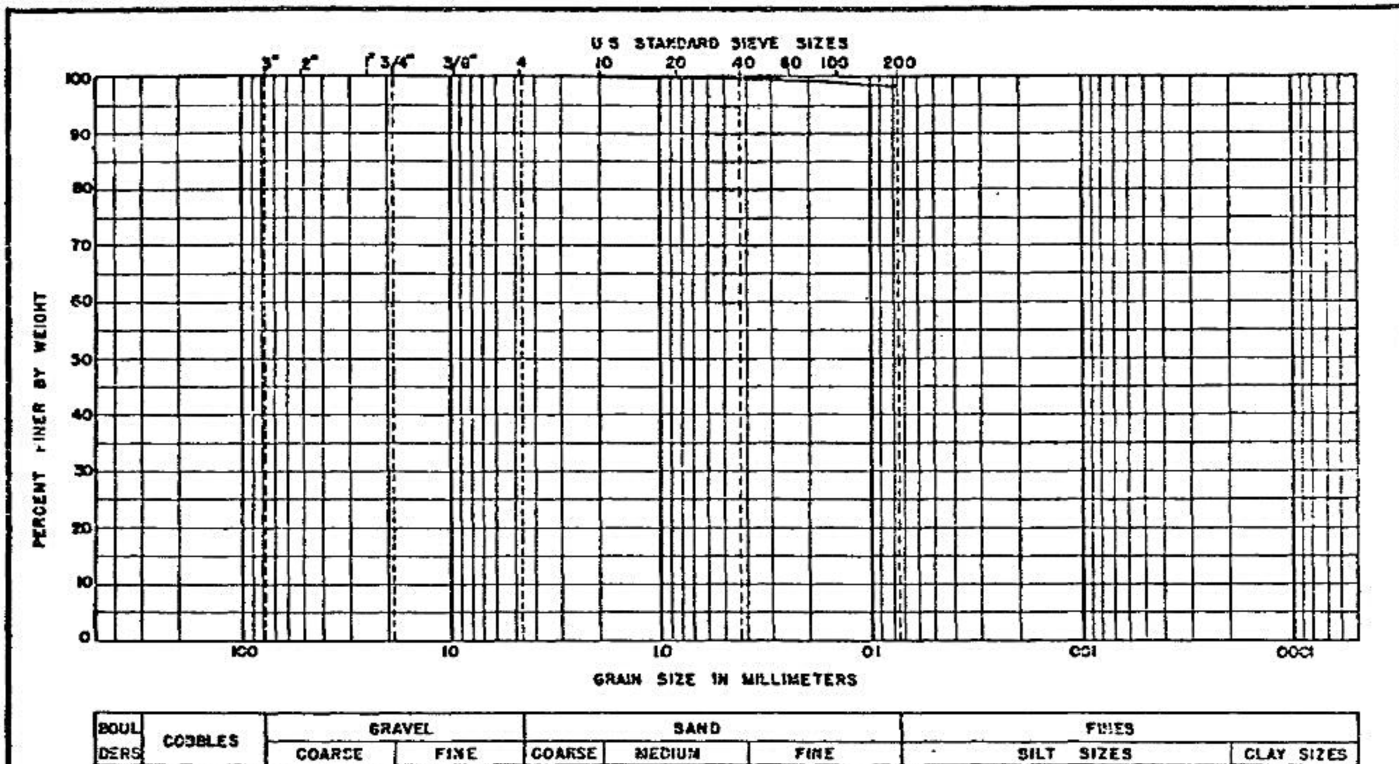
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-44

GRAIN SIZE DISTRIBUTION, BORING NO. BC-177, S-3



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BC-177	6.5'-11.5' S-3					Red Clayey SILT

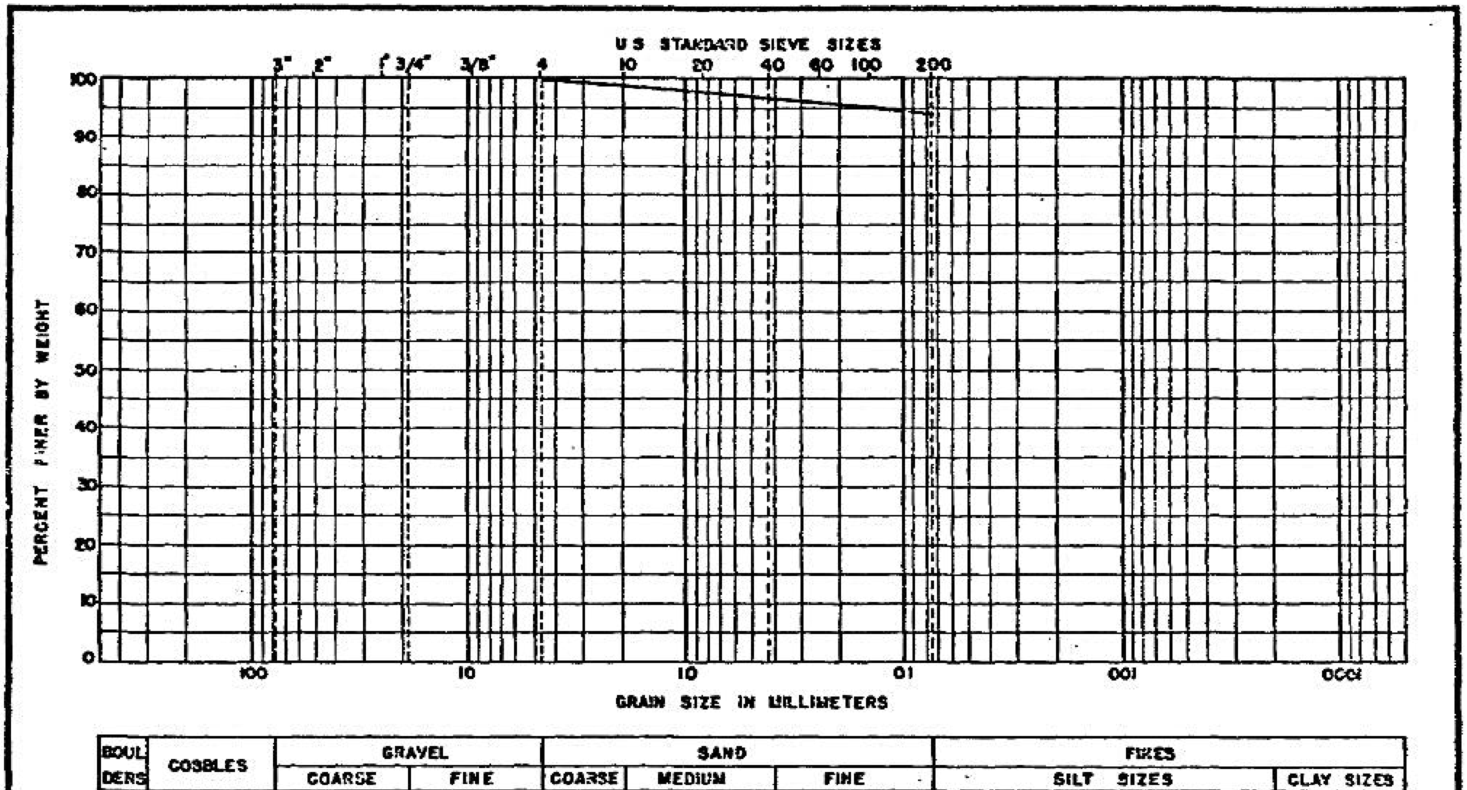
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-45

GRAIN SIZE DISTRIBUTION, BORING NO. BC-180, S-3



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BC-180 S-3	10.0'-15.0'					Red Brown Fine Sandy Clayey SILT

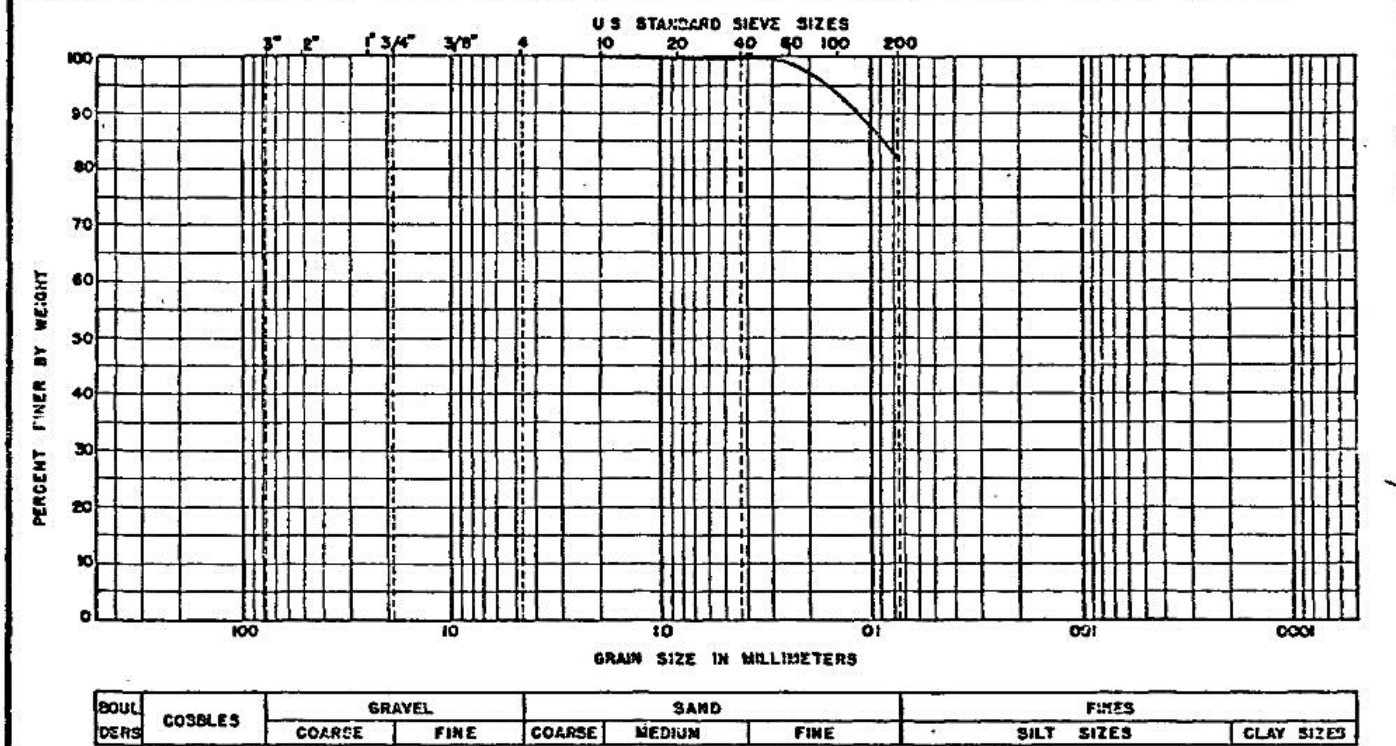
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-46

GRAIN SIZE DISTRIBUTION, BORING NO. BC-182, S-3



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BC-182 S-3	6.0'-7.5'					Brown and Gray Clayey SILT

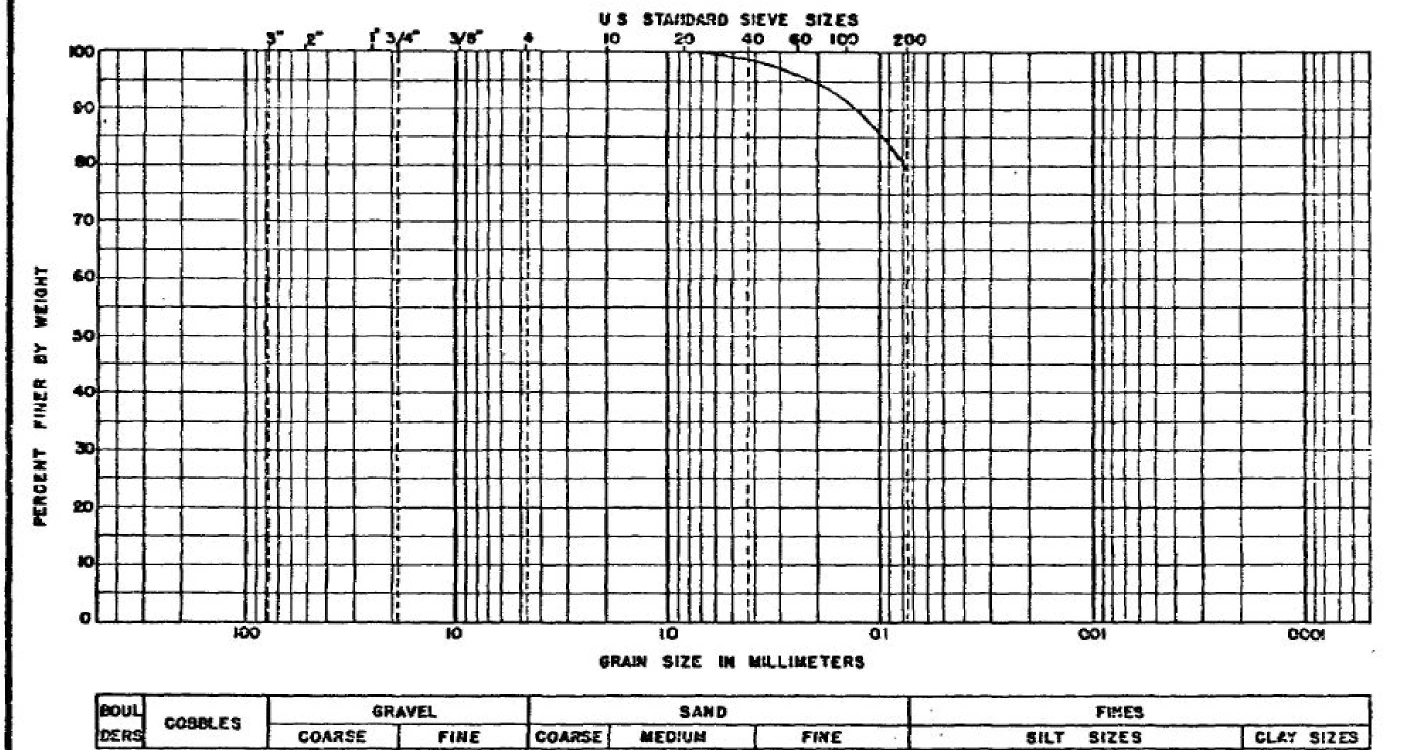
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-47

GRAIN SIZE DISTRIBUTION, BORING NO. BC-182, S-5



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BC-182	13.5'-15.0' S-5					Red Fine Sandy SILT

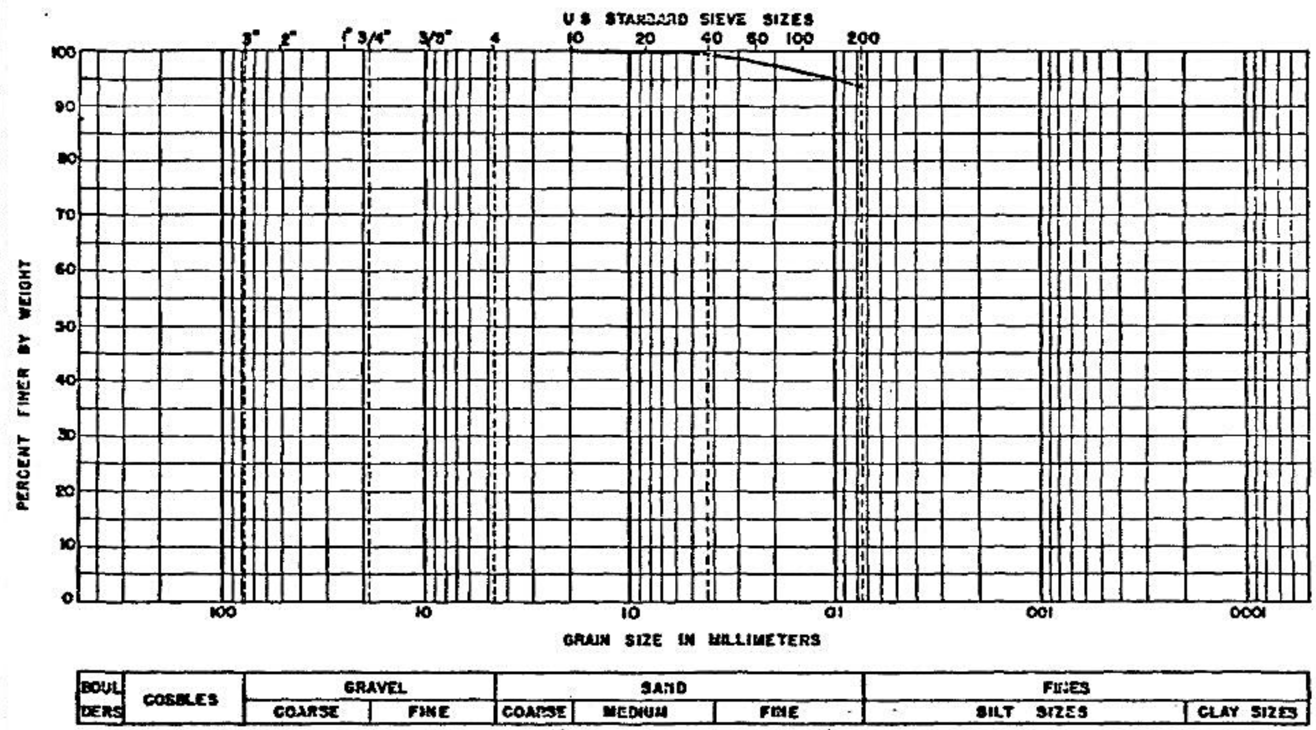
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-48

GRAIN SIZE DISTRIBUTION, BORING NO. BC-184, S-3



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BC-184 S-3	8.5'-13.5'					Red Brown Slightly Clayey Fine Sandy SILT

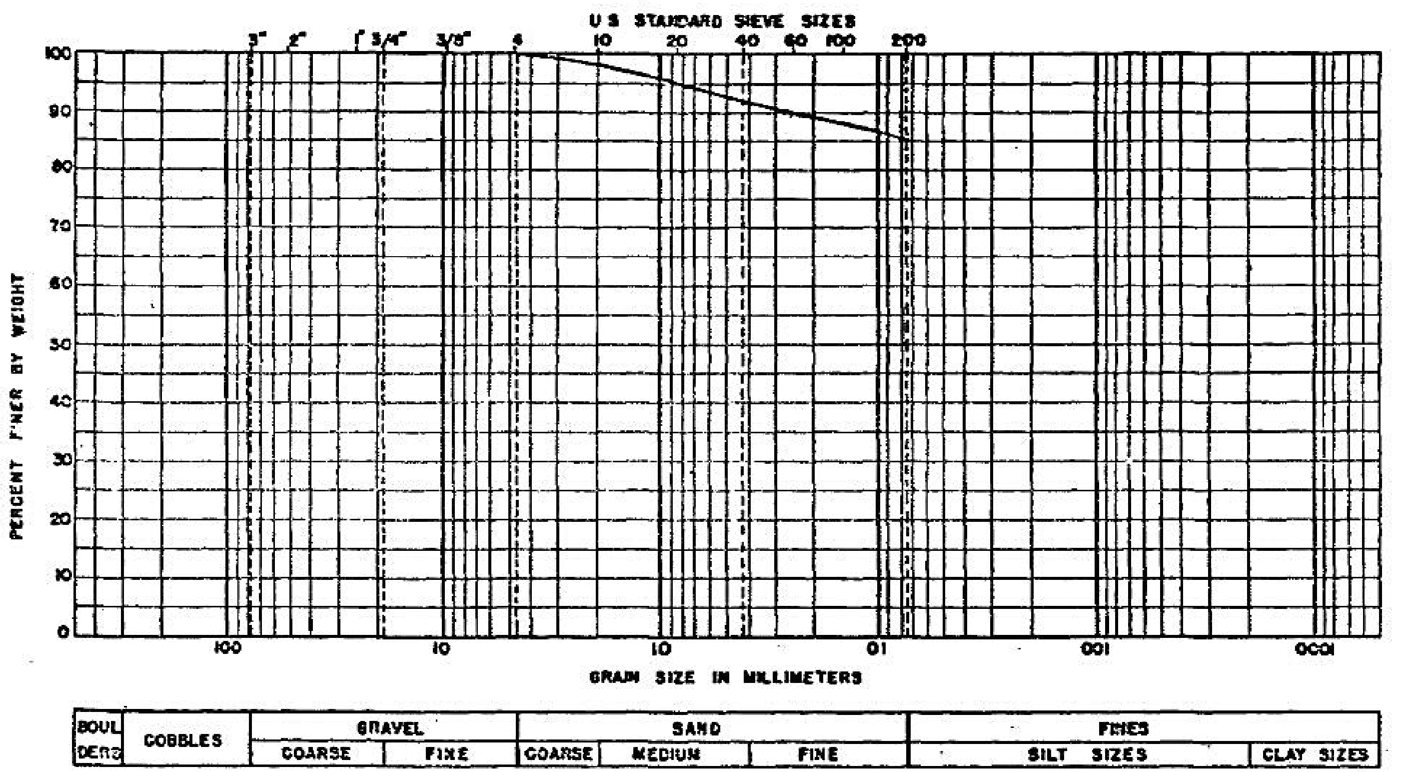
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-49

GRAIN SIZE DISTRIBUTION, BORING NO. BC-187, S-2



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BC-187 S-2	3.5'-8.5'					Red Brown Fine Sandy Clayey SILT

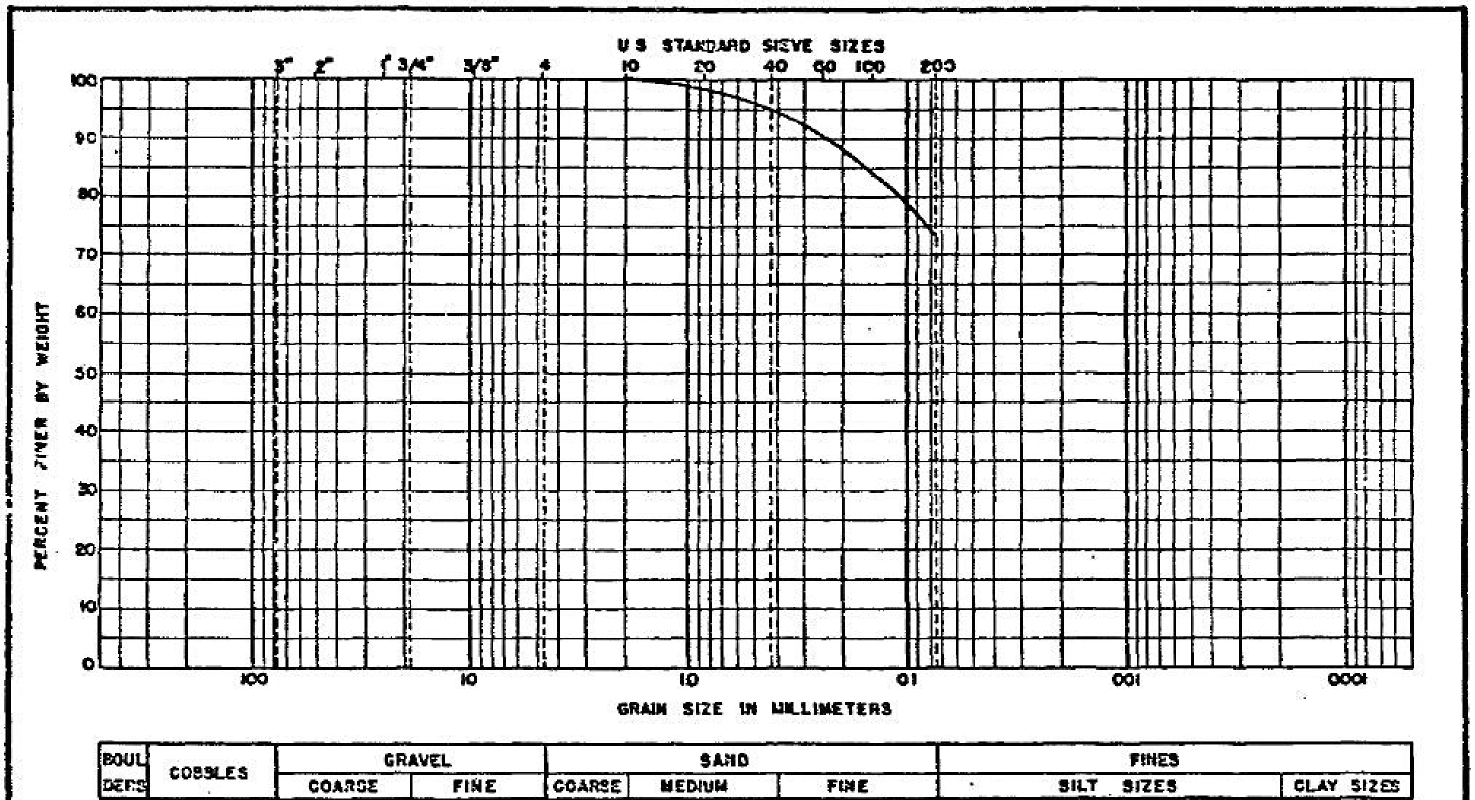
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-50

GRAIN SIZE DISTRIBUTION, BORING NO. BC-188, S-2



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BC-188	5.0'-10.0' S-2					Red Brown Fine Sandy SILT

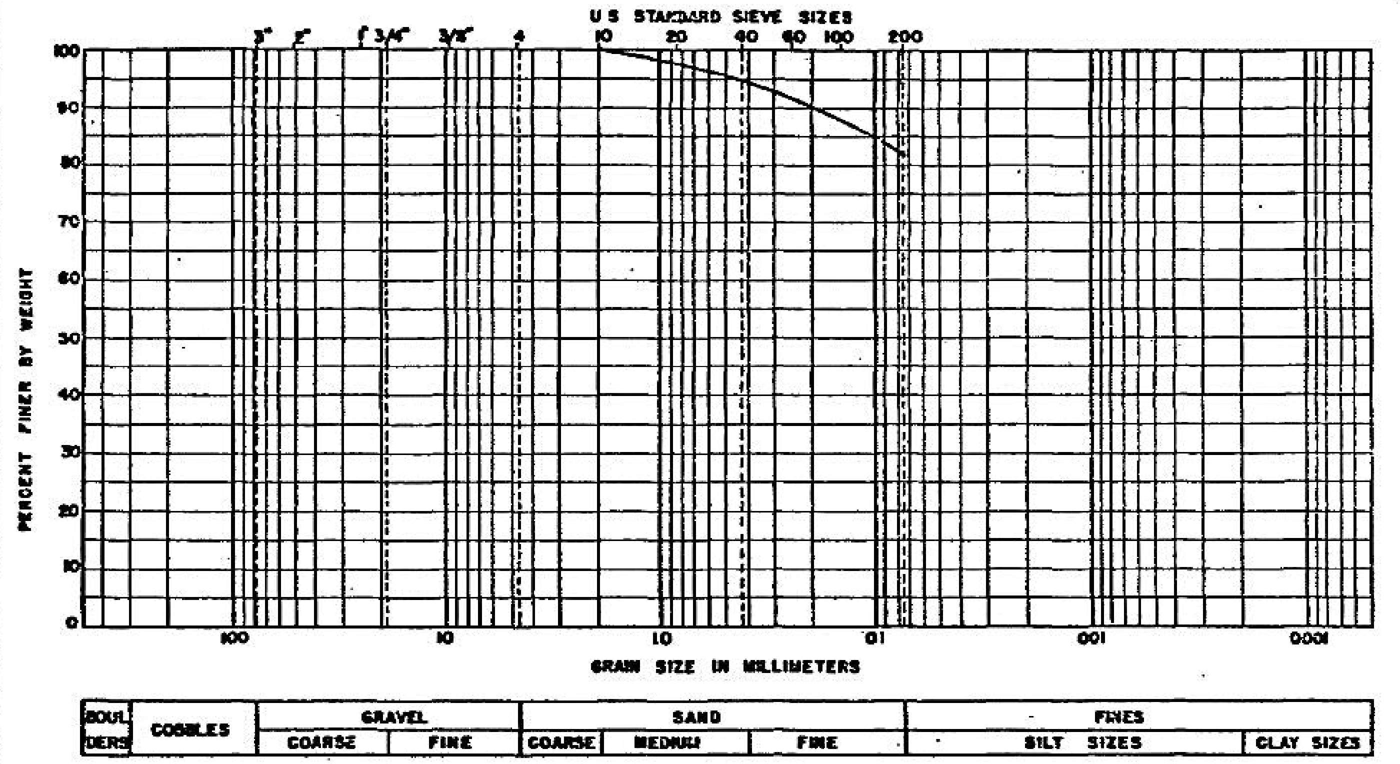
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-51

GRAIN SIZE DISTRIBUTION, BORING NO. BC-190, S-4



BORING NO	ELEV OR DEPTH	NAT WC	LL	PL	PI	DESCRIPTION OR CLASSIFICATION
BC-190 S-4	8.5'-10.0'					Red and Gray Mottled Fine Sandy Clayey SILT

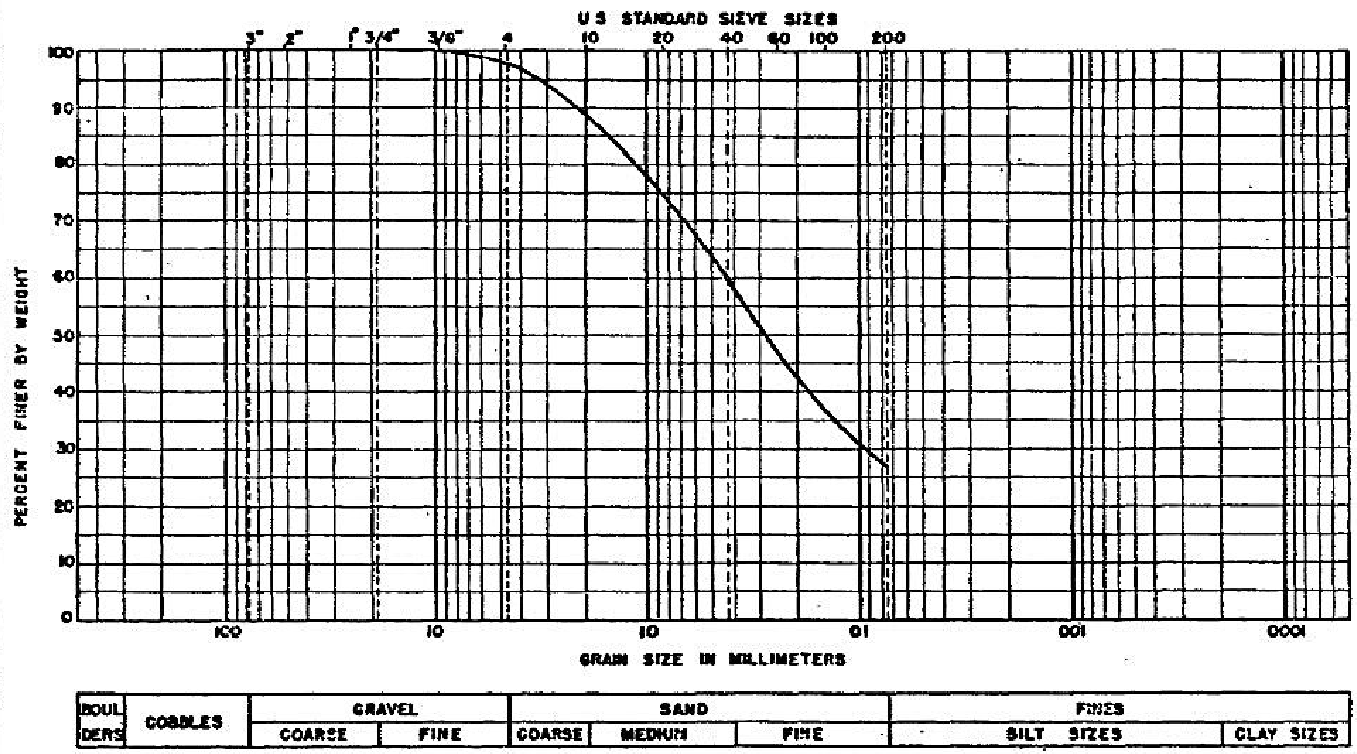
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-52

GRAIN SIZE DISTRIBUTION, BORING NO. BC-190, S-6



BORING NO	ELEV OR DEPTH	NAT WC	L L	P L	P I	DESCRIPTION OR CLASSIFICATION
BC-190 S-6	18.5'-20.0'					Gray and Red Very Silty Fine to Coarse SAND With Traces of Mica

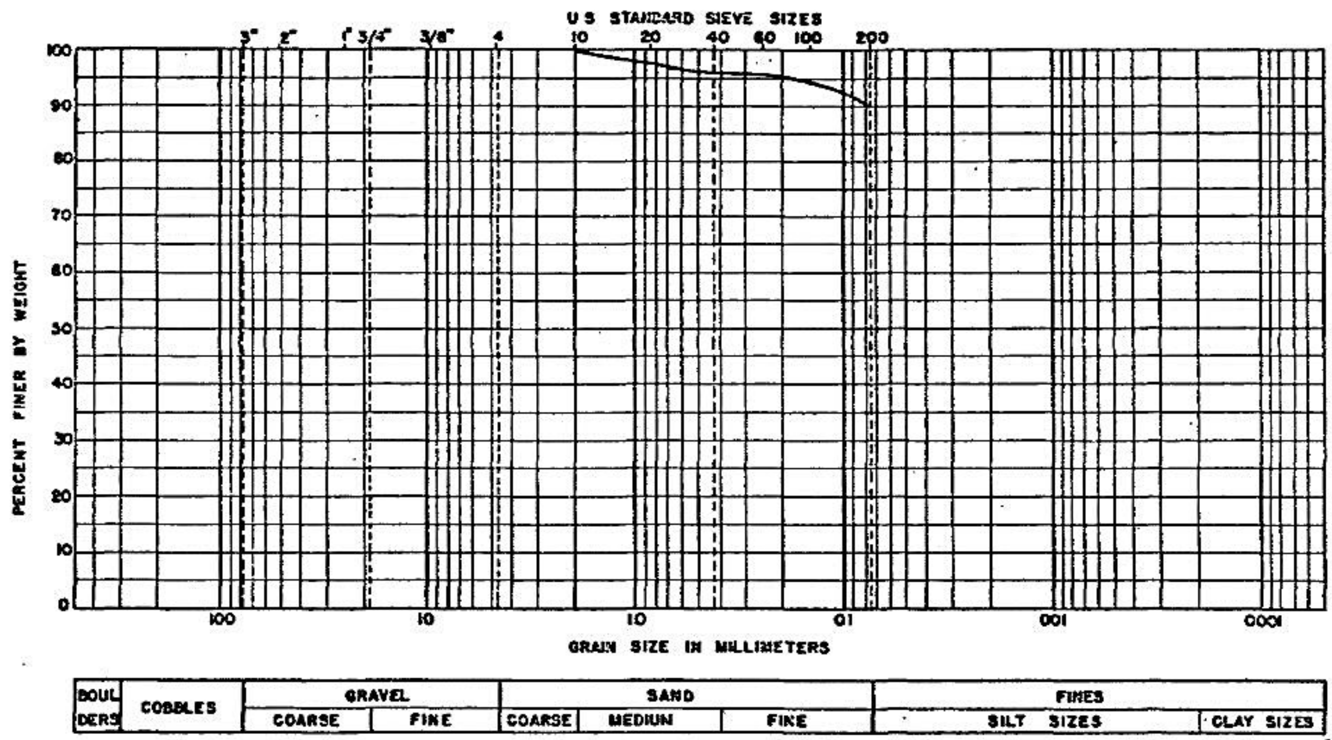
GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-53

GRAIN SIZE DISTRIBUTION, BORING NO. BC-191, S-3



BORING NO	ELEV OR DEPTH	NAT WC	L L	PL	P I	DESCRIPTION OR CLASSIFICATION
BC-191 S-3	6.0'-7.5'					Mottled Brown and Gray Sandy Clayey SILT

GRAIN SIZE DISTRIBUTION

JOB NO. RA-503

LAW ENGINEERING TESTING COMPANY

FIGURE 2.5.4-54

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-13

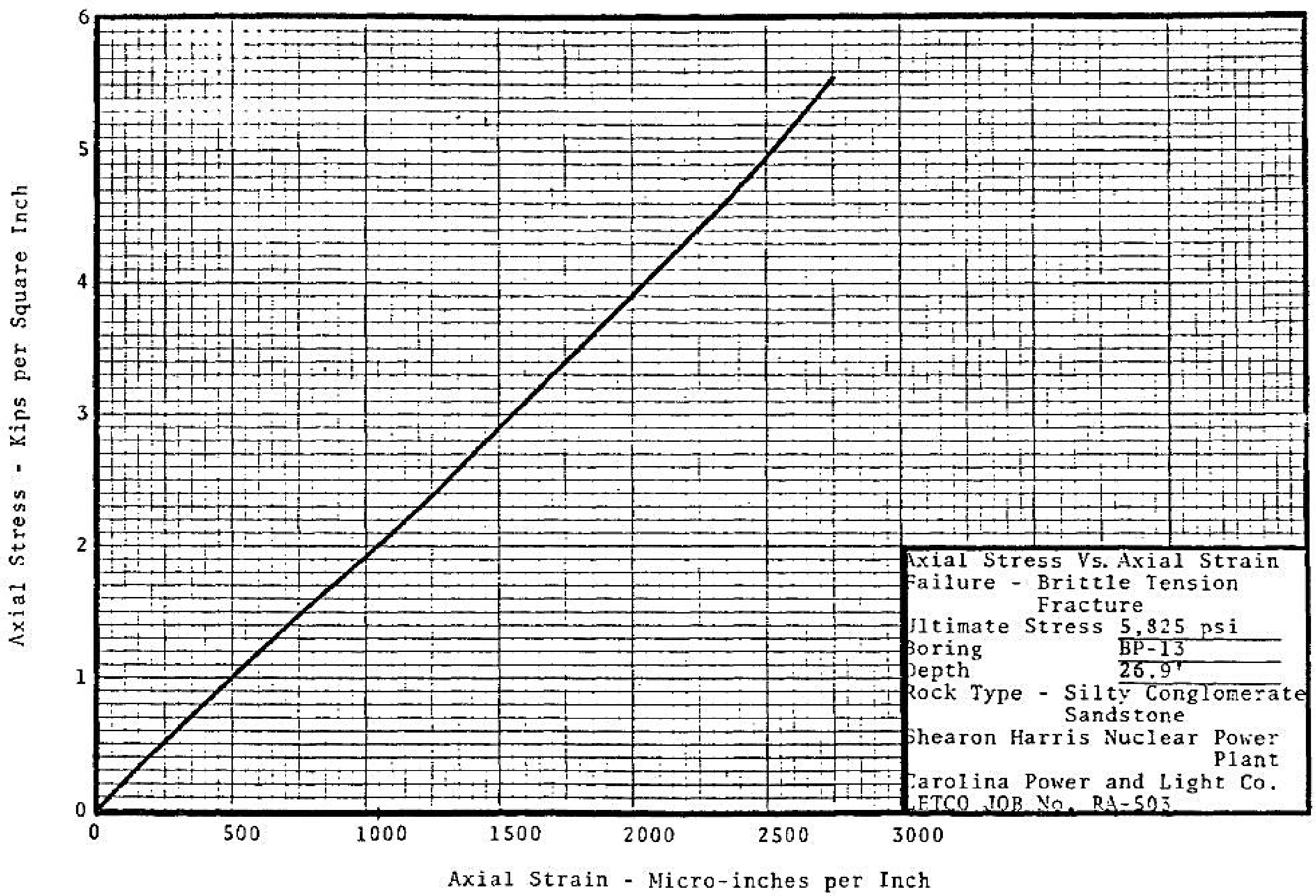


FIGURE 2.5.4-55

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-35

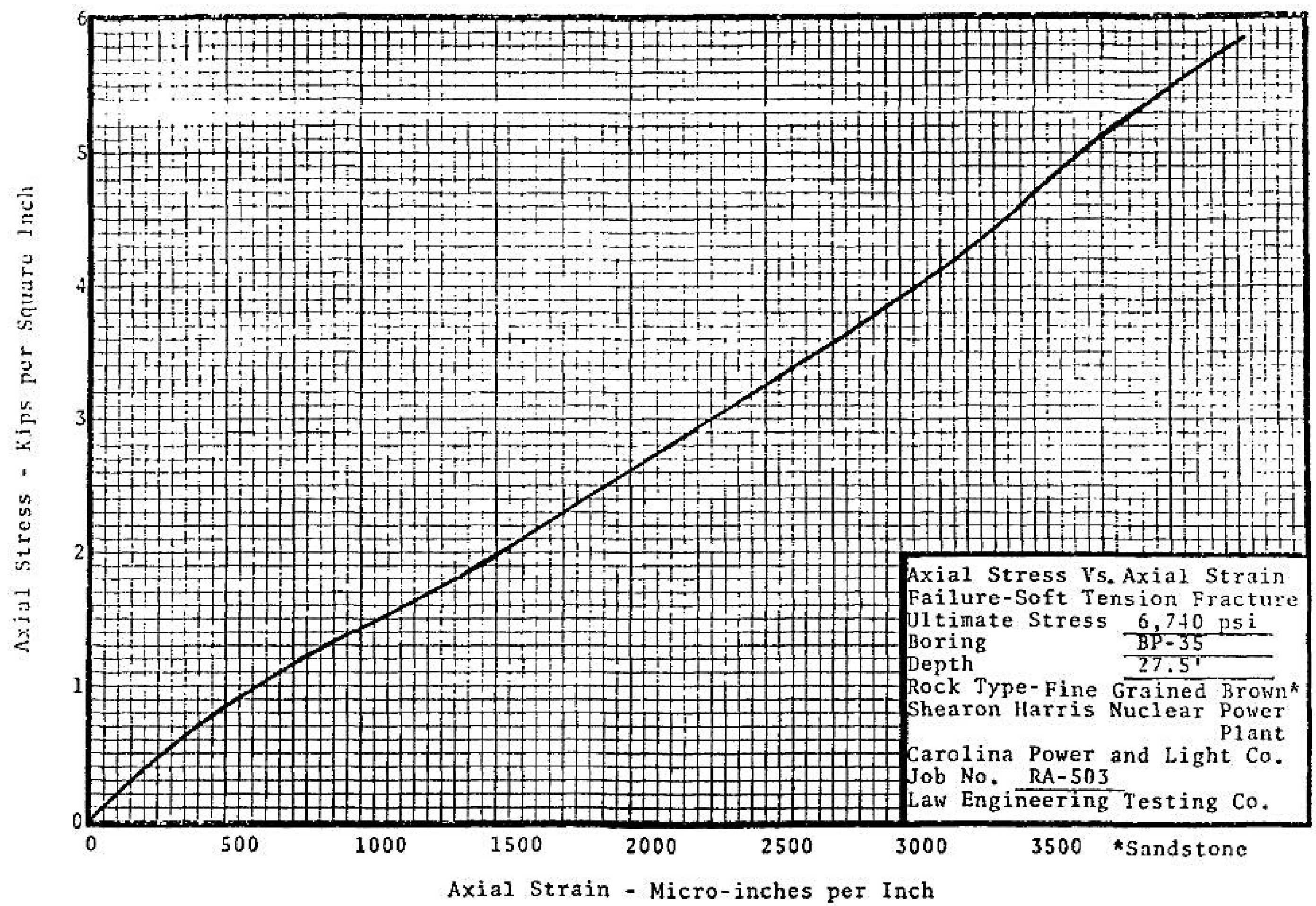


FIGURE 2.5.4-56

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-46

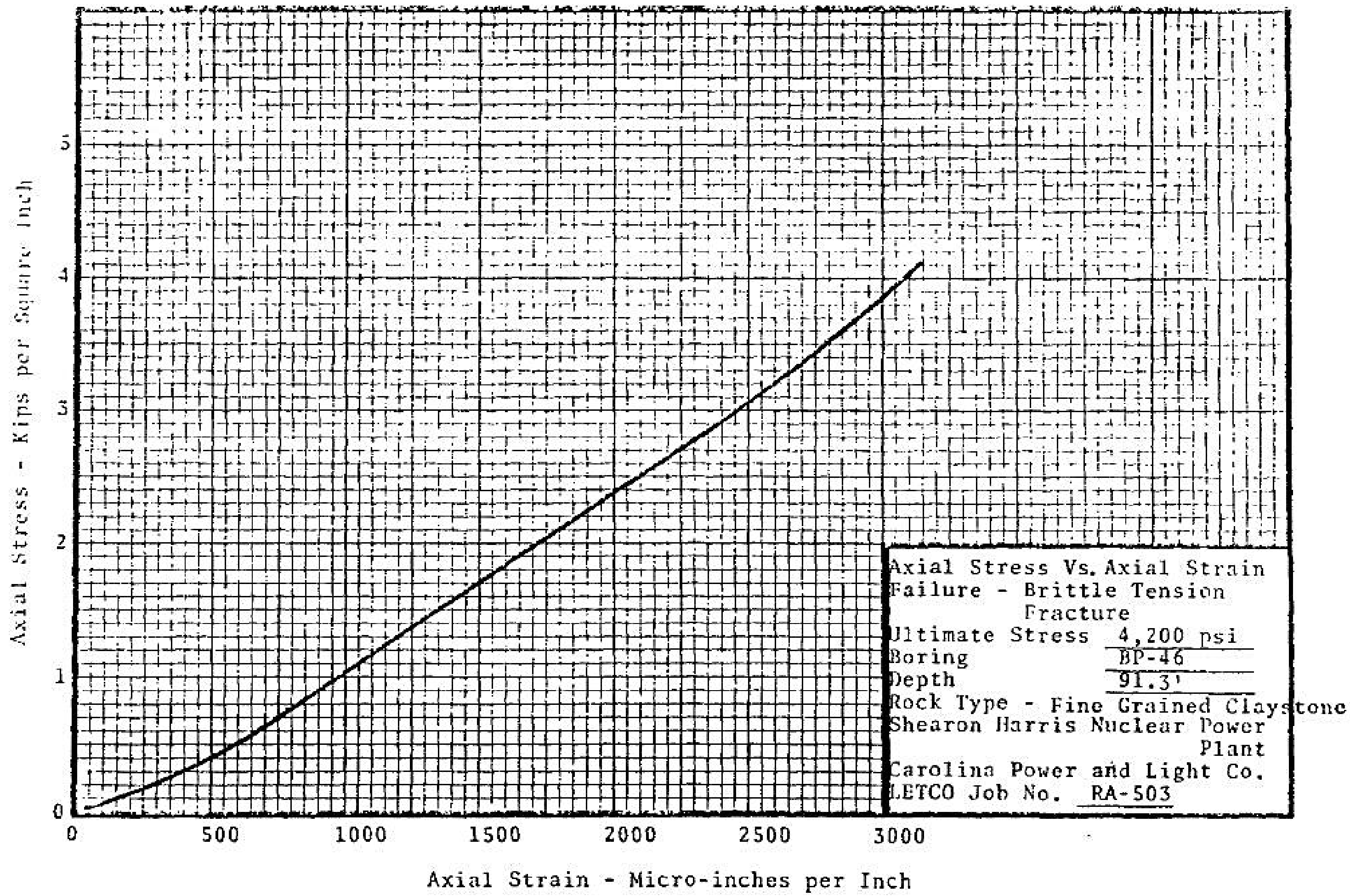


FIGURE 2.5.4-57

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-58

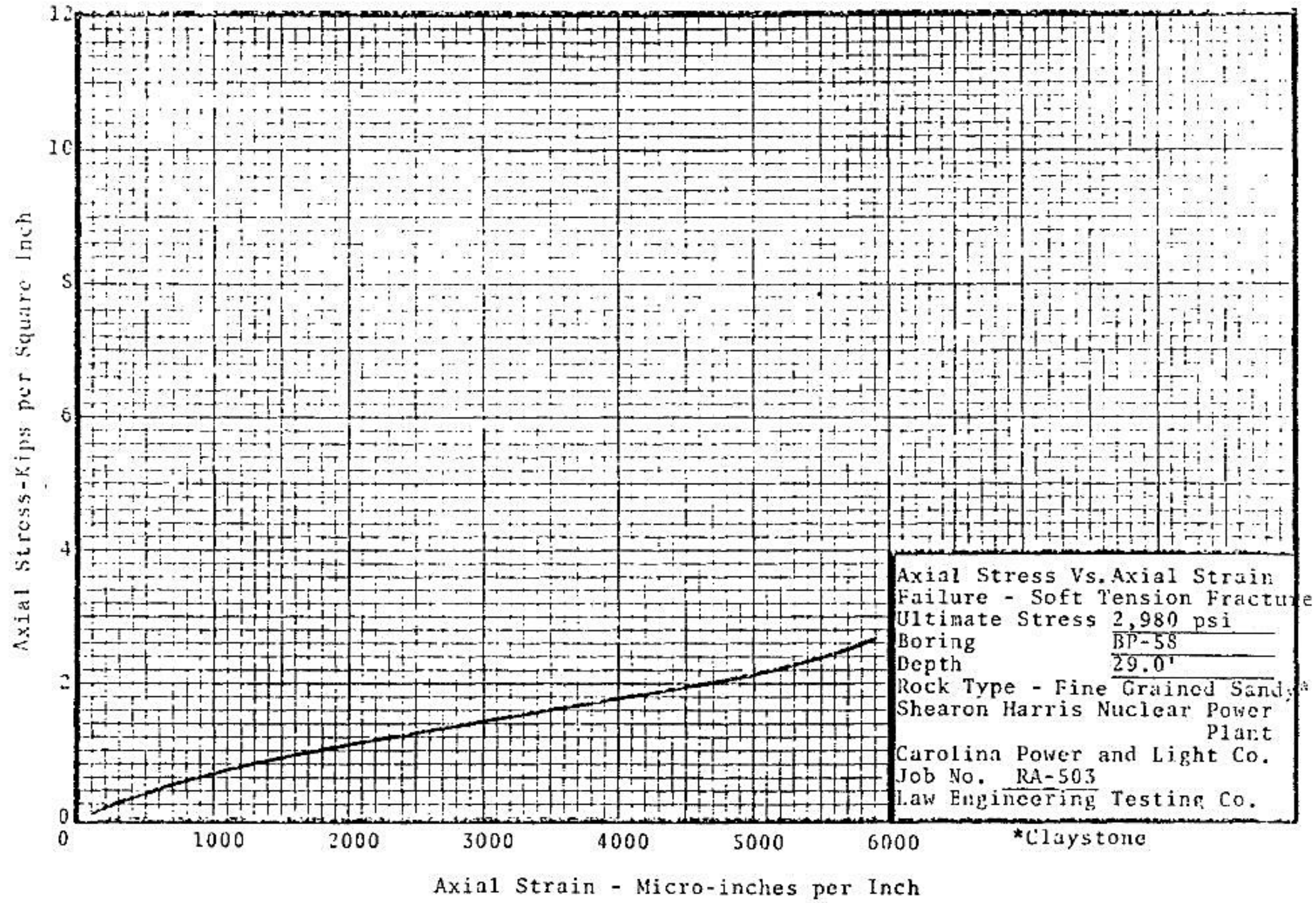


FIGURE 2.5.4-58

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-62

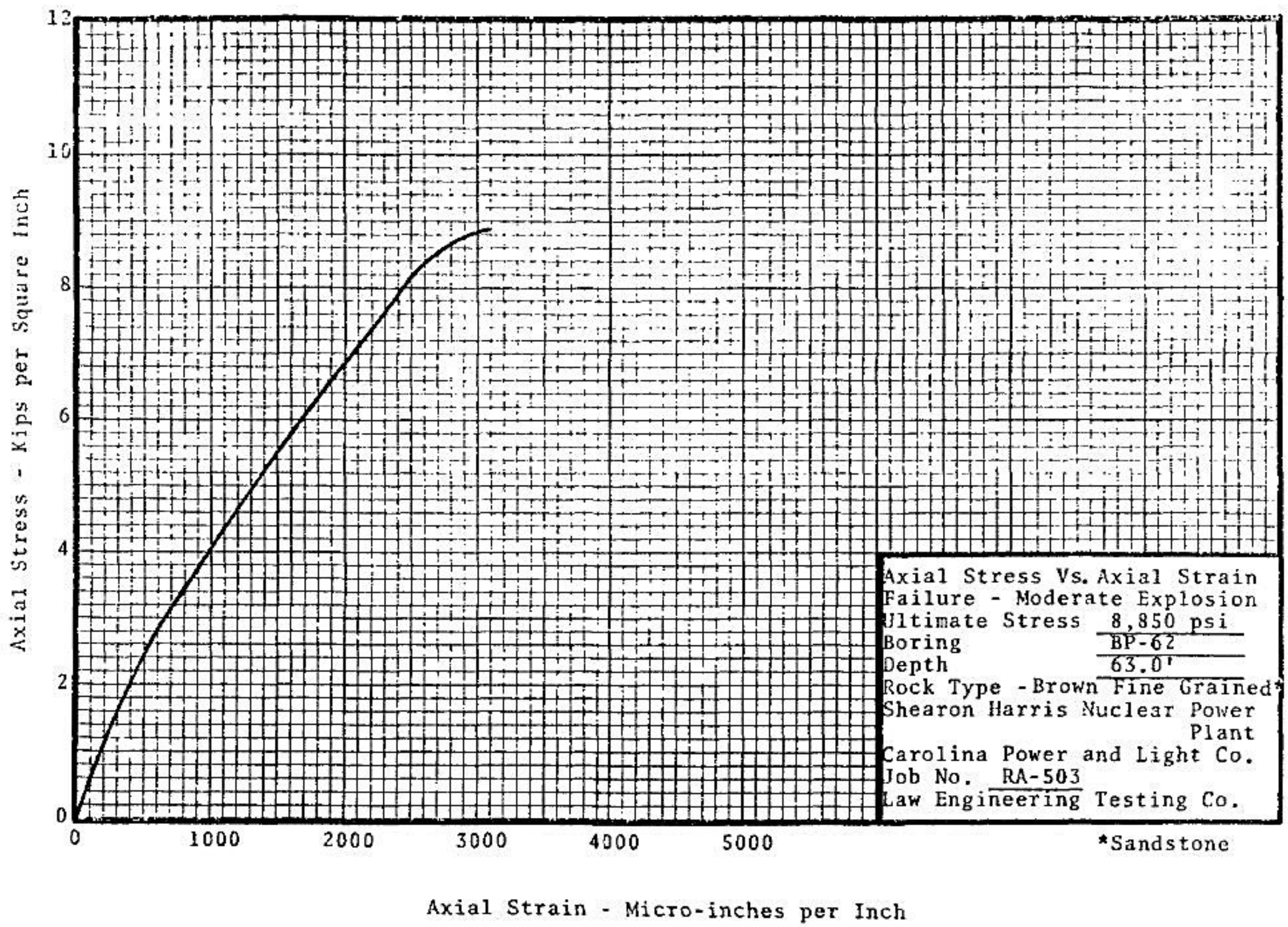


FIGURE 2.5.4-59

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-63

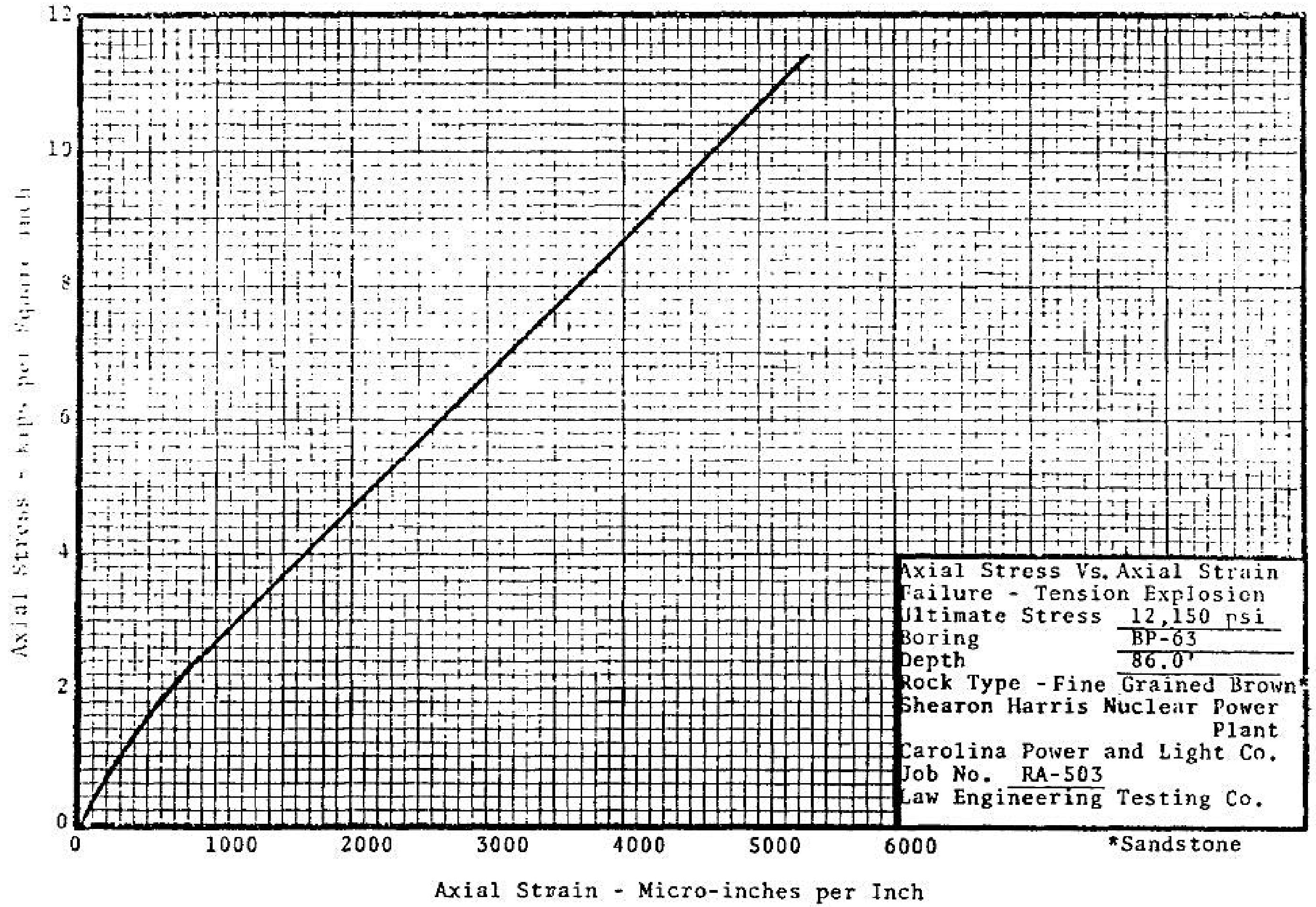


FIGURE 2.5.4-60

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-64

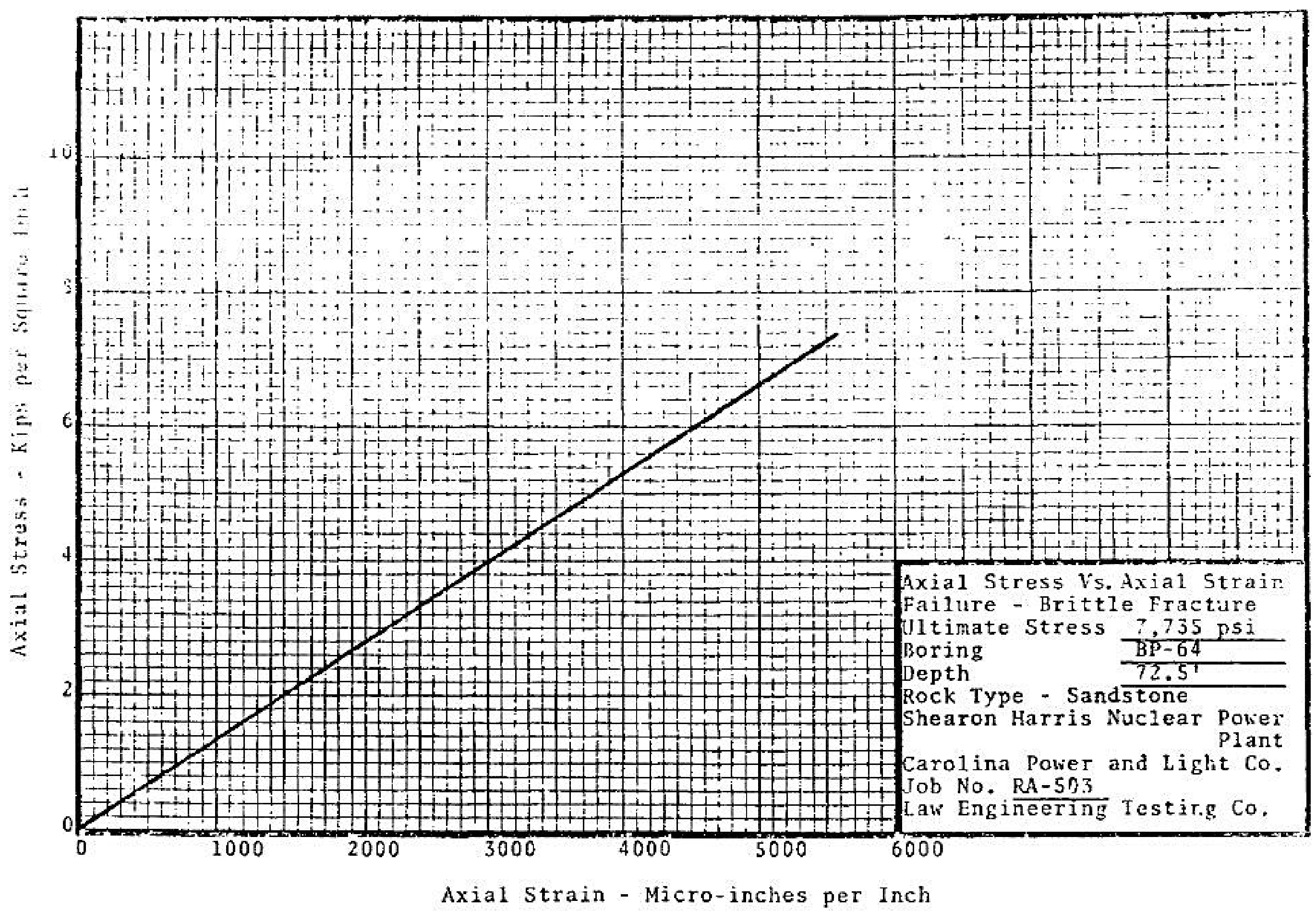


FIGURE 2.5.4-61

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-66

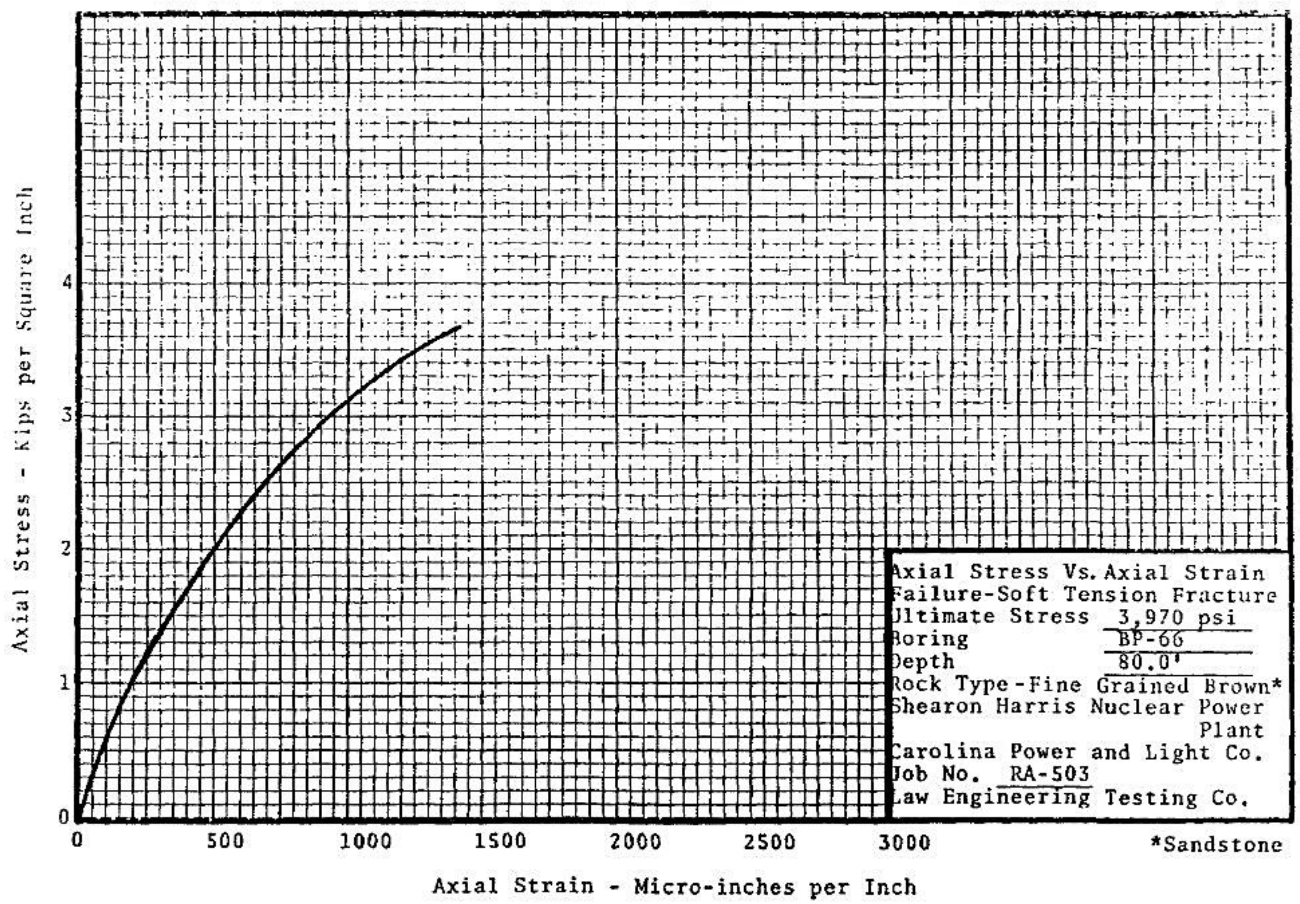


FIGURE 2.5.4-62

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-68

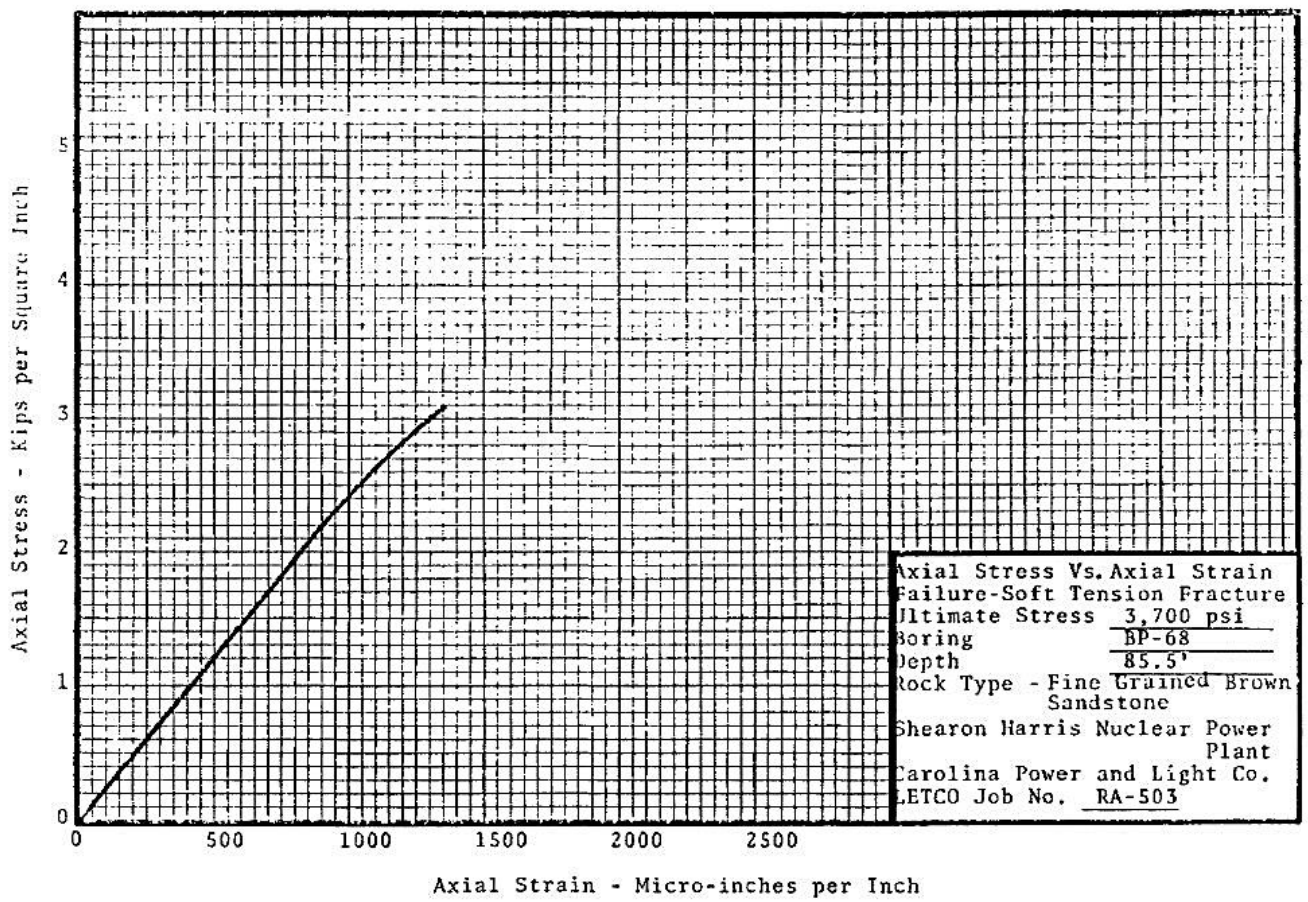


FIGURE 2.5.4-63

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-69

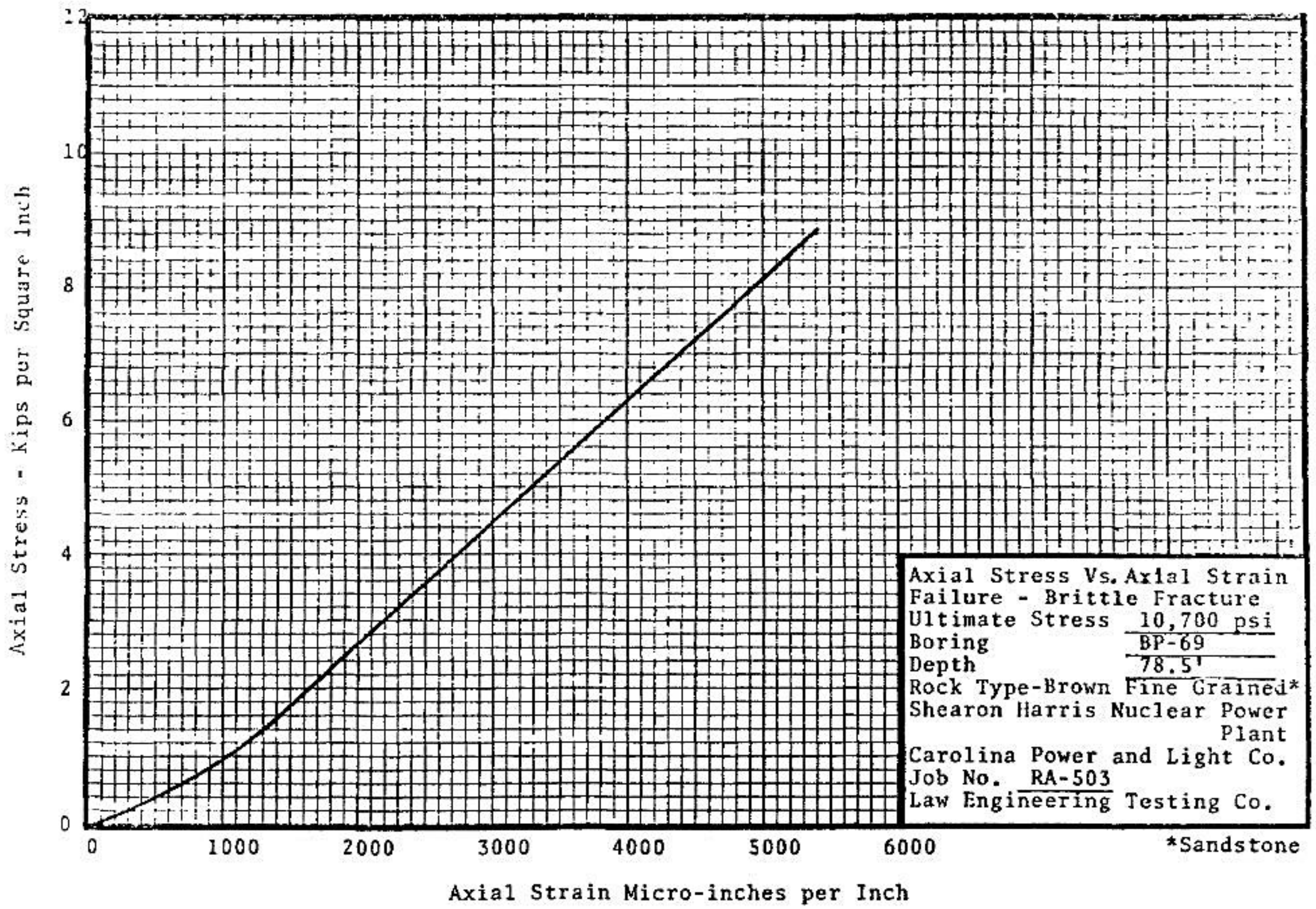


FIGURE 2.5.4-64

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-70

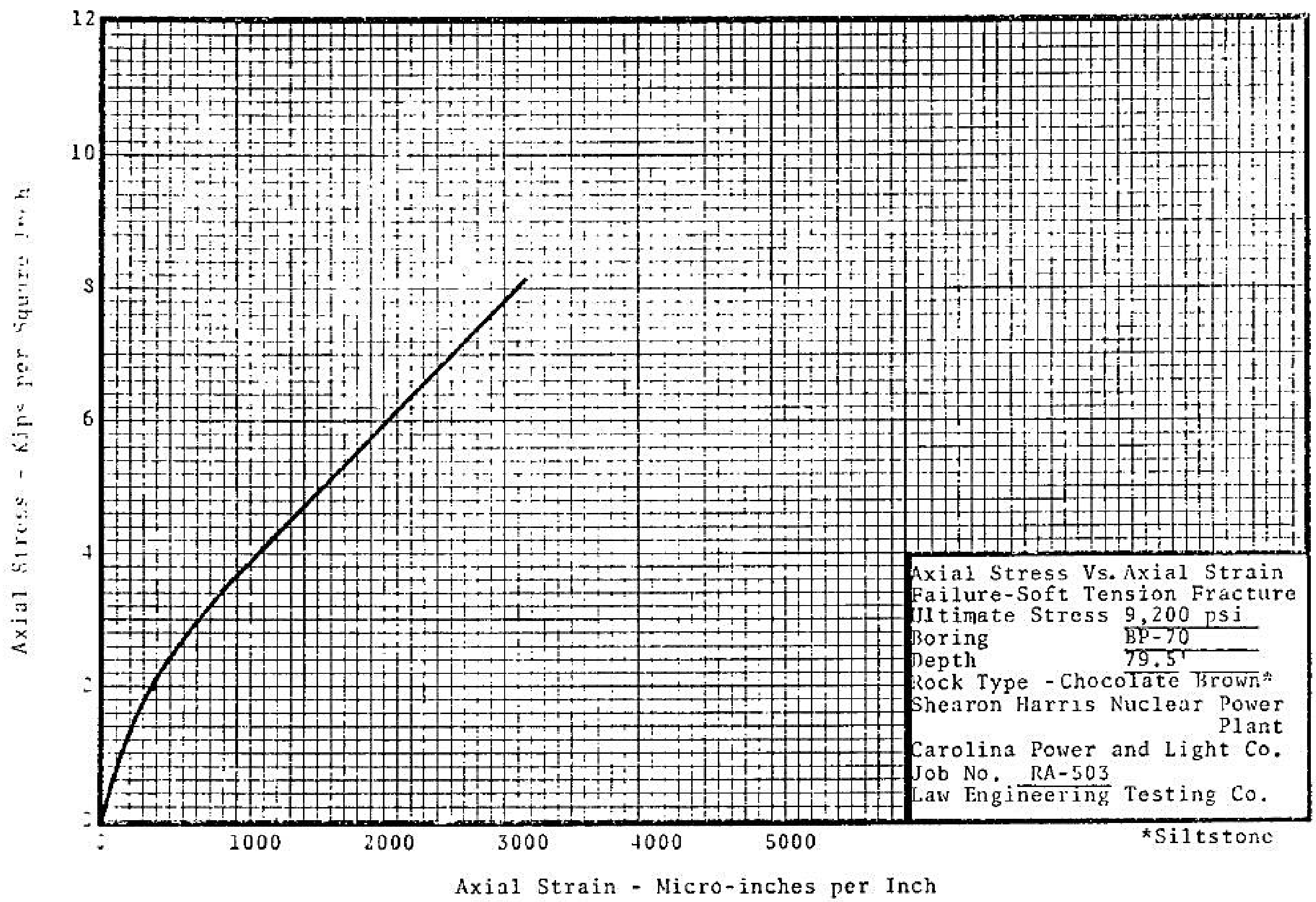


FIGURE 2.5.4-65

AXIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-74

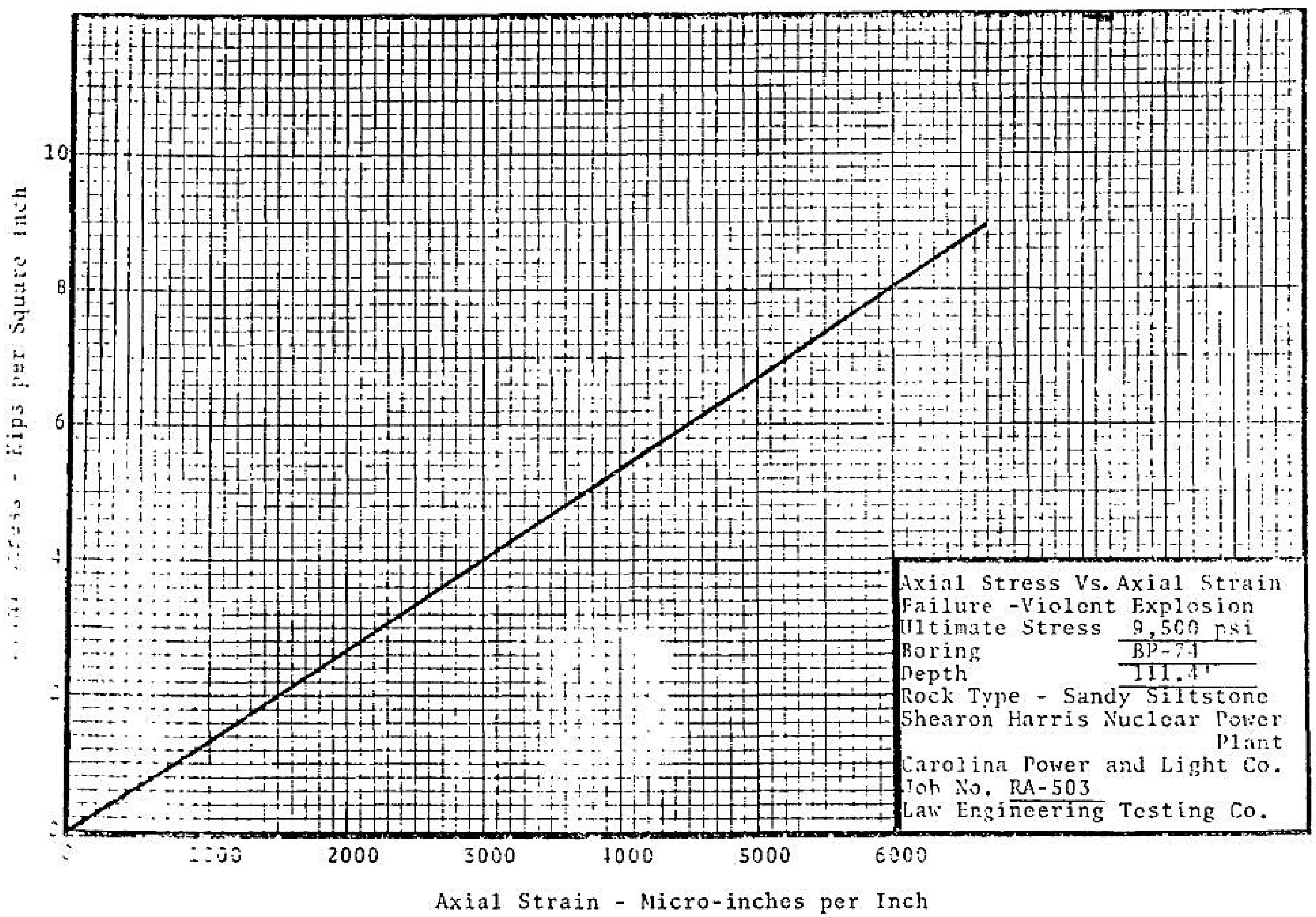


FIGURE 2.5.4-66

TRIAXIAL COMPRESSION

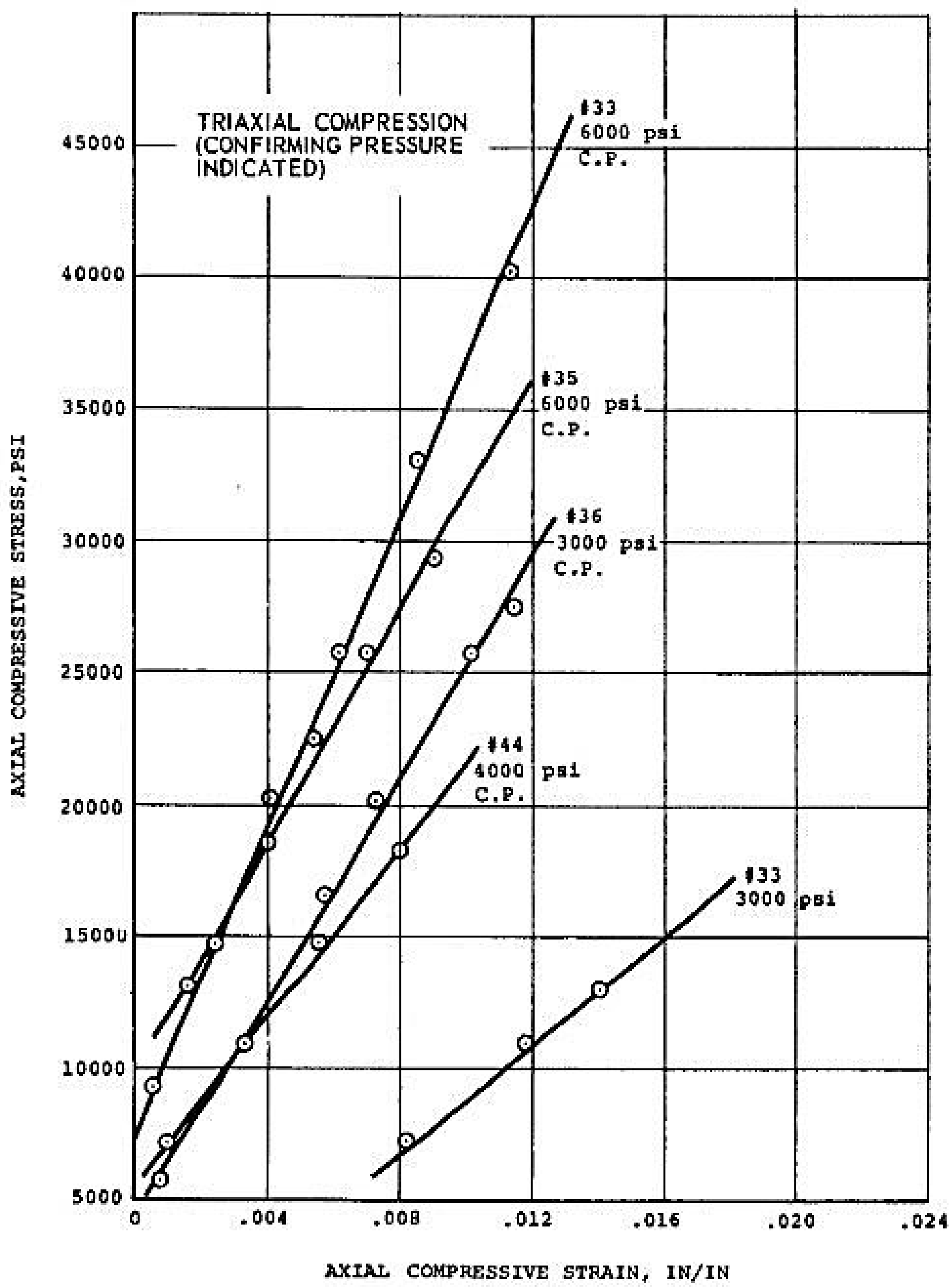


FIGURE 2.5.4-67

MOHR'S CIRCLE PLOT

BORING NO	DEPTH (FT)	SAMPLE NO	σ_3 (PSI) CONFINING PRESSURE	$\Delta\sigma$ (PSI)	σ_1 (PSI) NORMAL STRESS ($\sigma_1 = \sigma_3 + \Delta\sigma$)
P-6	64	44	4,000	22,200	26,200
P-7	85	35	6,000	35,900	41,900
P-7	117	33	6,000	49,600	55,600
P-6	64	44	0	9,200	9,200
P-7	85	35	0	9,800	9,800
P-7	117	33	0	15,350	15,350

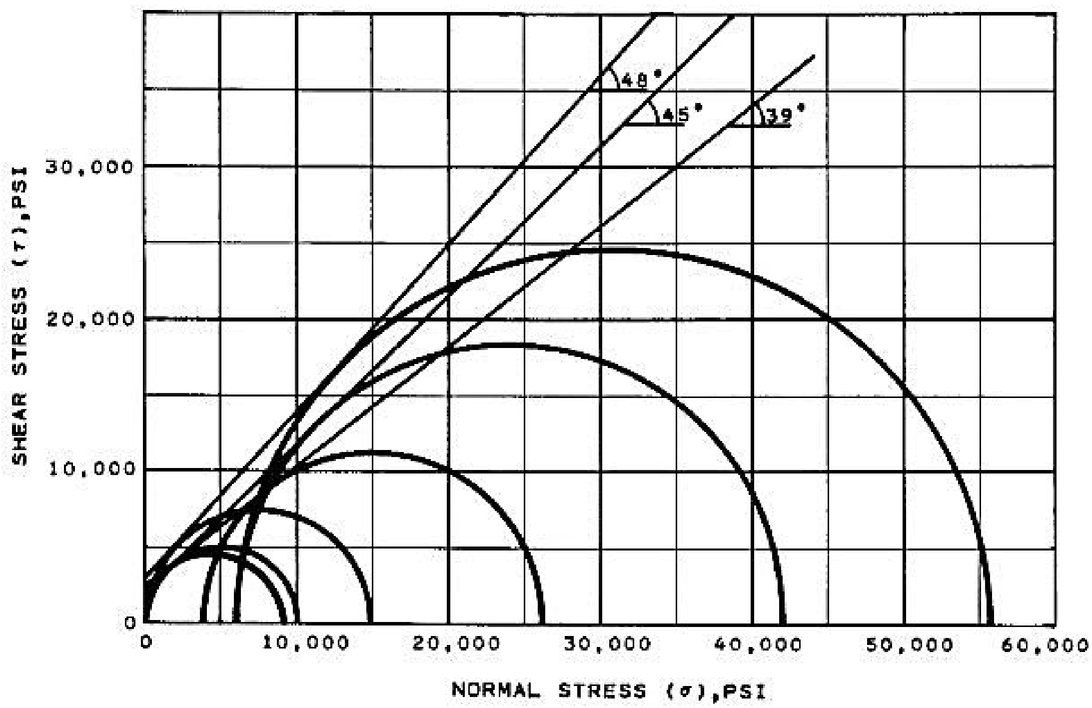


FIGURE 2.5.4-68

RADIAL STRESS VS. AXIAL STRAIN, BORING NO. BP-13

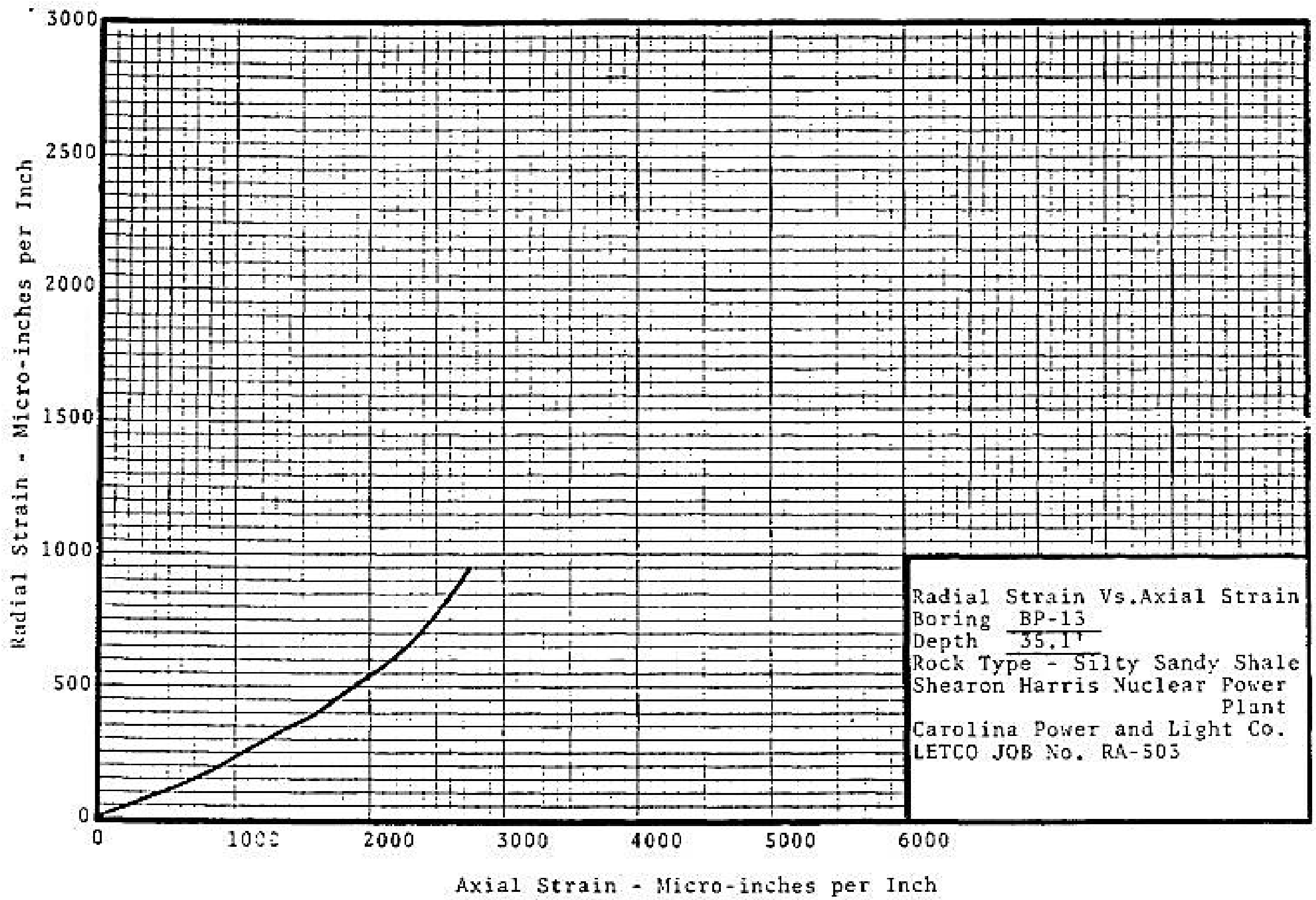


FIGURE 2.5.4-69

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-13

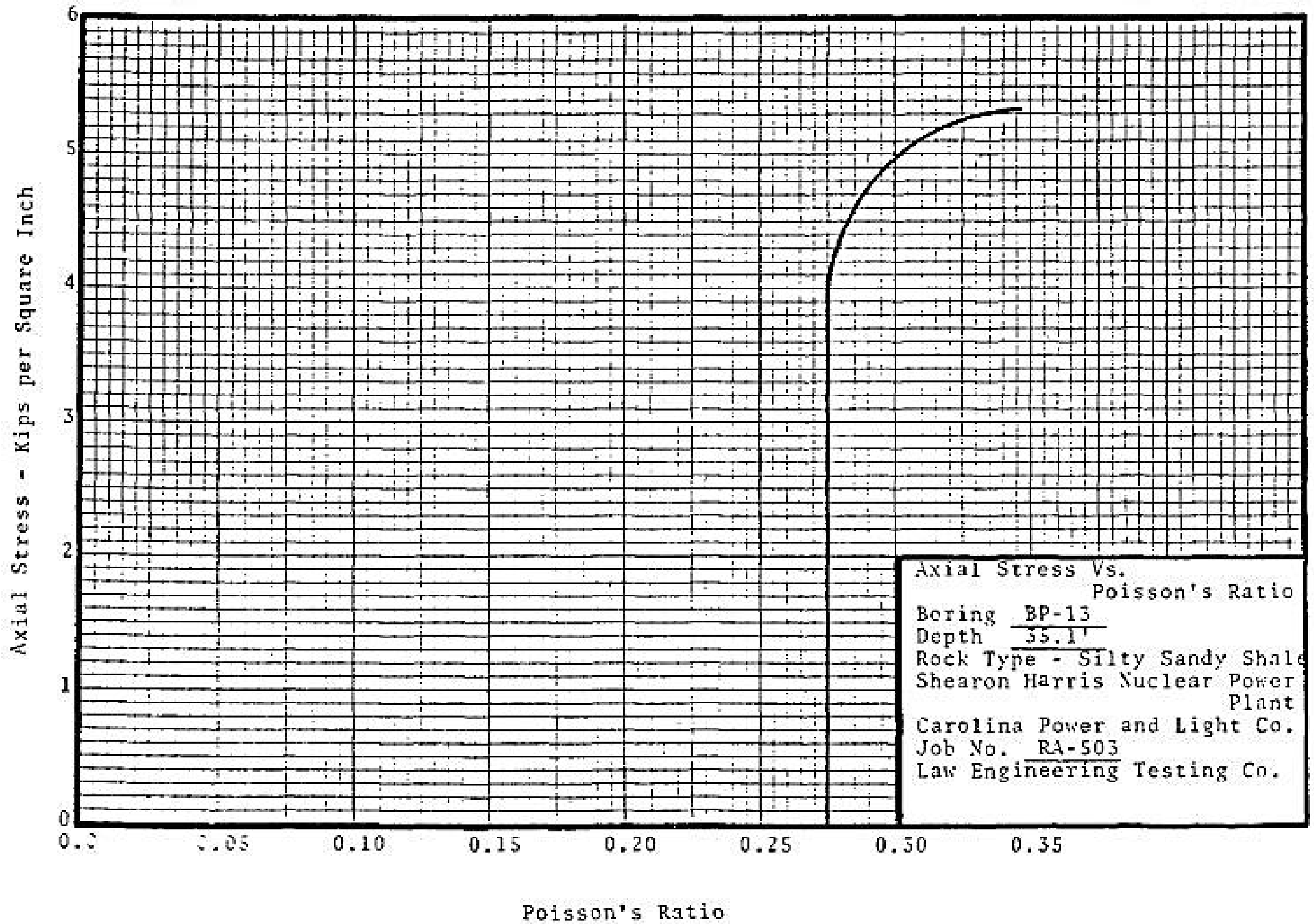


FIGURE 2.5.4-70

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-35

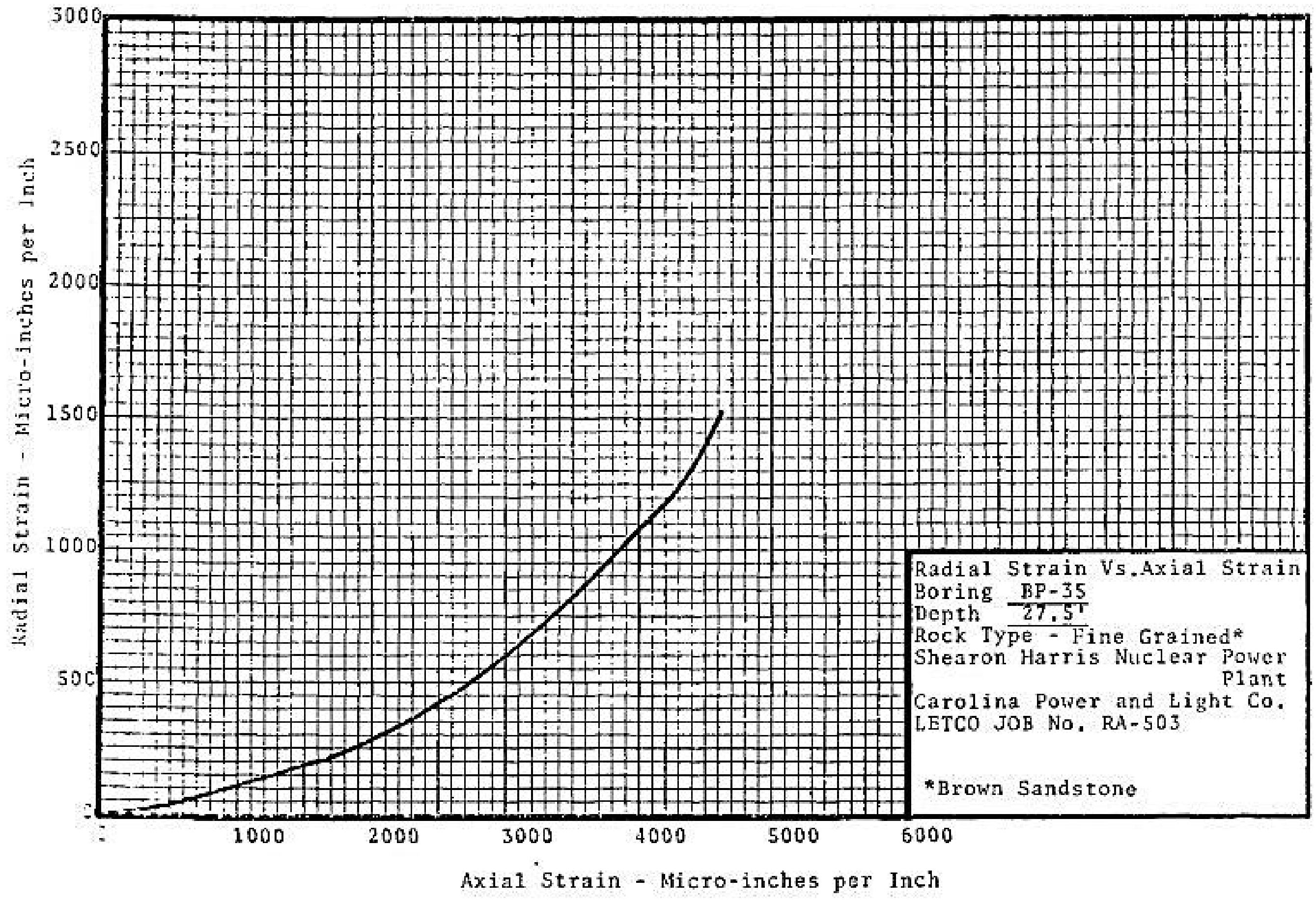


FIGURE 2.5.4-71

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-35

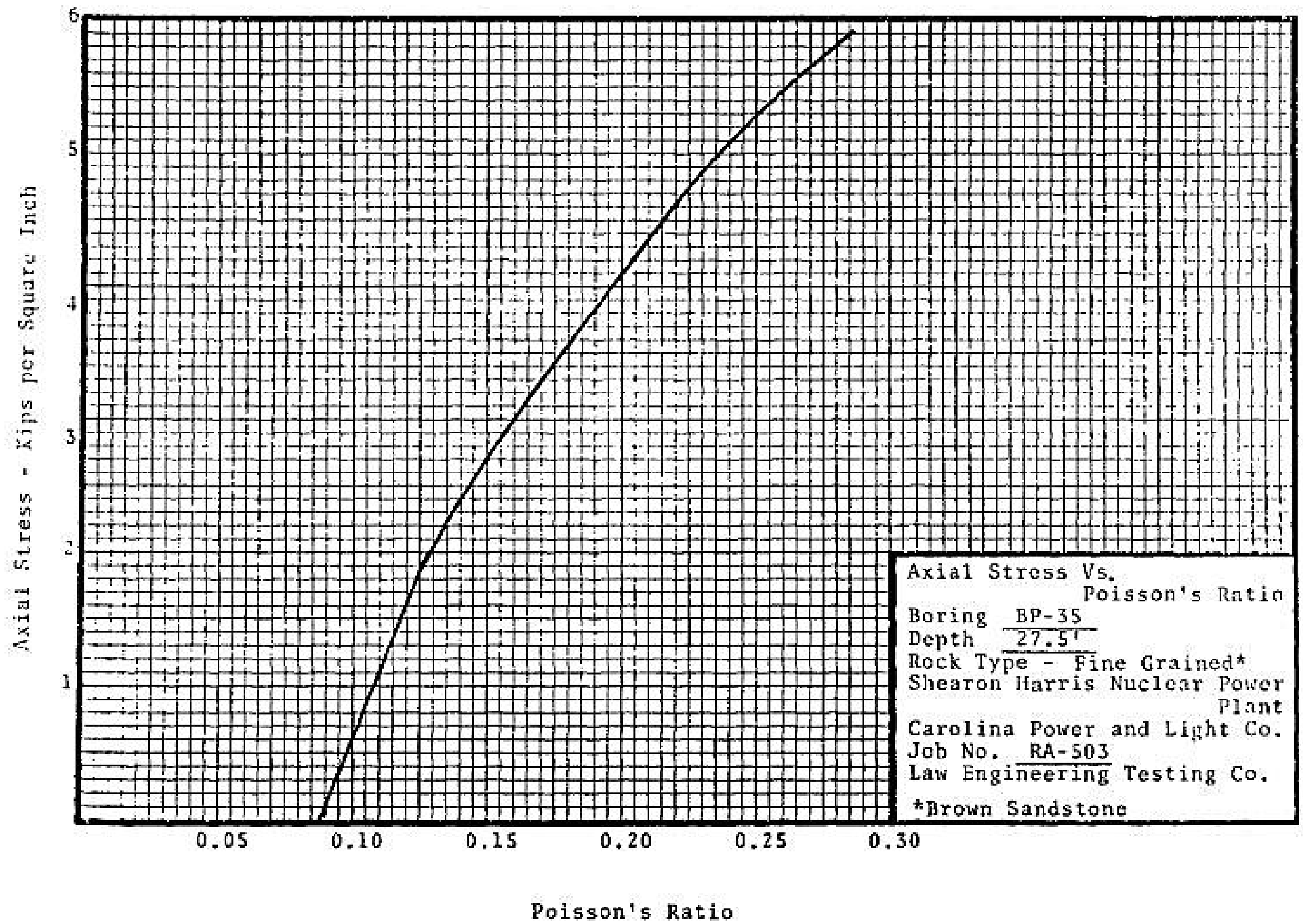


FIGURE 2.5.4-72

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-46

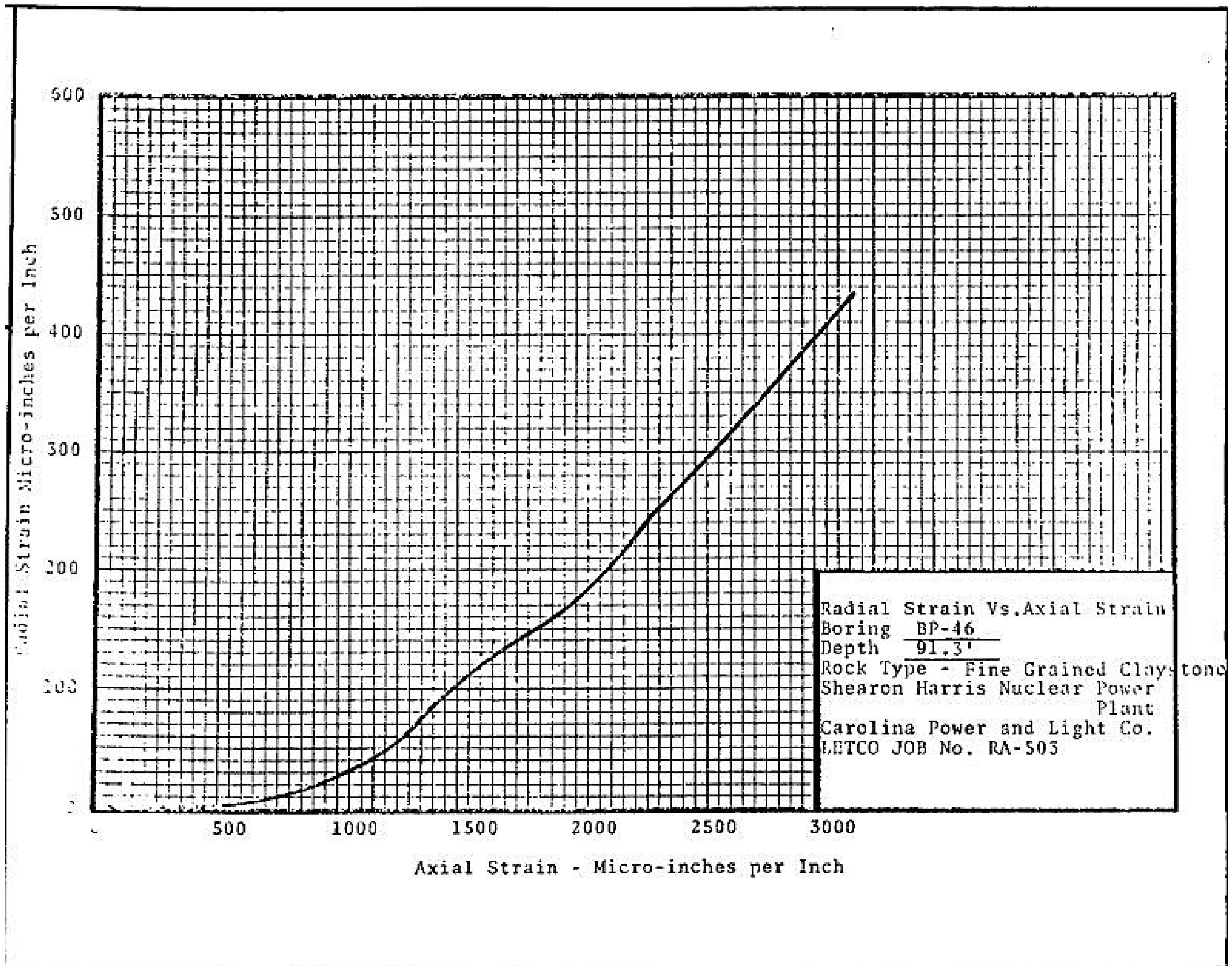


FIGURE 2.5.4-73

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-46

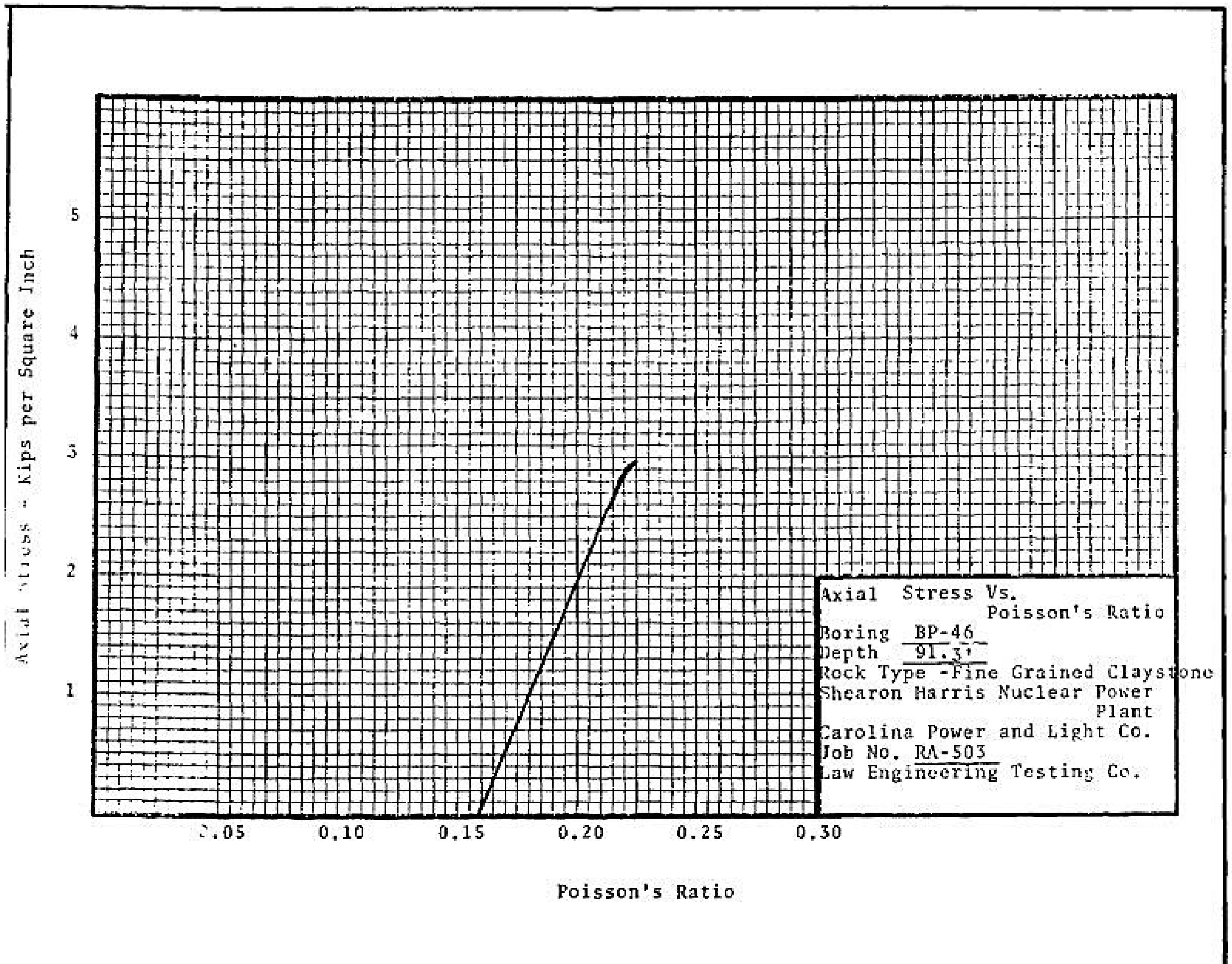


FIGURE 2.5.4-74

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-58

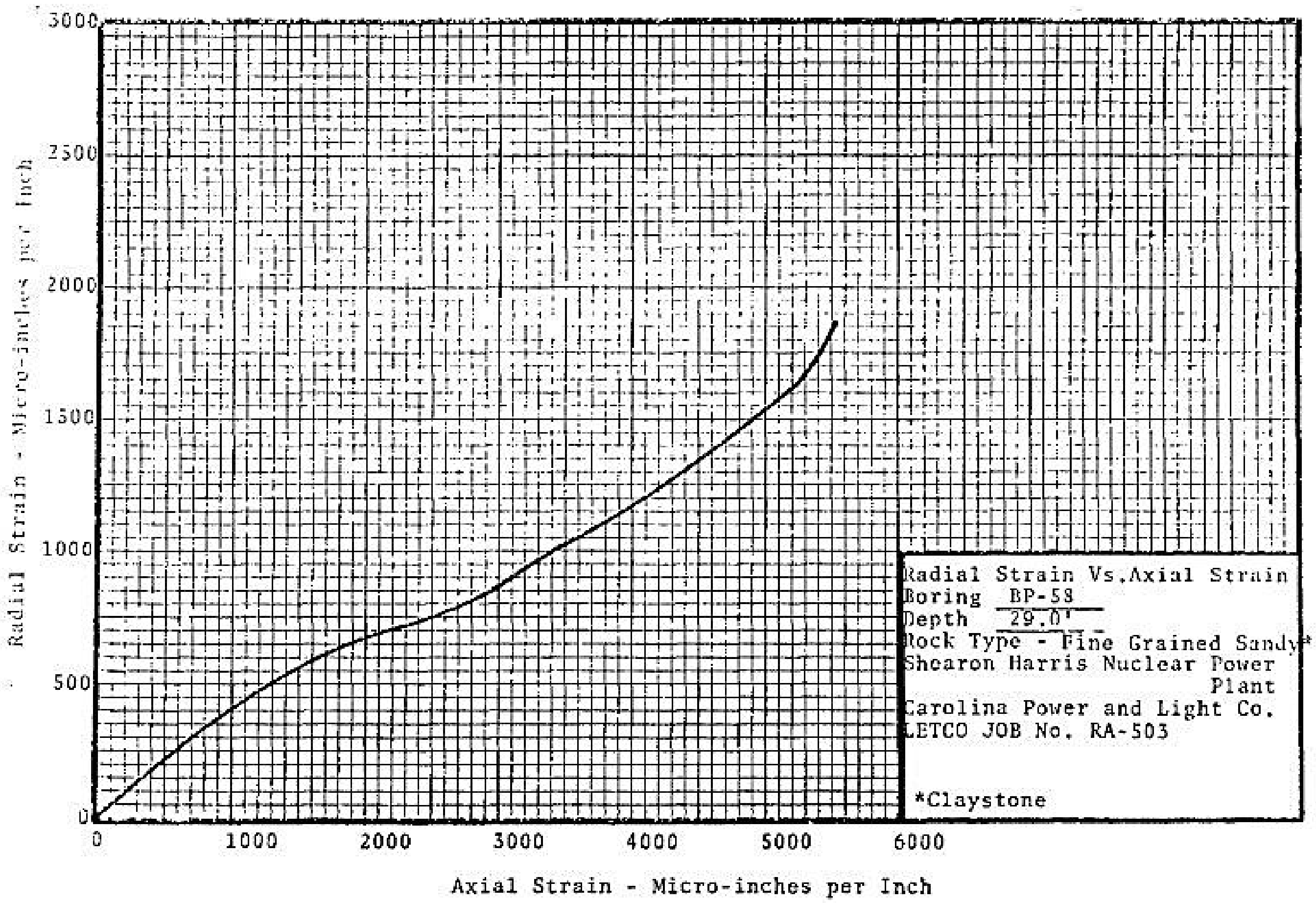


FIGURE 2.5.4-75

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-58

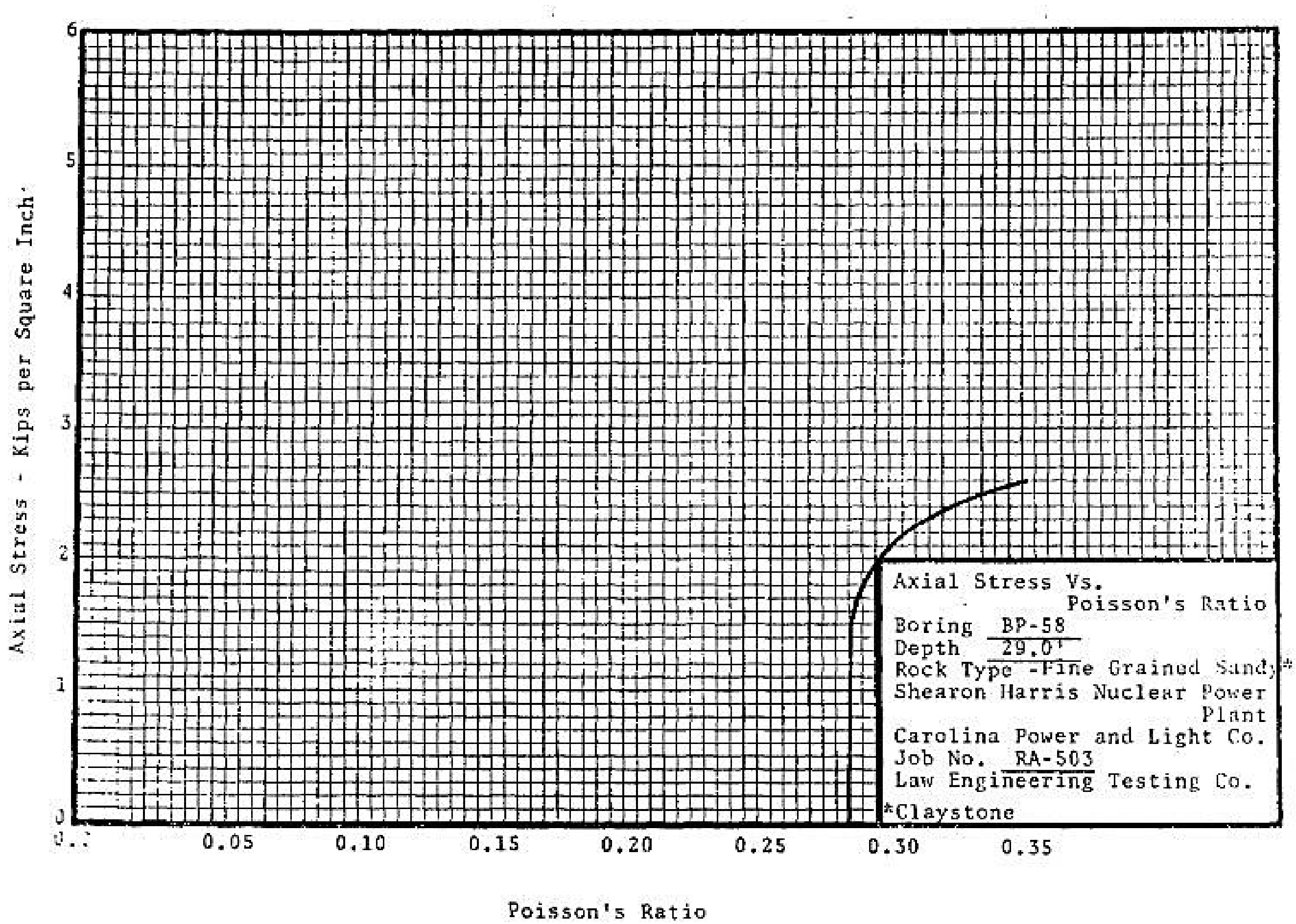


FIGURE 2.5.4-76

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-62

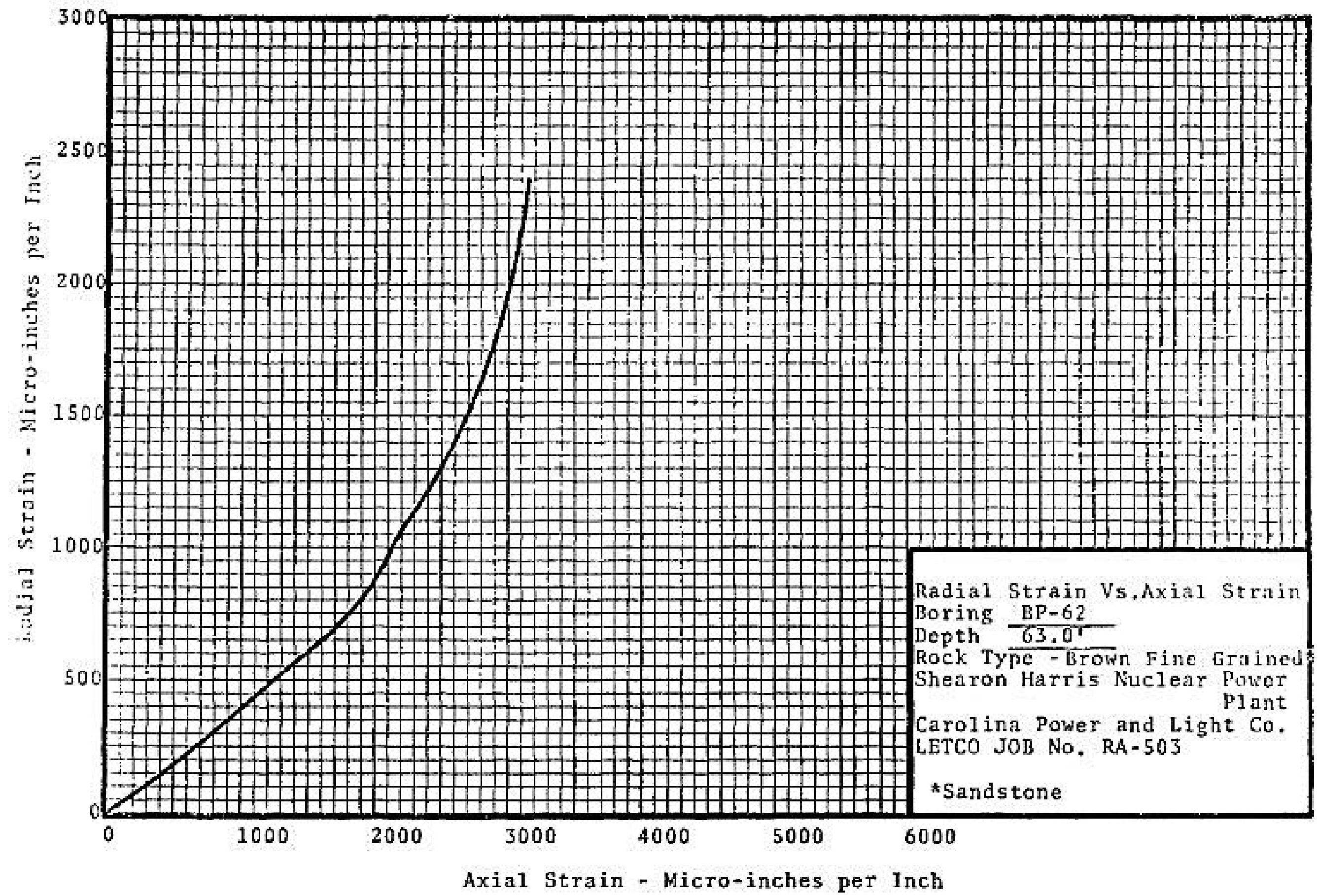


FIGURE 2.5.4-77

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-62

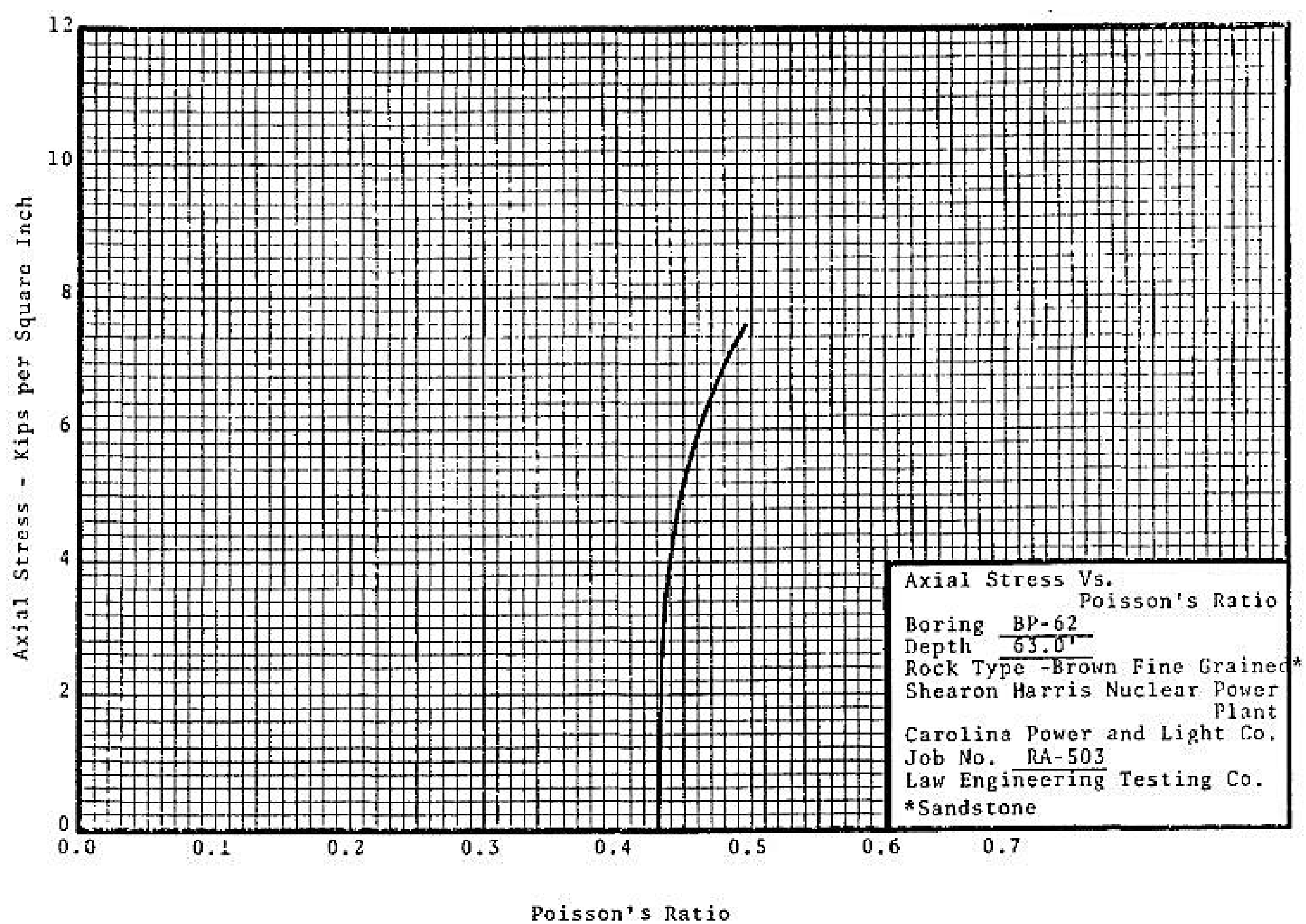


FIGURE 2.5.4-78

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-63

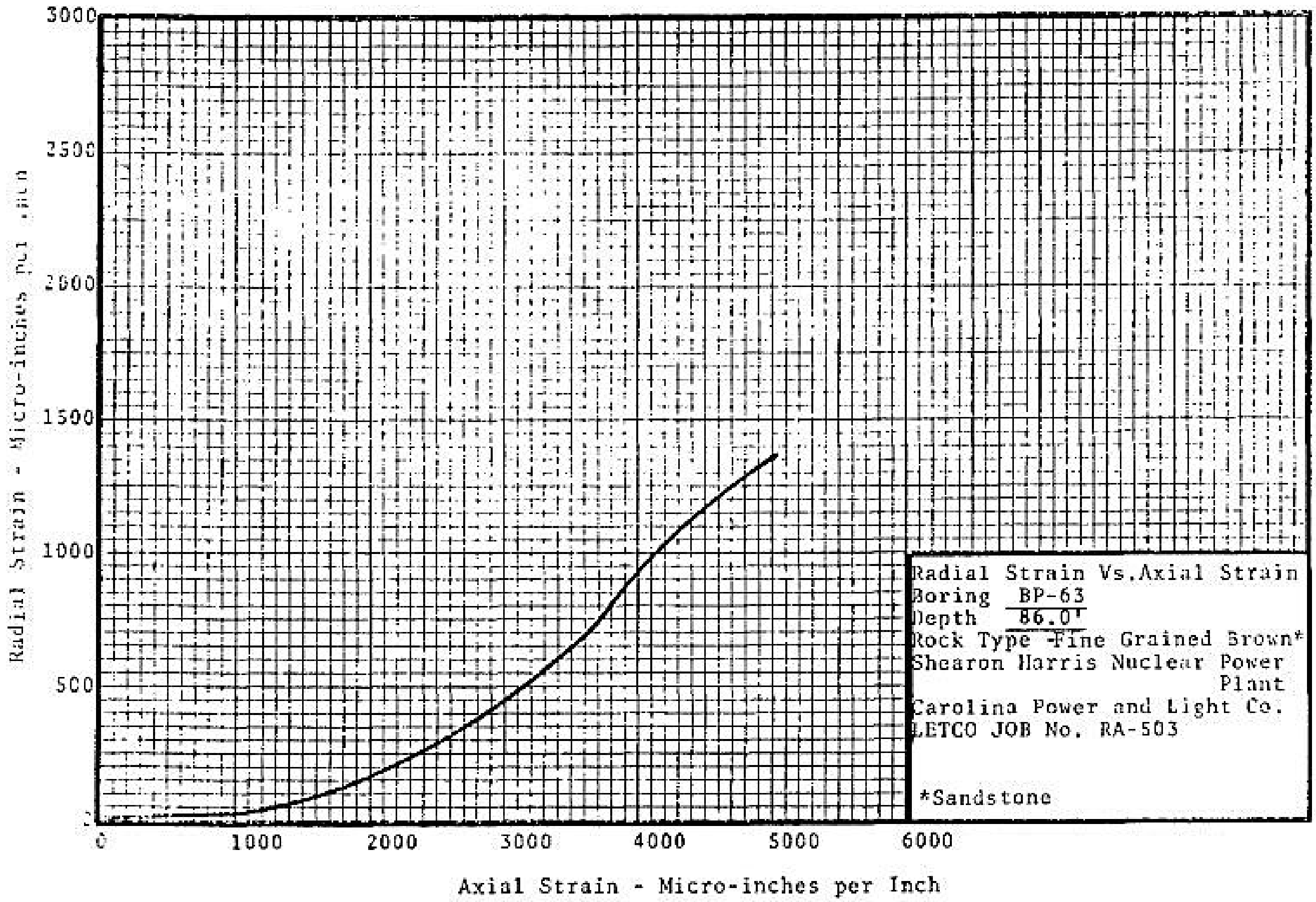


FIGURE 2.5.4-79

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-63

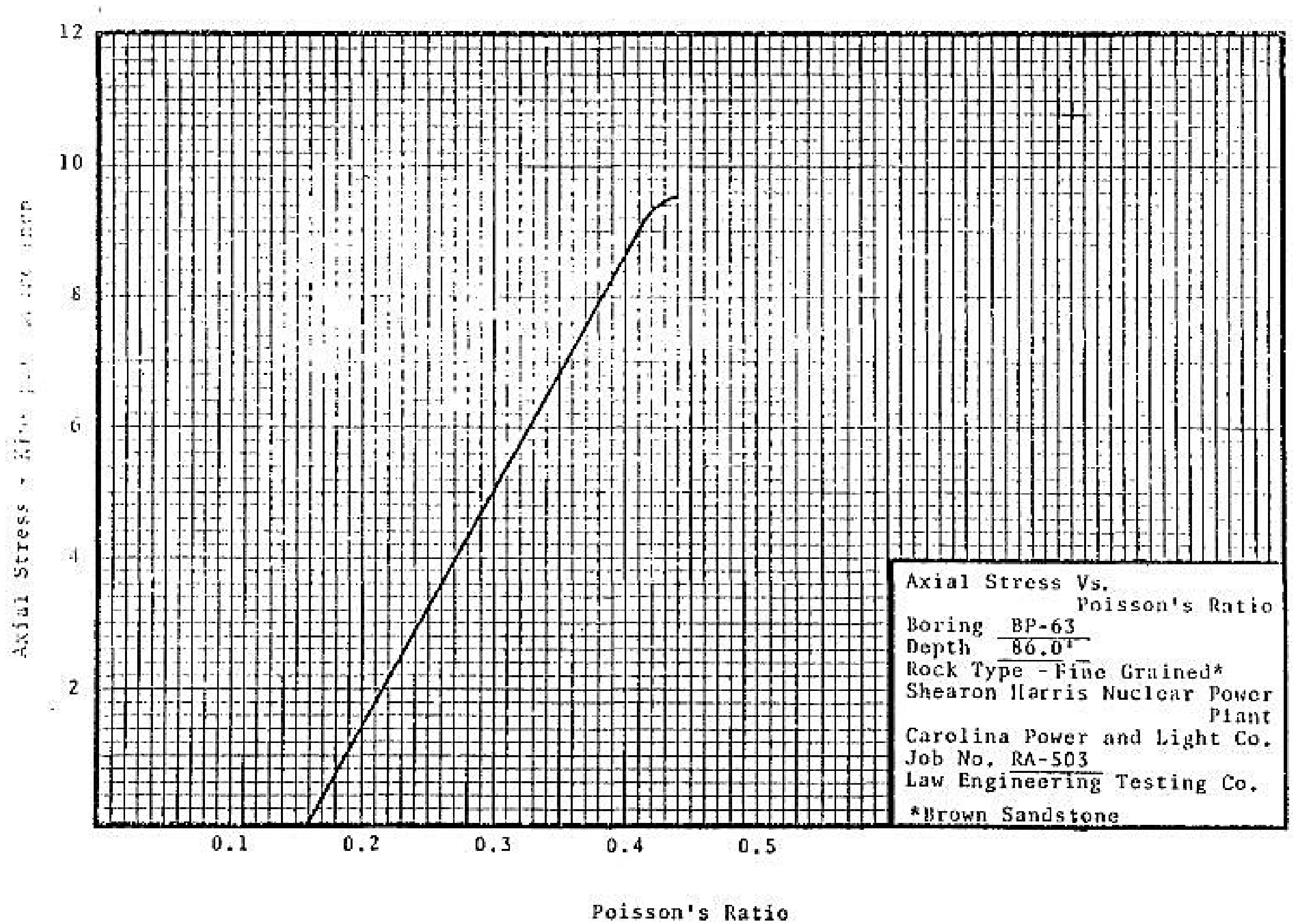


FIGURE 2.5.4-80

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-64

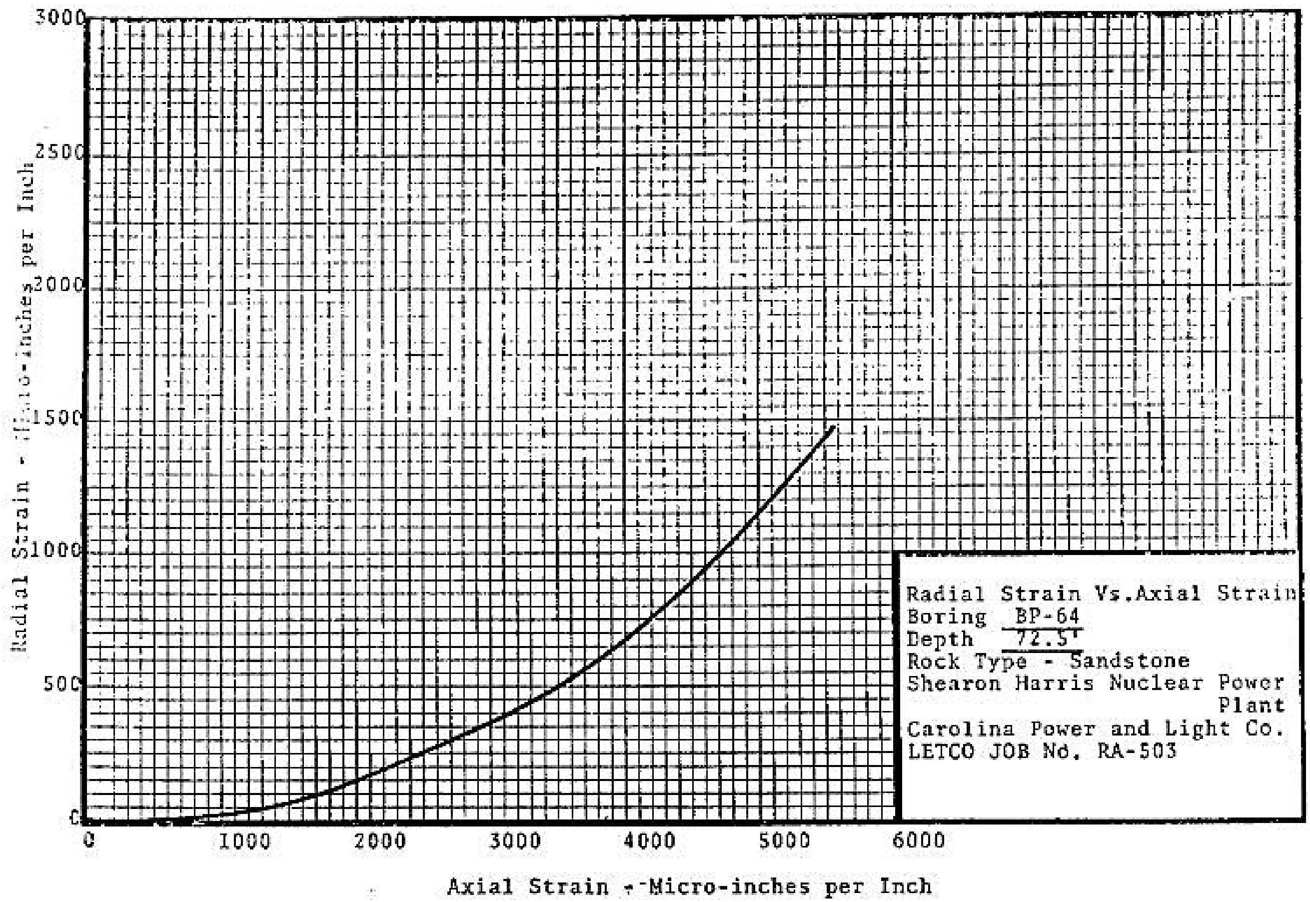


FIGURE 2.5.4-81

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-64

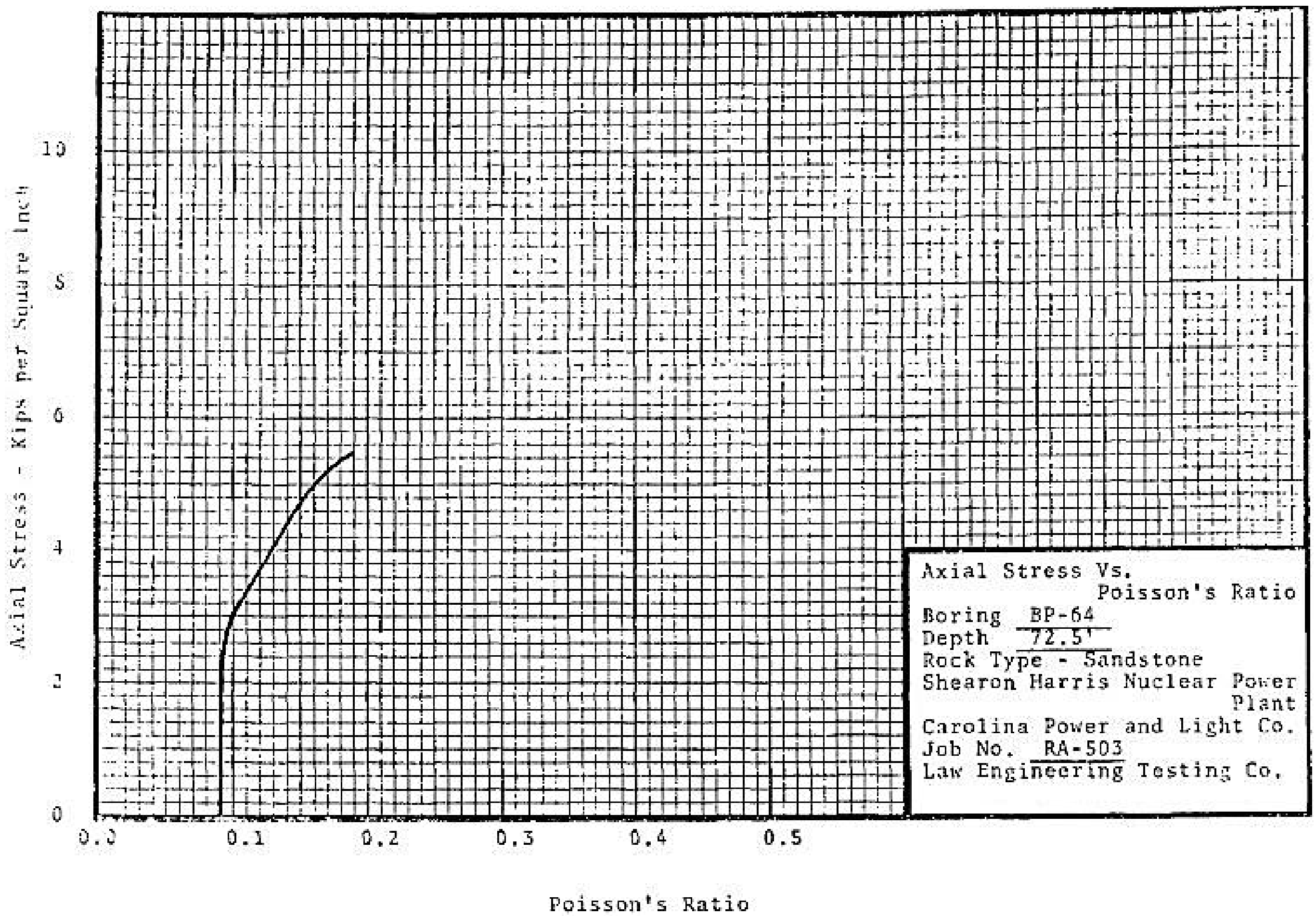


FIGURE 2.5.4-82

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-66

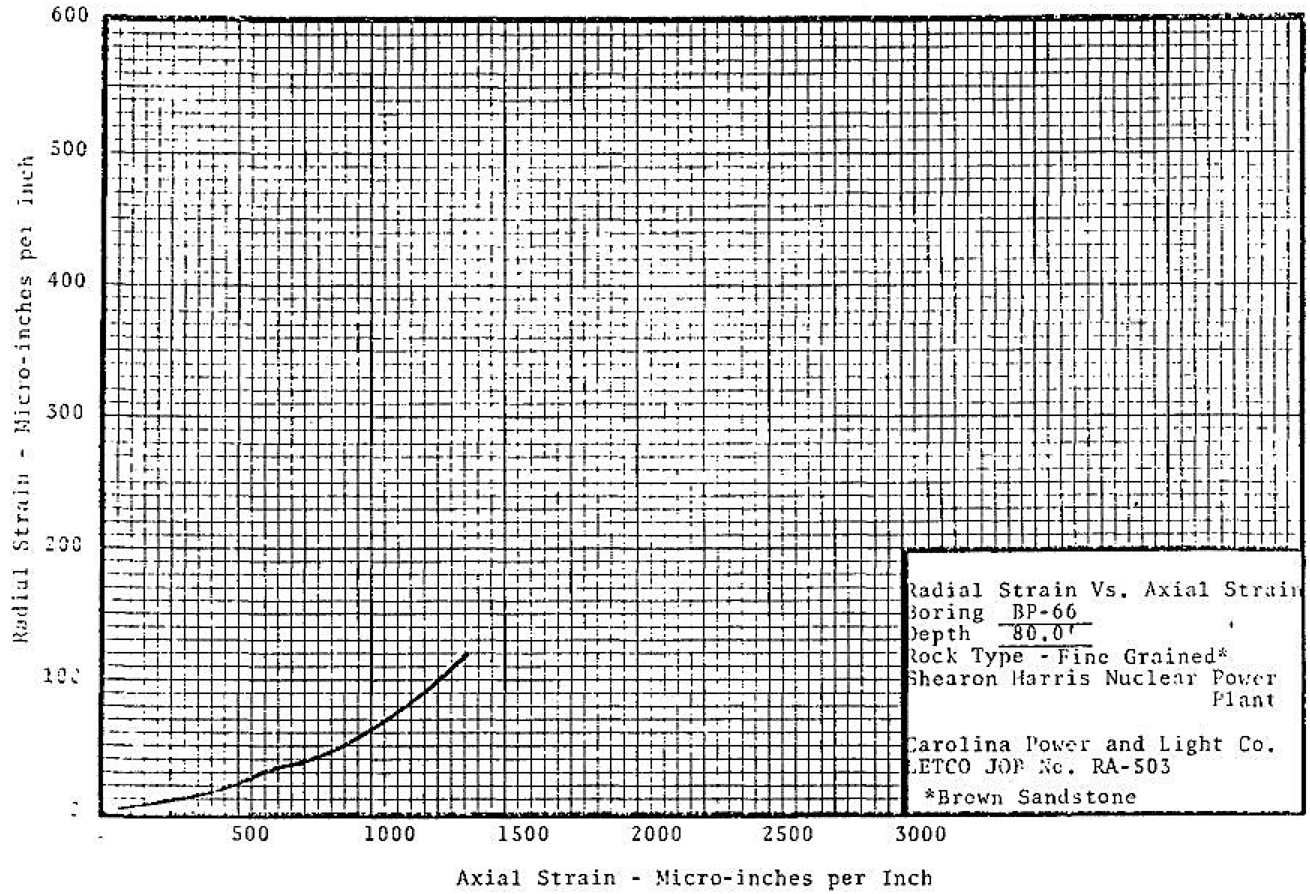


FIGURE 2.5.4-83

AXIAL STRESS VX. POISSON'S RATIO, BORING NO. BP-66

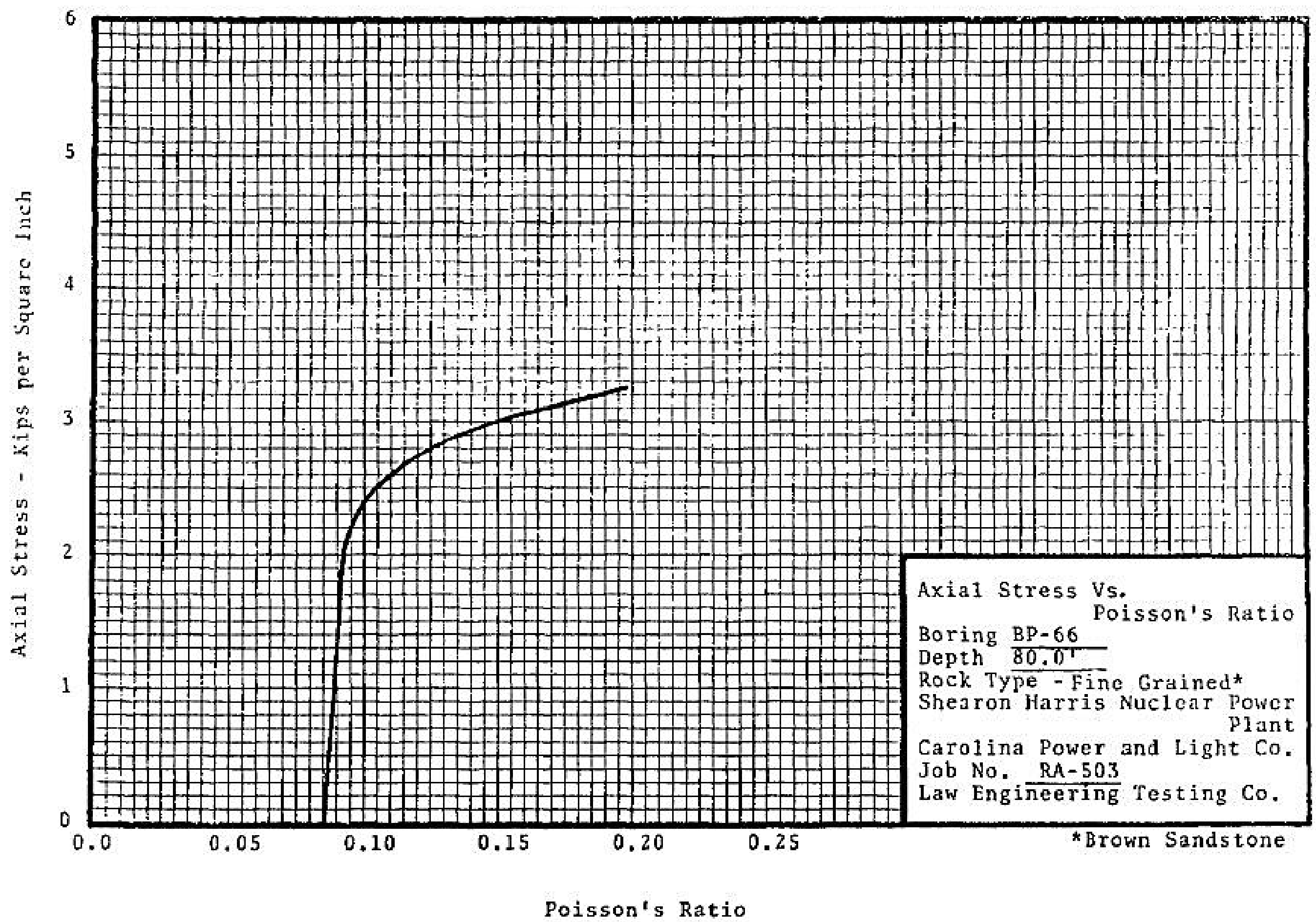


FIGURE 2.5.4-84

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-68

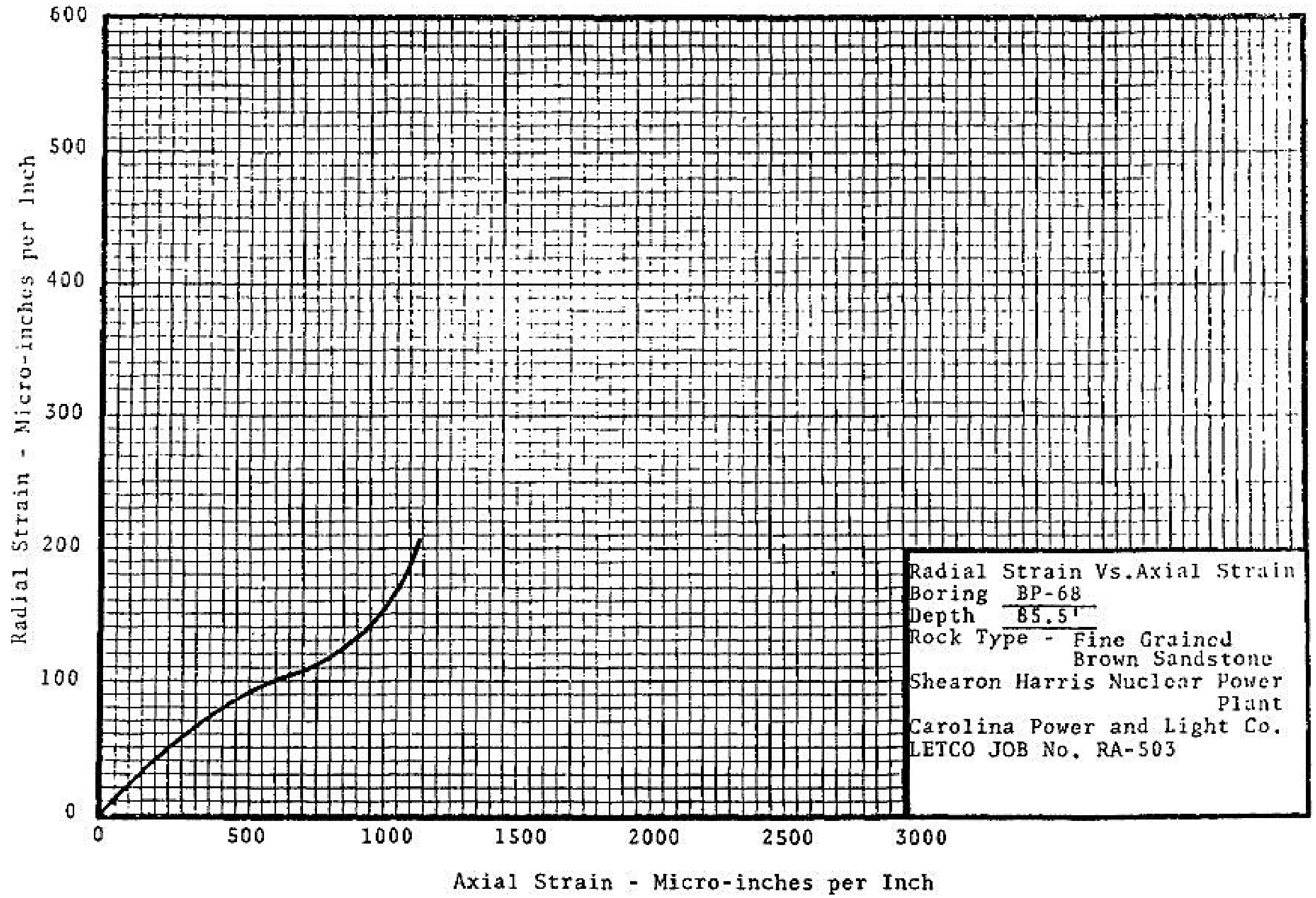


FIGURE 2.5.4-85

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-68

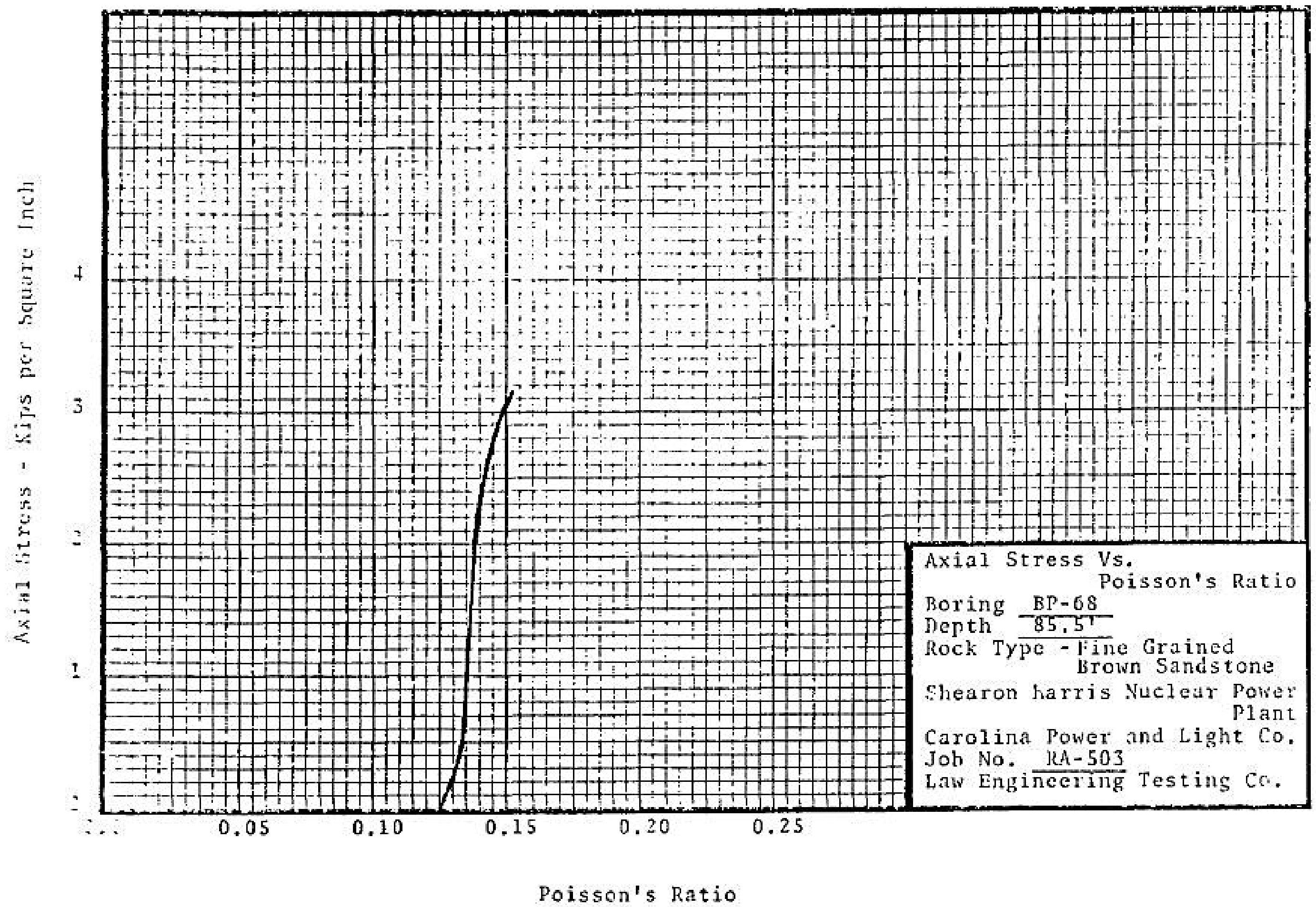


FIGURE 2.5.4-86

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-69

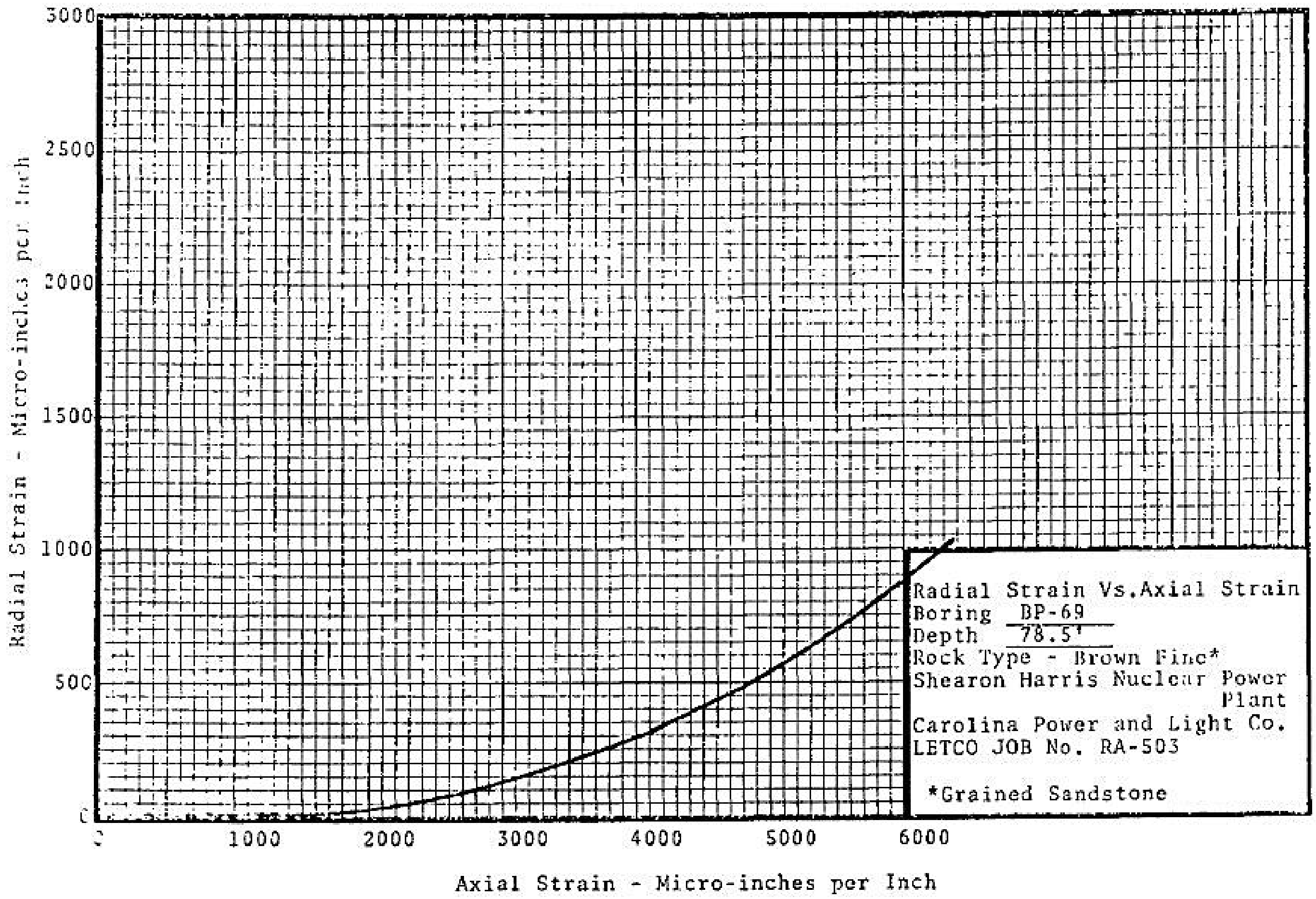


FIGURE 2.5.4-87

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-69

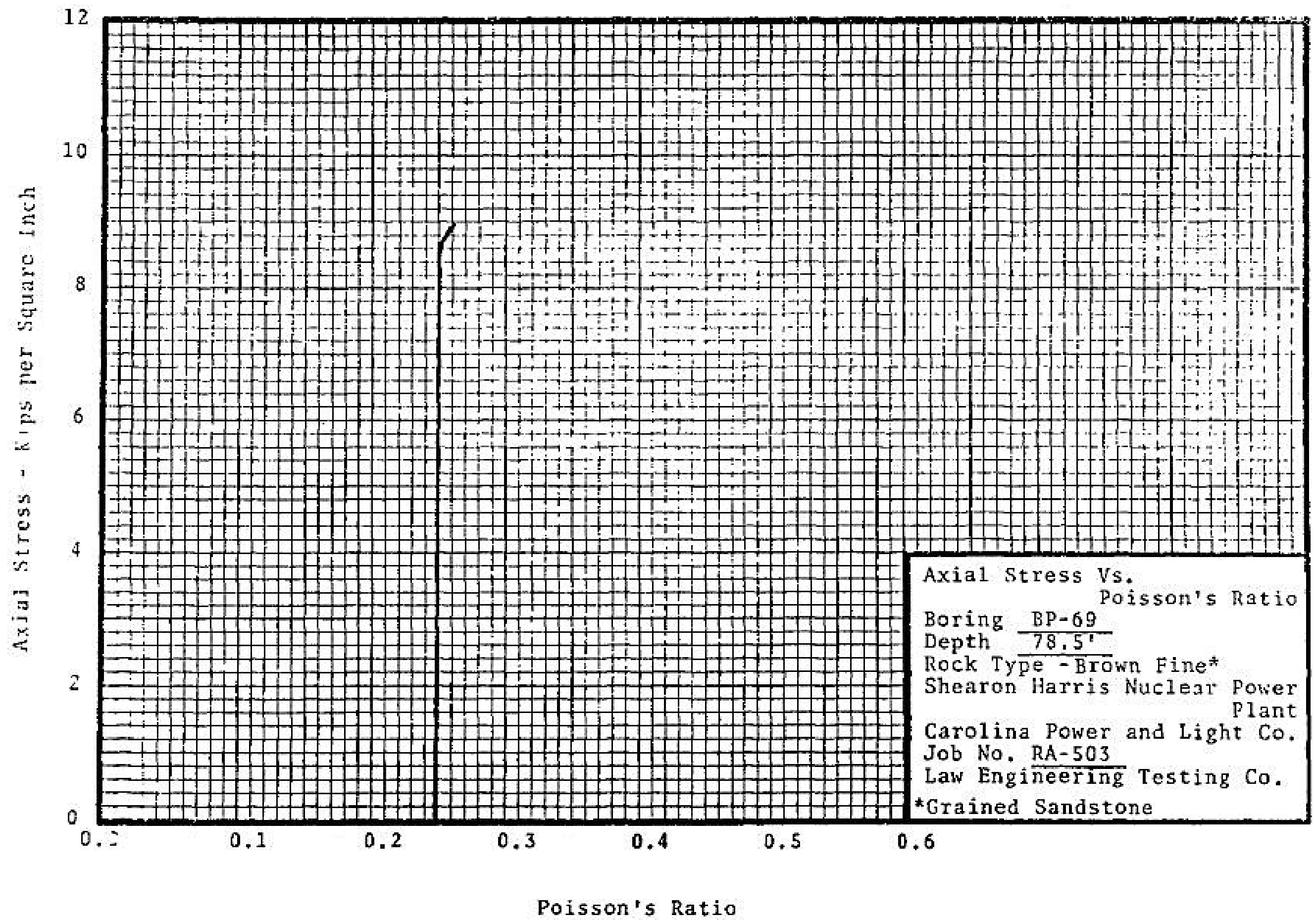


FIGURE 2.5.4-88

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-70

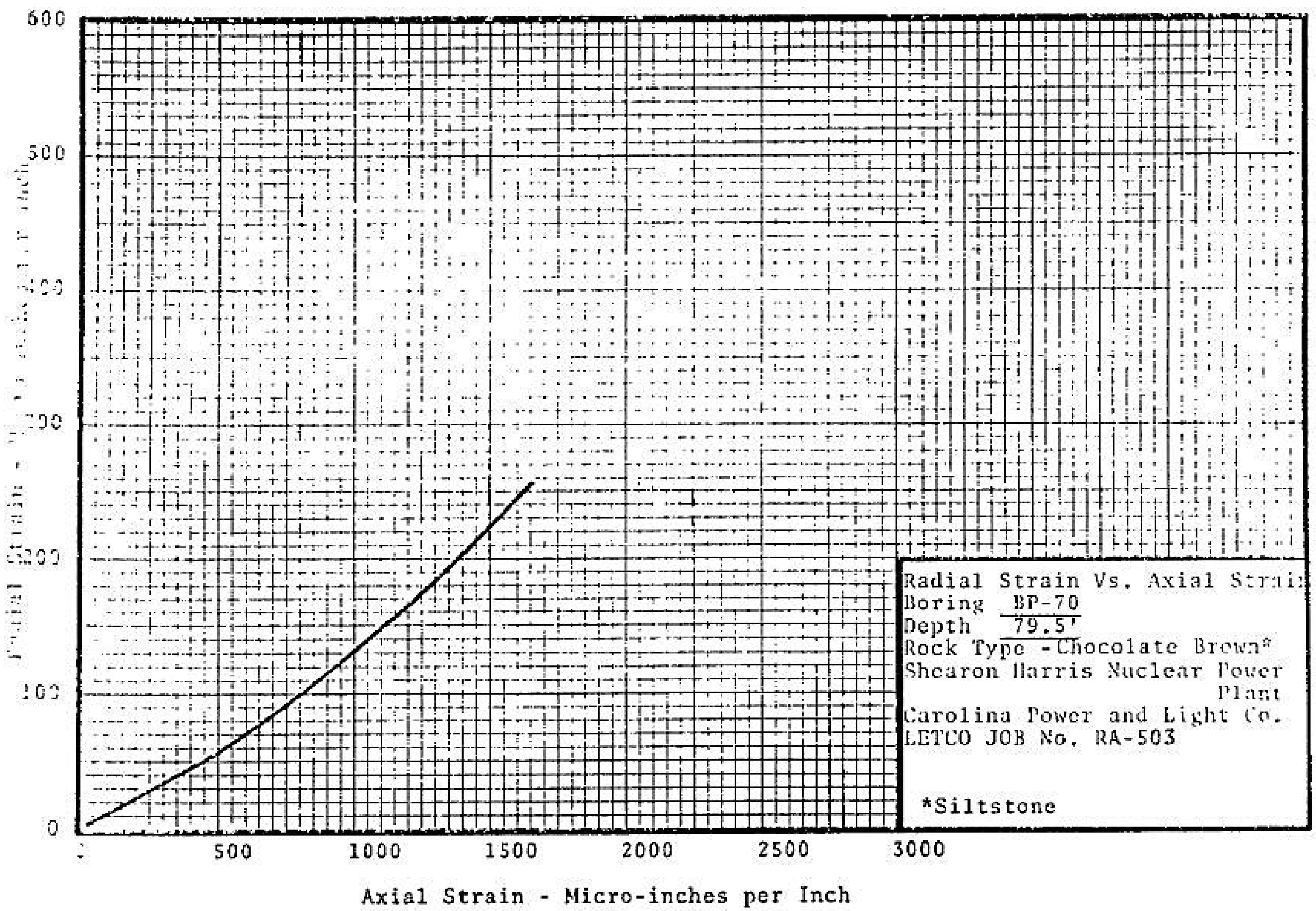


FIGURE 2.5.4-89

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-70

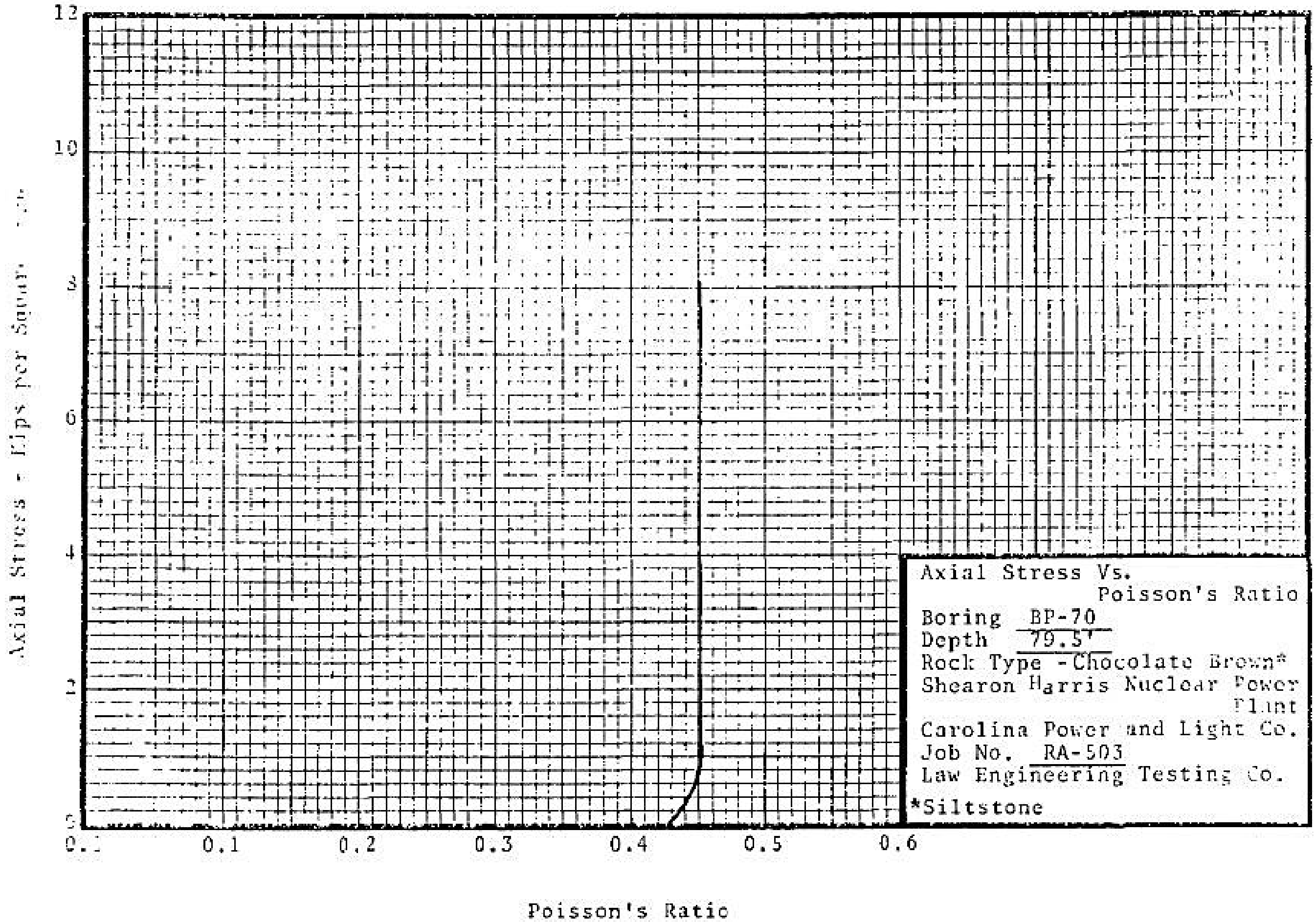


FIGURE 2.5.4-90

RADIAL STRAIN VS. AXIAL STRAIN, BORING NO. BP-74

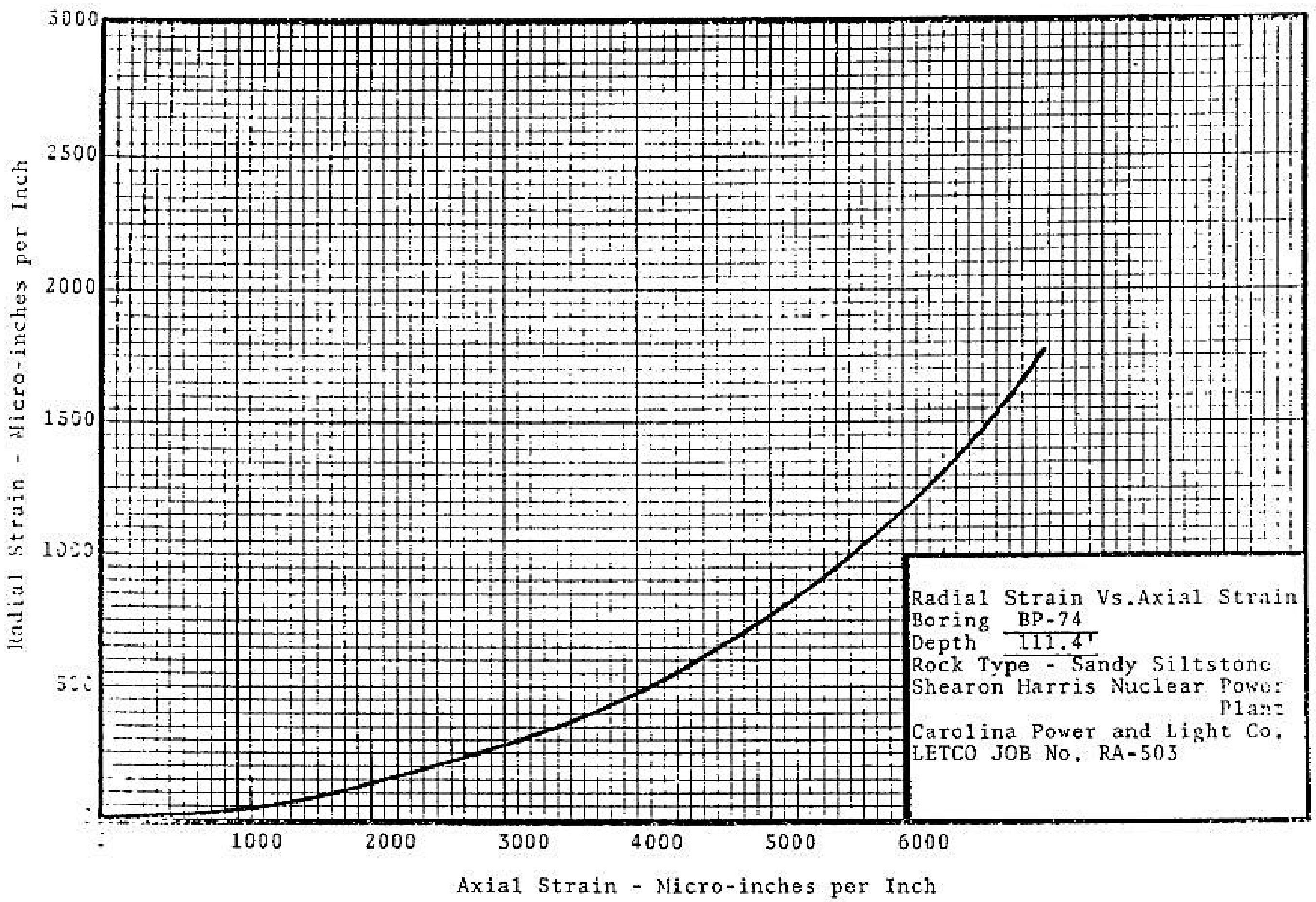


FIGURE 2.5.4-91

AXIAL STRESS VS. POISSON'S RATIO, BORING NO. BP-74

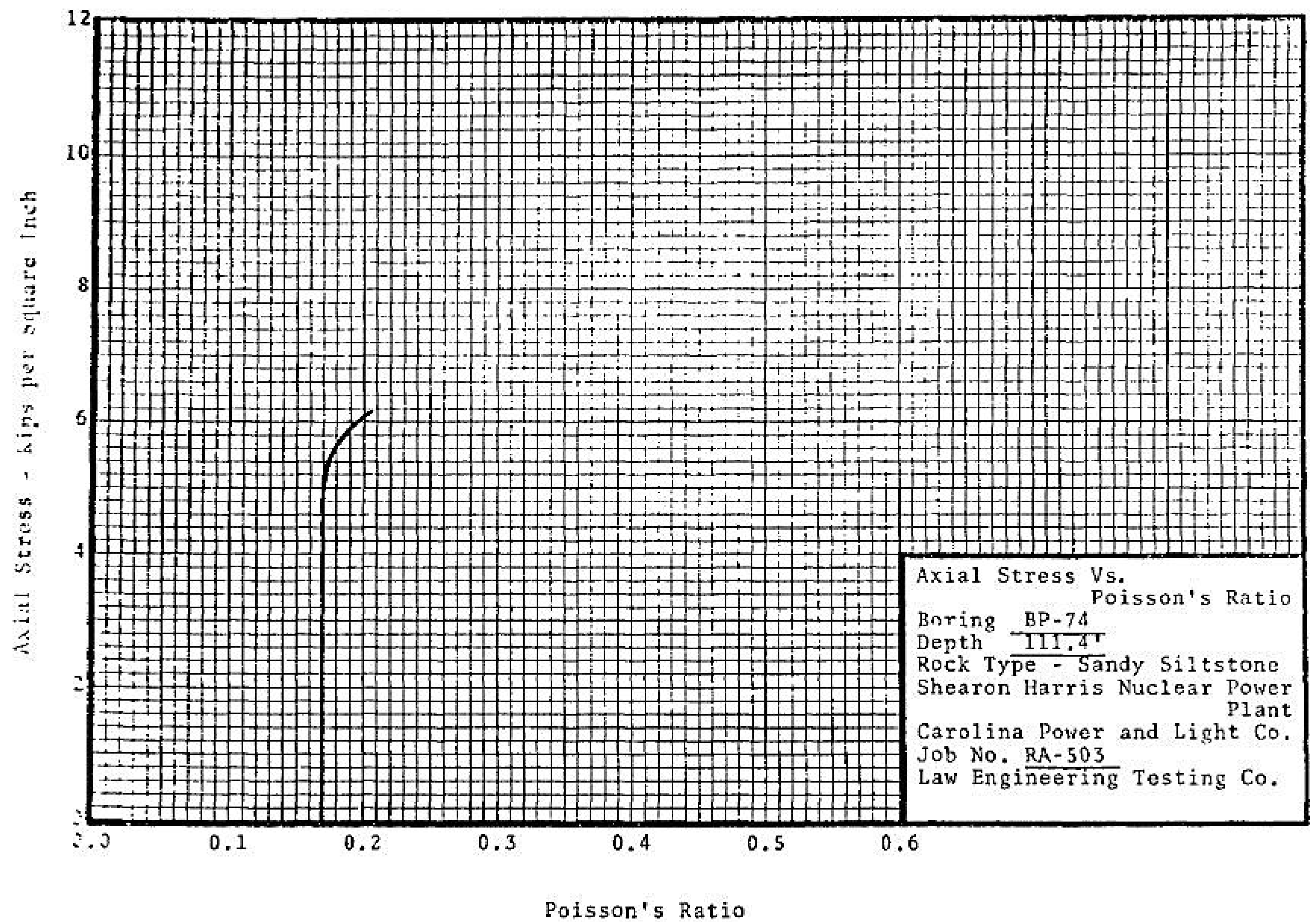


FIGURE 2.5.4-92

TRENCH CROSS SECTIONS, TRENCH NO. 1

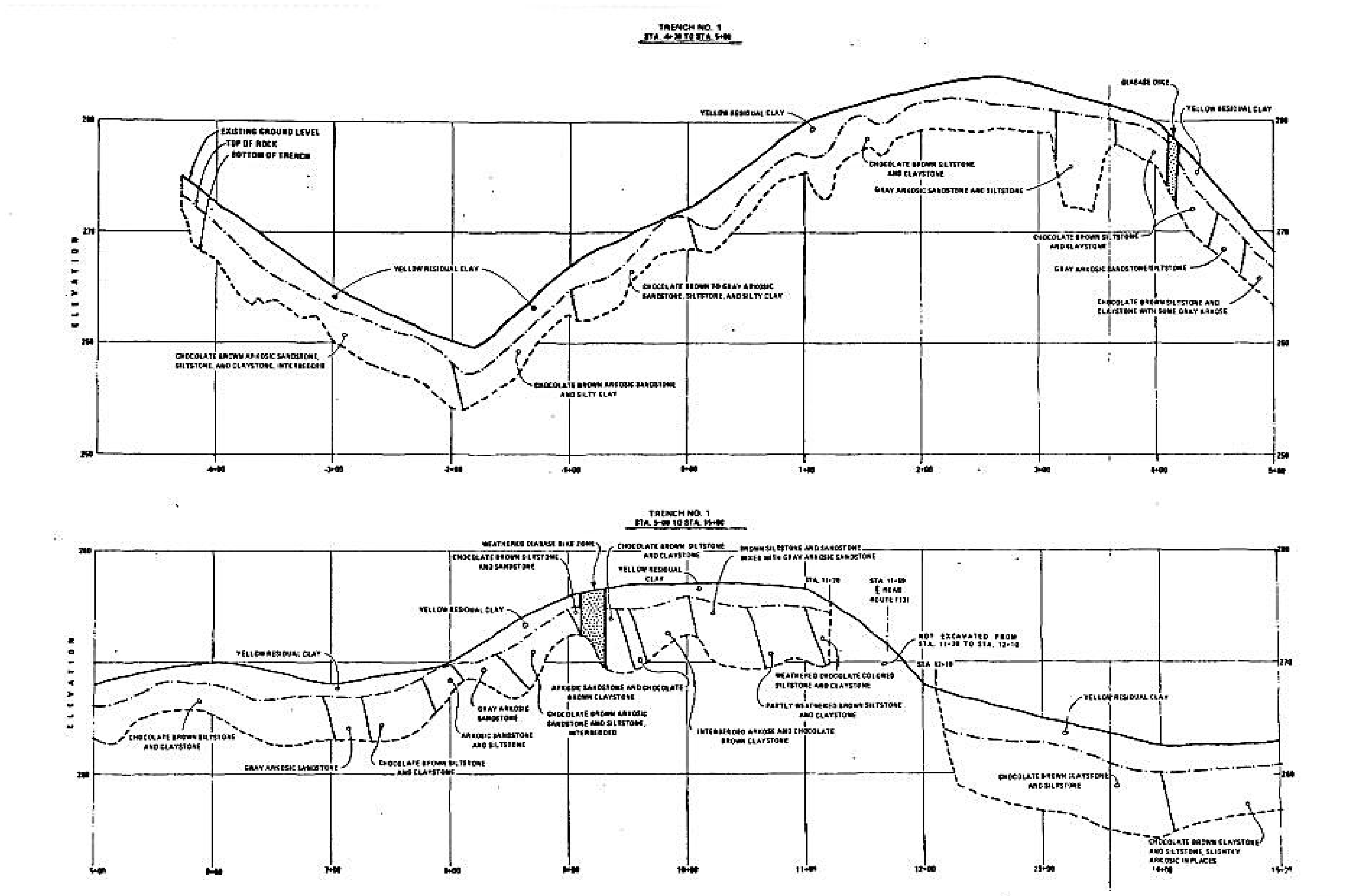


FIGURE 2.5.4-93

TRENCH CROSS SECTIONS, TRENCH NO. 1

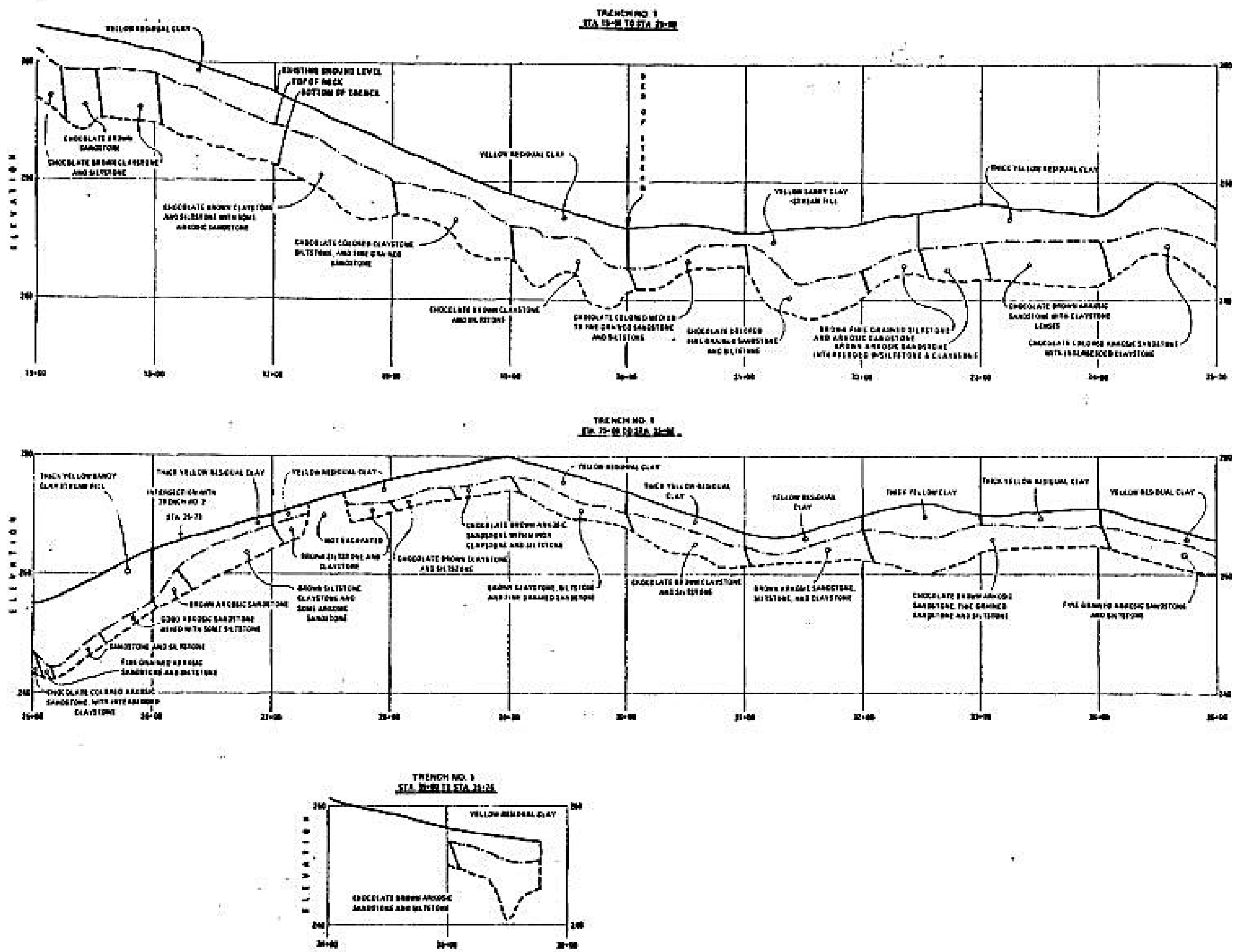


FIGURE 2.5.4-94

TRENCH CROSS SECTIONS, TRENCH NO. 2

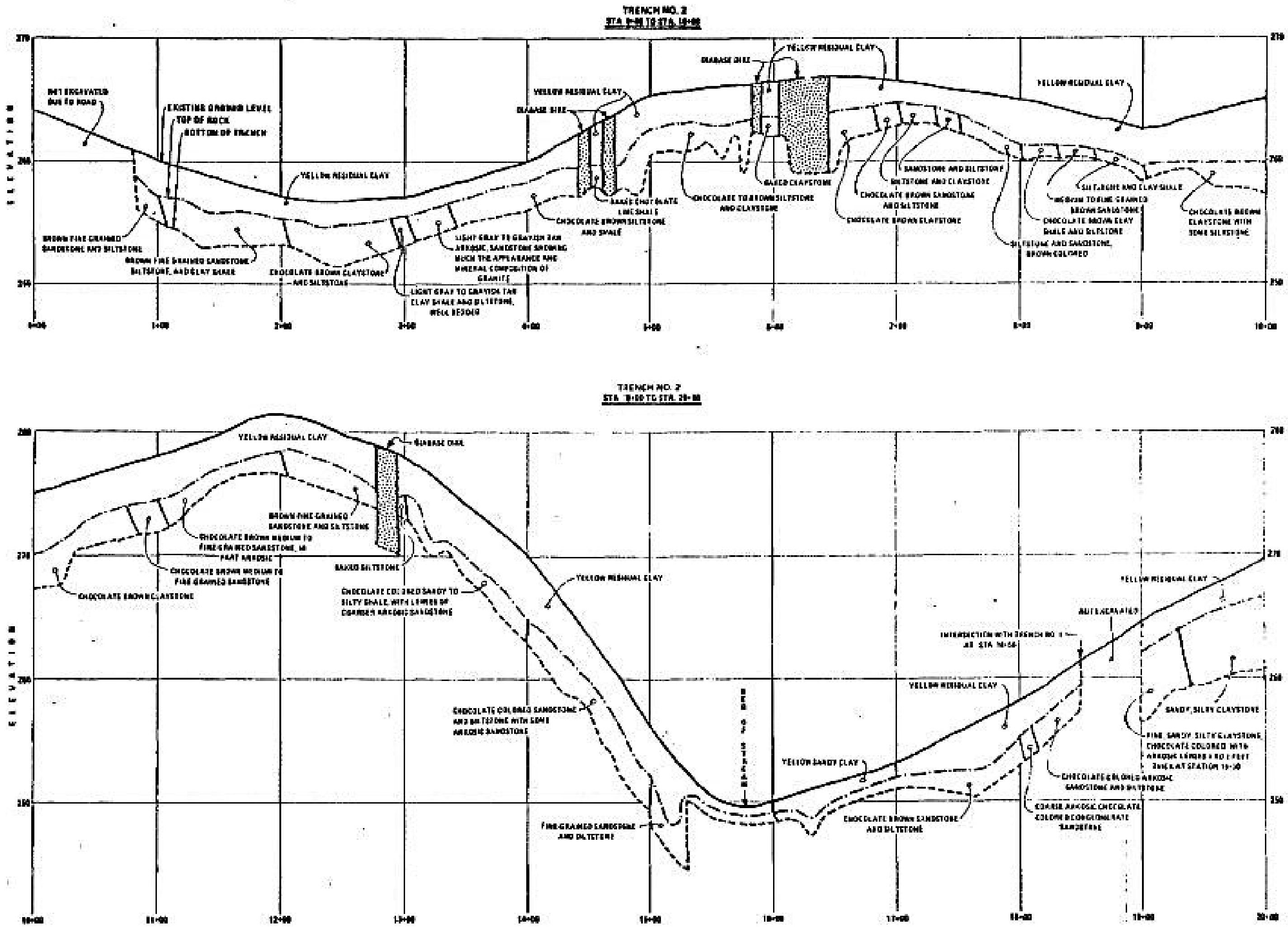


FIGURE 2.5.4-95

TRENCH CROSS SECTIONS, TRENCH NO. 2

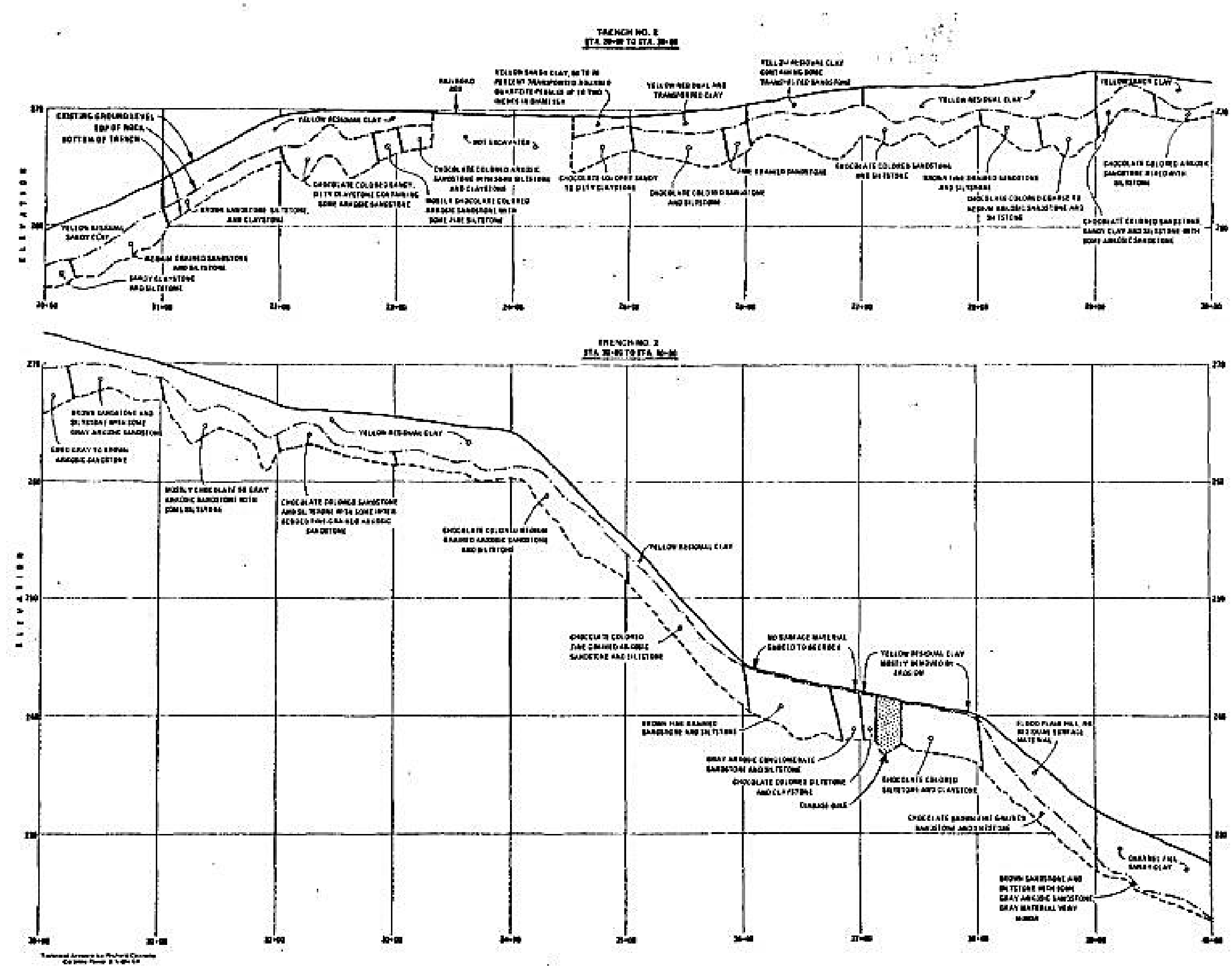


FIGURE 2.5.4-96

TRENCH CROSS SECTIONS, TRENCH NOS. 2 AND 3

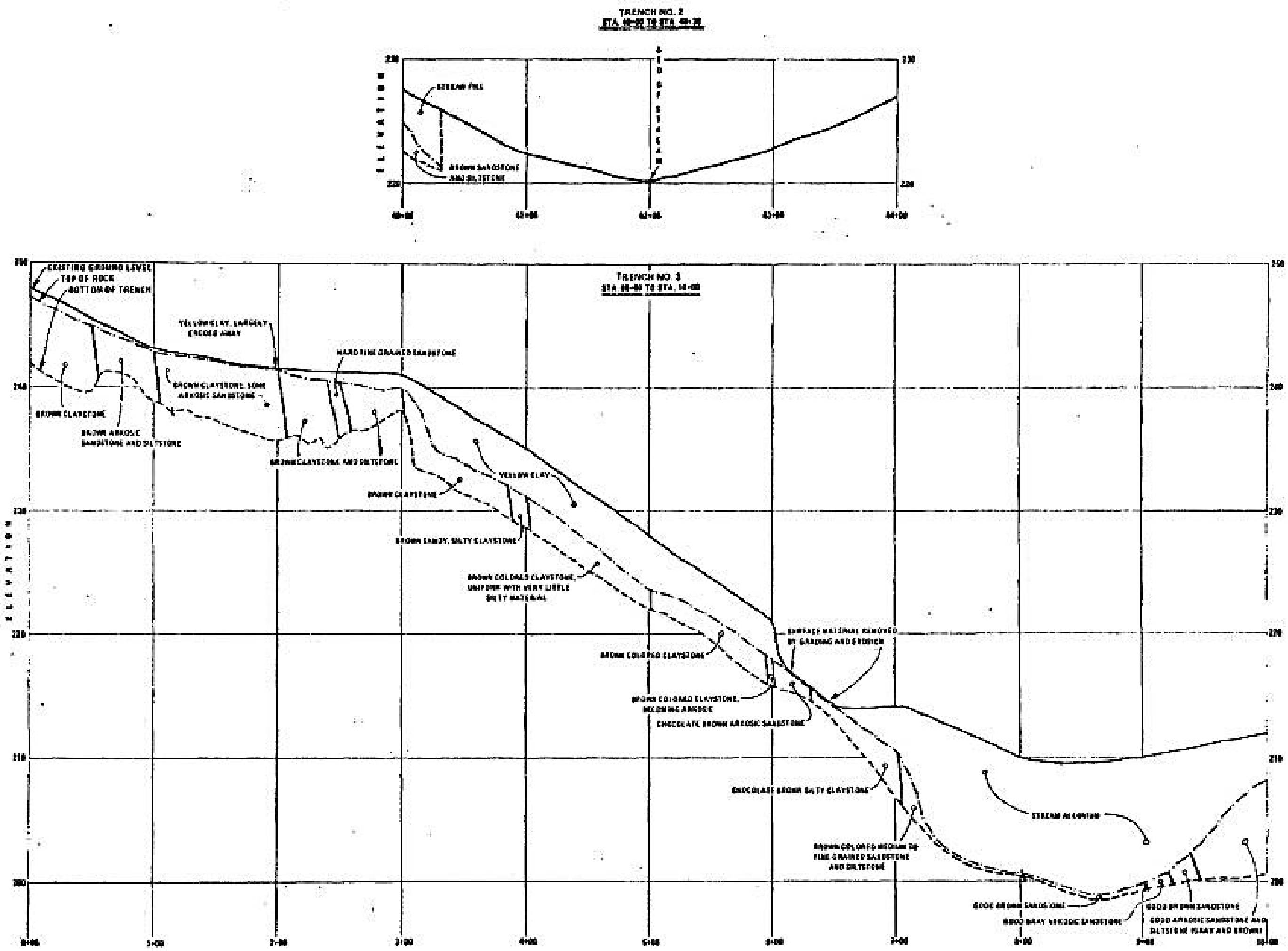


FIGURE 2.5.4-97

TRENCH CROSS SECTIONS, TRENCH NO. 3

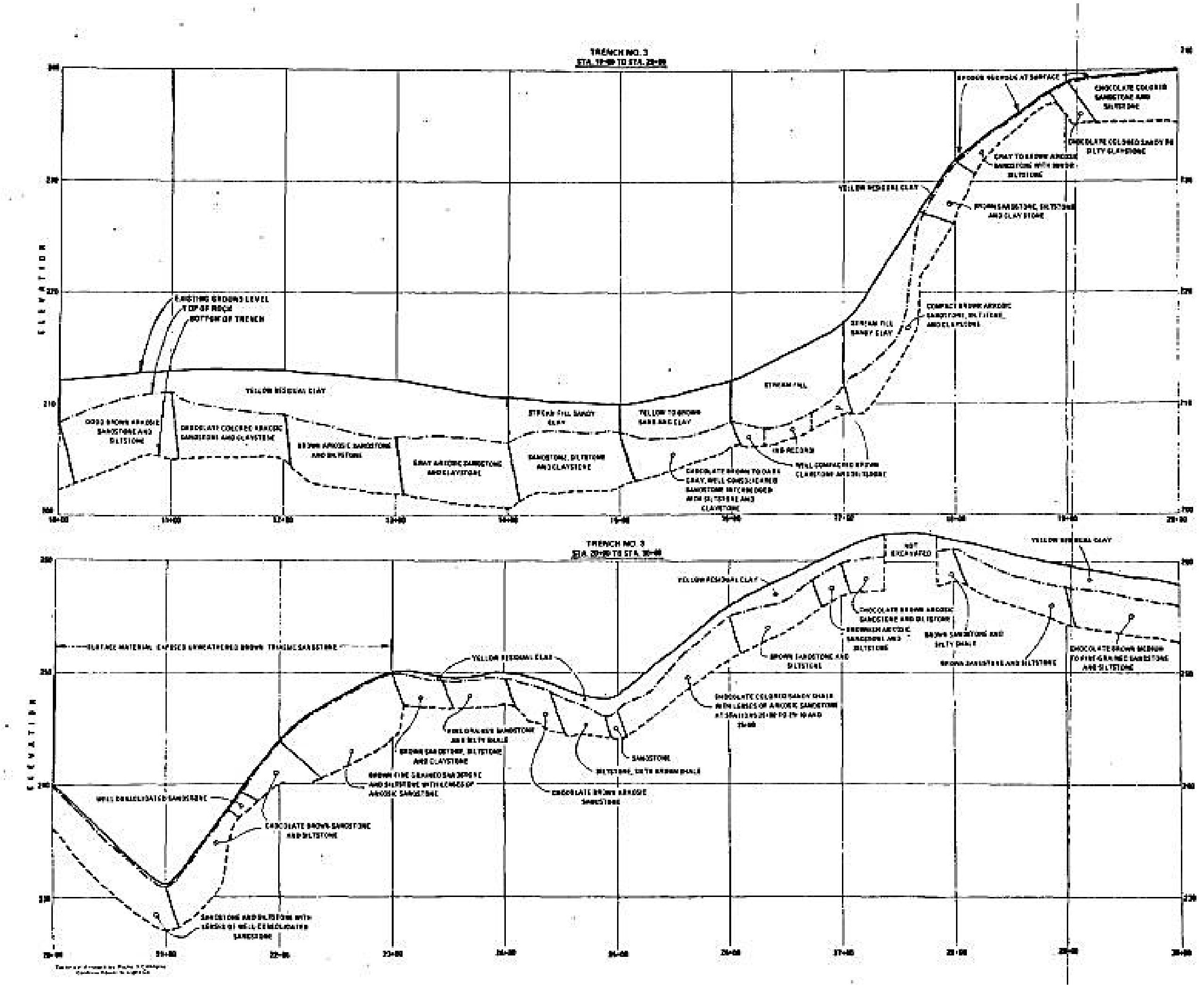


FIGURE 2.5.4-98

TRENCH CROSS SECTIONS, TRENCH NOS. 3 AND 4

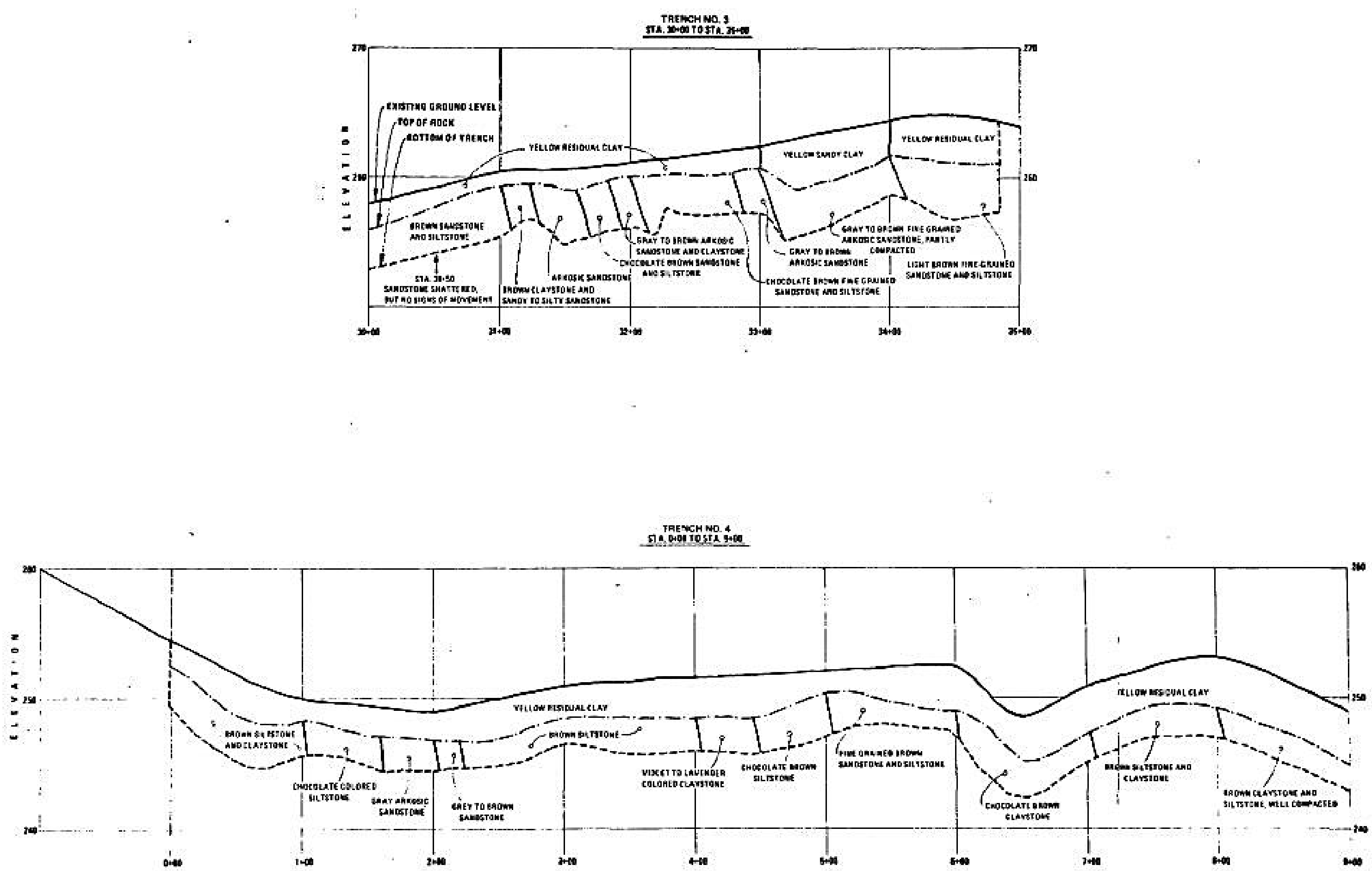
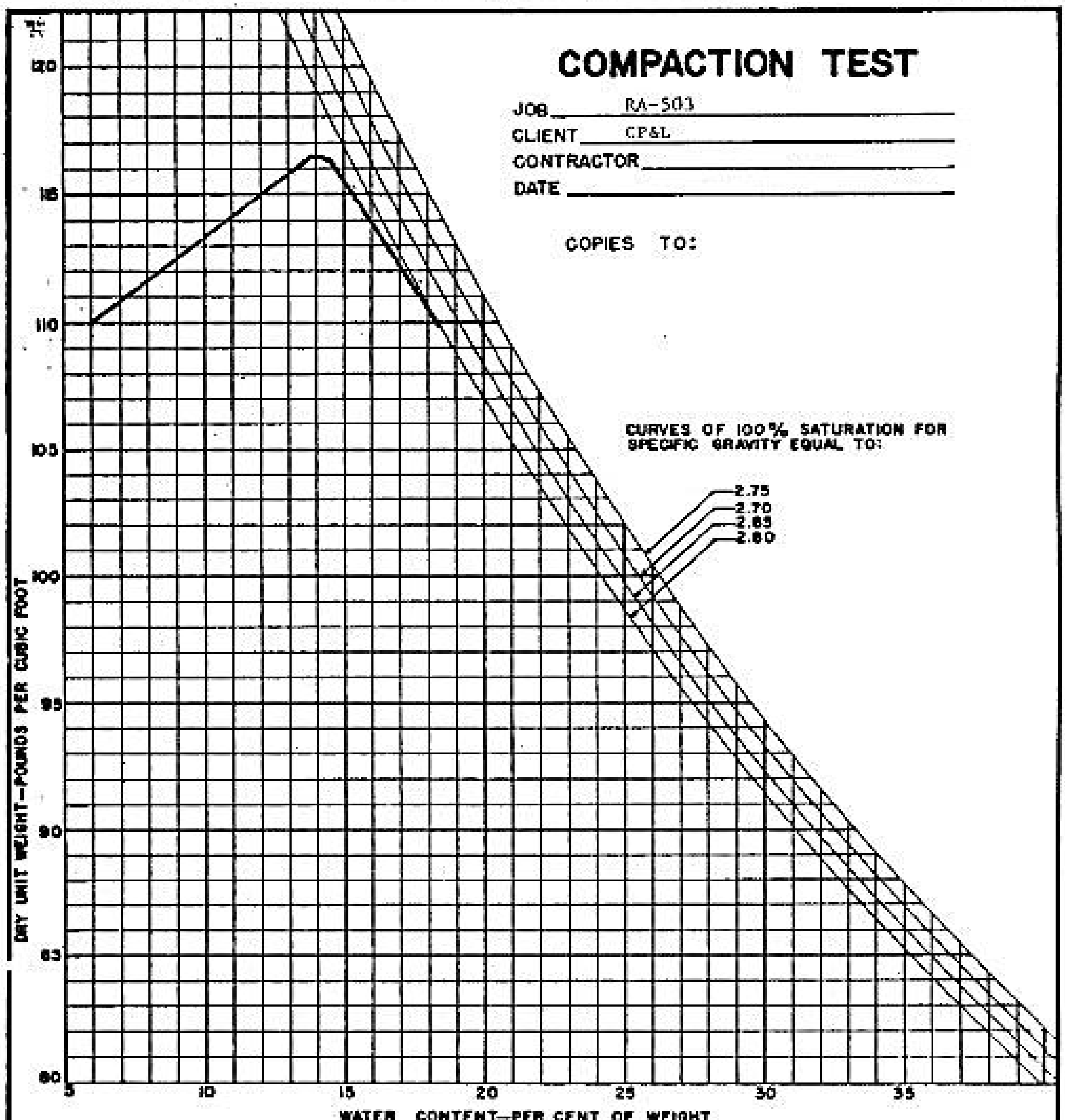


FIGURE 2.5.4-106

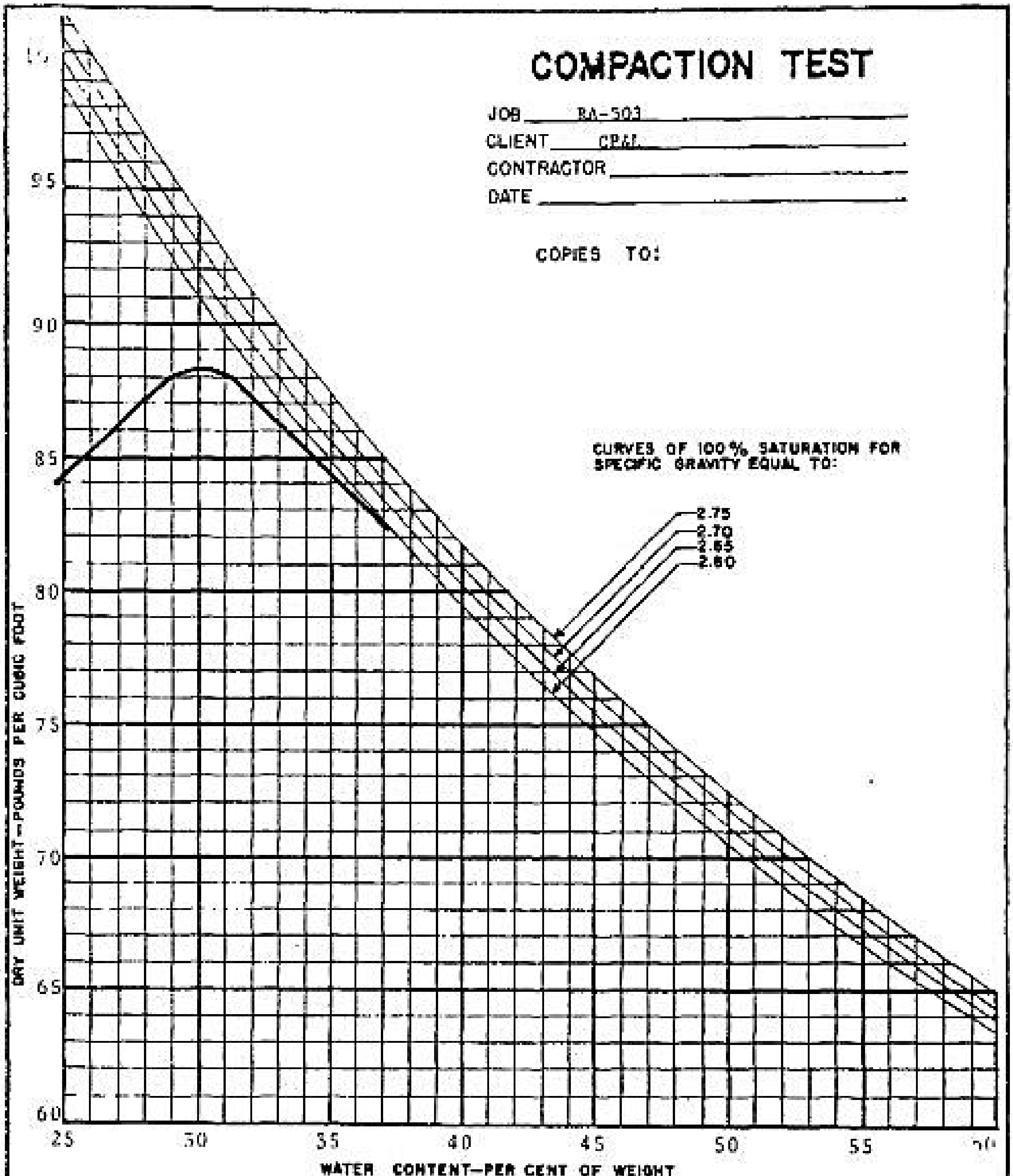
COMPACTION TEST, BORING NO. BC-153, S-3



MOISTURE DENSITY RELATION	METHOD OF TEST	MAX. DRY DENSITY PCF	OPTIMUM MOISTURE CONTENT %	SOIL DESCRIPTION OR CLASSIFICATION AND SAMPLE LOCATION
BC-153 S-3 8.0'-13.0'	STANDARD PROCTOR	116.5	14.2	RED BROWN FINE SANDY CLAYEY SILT

FIGURE 2.5.4-107

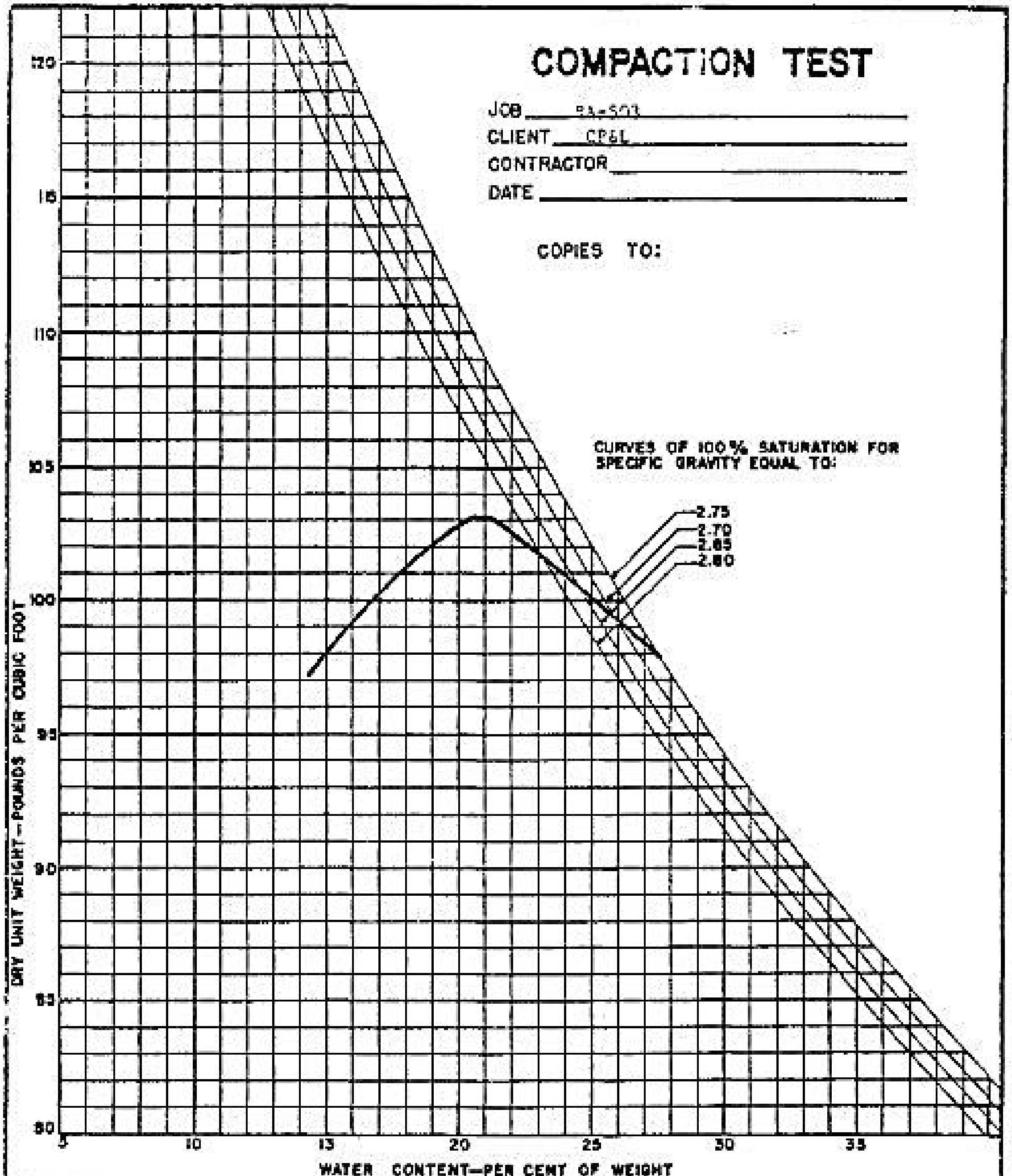
COMPACTION TEST, BORING NO. BC-158, S-2



MOISTURE DENSITY RELATION	METHOD OF TEST	MAX. DRY DENSITY PCF	OPTIMUM MOISTURE CONTENT %	SOIL DESCRIPTION OR CLASSIFICATION AND SAMPLE LOCATION
BC-158 S-2 5.0'-10.0'	STANDARD PROCTOR	88.5	30.0	RED BROWN SLIGHTLY FINE TO COARSE SANDY VERY SILTY CLAY

FIGURE 2.5.4-108

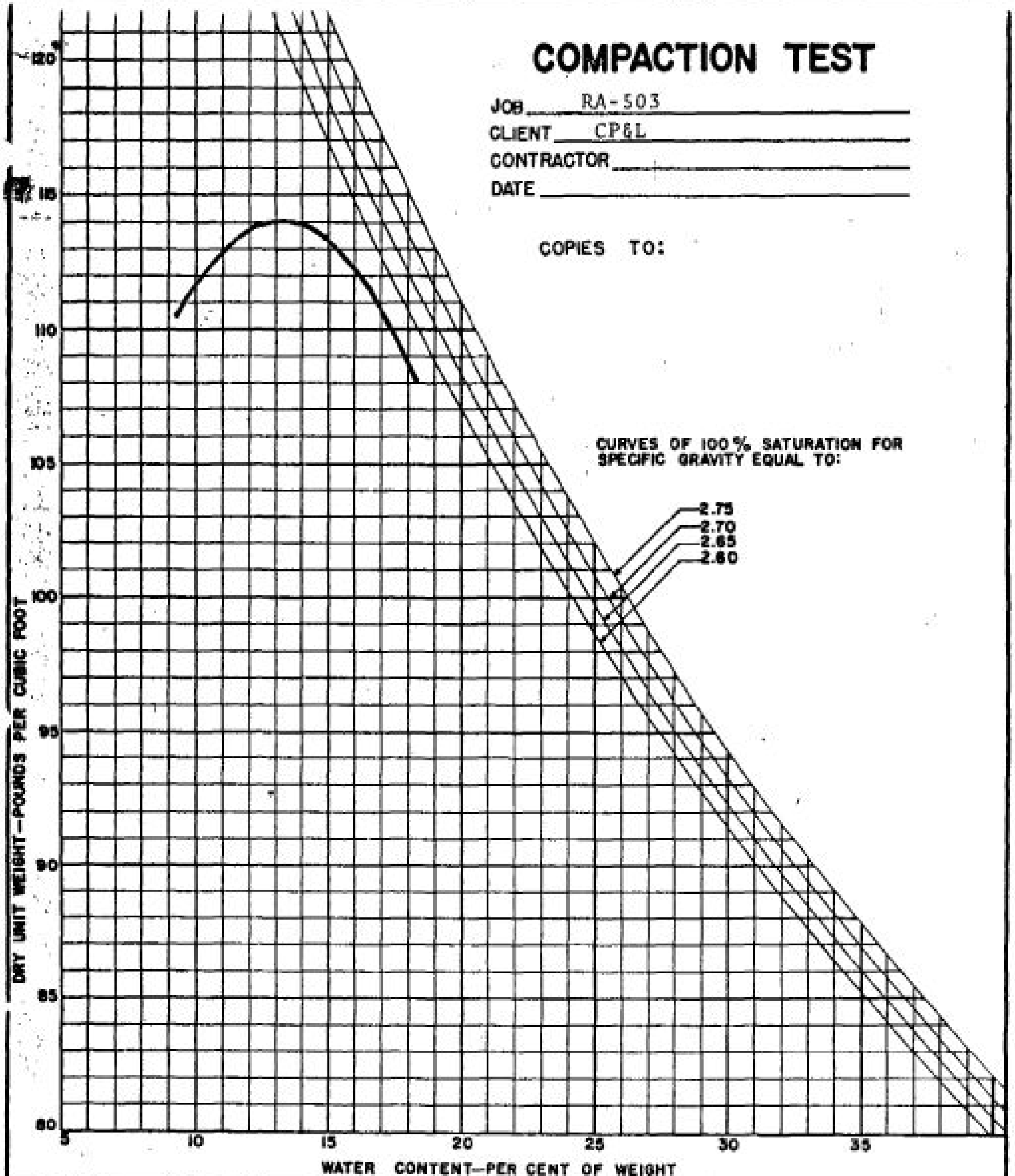
COMPACTION TEST, BORING NO. BC-158, S-3



MOISTURE DENSITY RELATION	METHOD OF TEST	MAX. DRY DENSITY PCF	OPTIMUM MOISTURE CONTENT %	SOIL DESCRIPTION OR CLASSIFICATION AND SAMPLE LOCATION
BC-158 S-3 10.0'-15.0'	STANDARD PROCTOR	103.0	21.0%	RED BROWN FINE SANDY VERY CLAYEY SIL

FIGURE 2.5.4-109

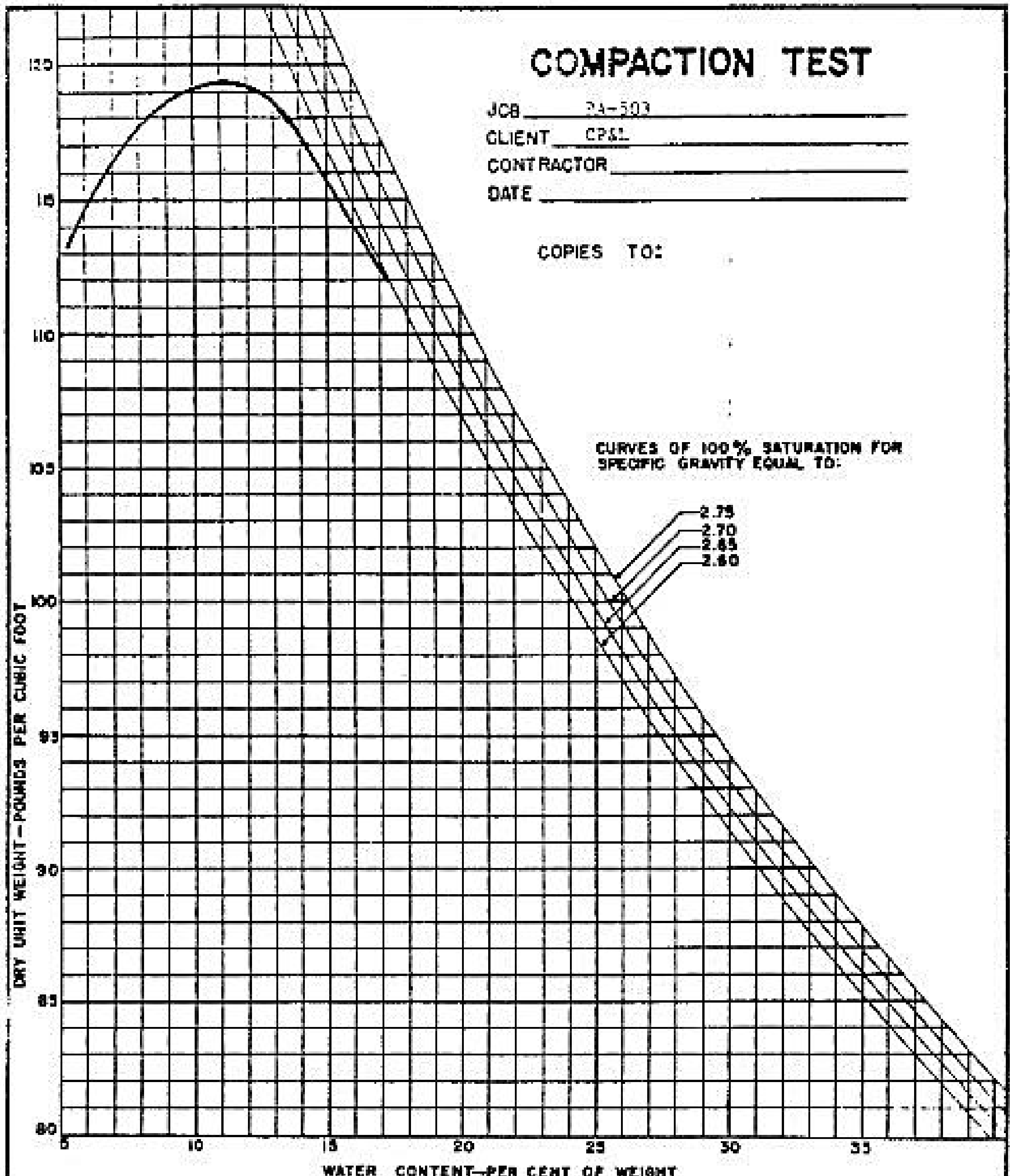
COMPACTION TEST, BORING NO. BC-161, S-2



MOISTURE DENSITY RELATION	METHOD OF TEST	MAX. DRY DENSITY PCF	OPTIMUM MOISTURE CONTENT %	SOIL DESCRIPTION OR CLASSIFICATION AND SAMPLE LOCATION
BC-161 S-2 5.0'-10.0'	STANDARD PROCTOR	114.0	13.2	BROWN RED SLIGHTLY FINE TO MEDIUM SANDY CLAYEY SILT

FIGURE 2.5.4-110

COMPACTION TEST, BORING NO. BC-161, S-3



MOISTURE DENSITY RELATION	METHOD OF TEST	MAX. DRY DENSITY PCF	OPTIMUM MOISTURE CONTENT %	SOIL DESCRIPTION OR CLASSIFICATION AND SAMPLE LOCATION
BC-161 S-3 10.0'-11.0'	STANDARD PROCTOR	119.4	11.2	BROWN RED SLIGHTLY FINE TO COARSE SANDY CLAYEY SILT

FIGURE 2.5.4-111

MOHR COULOMB FAILURE ENVELOPE FOR TRIAXIAL SHEAR TESTS ON BACKFILL MATERIAL

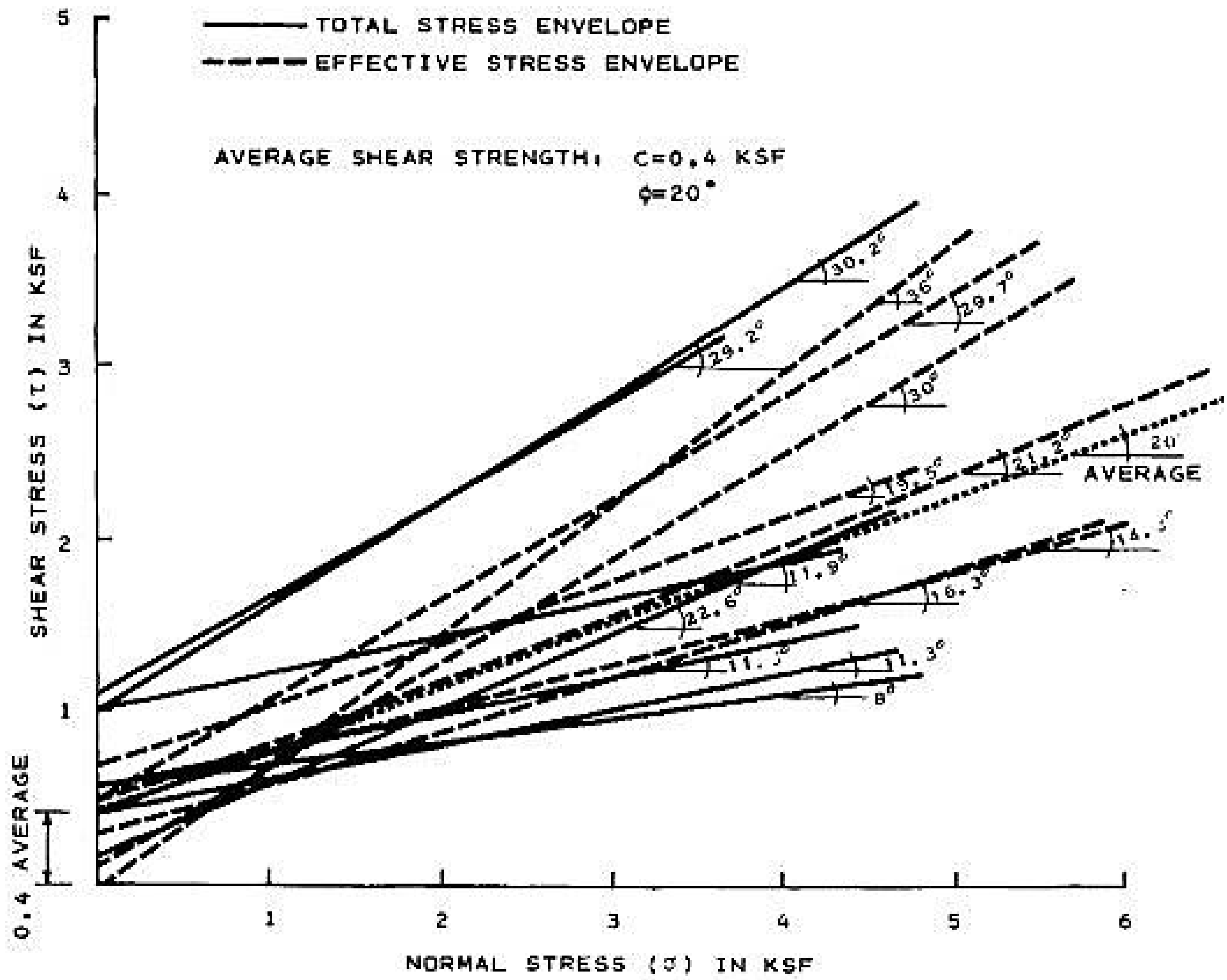
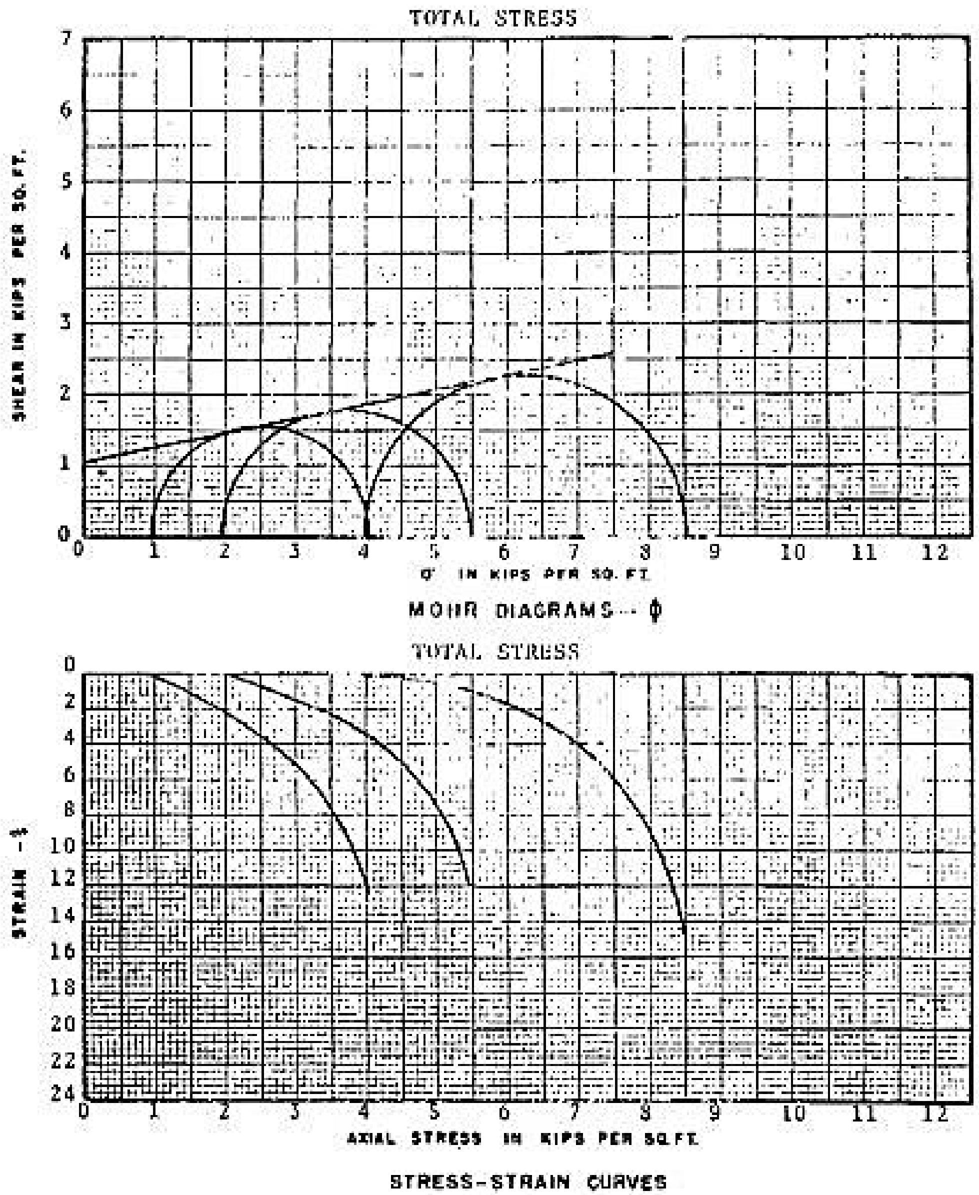


FIGURE 2.5.4-112

TRIAXIAL SHEAR TEST, BORING NO. BP-185, S-3



"COHESION", c 1.0 kips/ft^2
ANGLE OF SHEAR RESISTANCE, φ 11.9°
UNIT WEIGHT, γ Wet=131.7 pcf Dry=115.5 pcf
WATER CONTENT, w 14.0%
VOID RATIO, e 0.49
Compactive Effort 95% Standard Proctor Maximum Dry Density

Saturated, Consolidated, Undrained with Pore Pressure Measurement During Shear

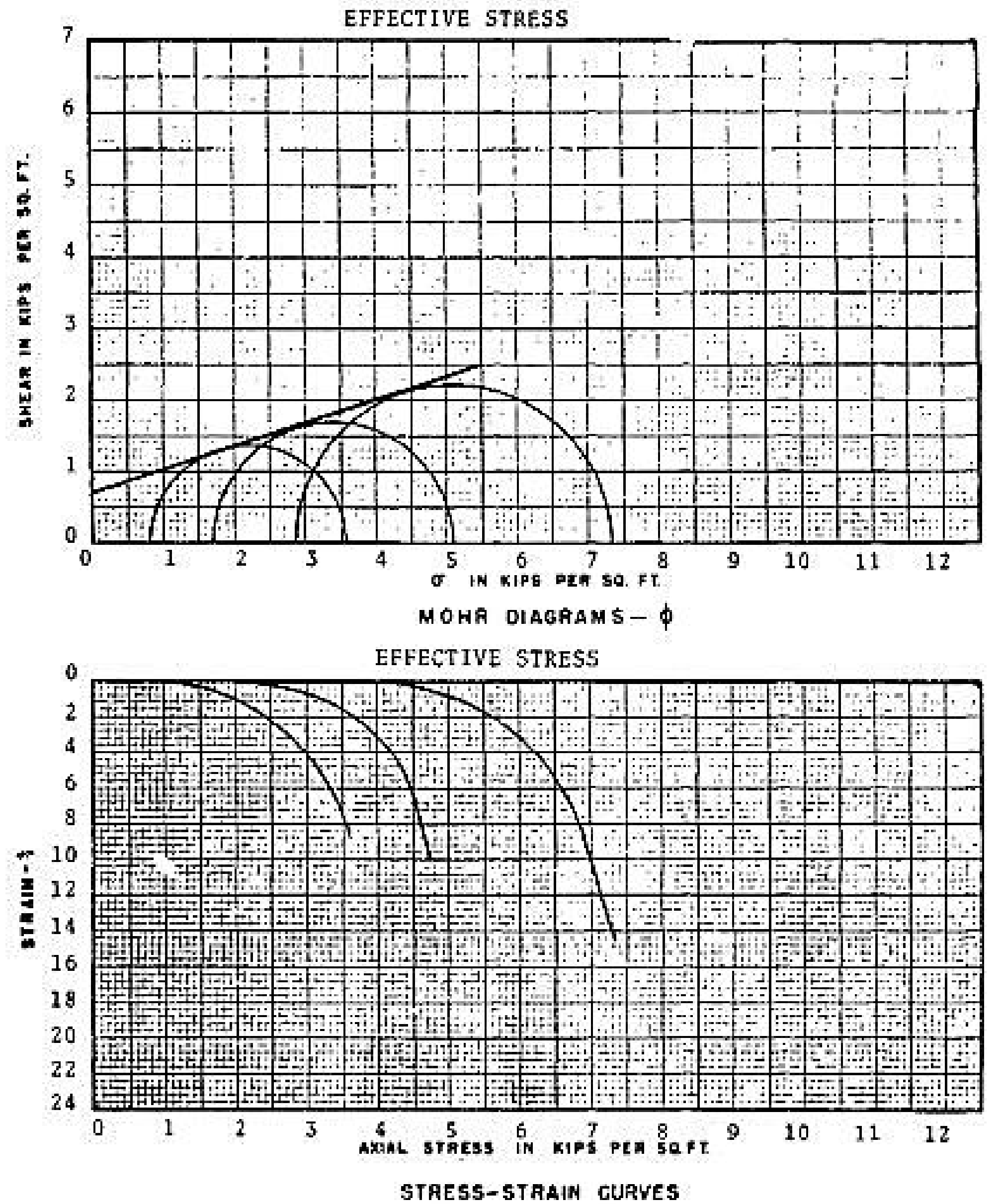
TRIAXIAL SHEAR TEST

BORING NO. BP-185 SAMPLE NO [illegible]
ELEV OR DEPTH 7.5'-12.5' JOB NO. RA [illegible]

LAW ENGINEERING TESTING CO.

FIGURE 2.5.4-113

TRIAXIAL SHEAR TEST, BORING NO. BP-185, S-3



"COHESION", c 0.69 kips/ft^2

ANGLE OF SHEAR RESISTANCE, ϕ 19.5°

UNIT WEIGHT, γ Wet=131.7 pcf Dry=115.5 pcf

WATER CONTENT, w 14.0%

VOID RATIO, e 0.49

Compactive effort 95% Standard Proctor Maximum Dry Density

Saturated, Consolidated, Undrained with Pore Pressure Measurement During Shear

TRIAXIAL SHEAR TEST

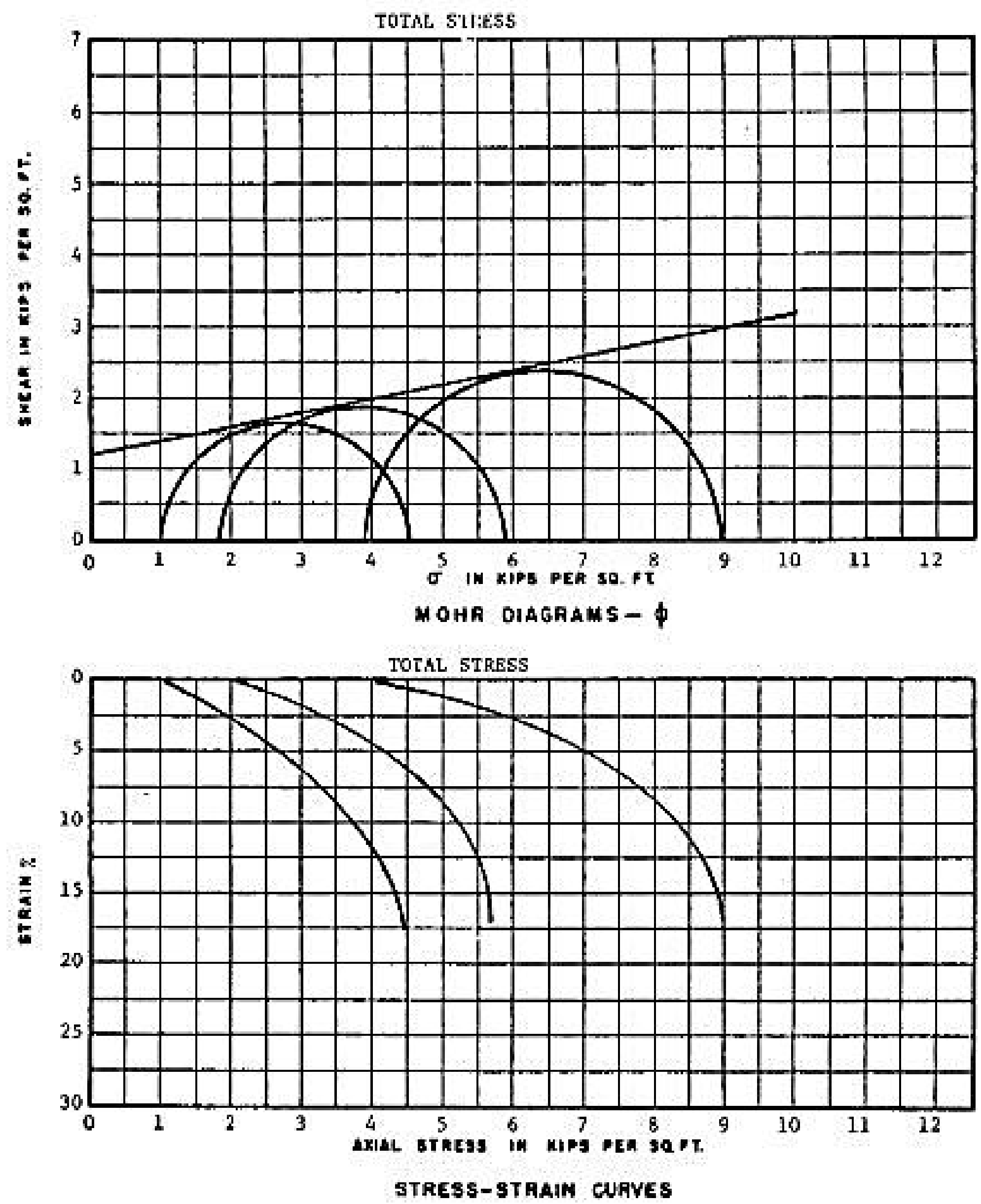
BORING NO. BP-185 SAMPLE NO. S-3

ELEV. OR DEPTH 7.5'-12.5' JOB NO. RA-503

LAW ENGINEERING TESTING CO.

FIGURE 2.5.4-120

TRIAXIAL SHEAR TEST, BORING NO. BC-153, S-3



"COHESION", c 1.2 kips/ft²

ANGLE OF SHEAR RESISTANCE, φ 11.3°

UNIT WEIGHT, γ WET=130.7 pcf DRY=110.7 pcf

WATER CONTENT, w 18.0%

VOID RATIO, e 0.52

COMPACTIVE EFFORT: 95% Standard Proctor Maximum Dry Density

Saturated, Consolidated, Undrained with Pore Pressure Measurement During Shear

TRIAXIAL SHEAR TEST

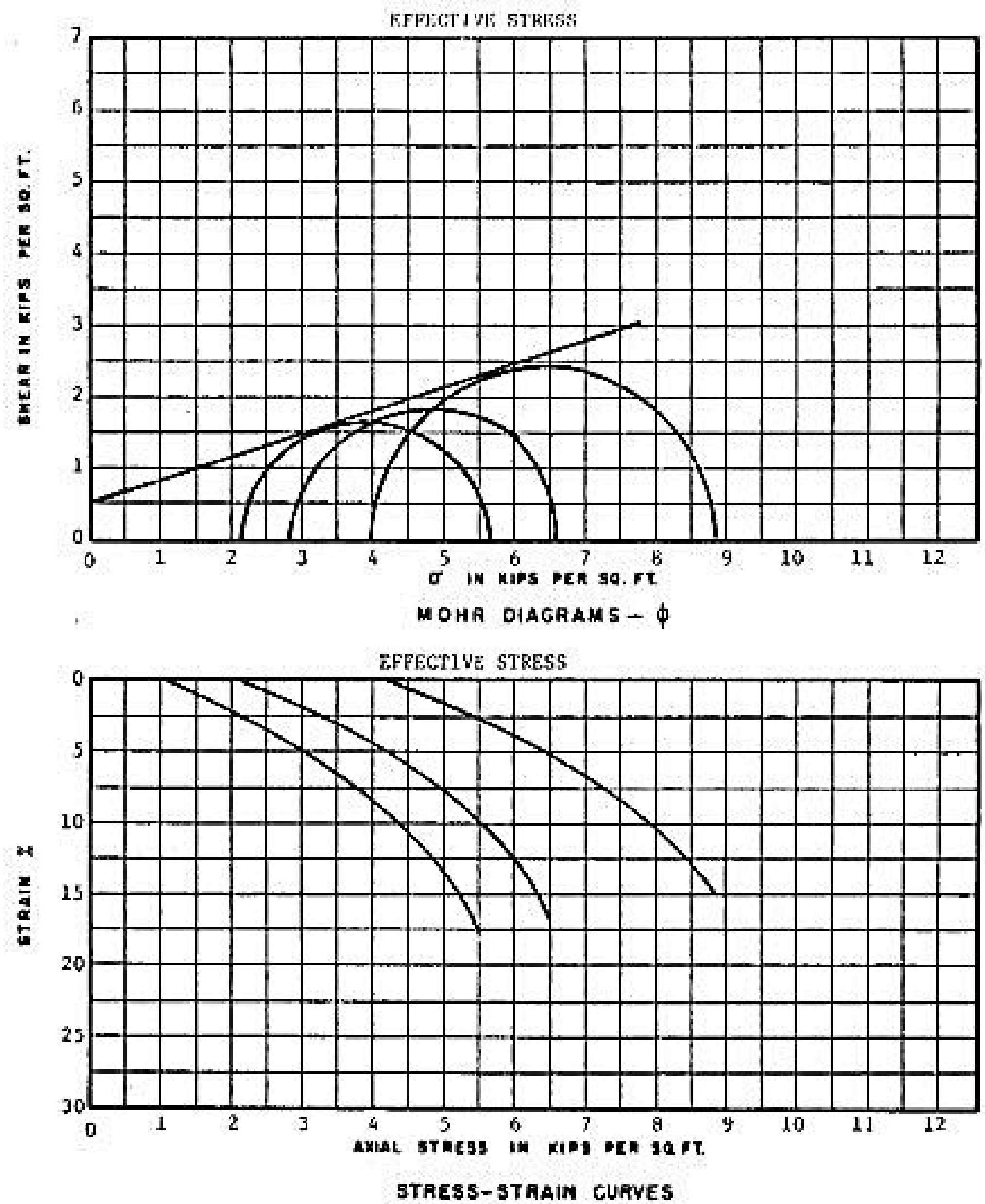
BORING NO. BC-15[illegible] SAMPLE NO. [illegible]

ELEV. OR DEPTH 8.0'-13.0' JOB NO. [illegible]

LAW ENGINEERING TESTING [illegible]

FIGURE 2.5.4-121

TRIAXIAL SHEAR TEST, BORING NO. BC-153, S-3



"COHESION", C 0.5 kips/ft^2

ANGLE OF SHEAR RESISTANCE, φ 19.8°

UNIT WEIGHT, γ WET=130.7 pcf DRY=110.7 pcf

WATER CONTENT, w 18.0%

VOID RATIO, e 0.52

COMPACTIVE EFFORT: 95% Standard Proctor Maximum Dry Density

Saturated, Consolidated, Undrained with Pore Pressure Measurement During Test

TRIAXIAL SHEAR TEST

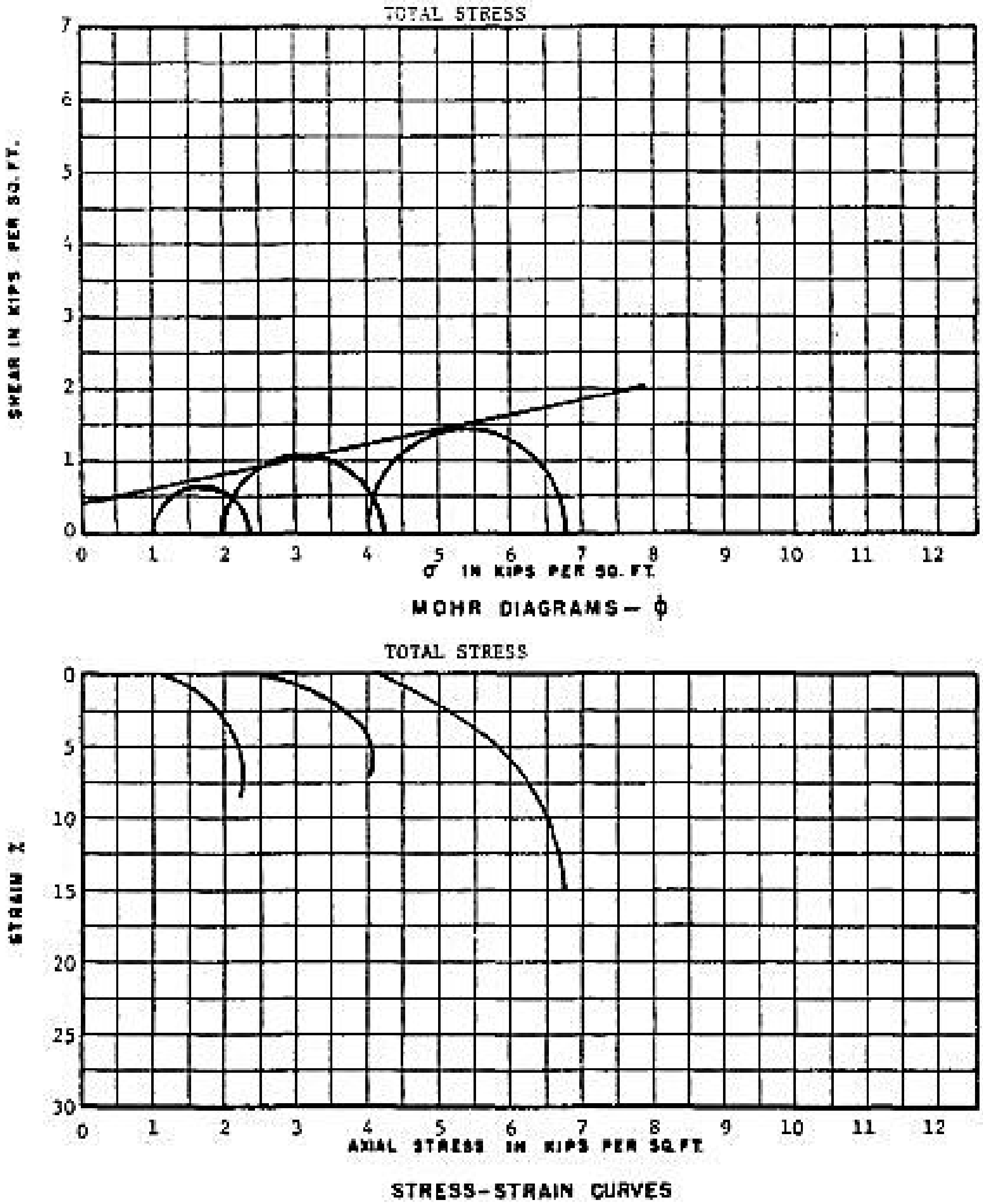
BORING NO. BC-153 SAMPLE NO. S-3

ELEV. OR DEPTH 8.0'-13.0' JOB NO. [illegible]

LAW ENGINEERING TESTING CO.

FIGURE 2.5.4-122

TRIAXIAL SHEAR TEST, BORING NO. BC-158, S-2



"COHESION", C 0.4 kips/ft²

ANGLE OF SHEAR RESISTANCE, ϕ 11.3°

UNIT WEIGHT, γ WET=113.9 pcf DRY=84.0 pcf

WATER CONTENT, W 35.5%

VOID RATIO, e 1.00

COMPACTIVE EFFORT: 95% Standard Proctor Maximum Dry Density

Saturated, Consolidated Undrained with Pore Pressure Measurement During Shear

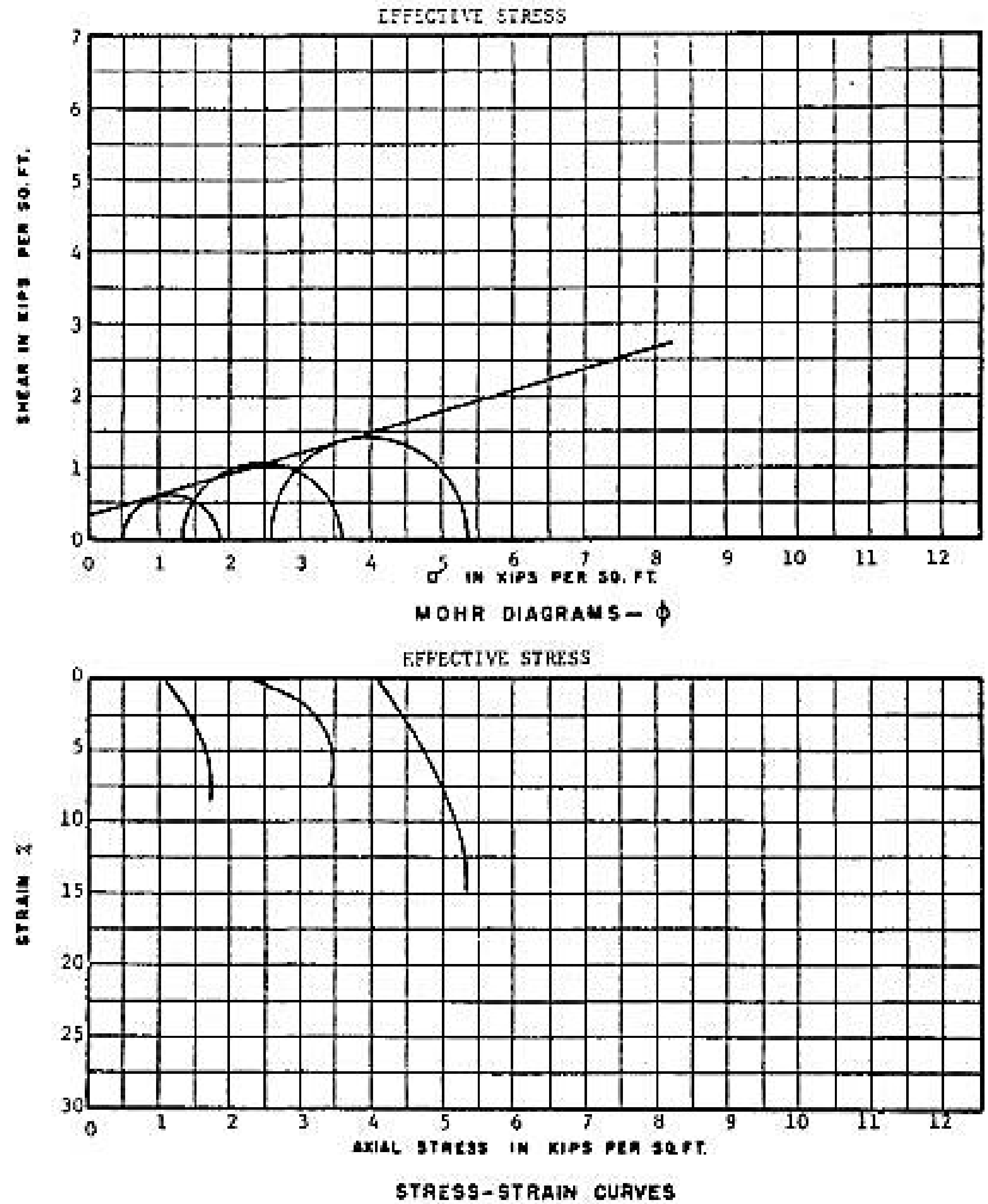
TRIAXIAL SHEAR TEST

BORING NO. BC-158 SAMPLE NO. S-2

ELEV. OR DEPTH 5.0'-10.0' JOB NO. RA-503

FIGURE 2.5.4-123

TRIAXIAL SHEAR TEST, BORING NO. BC-158, S-2



"COHESION", C 0.3 kips/ft^2

ANGLE OF SHEAR RESISTANCE, φ 16.3°

UNIT WEIGHT, γ WET=113.9 pcf DRY=84.0 pcf

WATER CONTENT, w 35.5%

VOID RATIO, e 1.00

COMPACTIVE EFFORT: 95% Standard Proctor Maximum Dry Density

Saturated, Consolidated, Undrained with Pore Pressure Measurement During Shear

TRIAXIAL SHEAR TEST

BORING NO. [illegible] SAMPLE NO. [illegible]

ELEV. OR DEPTH [illegible] JOB NO. [illegible]

FIGURE 2.5.4-124

TRIAXIAL SHEAR TEST, BORING NO. BC-158, S-3

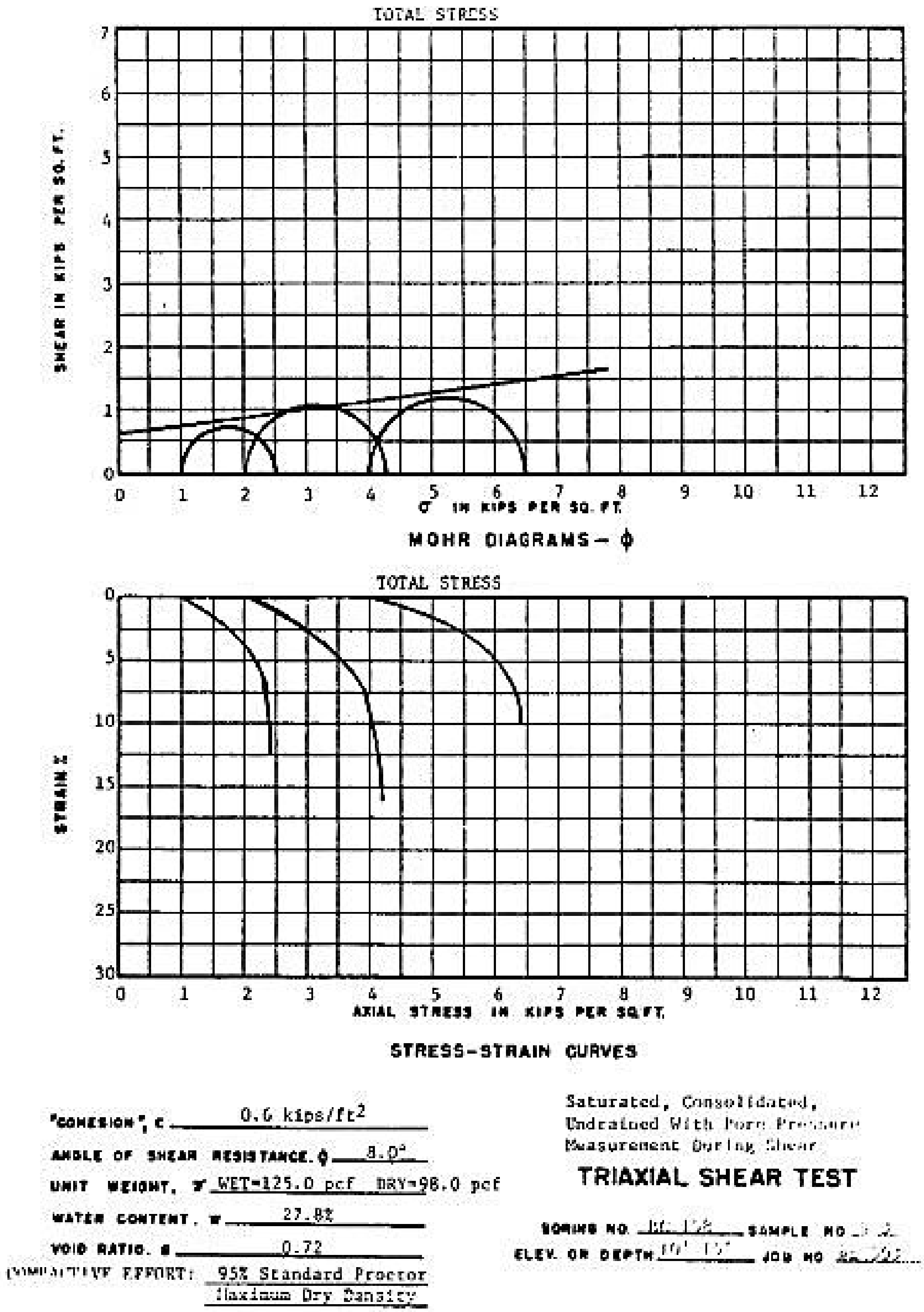
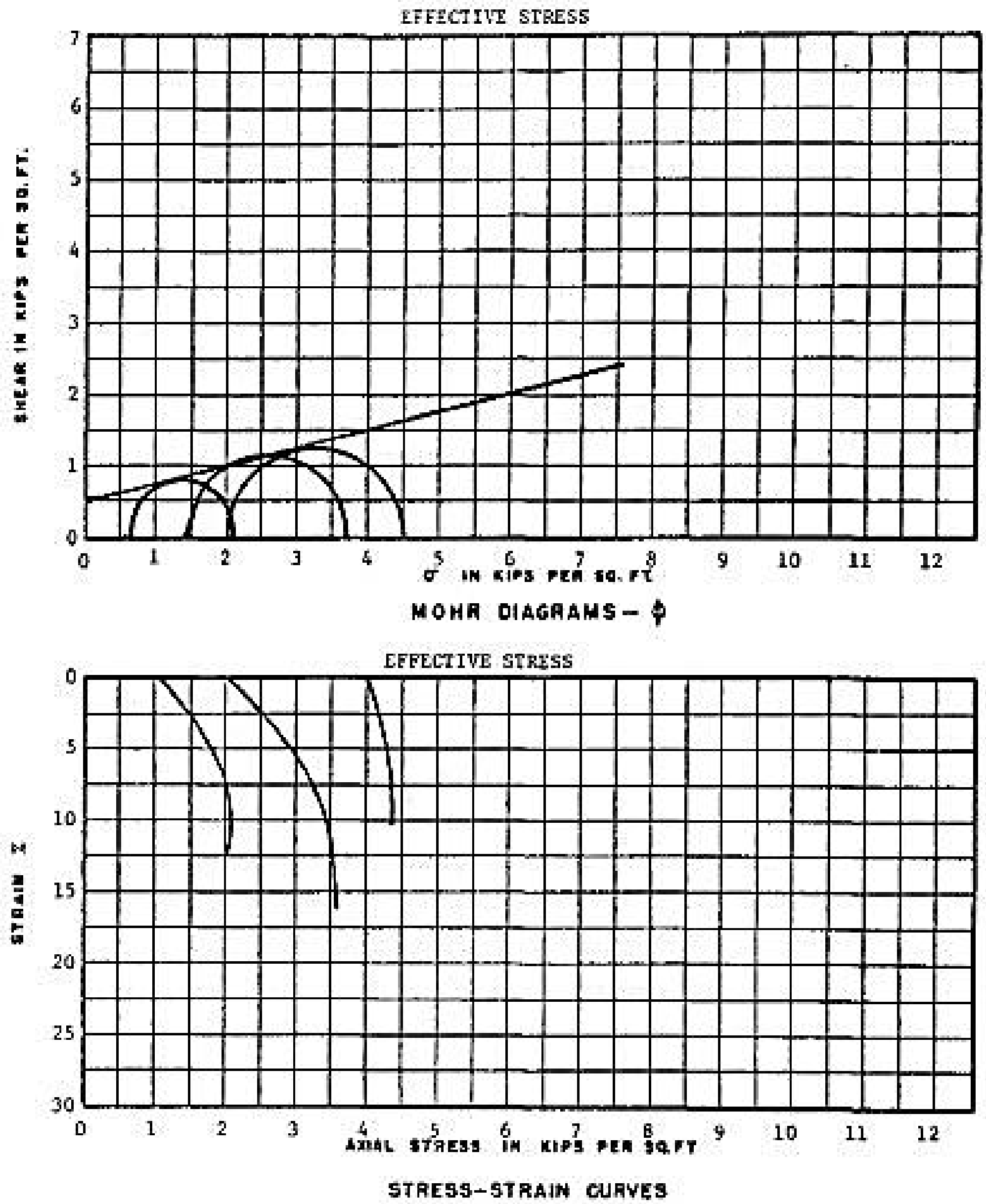


FIGURE 2.5.4-125

TRIAXIAL SHEAR TEST, BORING NO. BC-158, S-3



"COHESION", c 0.5 kips/ft^2

ANGLE OF SHEAR RESISTANCE, φ 14.3°

UNIT WEIGHT, γ WET=125.0 pcf DRY=98.0 pcf

WATER CONTENT, w 27.8%

VOID RATIO, e 0.72

COMPACTIVE EFFORT: 95% Standard Proctor Maximum Dry Density

Saturated, Consolidated, Undrained with Pore Pressure Measurement During Shear

TRIAXIAL SHEAR TEST

BORING NO. BC-158 SAMPLE NO. S-3

ELEV. OR DEPTH 10'-15' JOB NO. RA-503

FIGURE 2.5.4-126

TRIAXIAL SHEAR TEST, BORING NO. BC-161, S-2

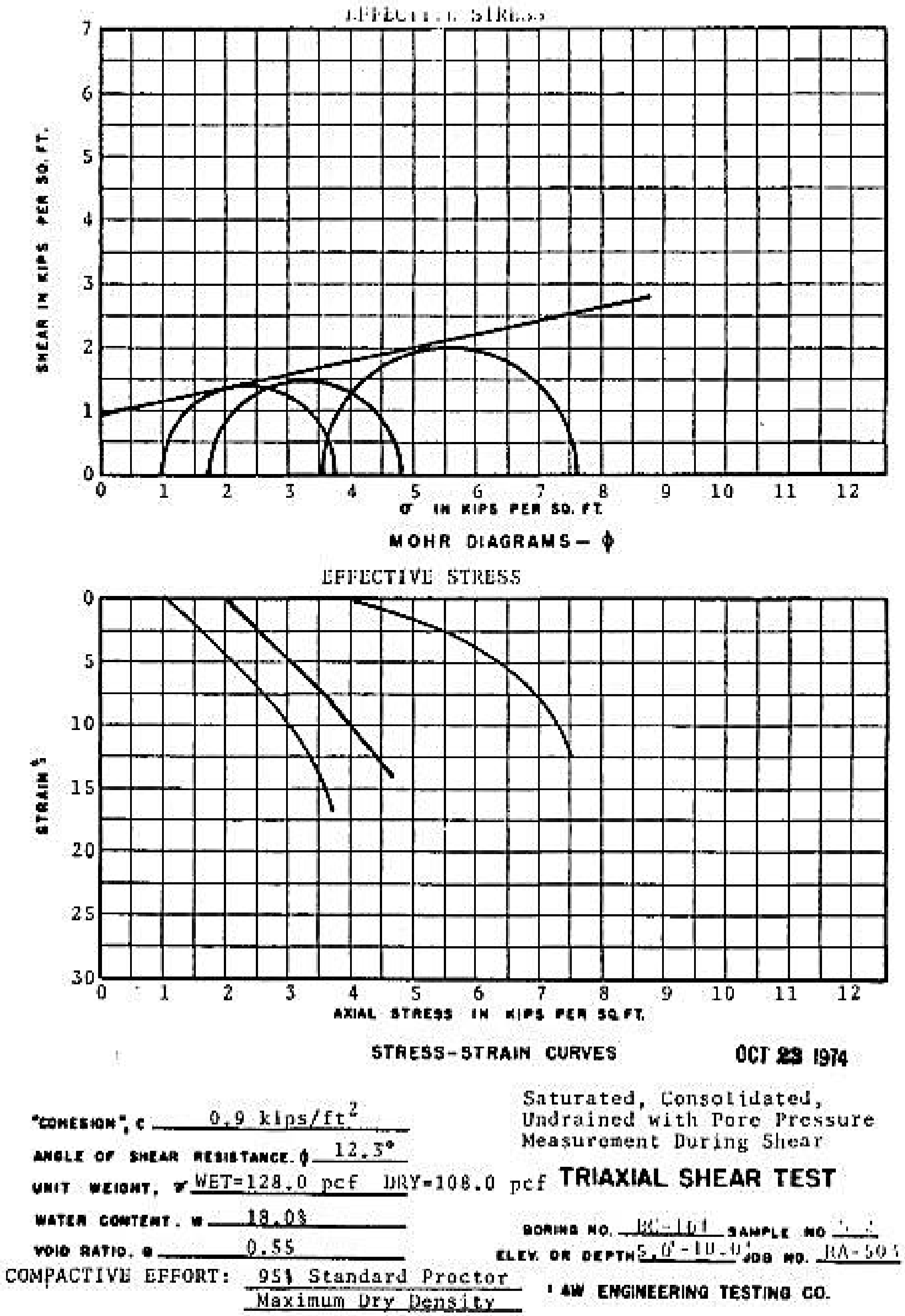
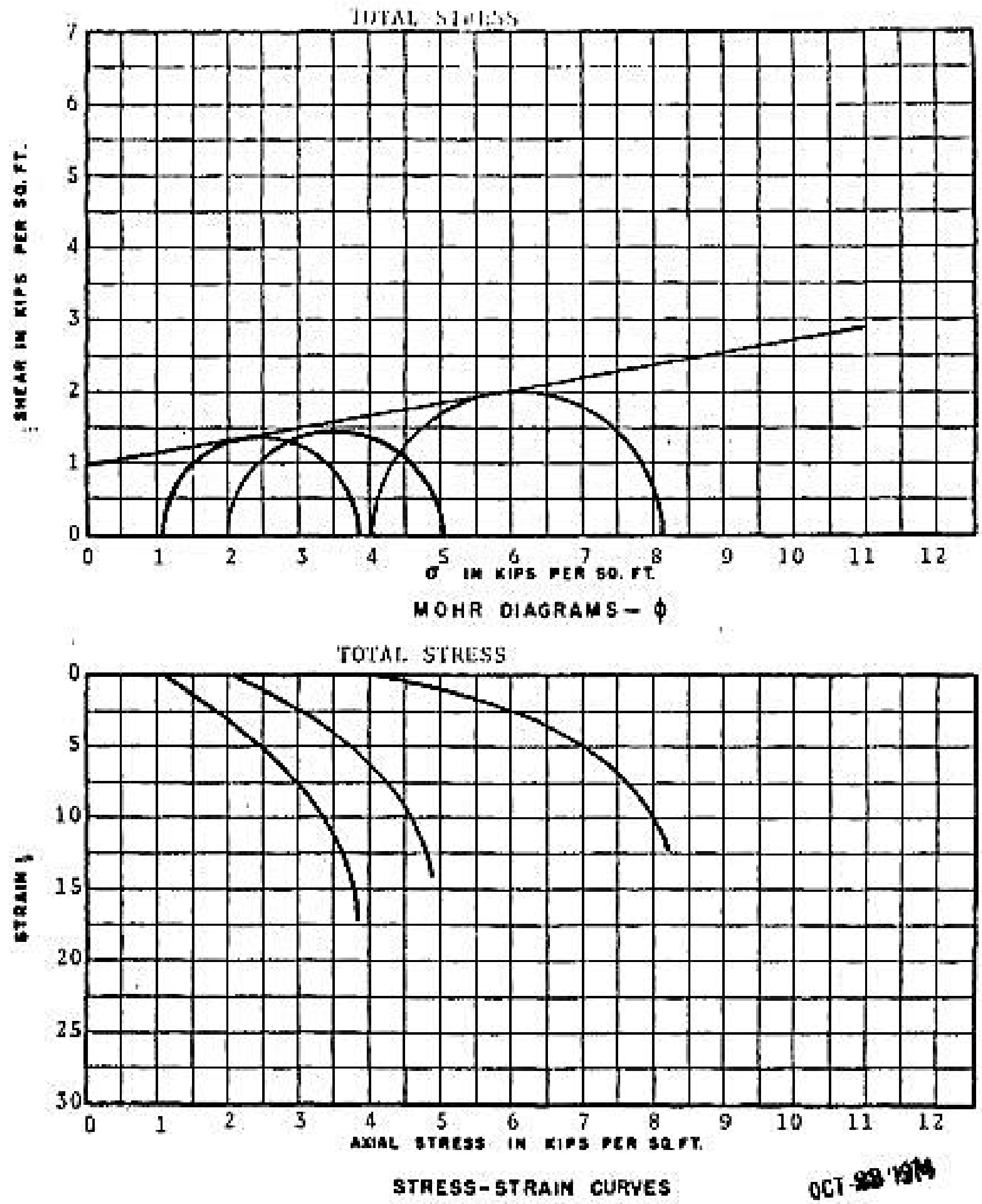


FIGURE 2.5.4-127

TRIAXIAL SHEAR TEST, BORING NO. BC-161, S-2



"COHESION", c 1.0 kips/ft^2

ANGLE OF SHEAR RESISTANCE, ϕ 11.5°

UNIT WEIGHT, γ WET=128.0 pcf DRY=108.0 pcf

WATER CONTENT, w 18.0%

VOID RATIO, e 0.55

COMPACTIVE EFFORT: 95% Standard Proctor Maximum Dry Density

Saturated, Consolidated, Undrained with Pore Pressure Measurement During Shear

TRIAXIAL SHEAR TEST

BORING NO. BC-161 SAMPLE NO. S-2

ELEV. OR DEPTH 5.0'-10.0' JOB NO. RA-503

LAW ENGINEERING TESTING CO.

FIGURE 2.5.4-128

TRIAXIAL SHEAR TEST, BORING NO. BC-161, S-3

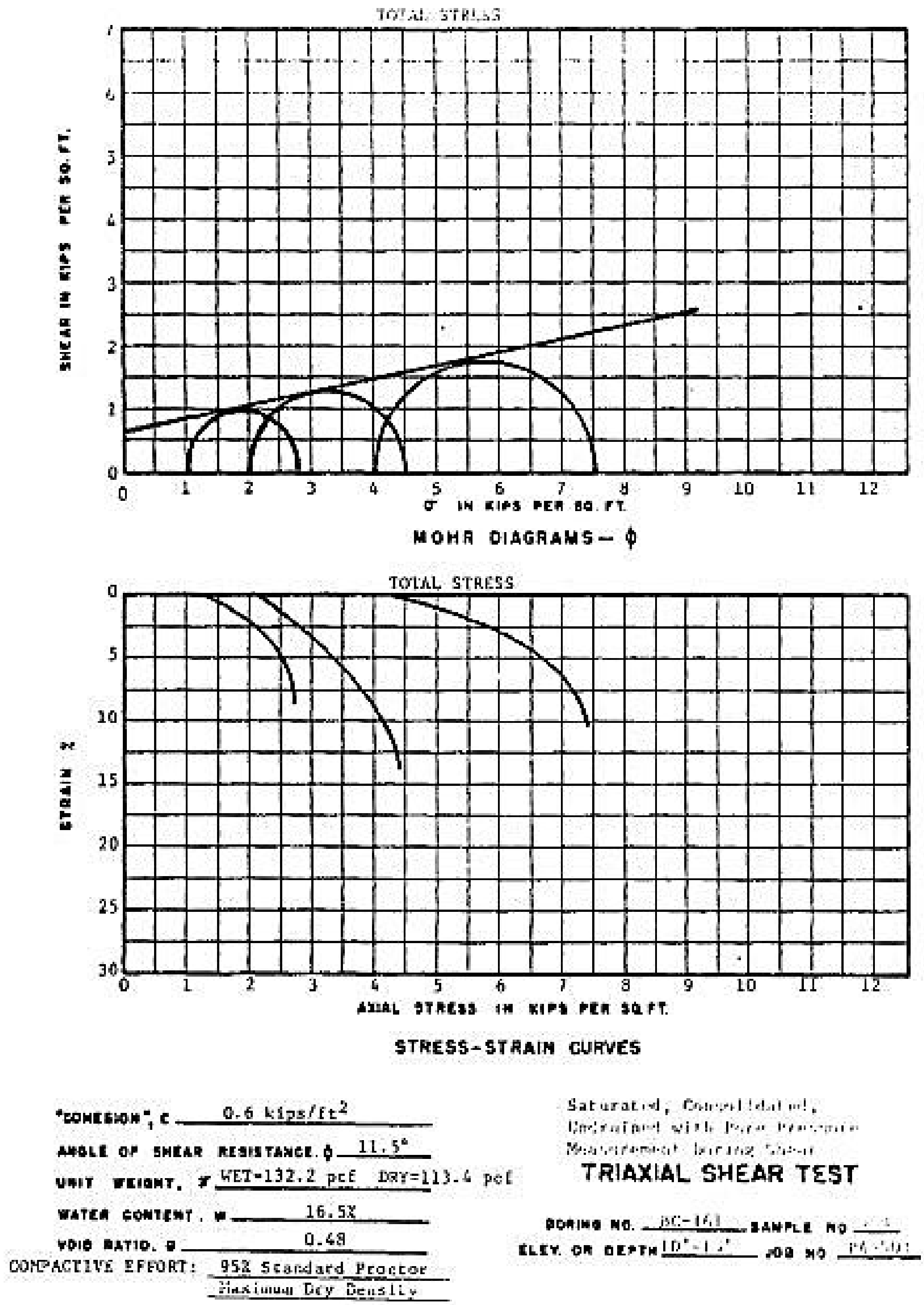
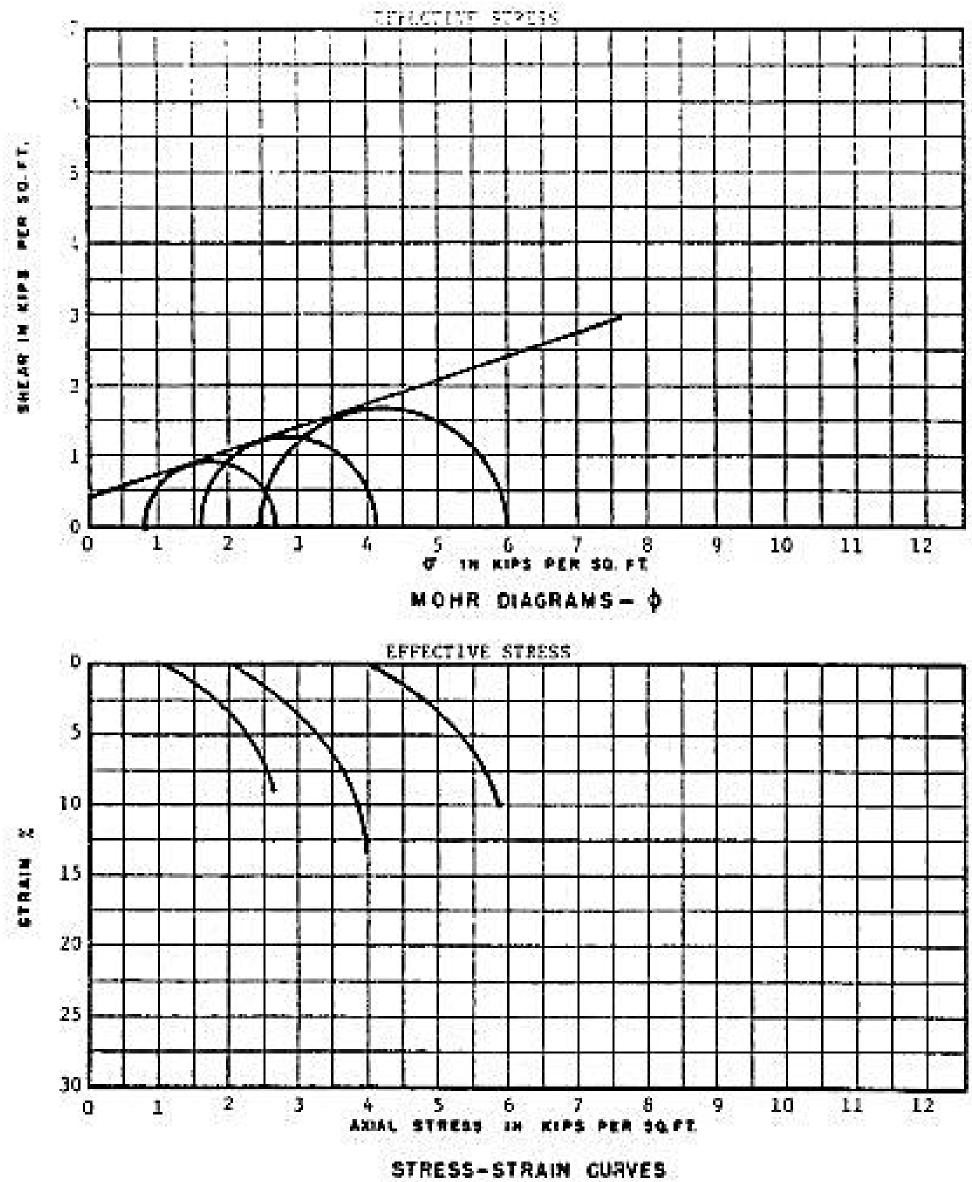


FIGURE 2.5.4-129

TRIAXIAL SHEAR TEST, BORING NO. BC-161, S-3



"COHESION", c 0.4 kips/ft^2

ANGLE OF SHEAR RESISTANCE, φ 21.2°

UNIT WEIGHT, γ WET=132.2 pcf DRY=113.4 pcf

WATER CONTENT, w 16.5%

VOID RATIO, e 0.48

COMPACTIVE EFFORT: 95% Standard Proctor Maximum Dry Density

Saturated, Consolidated, Undrained with Pore Pressure Measurement During Shear

TRIAXIAL SHEAR TEST

BORING NO. BC-161 SAMPLE NO. S-3

ELEV. OR DEPTH 10'-15' JOB NO. [illegible]

FIGURE 2.5.4-130

BACKFILL AGAINST EXTERIOR WALLS OF STRUCTURES

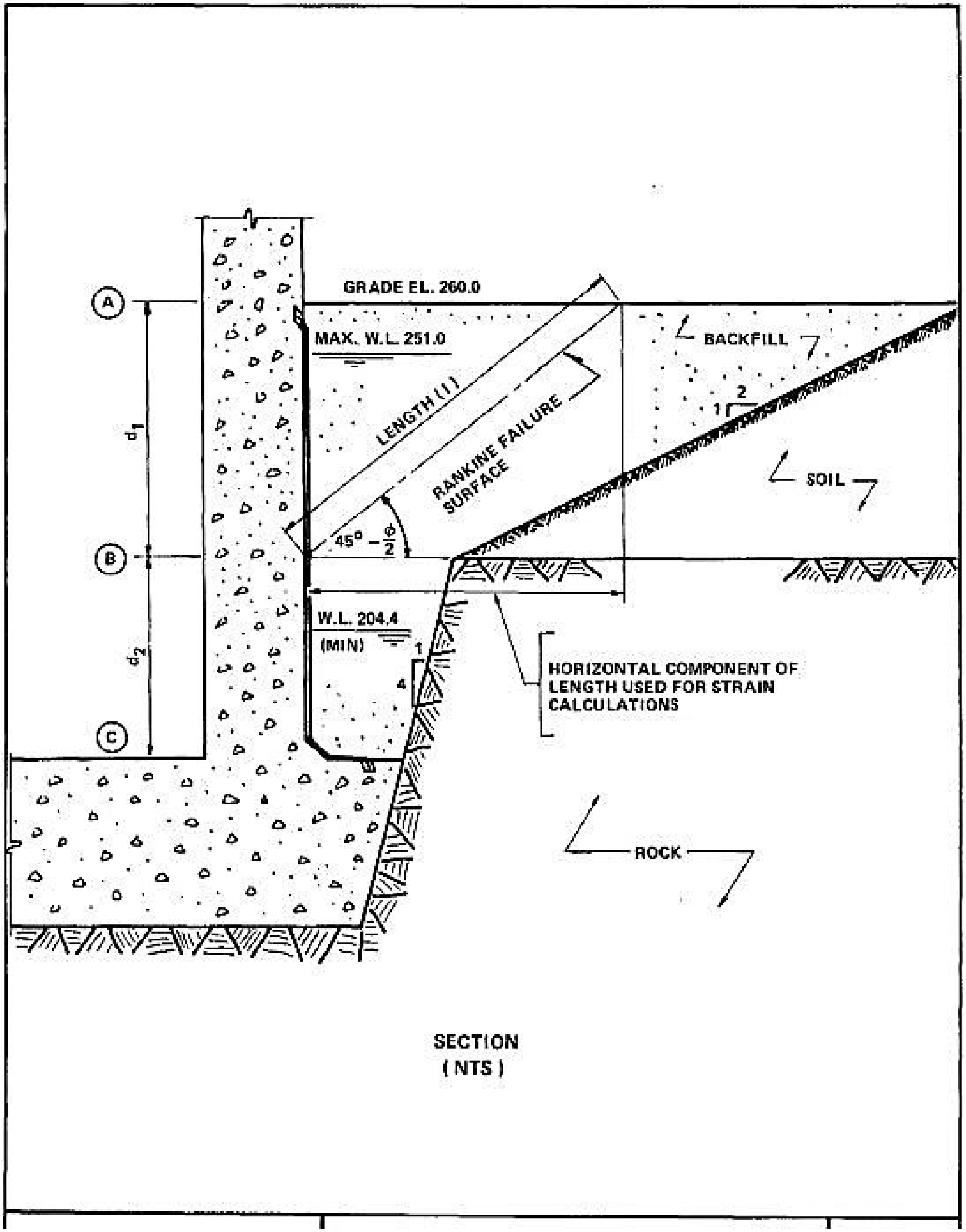
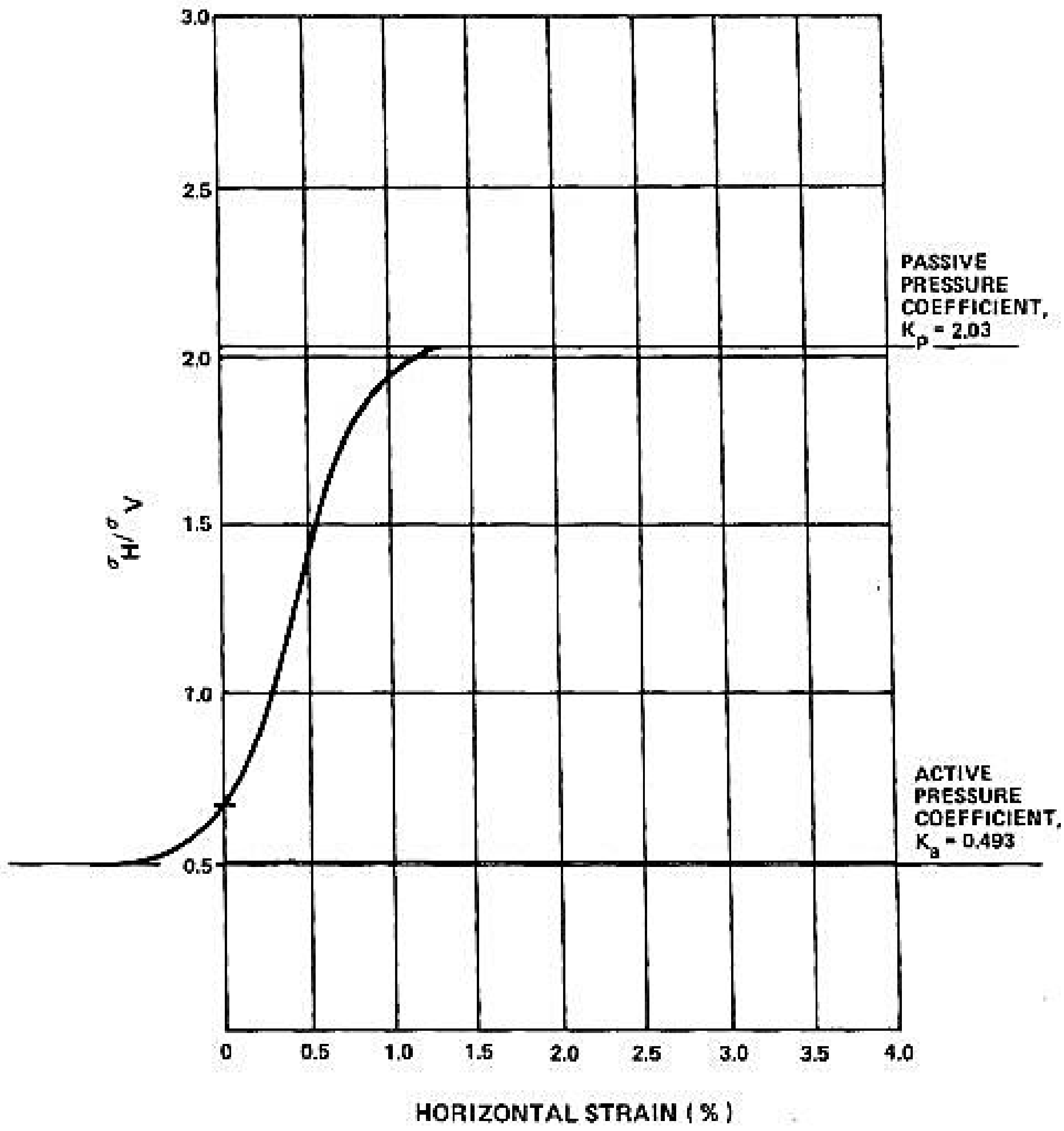


FIGURE 2.5.4-131

HORIZONTAL SOIL PRESSURE UNDER EARTHQUAKE CONDITIONS



NOTE:
THE ABOVE CURVE WAS BASED ON FIGURE 13.7 IN SOIL MECHANICS BY LAMBE & WHITMAN, CORRECTED FOR SOIL PROPERTIES.

FIGURE 2.5.6-9

MAIN DAM VICINITY EXPLORATION, SHEET 1

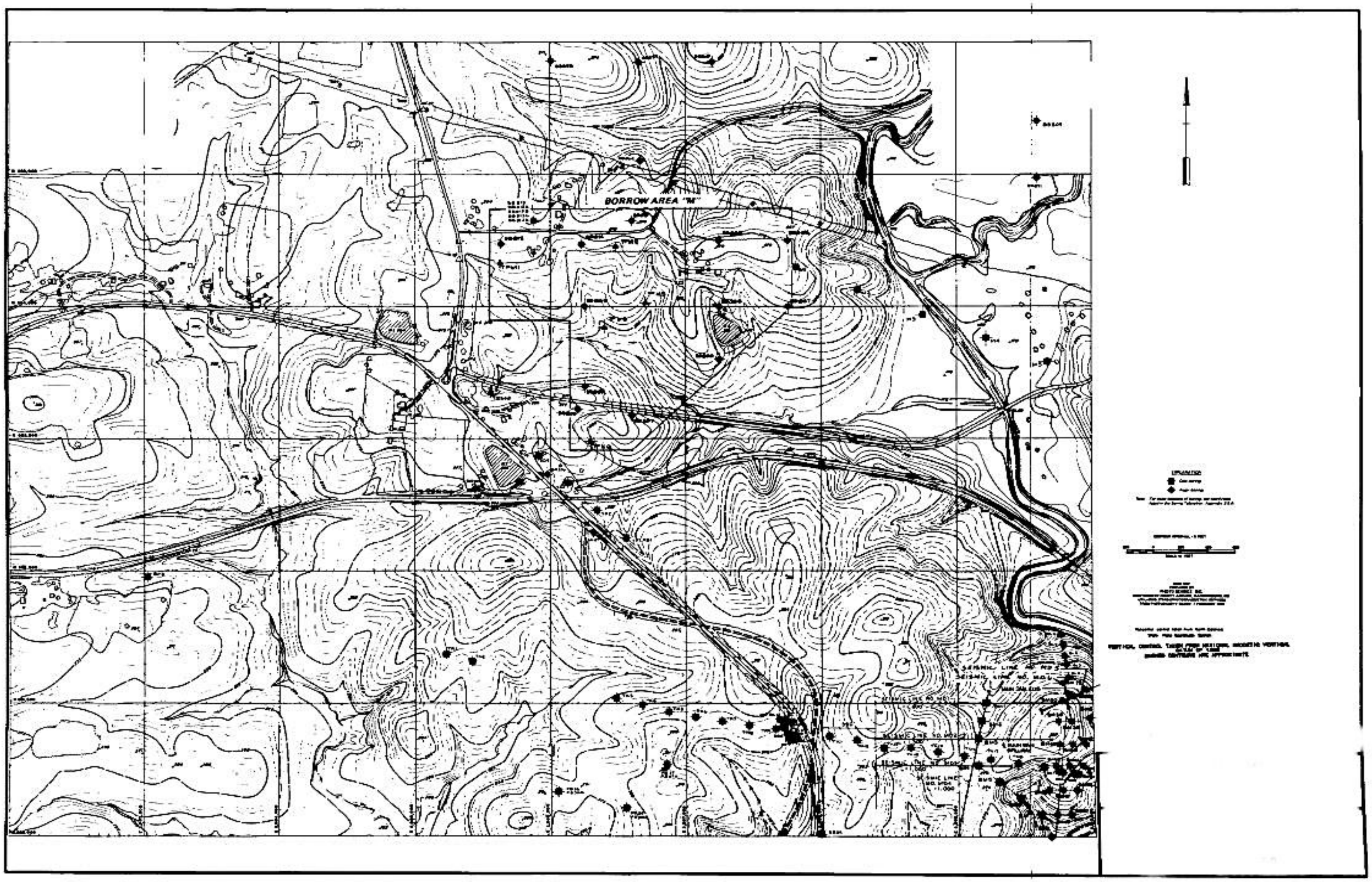


FIGURE 2.5.6-10

MAIN DAM VICINITY EXPLORATION, SHEET 2

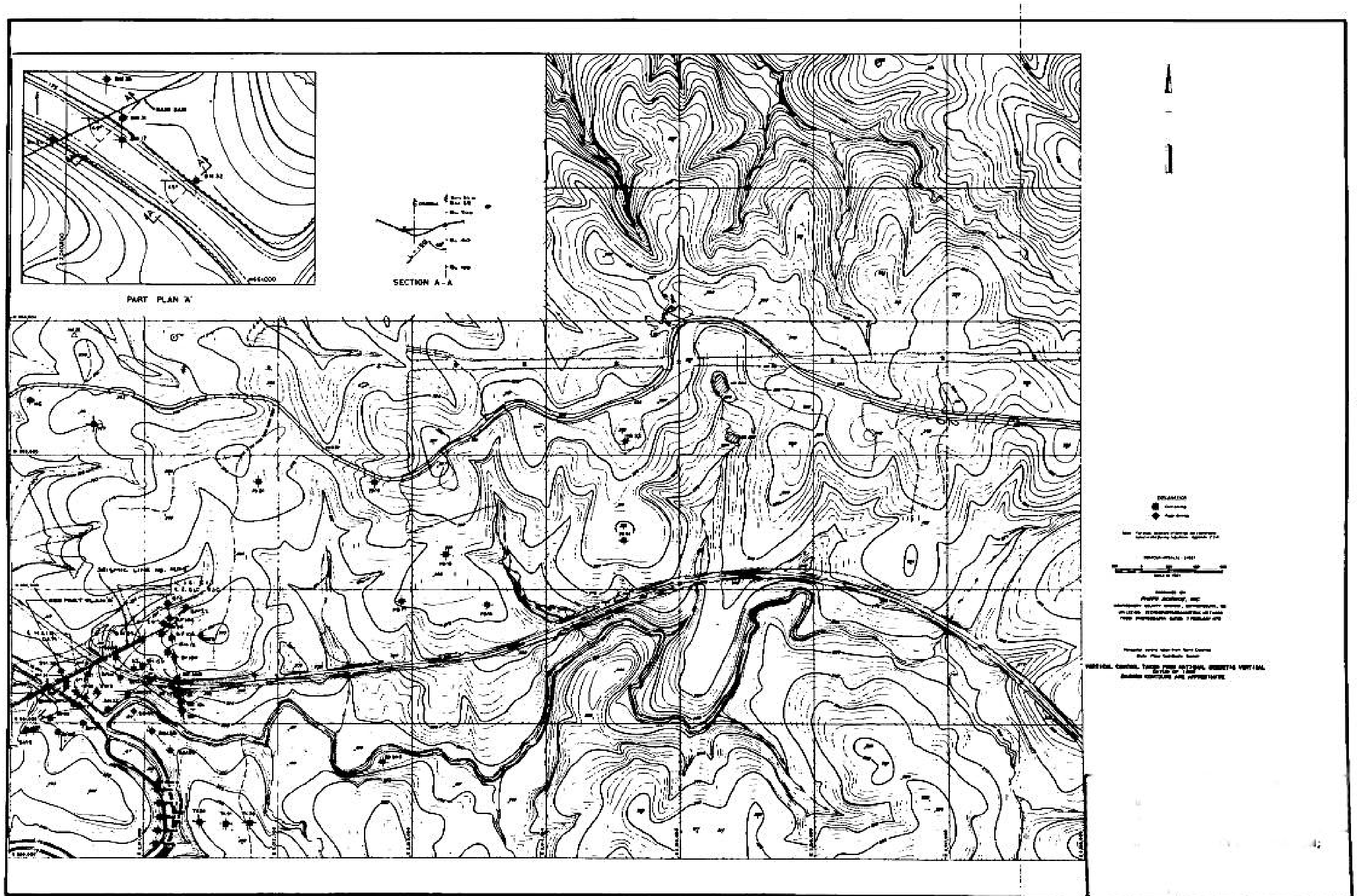


FIGURE 2.5.6-11

AUXILIARY DAM VICINITY EXPLORATION, SHEET 1

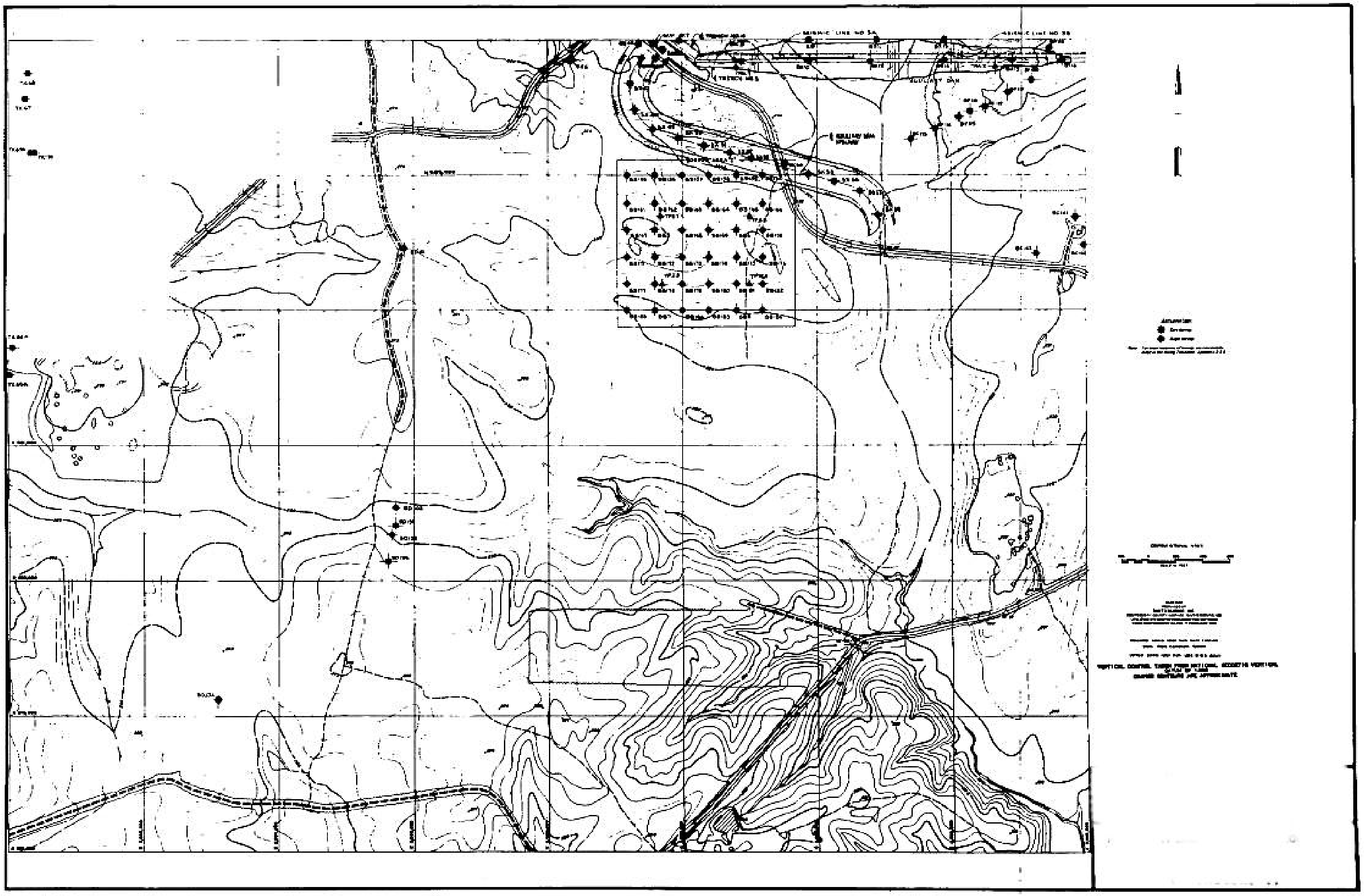


FIGURE 2.5.6-12

AUXILIARY DAM VICINITY EXPLORATION, SHEET 2

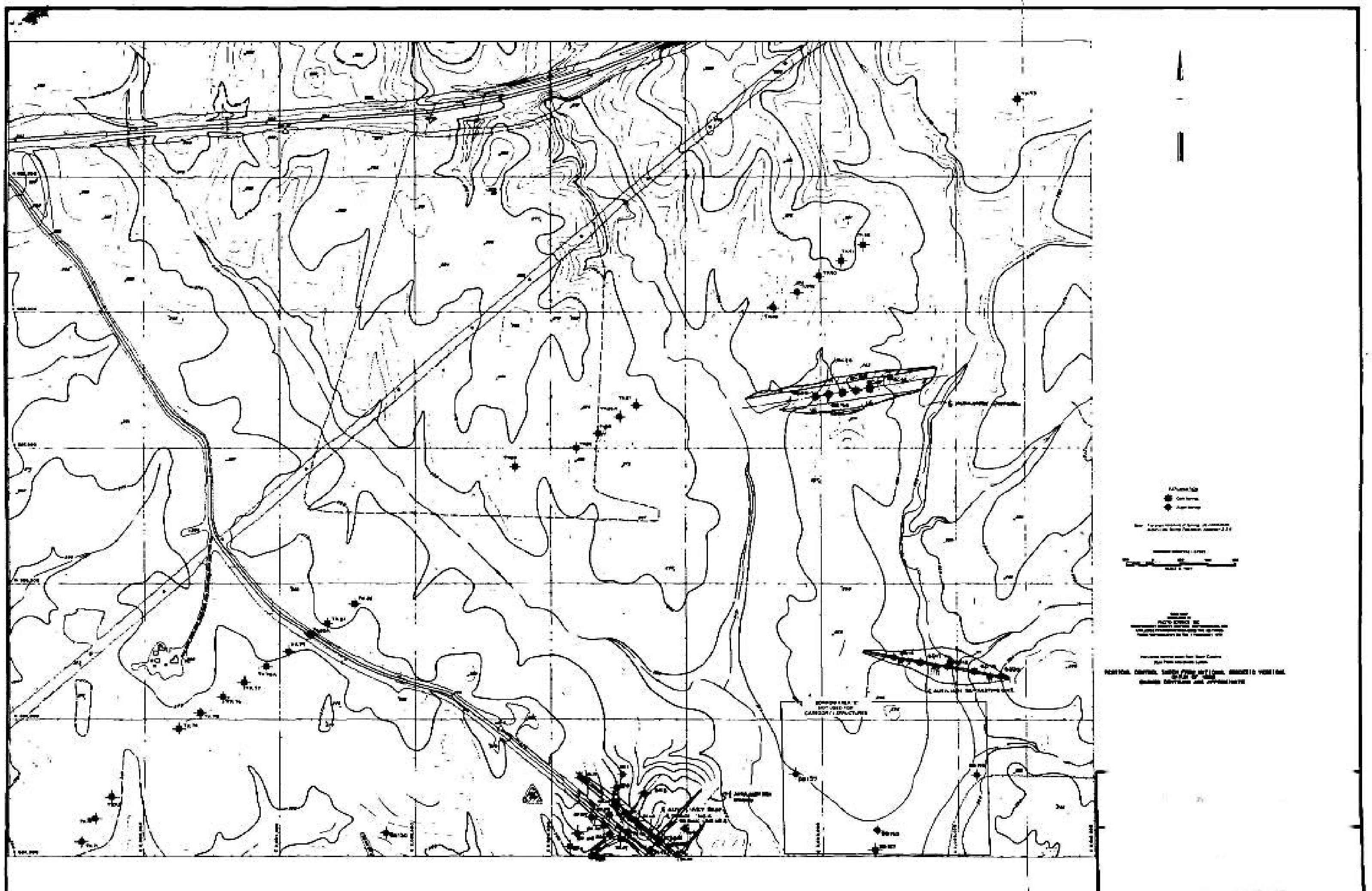


FIGURE 2.5.6-13

TYPICAL CURVES OF MODULUS AND DAMPING VALUES VERSUS STRAIN

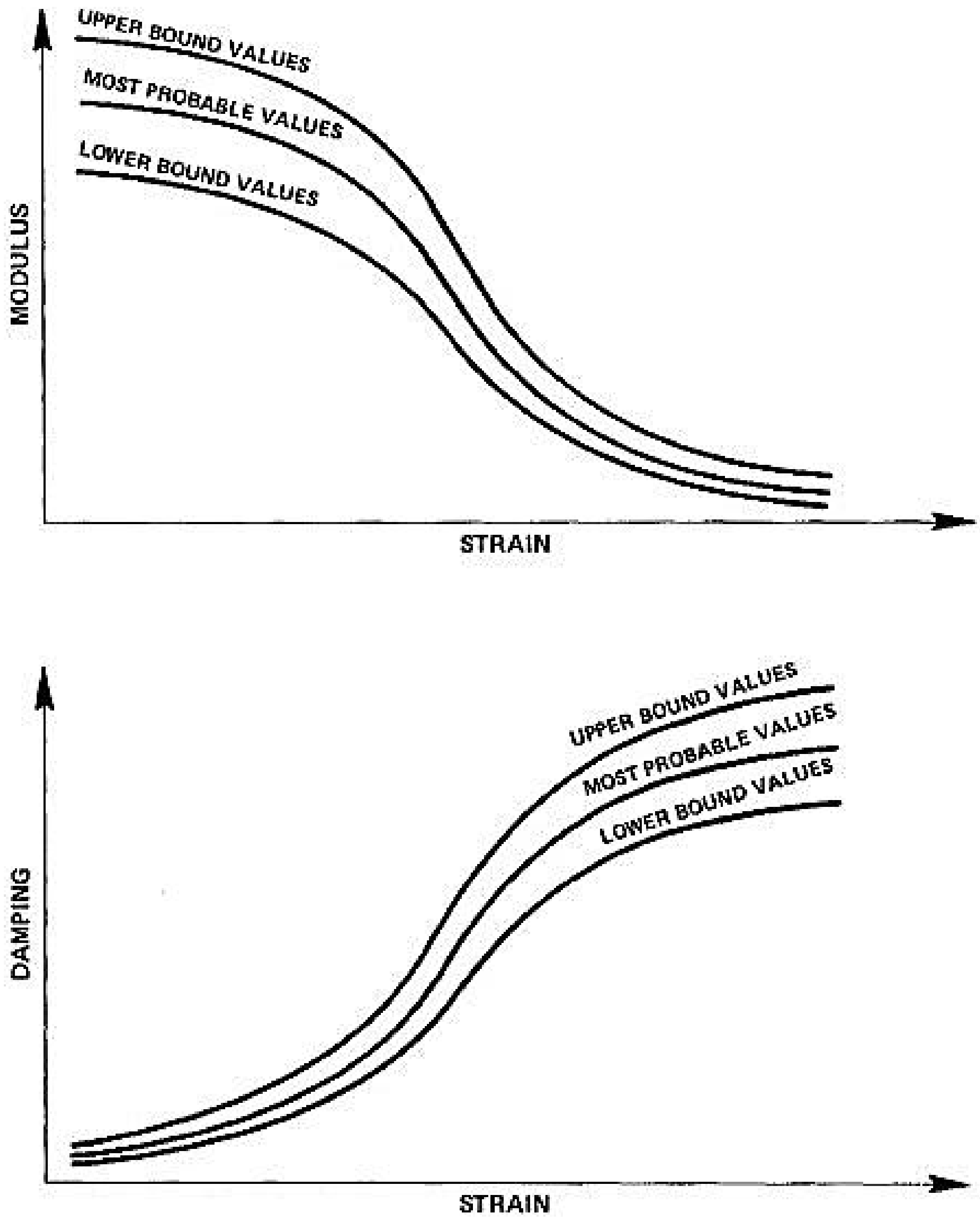


FIGURE 2.5.6-14

TYPICAL CYCLIC STRENGTH CHARACTERISTICS

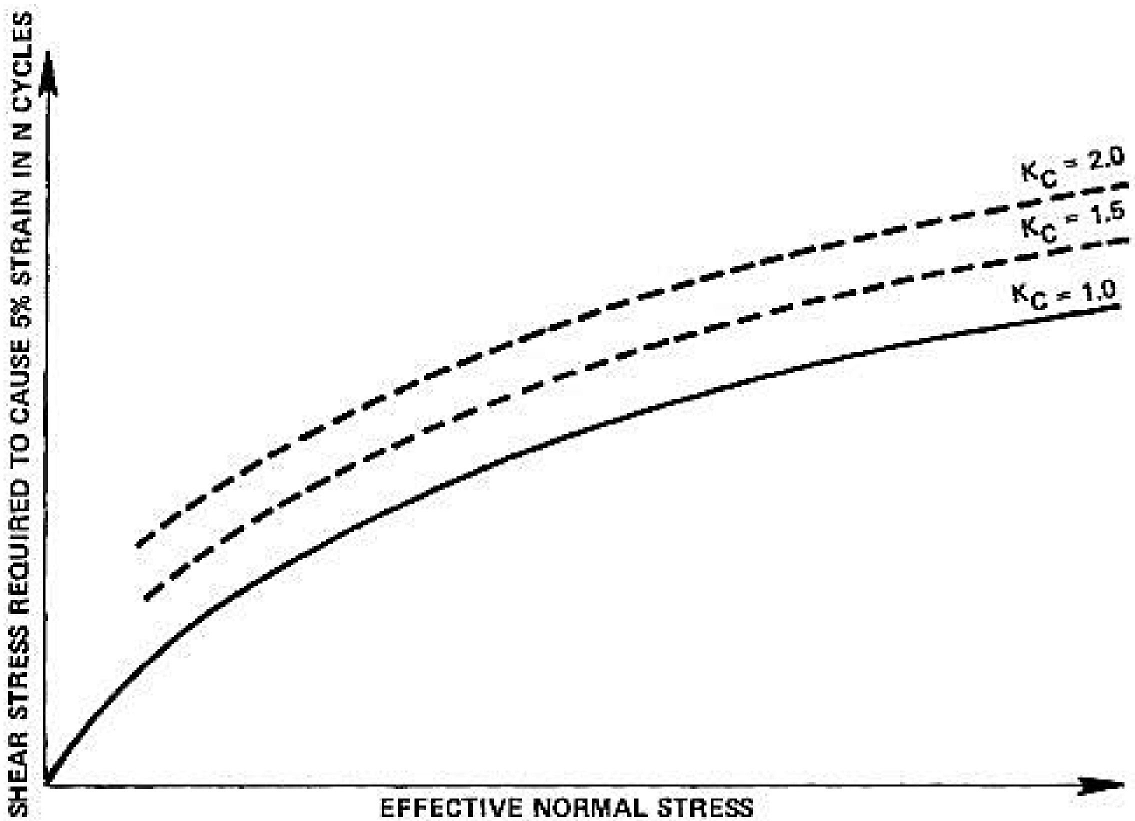


FIGURE 2.5.6-15

BLOW COUNTS OF THE STANDARD PENETRATION TESTS VERSUS DEPTH INSITU RESIDUAL SOIL IN CHANNEL AREAS

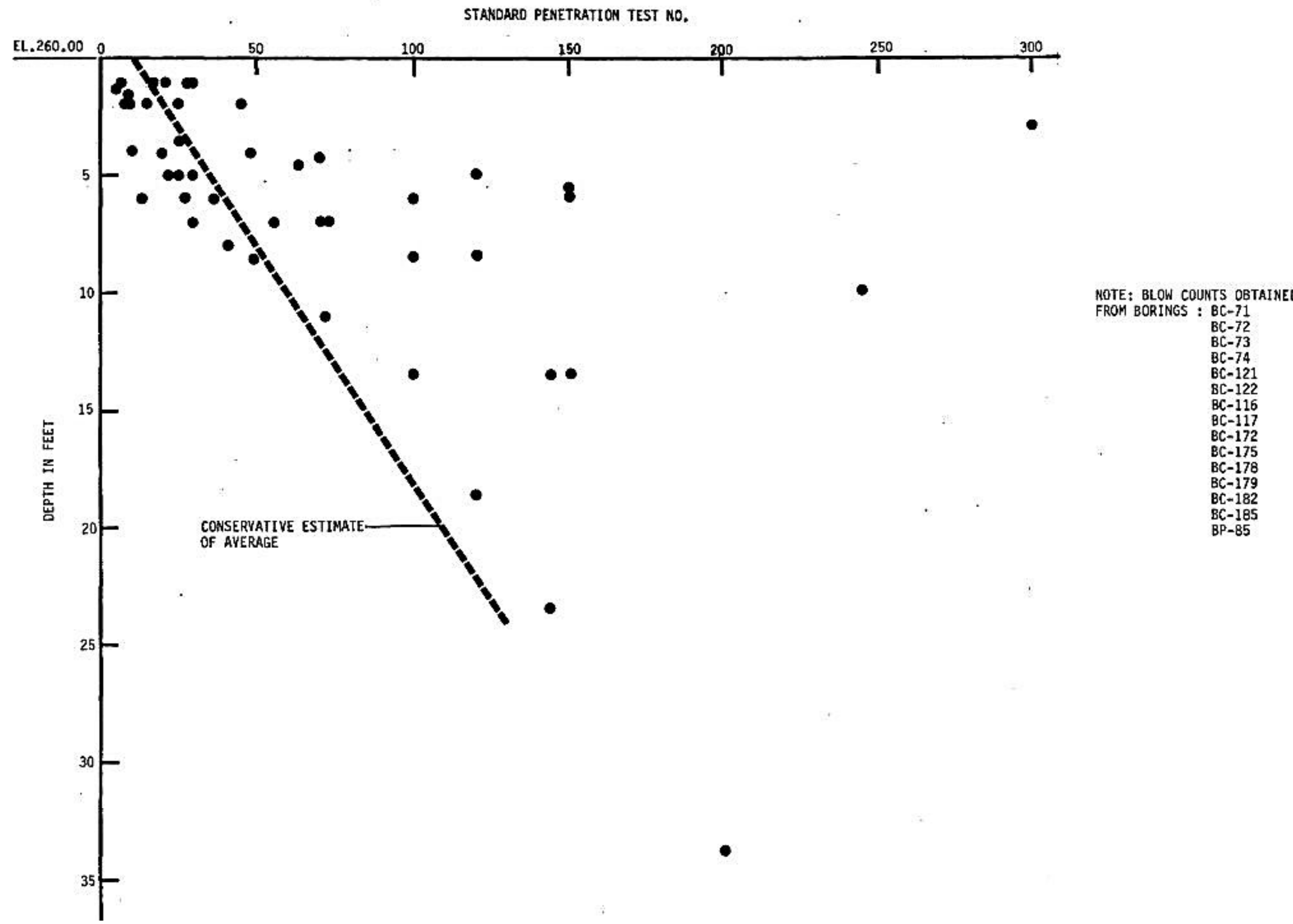


FIGURE 2.5.6-16

DAM MODELING

Security-Related Information - Figure Withheld Under 10 CFR 2.390

A. TYPICAL CROSS SECTION OF THE MAIN DAY

Security-Related Information - Figure Withheld Under 10 CFR 2.390

B. TYPICAL FINITE ELEMENT REPRESENTATION FOR DYNAMIC ANALYSIS OF THE MAIN DAY

FIGURE 2.5.6-17

TYPICAL DISTRIBUTION OF SHEAR STRESSES AND EVALUATION OF FAILURE POTENTIAL ALONG A SELECTED PLANE

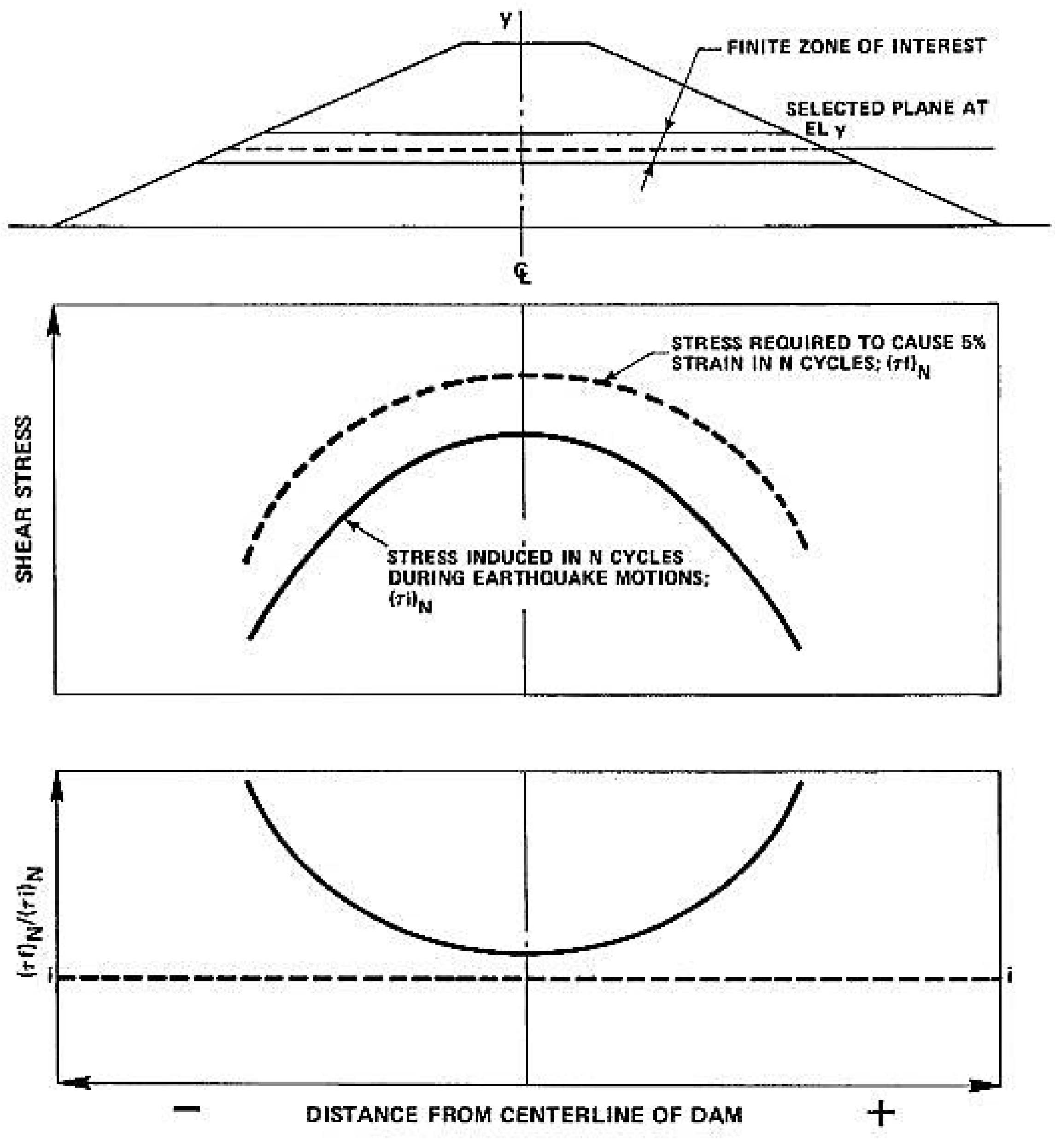


FIGURE 2.5.6-18

AUXILIARY DAM – WEDGE ANALYSIS

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.6-19

MAIN DAM STABILITY ANALYSIS STEADY SEEPAGE

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.6-20

MAIN DAM STABILITY ANALYSIS DOWNSTREAM DRAWDOWN

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.6-21

AUXILIARY DAM STABILITY ANALYSIS

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.6-22

AUXILIARY DIKE STABILITY ANALYSIS

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.6-23

AUXILIARY RESERVOIR CHANNEL SLOPE STABILITY

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.6-24

EMERGENCY SERVICE WATER INTAKE CHANNEL – “FILL” SECTIONS SLOPE STABILITY

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.6-25

EMERGENCY SERVICE WATER DISCHARGE CHANNEL AND EMERGENCY SERVICE WATER INTAKE CHANNEL “CUT” SECTION SLOPE STABILITY

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5.6-27

Security-Related Information - Figure Withheld Under 10 CFR 2.390

REF DWG: PART OF CAR-2167-G-6233 (REV 2)

FIGURE 2.5K-1

SERVICE WATER CHANNELS PLAN

Security-Related Information - Figure Withheld Under 10 CFR 2.390

FIGURE 2.5K-2

AUXILIARY RESERVOIR CHANNEL

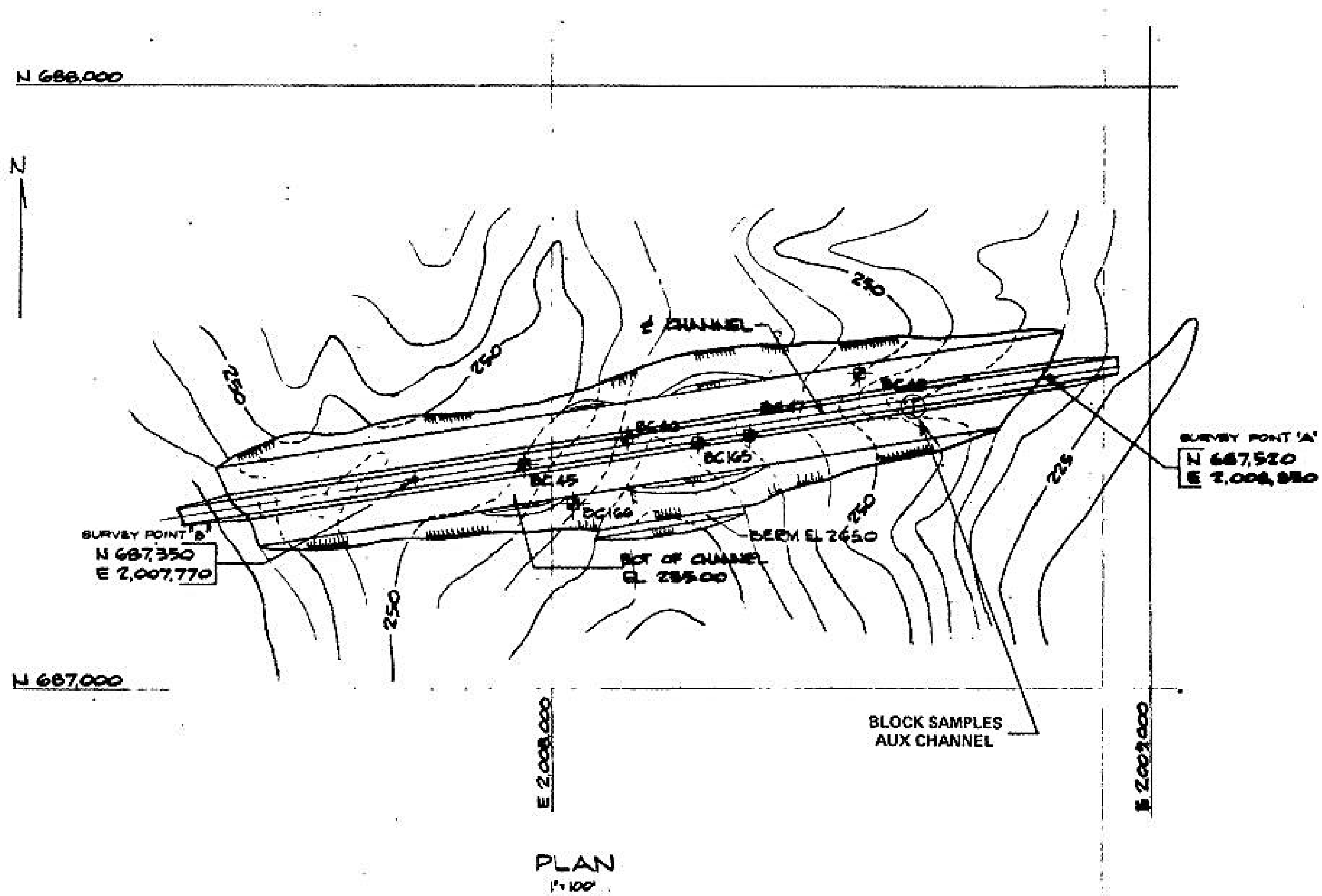


FIGURE 2.5K-3

CATEGORY I CHANNEL LINING STUDY – DRAINED TRIAXIAL TESTS EMERGENCY SERVICE WATER INTAKE CHANNEL – COMPOSITE SAMPLES

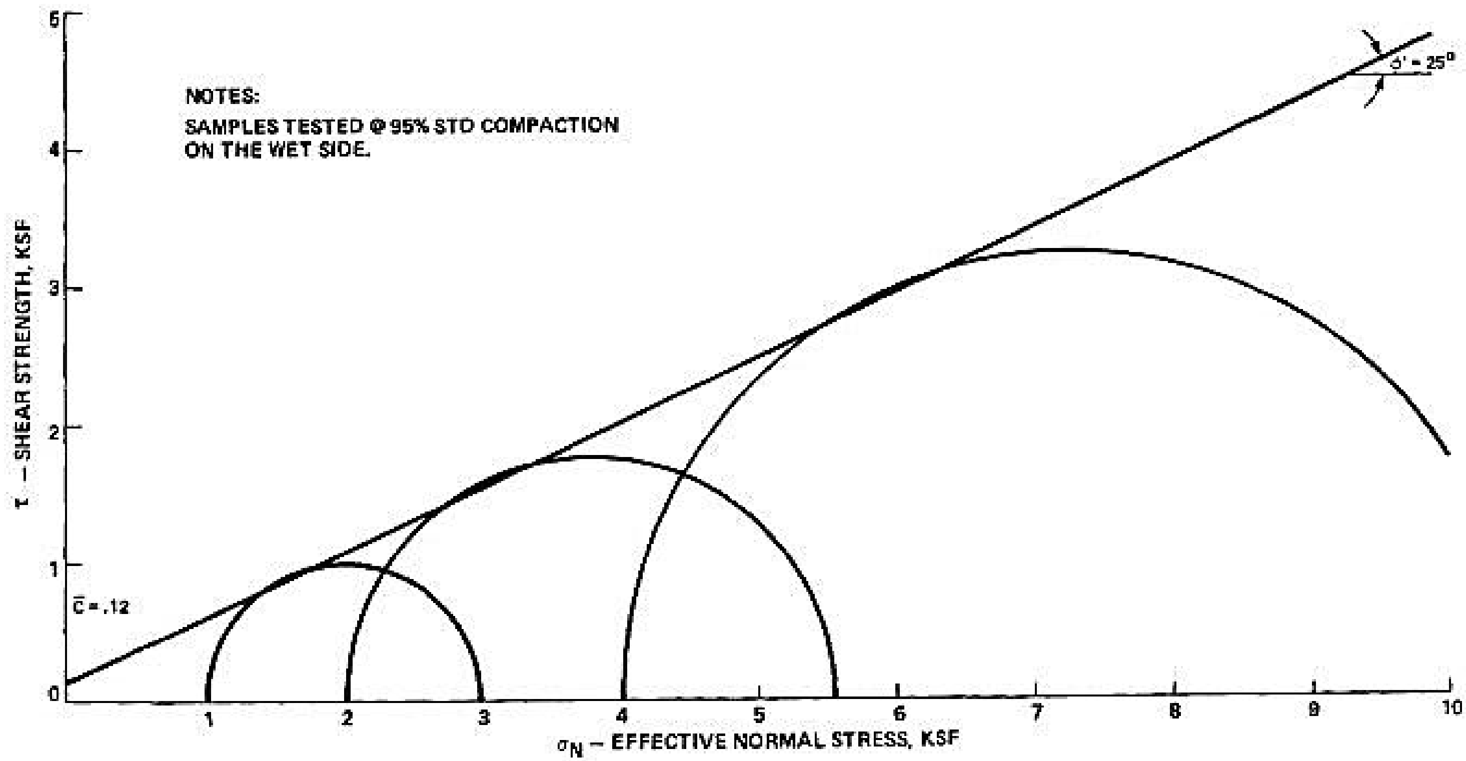


FIGURE 2.5K-4

CATEGORY I CHANNEL LINING STUDY CONSOLIDATED – UNDRAINED TRIAXIAL TESTS EMERGENCY SERVICE WATER INTAKE CHANNEL BLOCK SAMPLE

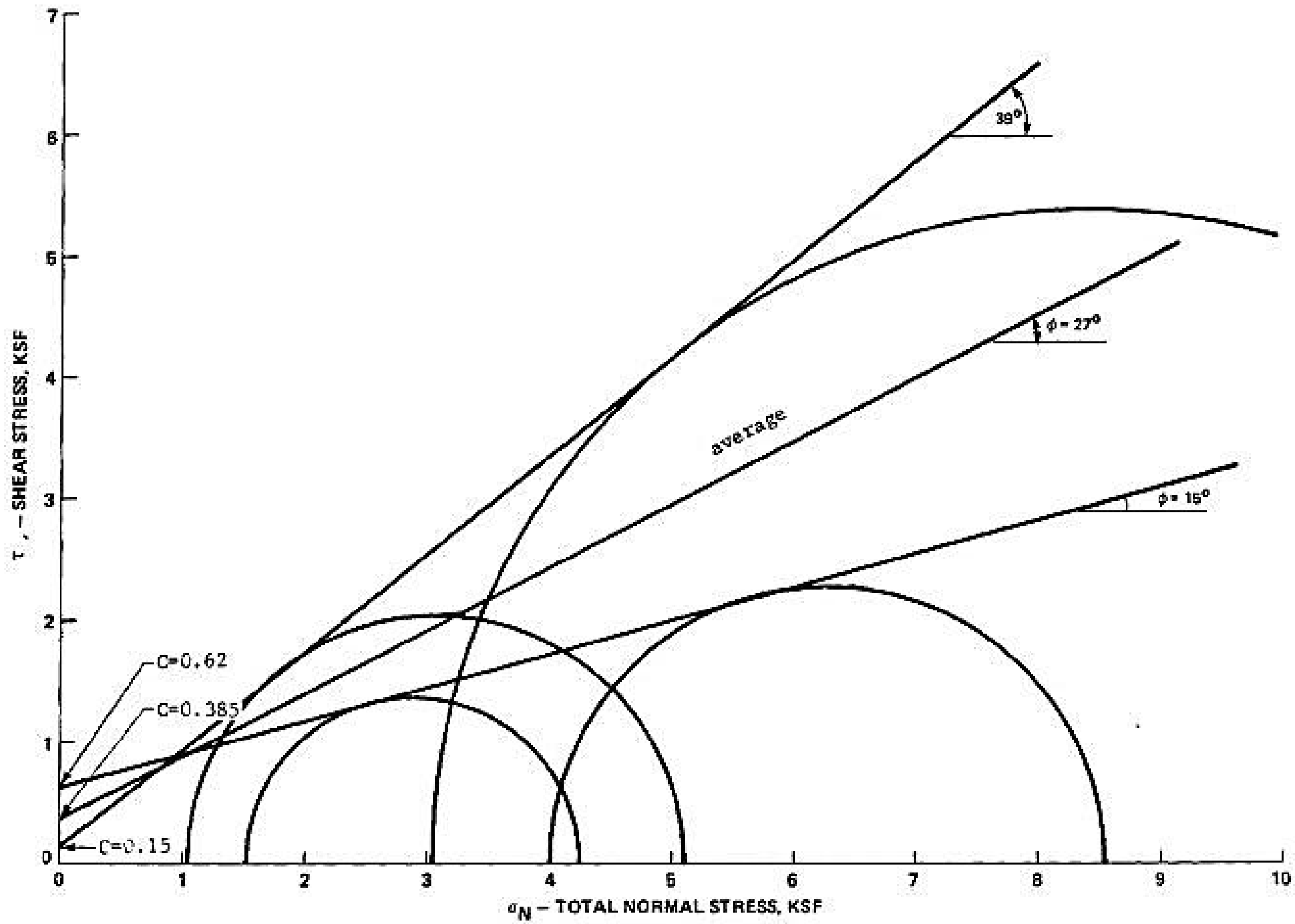


FIGURE 2.5K-5

CATEGORY I CHANNEL LINING STUDY UNCONSOLIDATED – UNDRAINED TRIAXIAL TEST SERIES ESWIC BLOCK SAMPLE

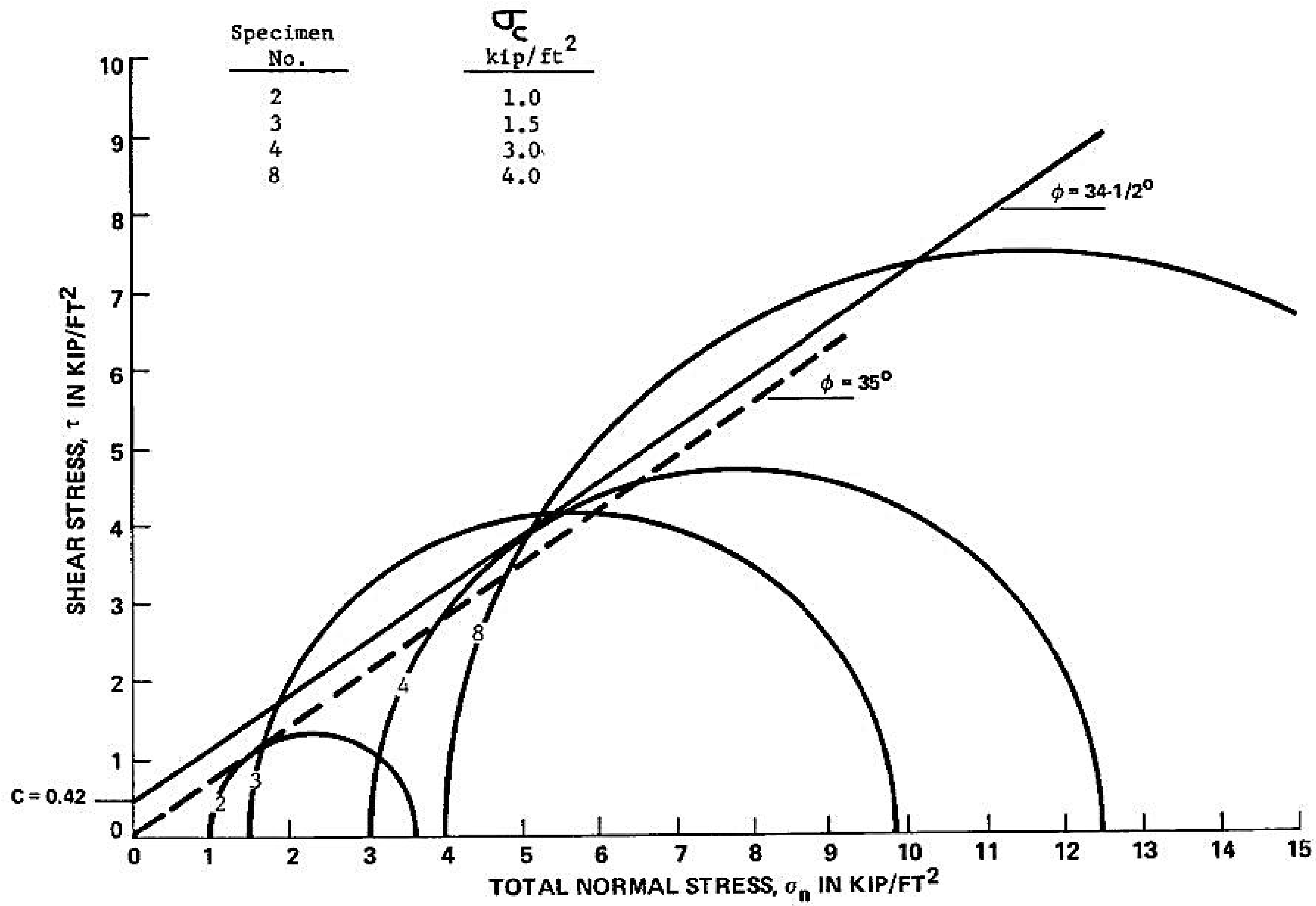


FIGURE 2.5K-6

CATEGORY I CHANNEL LINING STUDY CONSOLIDATED – ISOTROPICALLY UNDRAINED TRIAXIAL TEST SERIES ESWIC BLOCK SAMPLE

Spec. No.	w %	γ_d pcf	$\bar{\sigma}_c$ ksf
1	16.4	118.3	1.0
7	18.9	111.8	1.5
5	14.9	122.3	3.0
6	21.5	107.4	4.0

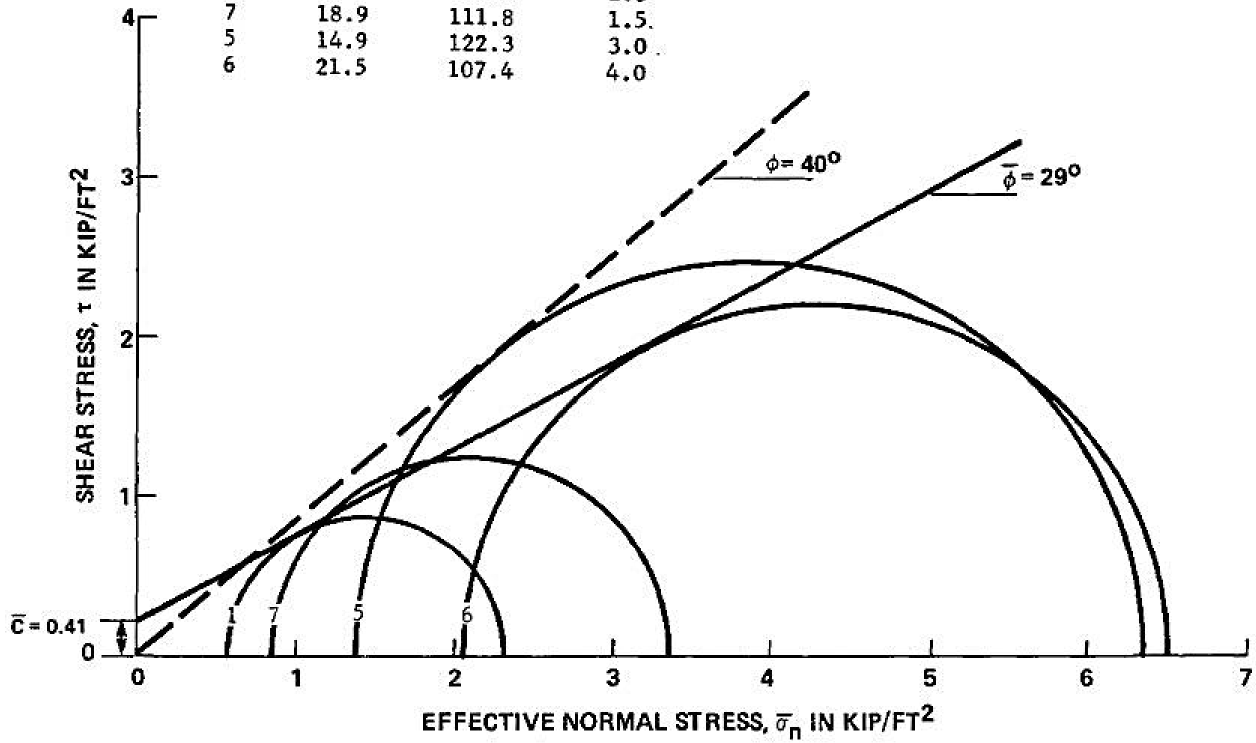


FIGURE 2.5K-7

CATEGORY I CHANNEL LINING STUDY – UNDRAINED TRIAXIAL TESTS EMERGENCY SERVICE WATER INTAKE CHANNEL – COMPOSITE SAMPLES

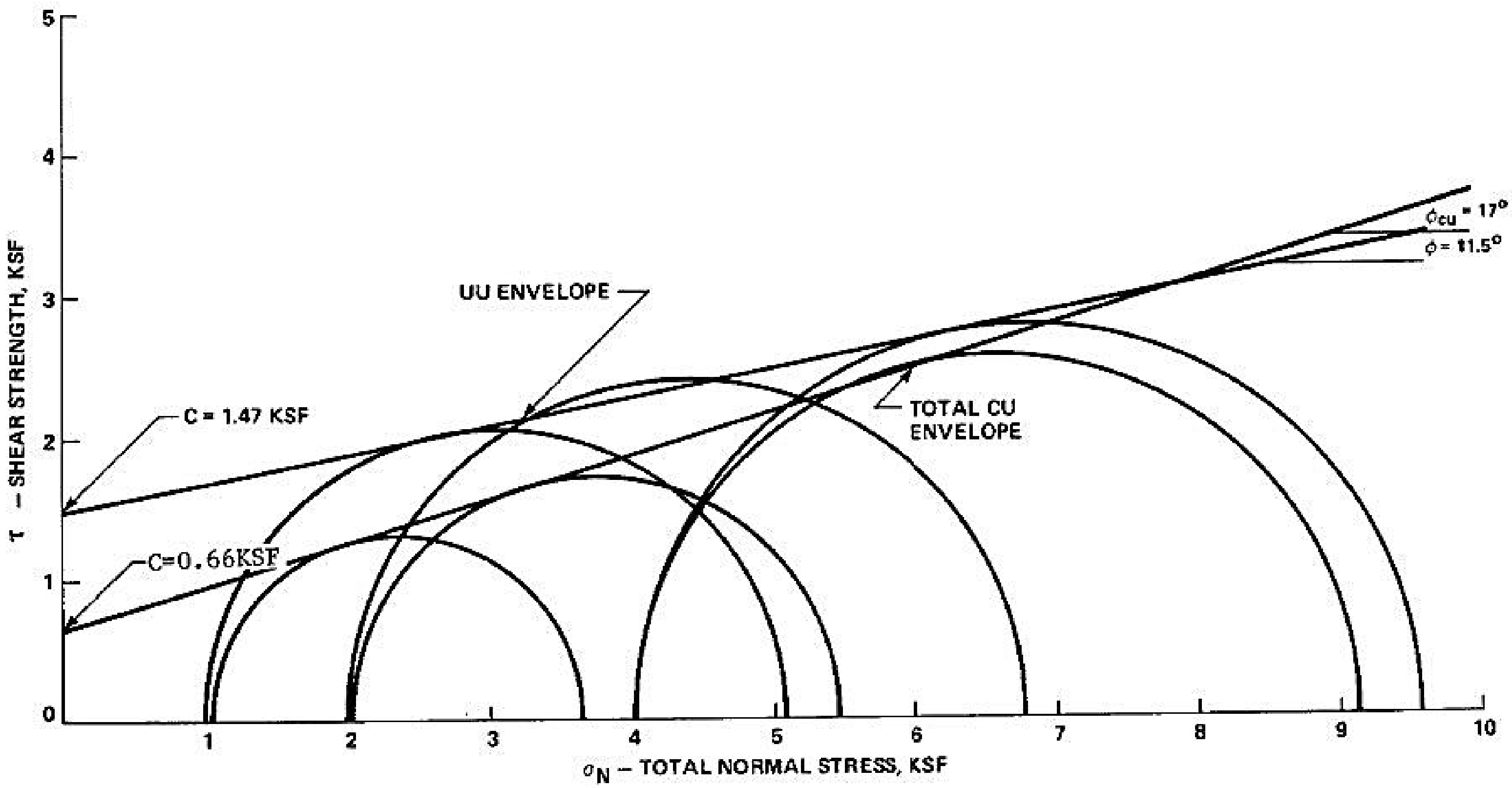
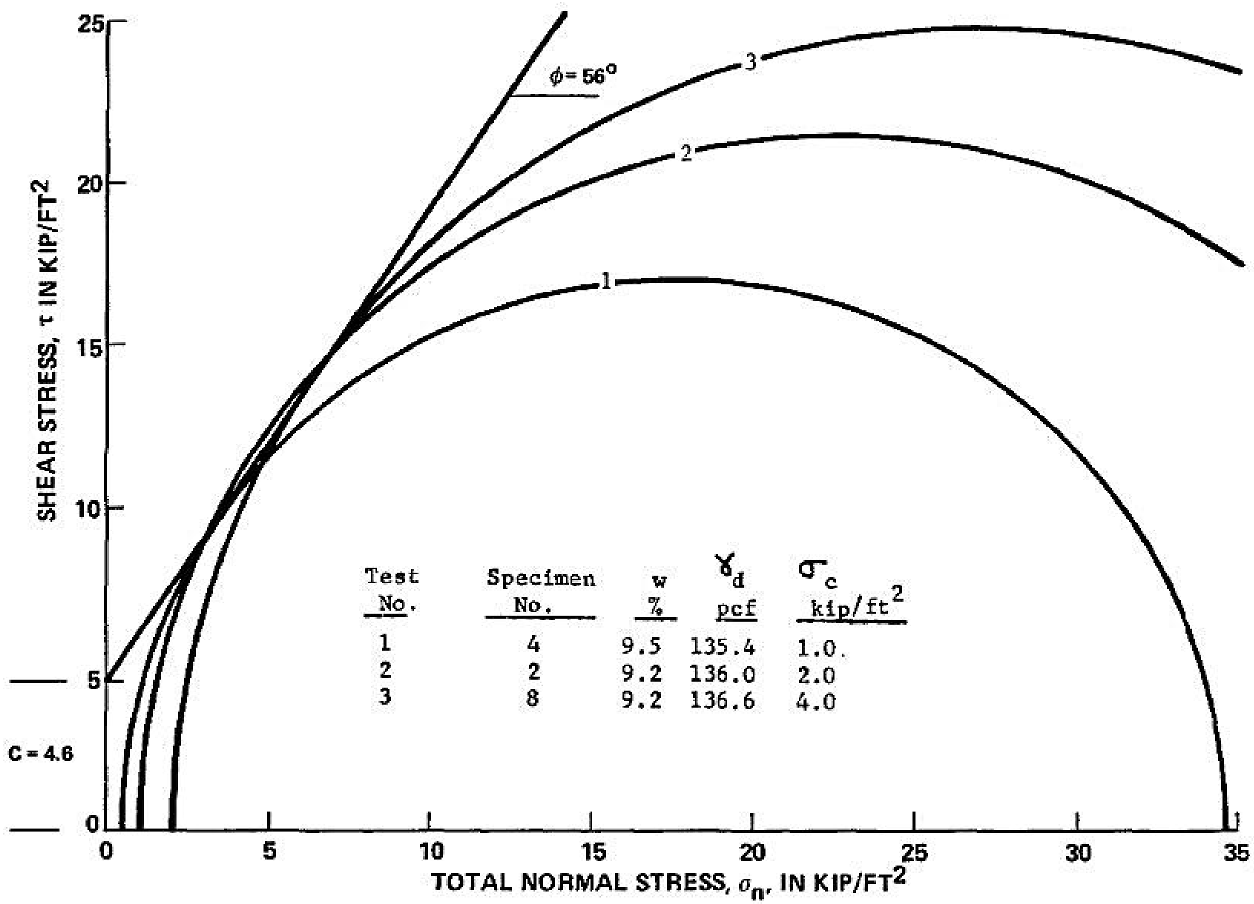


FIGURE 2.5K-8

CATEGORY I CHANNEL LINING STUDY – UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST SERIES AUX. BLOCK SAMPLE



Test No.	Specimen No.	w %	γ_d pcf	σ_c kip/ft^2
1	4	9.5	135.4	1.0
2	2	9.2	136.0	2.0
3	8	9.2	136.6	4.0

FIGURE 2.5K-9

CATEGORY I CHANNEL LINING STUDY CONSOLIDATED – ISOTROPICALLY DRAINED TRIAXIAL TEST SERIES AUX. BLOCK SAMPLE

Specimen No.	ω %	γ_d lb/ft^3	$\bar{\sigma}_c$ kip/ft^2
5	10.2	136.1	1.0
7	10.1	136.2	2.0
6	10.3	135.6	4.0