SEABROOK STATION
UPDATED FINAL SAFETY ANALYSIS REPORT

CHAPTER 2
SITE CHARACTERISTICS

SECTIONS

APPENDICES

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 Site Location and Description

2.1.1.1 Specification of Location

The site property consists of approximately 889 acres near the northern boundary of the town of Seabrook, Rockingham County, New Hampshire. The site is situated about 8 miles southeast of the county seat of Exeter; 5 miles northeast of Amesbury, Massachusetts; and 2 miles west of Hampton Harbor inlet. The site is bordered on the east by an extensive saltwater marsh and is located on a point of land called "The Rocks," between two small tidal estuaries, the Brown's River and the Hunt's Island Creek. The center of the Boston metropolitan area is approximately 40 miles south-southwest of the site. Figure 2.1-1 shows the site location in relation to principal cities and towns within a 50-mile radius.

Geographical coordinates of the reactor unit are as follows:

	Latitude and Longitude	Universal Transverse Mercator Coordinates
Unit No. 1	N 42° 53' 55.4" W 70° 50' 58.7"	4751005 m N (Zone 19) 348994 m E

2.1.1.2 Site Area

Figure 2.1-2 shows major transportation arteries and prominent natural features within 10 miles of the site. The details of the area within 5 miles of the site are shown on Figure 2.1-3. The site property plan, showing the site boundary and the location and orientation of principal structures within the site, is shown on Figure 2.1-4 and Figure 2.1-5. The site boundary will also be the exclusion area, as defined in 10 CFR Part 100. The minimum exclusion radius is 3000 feet measured from the center of the Containment Building to the nearest property line.

There are no industrial or recreational facilities located within the site boundary. There are no residential homes within the 3,000-foot exclusion radius measured from the center of the Unit I Containment Building. However, New Hampshire Yankee operates a public information center (Science and Nature Center) onsite approximately 1500 feet southwest of the reactor.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

The boundary for establishing gaseous effluent release limits is the site boundary. The area within that boundary consists of:

a. Land and marsh surface area above the mean high water line

b. Two tidal streams, Brown's River and Hunt's Island Creek, which are public waterways of the State of New Hampshire.

The area referred to in a. above consists of real property with ownership as described in Updated FSAR Subsection 1.2.1.2. Public Service Company of New Hampshire has full legal right to control access to that area for all purposes. The area referred to in b. above consists of two tidal streams, both of which are virtually dry at low tide. While the public has the right to use these waters for boating and fishing, the actual occupancy rate is necessarily low and of short duration because of the small size and tidal nature of these streams which make them impassable at low tide. Numerous observations, as described in Subsection 2.1.3.3e.3, of boating activity on these streams during the summer boating season has shown no significant use with only an occasional boat passing through the 3000-foot exclusion area.

Access to the area within the site boundary is controlled by signs at normal access points, e.g., Rocks Road, the main access road, and Brown's River and Hunt's Island Creek, and by visual observation where practical. The presence of individuals within the site boundary who are using the public waterways would necessarily be of short duration because of the tidal nature of these waterways.

Figure 2.1-4 shows the location of the site with respect to adjacent bodies of water and the distance from the plant's gaseous effluent vent located on top of the Reactor Containment Building to the nearest point on the site boundary in all directions.

All liquid radwaste effluents are discharged from the station to the Atlantic Ocean via a submerged multiport diffuser beginning approximately 1.1 miles off Hampton Harbor inlet. The concentration of all radioactive liquid effluents at the point of discharge from the diffuser will be below the limits specified in the Offsite Dose Calculation Manual (ODCM). The dose objectives of Appendix I to 10 CFR Part 50 will be met at the edge of the initial mixing area where the effluents have undergone immediate mixing (prompt dilution) only.

The radioactive releases expected for normal operation of the station are given in Subsections 11.2.3 and 11.3.3.

2.1.2 Exclusion Area Authority and Control

2.1.2.1 Authority

As shown in Figure 2.1-4, the exclusion area for the Seabrook site is the site boundary. The area within the site boundary will provide a minimum distance from the Containment Building to the site boundary of 3000 feet. The exclusion boundary presently includes, in addition to lands owned by the Public Service Company of New Hampshire, the following:

a. A section of railroad track owned by the Boston and Maine Railroad

b. A section of power transmission line owned by the Exeter and Hampton Electric Company

c. Portions of the Brown's River and Hunt's Island Creek.

The Boston and Maine Railroad line, which approaches within about 2100 feet of the reactor containment, is not used for passenger service, and is used only infrequently for freight service. The line is currently terminated to the south by an inoperable bridge between Newburyport, Massachusetts and Salisbury, Massachusetts. Provisions have been made with the Boston and Maine Railroad to control traffic on the railroad right-of-way passing through the site in case of an emergency.

The power transmission line passing through the site is owned by the Exeter and Hampton Electric Company. The line is routed overhead outside of the protected area north of the site (Figure 2.1-4). Provisions have been made with the Exeter and Hampton Electric Company to control access to the right-of-way in case of an emergency.

The control of traffic in case of an emergency on those portions of the Brown's River and Hunt's Island Creek that fall within the exclusion area comes under the authority of the director of the New Hampshire Office of Emergency Management.

2.1.2.2 Control of Activities Unrelated to Plant Operation

Other than transit through the exclusion area by the Boston and Maine Railroad, and infrequent boat traffic along the Brown's River and Hunt's Island Creek, along with the access to Brown's River provided by Rocks Road, activities on the site that are not directly related to plant operation occur at the Science and Nature Center, the firing range, and the fitness center.

The Science and Nature Center, located about 1500 feet southwest of the plant, is open specific hours and provides a focal point for the general public to become familiar with nuclear power and Seabrook Station. It is not expected that more than a few hundred people would normally be present at the Science and Nature Center at any one time.

A portion of the firing range is inside the exclusion zone, and the facility will be available for use by local law enforcement agencies. However, such use is expected to occur only several times annually, and Seabrook Station security personnel will be present and will control all activities during operation of the facility. No more than approximately 40 people would normally be expected to be present at the firing range at any one time.

The fitness center is located inside the exclusion zone. Administrative controls address evacuation of this facility if required.

Access to the Science and Nature Center, the firing range, and the fitness center is via the main access road off of Route 1. It is expected that, upon notification, individuals in these facilities could be moved to beyond the exclusion boundary in less than fifteen minutes.

2.1.2.3 Arrangements for Traffic Control

A letter agreement has been reached with the Seabrook Station Emergency Response Organization and the State of New Hampshire so that in case of an emergency, the State of New Hampshire will notify the Boston and Maine Railroad to prohibit rail traffic through the exclusion area.

2.1.2.4 Abandonment or Relocation of Roads

The existing Rocks Road will be abandoned approximately 1200 feet west of the Boston and Maine Railroad track. Relocated Rocks Road will be accessible from the north plant access road off of U.S. Route 1 at a point near the permanent Guardhouse. It will traverse parallel to the plant fence in an east and then southeast direction to the Rocks Road dock.

2.1.3 Population Distribution

Data from numerous sources were used in developing distributions and projections of permanent resident and transient populations within 50 miles of the Seabrook site. This area includes portions of New Hampshire, Massachusetts, and Maine. Each data source is identified in the applicable section. Census data from 1970 and more recent years, where available, have been used in this activity.

2.1.3.1 Permanent Population within 10 Miles

The area within 10 miles of the site includes portions of the states of New Hampshire and Massachusetts. Table 2.1-1 lists municipalities in each state which are located wholly or partly within 10 miles of the site. Also, shown on Table 2.1-1 are the 1970 resident population and estimated permanent resident populations for 1980 and 1983. Figure 2.1-2 is a map of the area within 10 miles of the site. Concentric circles have been drawn at radii of 1, 2, 3, 4, 5, and 10 miles, centered among plant structures approximately 250 feet southwest of the Containment Building. The circles have been divided into twenty-two sectors with each sector centered on one of the 16 cardinal compass points. The population distribution has been developed for each area formed by the series of concentric circles and radial lines.

Figure 2.1-6 and Figure 2.1-7 shows the projected resident population distribution within 10 miles of the site for the years 1983 and 2025, respectively. These times are the estimated year of Unit 1 startup and the approximate end of the life of the facility. Table 2.1-2 breaks down the resident population by segment for the two dates stated above and for the census decades between 1980 and 2020. This table also presents cumulative populations for annular rings and for radial distances. Those subdivisions of Figure 2.1-6 and Figure 2.1-7 and Table 2.1-2, which show a zero resident population, indicate an uninhabited area or water.

The distribution of the permanent resident population for 1983 within 5 miles of the site was determined using population projections and residential electric meter use data for 1978 and 1979, a mosaic of aerial photographs taken in July 1978, and the results of a field survey performed in December 1978. The procedures that were used are described below.

Within 1.25 miles of the site, the current resident population was estimated from an aerial photomosaic supplemented by a count of houses made during a field survey conducted in December 1978. An average household occupancy factor based on the 1970 U.S. Census of Housing data was applied. The rates used were 3.25 persons per household for Seabrook and 3.75 persons per household in Hampton Falls (Reference 1).

A second method was used to determine the population distribution within the 5-mile radius. A system of concentric circles and radial lines were superimposed on a map of electric meter reading routes (or pattern areas) within towns included within the 5-mile radius of Seabrook Station, excluding a small portion of North Hampton located between 4½ and 5 miles north of the site. For those communities within the 5-mile radius of the site, the residential electric meter data provided the basis for allocation of both the current and projected future resident populations to the defined sectors. First, portions of meter reading routes were assigned to the various sectors and counts of residential electric meters were made. Counts were made from electric use data collected over an extended record period (12 months) during 1978 and 1979. Table 2.1-3 provides a summary of the total residential living units based on counts of electric meters by town as well as estimates of those associated with seasonal and year-round residences. The proportion of year-round residential meters in a pattern to those included within an entire town was determined. These same proportions were used to distribute the permanent population estimates within the 5-mile radius for each of the above noted years. For that portion of North Hampton within the 5-mile radius, an equal area allocation approach was used to distribute the projected resident population since electric meter information was not available. Growth rates described below were used to project the population for 1983 and later years.

The distribution of population in the area between 5 and 10 miles from the site was made by area allocation in conjunction with review of local USGS maps. The fraction of a town's populated areas within each sector defined by the grid of concentric circles and radial lines was determined. The same fraction of each town's population was assigned to that segment.

The projected populations for towns in New Hampshire through the year 2000 were obtained from the report, "Interim Revisions - New Hampshire Population Projections for Towns and Cities to the Year 2000" (Reference 2). This document, dated August 1977, was prepared by the New Hampshire Office of Comprehensive Planning. The interim revisions were issued to account for observed trends that did not conform to the population projections made in 1975. Consequently, updated projections for southeastern New Hampshire, which includes the towns around the site, were included in the interim revisions. The projections were made by a three-step procedure. The steps include (1) statewide population forecasts by age and sex, (2) regional forecasts by age and sex, and (3) town and city total population forecasts.

The state and regional projections account for population changes over time resulting from births, deaths, and net migration. Fertility, survival, and net migration rates were developed from historic data and anticipated trends.

The population was allocated on the basis of a relative growth index. The index accounts for (1) the potential saturation population of the town, (2) its accessibility to nearby urban centers, and (3) its competitive advantage for attracting growth as compared to the region's other towns. Populations of towns for periods after the year 2000 were estimated by assuming the same rate of change in population projected to occur between 1990 and 2000.

The future populations of Massachusetts towns were taken from two sources. A town-by-town projection was prepared for 1980 and 1985 by the Massachusetts Office of State Health Planning (Reference 3). The methodology is similar to that described above for the New Hampshire projections with the exception that birth, death, and net migration rates were estimated on a town-by-town basis. Reference 4 describes the methodology in detail. For the years 1990 and 2000, the projected change in population is based on the estimates of the Bureau of Economic Analysis (Reference 5). These projections have been developed for individual states. For time periods beyond 2000, the projected growth rate from 1990 to 2000 was used.

2.1.3.2 Permanent Population between 10 and 50 Miles

The 50-mile radius around the site includes portions of New Hampshire, Massachusetts, and Maine. Concentric circles have been drawn with radii of 10, 20, 30, 40, and 50 miles centered on the site. The circles have been divided into twenty-two sectors with each sector centered on one of the 16 cardinal compass points. The population distribution has been developed for each area formed by the series of concentric circles and radial lines.

Figure 2.1-8 and Figure 2.1-9 shows the projected resident population distribution within 50 miles of the site for the years 1983 and 2025, respectively. Table 2.1-4 breaks down the resident population by sectors for the dates stated above and for census decades between 1980 and 2020. Those sectors on Figure 2.1-7 and Figure 2.1-8 and Table 2.1-4, which show a zero population, indicate uninhabited areas, primarily water.

The distribution of population in the area between 10 and 50 miles was made by equal area allocation.

The bases for projecting populations in cities and towns in Massachusetts and New Hampshire have been described in the previous section. For cities and towns in Maine, 1980 population estimates were obtained from Reference 6. These projections are provided for use by local, regional, and state agencies for planning purposes. The projections are based on population trends between 1970 and 1975 observed in the municipality itself, the surrounding cluster of municipalities, and the county in which the municipality is located. For 1990 and 2000 projections, the rates of change were taken from the Bureau of Economic Analysis for the State of Maine (Reference 5) and applied to the cities and towns. The rate of change between 1990 and 2000 was used to project for later times.

2.1.3.3 Transient Population

a. Seasonal Resident Population (0 to 10 miles)

The seasonal resident population has been estimated from the number of seasonal dwelling units in the area. The number and distribution of seasonal homes were based on a review of several sources of data including (1) 1961 general highway maps for Rockingham County, New Hampshire, (2) 1970 U.S. Census of Housing data on vacant seasonal and migratory units for towns within the 10-mile area, (3) 1978-1979 electric meter use data for New Hampshire towns within 5 miles (excluding North Hampton), (4) 1978 weekday/weekend occupancy survey of beach area housing, (5) 1978 aerial photography, and (6) 1979 telephone survey of town assessors and building inspectors. These data indicate that seasonal residential units are concentrated in the beach area sectors.

Figure 2.1-10 provides a current estimate and distribution of seasonal dwelling units within 10 miles based on annual electric use distribution patterns within 5 miles and 1970 U.S. Census of Housing estimates for the areas between 5 and 10 miles. Total seasonal residential units within the 5-mile radius of Seabrook Station are estimated at 4,013. Approximately 71 percent (2,843) of these units are located in the Seabrook-Hampton Beach area, 19 percent (755) in the Salisbury Beach area, and 10 percent in nonbeach areas. A comparison of the 1961 county highway map data (New Hampshire), 1970 Census data, and 1978-1979 electric meter use data indicated an increase in New Hampshire total units of about 22 percent between 1961 and 1970 and a 22 percent decrease between 1970 and 1978-79. Thus, more recent growth of seasonal units is believed limited in the vicinity of the site with the decrease in seasonal units, probably due to conversion of seasonal homes to year-round use. A 1978 survey of beach area housing indicated weighted average weekday and weekend occupancies per seasonal residents of 5.4 and 7.6 persons, respectively. Figure 2.1-11 and Figure 2.1-12 provides estimates of the respective current seasonal resident populations for typical summer weekday and weekend conditions within a 10-mile radius. It is estimated that approximately 30,555 persons are associated with the seasonal homes on a weekday and about 43,012 on a weekend day within 10 miles.

A 1979 telephone survey of personnel in the building inspector and town assessor offices for communities within the 10-mile study area was undertaken. The limited survey indicated that only small numbers of new seasonal dwelling units have been constructed in either the beach area or inland areas in recent years. In both the towns of Seabrook and Salisbury, the total number of seasonal dwelling units has decreased from 1970 to 1979 by approximately 400 and 200 units, respectively. These decreases were largely attributed to conversions of seasonal units to permanent, year-round units. In the town of Hampton, there has been an estimated increase of several hundred seasonal dwelling units between 1970 and 1979. Most other town officials surveyed commented that the total stock of seasonal units has remained relatively constant in the last few years.

b. Overnight Accommodations (0 to 10 miles)

Survey work was undertaken to determine the locations and estimated capacities of major overnight accommodations within 10 miles of Seabrook Station. Such accommodations included hotels, motels, and a number of guest houses. Information for the 0- to 5-mile area was based on survey work undertaken during the summer of 1978. The 5-to 10-mile information was developed as part of survey work during the summer of 1979. Several hundred facilities were identified with an estimated capacity of 11,024 within the 10-mile radius.

The majority of facilities surveyed were concentrated in the "beach area" and within the 5-mile radius. A total of 210 facilities were identified in the 0- to 5-mile study area with an estimated capacity of 10,019 people as compared to only 20 such facilities identified in the larger 5- to 10-mile study area with an estimated capacity of 1,005. Thus, 91 percent of the total overnight accommodation capacity is within a 5-mile radius of the site with the remaining 9 percent being located within the 5- to 10-mile study area. Approximately 82 percent of the total capacity within the 5-mile radius is found in the beach area, primarily in Hampton Beach.

Table 2.1-5 is a summary listing of accommodations identified during both the 1978 and 1979 survey periods. Information including sector location and capacity is noted in this table for each facility. Information on facilities was obtained from a variety of sources, including local telephone directories, tourist information brochures, and limited telephone and field survey work. Estimated peak overnight populations associated with hotels, motels and guest houses have been summarized on Figure 2.1-13 for a 10-mile radius.

c. Campgrounds

An inventory of both public and private campgrounds was made for the area within 10 miles of Seabrook Station. Table 2.1-6 summarizes information collected on this subject. Information on location, number of campsites and estimated capacity of surveyed camping facilities is included in the table. Figure 2.1-14 summarizes the distribution of the estimated capacity of the camping population for the 10-mile study area. A total of 17 facilities with an estimated capacity of about 7,648 was identified as part of this inventory. Six of these facilities, with a total estimated capacity of approximately 3,160 campers, are located within a 5-mile radius.

Several sources were consulted to assemble campground information for the New Hampshire and Massachusetts portions of the 10-mile study area. References included local telephone directories, 1979 New Hampshire Camping Guide, 1977 New Hampshire Outdoor Recreation Plan, New Hampshire Campground Owner's Association Guide - 1979, Massachusetts Department of Environmental Management's Space Inventory - 1978, Camping in Massachusetts (Division of Tourism), Massachusetts Department of Commerce and Development, and Massachusetts Outdoors - 1978 Massachusetts Department of Environmental Management. Limited field observations (Exeter, Kingston, Hampton Falls, North Hampton, and Seabrook) and telephone communications (Rye and Exeter) with local town offices provided additional information on camping facilities.

d. Beach Area Daily Transient Population

1. Parking Lot Capacity Estimates

A substantial daily transient population during the summer period is associated with the beaches and other recreational facilities in the vicinity of Seabrook Station. The coastal beaches within 10 miles of Seabrook Station extend from the Plum Island Beach in Newbury, Massachusetts to Wallis Sands Beach in Rye, New Hampshire. The beaches are generally readily accessible to the public in this area. Estimates of available parking spaces in the beach area provided the basis for estimating this daily transient population category since the majority of daily transients are assumed to arrive by automobile. During the summers of 1978 and 1979, beach area parking lots were identified by field inspection. Capacity estimates were developed from interviews with parking lot operators and by review of current aerial photography of the beach area parking lots.

An average automobile occupancy factor was estimated at 3.2 persons per vehicle. This figure is based on a survey conducted in July 1978 at Hampton and Salisbury Beaches. Table 2.1-7 provides a summary of beach area parking facilities, capacity estimates, and the maximum number of vehicles observed as part of a 1979 summer survey program for these lots. Within the 10-mile radius, it is estimated that 13,336 parking lot spaces (including metered street parking spaces, but excluding leased spaces) exist in the beach area. The maximum observed number of vehicles in the fee and free parking lots commonly used by daily transients occurred on July 22, 1979 during the summer survey period. Figure 2.1-15 shows the distribution of vehicles for this date.

Figure 2.1-16 is a capacity-type estimate of the peak population associated with surveyed parking lot spaces.

Some parking spaces in the beach area are leased during the summer months by seasonal residents and by motels for their lodgers; and, therefore, have not been included in the above capacity estimate. Leased spaces identified were located in five municipal parking lots: one in Salisbury and four in Hampton. A total of 559 leased parking spaces were identified in the beach area within 10 miles of the site.

2. <u>Parking Lot Aerial Survey Data</u>

To more accurately estimate the total numbers of daily transients entering the beach area and temporal variations in the daily transient population, data were collected and analyzed from June through September 1979. Aerial photographs (color slides) were taken from a light aircraft flying over the beach area on selected days throughout the summer. Vehicles parked in the major lots available to the public were counted from these slides. In addition, vehicles parked on the streets in the beach area were counted. However, many vehicles parked on the street are associated with other population categories (i.e., seasonal and year-round residents as opposed to daily transients). No distinction between vehicles was made in the counting process for on-street vehicles. Aerial photographs were taken and analyzed for a total of 64 days during the summer of 1979: 42 weekdays and 22 weekend days. This includes the 4th of July and Labor Day holidays.

3. Analysis of Parking Lot Data - Summer 1979

Beach area parking facilities available to the public are concentrated within a 6-mile radius of the site. The total capacity of available beach parking lot facilities in the Salisbury/Seabrook/Hampton Beach area is estimated at 11,434 parking spaces (includes Salisbury Beach State Park estimated at 1,921 spaces. Available capacity excludes leased parking spaces). This estimate was based on a survey of lots during the summer of 1979 and represents about 86 percent of the total parking lot spaces within the larger 10-mile radius of the site. During the summer of 1979, beach area lots were observed for 22 weekend and holiday periods and 42 weekday dates. Table 2.1-8 and Table 2.1-9 provide a summary of these survey dates. Observations were made primarily between noon and 3:00 p.m. when greatest use by daily transients can be noted. Figure 2.1-16 illustrates the distribution by time of day of vehicles in beach parking lots for several survey dates for which multiple observations were made.

The maximum number of vehicles observed in the major parking lots in the beach areas of Salisbury, Seabrook and Hampton, for all survey dates, occurred on a fair weather Sunday, July 22, 1979, between 1:00 and 2:00 p.m. At this time, a total of 9,077 vehicles were observed in the various beach area lots, representing 79 percent of the total capacity within approximately 6 miles of the site. For this peak observation during the survey period, beach area lots can be characterized as being either at or near capacity with the exception of several of the larger parking facilities in the Salisbury Beach area. For example, the Salisbury Beach State Park parking lot only reached 59 percent of its estimated 1,921-vehicle capacity on one of the 22 weekend and holiday survey periods in 1979. It was common to observe this facility at only 20-25 percent capacity during fair weather weekend beach days. Likewise, the observed use of this lot was lower for weekday periods than for weekend periods.

For lots within the 6-mile radius, the average number of vehicles observed for the 22 weekend periods was 4,945. This represents about 43 percent of the total available capacity of beach area lots in this radius. The level of observed weekday use was somewhat lower for these same facilities. The average number of vehicles observed for the 42 weekday survey dates was 3,073 or 26 percent of the total available capacity. The maximum number of vehicles in the available parking lots within a 6-mile area of the site for all weekday periods was 5,099 or 45 percent of capacity.

The total capacity of beach area parking lot facilities within the 6- to 10-mile radius is estimated at 1,902 spaces. The maximum number of vehicles observed also occurred on July 22, 1979. Approximately 1,444 vehicles representing about 76 percent of the available lot parking were observed.

The summer survey period extended from the second week in June through the third week in September. Figure 2.1-18 further illustrates the variation in total numbers of vehicles associated with surveyed parking facilities in the coastal area extending for approximately 28 miles along the New Hampshire-Massachusetts shore. Figure 2.1-18 also shows the variation in daily transients between weekend and weekday survey dates.

4. <u>On-Street Parking</u>

(a) <u>Review of Aerial Photography</u>

Survey work was also undertaken to determine the daily transient population using on-street parking in the beach area during the summer by estimating the total capacity of on-street parking spaces available to daily transients.

The number of daily transients in the beach area was estimated by comparing color high resolution aerial photography taken of the beach area at 8:00 a.m. and 1:00 p.m. on a high-use beach day (i.e., a clear and sunny Sunday, July 8, 1979). The scale of this aerial photography is approximately 1:2000.

The beach area was divided into sectors to record the number of vehicles observed parked on local streets. Similar sectors were used for both the on-street parking survey work and the beach area parking lot survey work previously described. These sector boundaries were delineated on the aerial photography and a review of vehicles observed and the number of available on-street parking spaces undertaken.

On-street parking was defined as the number of available parking spaces which are in the right-of-way of a road or within approximately ten feet of the edge of the traffic lane. It was also assumed that vehicles observed parked on-street in the 1:00 p.m. aerial photographs and not found in the same location or in nearby driveways in the 8:00 a.m. aerial photographs belonged to daily transients (e.g., persons making day trips to the beaches). Conversely, it was assumed that vehicles observed in both the 8:00 a.m. and 1:00 p.m. photos belonged to the "seasonal or permanent residents" or "overnight" beach area populations and thus were not associated with the daily transient population.

The number of daily transients may have been conservatively estimated in this manner. For example, some overnight, permanent, or seasonal residents may have moved their vehicles from off-street locations to on-street parking places or may have moved to another on-street parking place some distance away between the 8:00 a.m. and 1:00 p.m. periods. Such vehicles were counted as daily transients. However, most of the vehicles observed parked on-street in the 8:00 a.m. photographs were present, in the same place, in the 1:00 p.m. series.

The extent of illegal parking in the beach area was also considered as part of this survey. Parking ordinances were obtained from town police departments and reviewed. "No Parking" zones were outlined on local street maps from field observations of existing signs. By comparing on-street parking in the above noted 1:00 p.m. aerial photographs against the parking maps, a tabulation of illegally parked vehicles in the beach areas was made for a high beach use day. The amount of illegal parking was determined to be approximately 10 percent of the total number of vehicles observed parked on-street in the 1:00 p.m., July 8, 1979 aerial photography. Most of the "illegally parked" vehicles were observed in a small number of locations in the towns of Seabrook and Hampton.

(b) Estimate of On-Street Parking

Estimating the total capacity of on-street parking involved an examination of the 1:00 p.m., July 8, 1979 aerial photography. Spaces occupied by vehicles as well as empty spaces observed on the streets were recorded. The estimate of "on-street parking" required judgment regarding the distance daily transients would park from the beach. Thus, in all areas, except Hampton Beach, a maximum distance of approximately 600 feet from the beach was assumed as the boundary of "on-street parking" used by daily transients. All streets in Hampton Beach were assessed for their on-street parking capacity since it was observed that more parking at greater distances was common in this area.

From a detailed study of vertical aerial photographs for the beach area, a total of 4,574 vehicles was observed parked on-street at 1:00 p.m., July 8, 1979 within the beach area and within the 5-mile radius. This is 87 percent of the total on-street parking capacity (5,262). Of this total, 2,514 or 48 percent were defined as daily transients since these vehicles were not observed in the 8:00 a.m. photo series. Approximately 10 percent of the total numbers of vehicles parked on-street were parked illegally. Most areas where on-street parking occurred were at or near capacity for the observed date. Half of the empty on-street parking spaces were located north of the more popular Hampton Beach area (in sectors NE 3-4 miles and NE 4-5 miles). The total on-street parking capacity (including legal and illegal parking) for daily transients was estimated to be about 3,202 vehicles within 5 miles. The daily transient population associated with this parking is estimated at 10,246 persons (assuming 3.2 persons per vehicle).

Figure 2.1-19 shows the distribution of on-street parking estimated to be available to daily transients within the 10-mile radius. A beach area on-street parking capacity population estimate is included as Figure 2.1-20.

5. Origin/Destination Survey

An analysis was performed to determine if significant "double- counting" of the "permanent population" within capacity estimates for "daily transients" in the beach area occurred.

Survey work was directed at estimating the percentage of permanent residents residing within the 0- to 5-mile and 5- to 10-mile radii of the site and typically traveling to the Hampton/Seabrook/Salisbury beaches during the summer season for the day. A survey was designed and conducted during the summer of 1979 of individuals arriving by car at major parking lots in this beach area. Information on origins of trips for both weekdays and weekends was collected for selected lot locations. A summary of the survey results is included on Table 2.1-10.

A total of 3,000 questionnaires were completed over a 19-day period during the summer of 1979.

The results of the survey show that for all locations, averaged over all days of the week, about 5 percent of the people surveyed came from within 5 miles of the plant to the beach area, and 10 percent came from within 10 miles of the plant. On weekends, 3 percent of all beach area users came from within the 5-mile radius and 7 percent from within the 10-mile radius. On weekdays, the results are 6 percent from within 5 miles and 14 percent from within 10 miles of the site.

The survey results indicate that the daily beach population is made up primarily of "daily transients" with only a relatively small percentage of the area permanent and seasonal populations using the beach area parking lots. Since the percentage of permanent residents using the beach area parking lots is small, the total capacity of lots was attributed to the daily transient population category.

e. <u>Other Activities</u>

1. Seabrook Greyhound Park

A major commercial dog race track, Seabrook Greyhound Park, is located 2¼ miles west of the site. The facility operates year-round according to the following schedule (as of October 1979):

Evening Activity	Races scheduled nightly, except Sundays, from 7:45 to 11:00 p.m.
Daytime Activity	Races scheduled on Tuesdays and Thursdays between 1:15 and 4:00 p.m. and on Saturday between 12:00 noon and 4:00 p.m.

Track attendance data from January 1977 to September 1979 was reviewed. Highest recorded attendance during this 33-month period was on September 1, 1979, with an evening attendance of 7,027 persons. Observation of the facility on this day indicated that the track's parking lot with approximately 3,100 spaces was nearly full. Table 2.1-11 provides a summary of peak monthly attendance at this facility. The peak capacity of the track is estimated at 7,500 persons.

2. Route 1

Route 1 is a major north-south artery located in the 0- to 10-mile study area. A variety of commercial uses exist along Route 1 including shopping centers, gas stations, restaurants and fast food chains, motels, automobile dealers and repair shops, taverns, gift shops, and building supply stores. Shopping centers found along this route have the greatest concentrations of vehicles. Six shopping centers were identified along Route 1 within the 10-mile radius. These major shopping facilities include:

Vehicles parked at these facilities were recorded for ten days during the summer of 1979. Observations were made between 1:00 and 5:00 p.m. on both weekday and weekend periods. The maximum number of vehicles observed, as noted above, was less than the lot capacity estimates.

Shopping Center	Distance/ Direction	Lot Vehicle Capacity Estimate	Max Observed
Seabrook Plaza	W/0-1 mi	710	265
Seabrook Southgate	SW/1-2 mi	730	460
Convenience Shopping Center	2/3-4 mi	50	22
Hampton Court (lacks major tenant)	N/4-5 mi	750	67
North Hampton Village Shopping Center	N/5-6 mi	140	66

Shopping Center	Distance/ Direction	Lot Vehicle Capacity Estimate	Max Observed
Southgate Plaza	NNE/9-10 mi	550	286

3. Recreational Boating

Recreational boating is prevalent in the summer months in the Hampton Harbor vicinity. Observations of occupied boats (moving and stationary) within the 3,000-foot station exclusion area (Brown's River and Hunts's Island Creek), within a 2-mile radius (Hampton and Blackwater Rivers), and an approximate 5- to 10-mile radius in the Atlantic Ocean were made. Fifty-two observations were made during the summer of 1979 between 1:00 and 4:00 p.m. on both weekdays and weekends of the waters within the 5-mile radius. These observations are summarized on Table 2.1-12.

Only three boats were observed on three dates during the summer in Brown's River within the 3,000-foot exclusion zone of Seabrook Station. No occupied boats were observed on Hunt's Island Creek, the Boston and Maine Railroad Landing, Farm Dock Landing, and Walton Landing.

Boating activity on the Hampton and Blackwater Rivers within a 2-mile radius of Seabrook Station was concentrated within their lower stretches in the Hampton Harbor area. Many of the moving boats observed in Hampton Harbor were either departing for or returning from the Atlantic Ocean. The average weekend observation was 5 occupied boats on the Hampton River and one occupied boat on the Blackwater River. With the exception of the larger, charter deep-sea fishing vessels, the boats observed were small or medium-sized power boats. Few sailboats and no water skiers were observed in this area.

Boating activity in the Atlantic Ocean was largely concentrated within 2 or 3 miles of Hampton Harbor inlet. It is highly probable that many of the sailboats, which accounted for roughly half of all boats observed in the Atlantic, originated at points either north (Portsmouth) or south (Ipswich, Gloucester) of Hampton Harbor. No sailboats were observed in Hampton Harbor.

Boating activity was greatest on warm, sunny weekends and holidays. The largest number of boats observed within the 5-mile radius in the Atlantic Ocean was estimated to be 300. The average weekend observation during the summer in the Atlantic Ocean within 5 miles of Seabrook Station was 95 boats. Weekday boating activity was substantially less than weekend activity.

Boating activity in the 5- to 10-mile area is concentrated on the Merrimack River, approximately 6 to 7 miles south from Seabrook Station, and in Rye Harbor about 9 miles northeast of Seabrook Station.

4. Major Manufacturing

An inventory of major employers was undertaken for the 10-mile study area. Inventory work undertaken relates to populations associated with major manufacturing and industrial facilities. Four primary sources of data were used: (1) Directory of Massachusetts Manufacturers: 1981-82 Edition, George D. Hall Company, 1981, (2) Directory of New England Manufacturers: 1980, New England Council, 1980, (3) MacRae's New Hampshire State Industrial Directory: 1982, MacRae's Blue Book, Inc., 1981, and (4) "Exeter, N.H. Industrial & Manufacturer's Guide and Exeter Grown Products," Exeter Area Chamber of Commerce, 1981. Local telephone directories and limited telephone contacts were made to locate major employers. Major employers were defined as facilities listed in the manufacturers' guides.

Table 2.1-13 is a listing of the "major employers" with their associated reported employment levels. Figure 2.1-21 shows the distribution of this total major employer population within the 10-mile study area. Employment related to small manufacturing, commercial retail, and service-type business is not in this estimate.

Within the 0- to 5-mile radius, there are an estimated 3,547 employees associated with major industrial or manufacturing firms. The number of employees in the 5- to 10-mile radius is estimated to be 6,707. Therefore, in the 0- to 10-mile radius, there are an estimated 10,254 employees. Over 50 percent of these employees are located in a 3-mile ring (between the 4- and 7-mile radii) in the following sectors: WSW, SW, SSW, and S. This area includes the towns of Newburyport, Amesbury, and Salisbury. A total of about 18 percent of the estimated number of employees work in Seabrook, NH in a 2-mile ring (between the 1- and 3-mile radii) in the W, WSW, SW, and S sectors.

5. Educational Facilities

Information on the location of schools was developed for the 10-mile study area. An estimate of population for each facility was developed based on current levels of staffing and enrollment. Table 2.1-14 summarizes this information, while Figure 2.1-22 indicates the distribution of the total school population within 10 miles of the site. It indicates a total school population of 6,020 at 18 facilities within 0 to 5 miles of the site and an additional population of 15,469 at 53 facilities within the 5- to 10-mile radius.

Information on schools was obtained from directories prepared by New Hampshire's and Massachusetts' Departments of Education (i.e., Education Directory, Student Enrollment, 1977 School Population by Massachusetts Department of Education and Education Directory 1978-79 and Park II 1978 Student Enrollment by the New Hampshire Department of Education). Their directories provided enrollment figures for both public and approved nonpublic facilities. Local telephone directories were also used to identify other preschool and special schools. Telephone surveying of most school district offices and a number of schools directly was also undertaken.

6. Medical-Related Facilities

Information on the location and capacities of major medical- related facilities, including hospitals and nursing homes, was collected for the area within 10 miles of Seabrook Station. Table 2.1-15 provides a summary of the medical-related facilities including the estimated bed capacities and staffing at these facilities. The distribution of this estimated medical-related population is shown on Figure 2.1-22. The majority of the medical-related population within the 10-mile radius is found within the 5- and 10-mile radii. Approximately 10 percent (or 329 persons) of the total estimated medical-related population is located within a 5-mile area and 90 percent (or 3,146 persons) within 5 to 10 miles. Hospital staff was included in the estimates of this medical-related population estimate. However, estimates of visitors and other personnel were not included in the total estimate. Sources of information included the United States Health Systems Agency in Concord, New Hampshire, health planning services directories, and communication with many other facilities.

f. Seasonal Transients (10 to 50 miles)

For the area between 10 and 50 miles from the site, transient population elements have been investigated. Data at a level of detail similar to the information presented for the area within 10 miles are not readily available. This section describes the information on transient populations by state. For each of the three states - Maine, New Hampshire and Massachusetts - seasonal transient population data were investigated. Seasonal transient population data were examined for three components: beach users, campers and occupants of seasonal dwellings. Daily employment movements were assumed to correspond to commutation of the work force from residential locations to places of employment. Emphasis is placed on the seasonal transient element due to the recreational use of much of the study area.

1. Seasonal Transients (Maine)

(a) Swimming or Beach Use

The portion of Maine within 50 miles of the site comprises about 75 percent of York County's land area. A report entitled Maine Comprehensive Outdoor Recreation Plan (1978) describes recreational activities in various parts of the state. York County and a southern portion of Oxford County comprise what is referred to in this report as the Southern Maine District (Southern Maine Regional Planning Commission Area). Approximately 53 percent of the Southern Maine District is within the 50-mile study area. The report indicates that 57,862 feet of municipal and 8,975 feet of private beach exist in the planning district. Much of the summer beach activity occurs along the coast of York County where several of Maine's major beaches are located. Major York County beach areas include Old Orchard Beach, Ogunquit, Kennebunk, Kennebunkport, and Biddeford. A beach-capacity estimate was made by applying a standard of two feet of beach shoreline per person or 33,418 persons.

(b) Camping

The Maine Comprehensive Outdoor Recreation Plan also indicates that approximately 4,808 campsites exist in the York County-Southern Oxford County area. This estimate excludes summer camps. Applying a standard of four persons per campsite results in an estimated camping population of 19,232 persons in the entire southern Maine planning district. About half of the southern district's land area is included within the 50-mile study area. Thus, for estimating purposes 53 percent or 10,195 persons are located within the York County sectors within the 50-mile radius. The Maine Comprehensive Outdoor Recreation Plan estimates "peak day camping demand" at less than capacity with 6,717 persons for 1980 and 7,464 persons for 1990.

(c) Seasonal Residents

An estimate of the seasonal resident population was developed based on the 1970 U.S. Census of Housing data. An estimate of the seasonal resident population was developed by applying an average occupancy factor of 7.6 persons per "vacant-seasonal and migratory" dwelling unit. This is the weekend occupancy factor which was determined from a 1978 survey conducted in the Hampton, Seabrook, and Salisbury beach area. It was estimated that 7,030 of the 9,373 "seasonal units" reported were within the portion of Maine included within the 50-mile radius. This estimate was based on an equal area allocation approach. The estimated seasonal resident population is 53,428 persons within this same area. Table 2.1-16 presents 1970 U.S. Census of Housing data on vacant-seasonal and migratory units for counties in Maine, New Hampshire, Massachusetts, and Vermont. All or portions of these counties are included within the 50-mile radius. Figure 2.1-24 shows the estimated size and distribution of this seasonal resident population.

(d) General Employment Patterns

Daily transient population movements related to the work force also occur in this area. It is assumed that the primary movements of work force populations are to urban centers. The most densely developed areas within the 50-mile radius in York County include Biddeford, Sanford, Kittery, Berwick and South Berwick, Kennebunk, and Kennebunkport. A 1972 report entitled Future Land Use Plans, prepared by the Southeastern New Hampshire Regional Planning Commission (SNHRPC), indicates that a large part of the Southeastern New Hampshire region labor force[1] works in Maine, particularly at the Portsmouth Naval Shipyard, which is in Kittery.

Table 2.1-17 is a summary of employment statistics by place of residence for the counties within the 50-mile radius. Figure 2.1-25 shows the distribution of 1970 employment by place of residence based on equal area allocation. Total employment estimated in this manner for the 50-mile radius is 1,328,320 workers. Total employment in York County, Maine increased from 31,191 in 1940 to 44,780 in 1970.

[1] Towns included in the region are Epping, Fremont, Brentwood, Newfields, Exeter, East Hampton, South Hampton, Kensington, Hampton Falls, Seabrook, Stratham, Greenland, Newington, Portsmouth, Rye and North Hampton.

2. Seasonal Transients (New Hampshire)

(a) Swimming or Beach Use

A detailed estimate of the seasonal and daily transient populations within 10 miles of the site was presented above. Major transient populations in this area were associated with the beach areas of Seabrook and Hampton, New Hampshire and Salisbury, Massachusetts. The remainder of the New Hampshire coast is also subject to seasonal shifts in the population. For example, the "swimming" or beach area population for the remainder of the coastal area from Wallis Sands State Beach in Rye to New Castle is estimated at 4,873 to 14,058 persons in a report entitled Public Shorefront Access Planning Process for the State of New Hampshire (June 1978) by the Strafford Rockingham Regional Council.

(b) Camping

The 1977 New Hampshire Outdoor Recreation Plan indicates that approximately 6,180 campsites exist in Planning Regions 2, 5, and 6 which are contained in the area within 50 miles of the site (Reference 8). It is estimated by equal area allocation that approximately 2,441 campsites exist within 10 to 50 miles of the site. Applying a standard average occupancy factor of four persons per campsite results in an estimated camping population of 9,764 persons within the 10- to 50-mile radius.

(c) Seasonal Residents

It is estimated by equal area allocation that about 41 percent or 10,013 of the 24,251 vacant-seasonal and migratory units reported in the 1970 U.S. Census of Housing for Carroll, Belknap, Merrimack, Hillsborough, Strafford and Rockingham counties were in the 10- to 50-mile portion of New Hampshire. An occupancy factor of 7.6 was applied to the seasonal units in the 10- to 50-mile study area to estimate the size and distribution of the seasonal resident population. Thus, the seasonal resident population for this area of New Hampshire is estimated at 76,099 persons.

(d) General Employment Patterns

Daily work force population shifts also occur in New Hampshire. Major employment centers in the 10- to 50-mile portion of New Hampshire include Manchester, Nashua, Concord, Salem, Derry, Goffstown, Milford, Wilton, Suncook, and Penacook. Some employment shifts from New Hampshire to Massachusetts and southern Maine likely exist. No detailed information of the types of daily work force employment shifts was available. Table 2.1-17 indicates that total employment in Belknap, Carroll, Hillsborough, Merrimack, Rockingham and Strafford counties increased from 125,224 in 1940 to 232,131 workers in 1970.

3. Seasonal Transients (Massachusetts)

(a) Swimming or Beach Use

The seasonal transient population within Massachusetts and the 10- to 50-mile radius of the site is significantly influenced by the permanent population within this same area. For example, the permanent population of the Boston Metropolitan Area and Lawrence, Haverhill, and Lowell, as well as persons in other less densely settled communities may use the nearby coastal beaches.

(b) Camping

A 1978 report entitled Massachusetts Outdoors provided the basis for estimating the camping population within the 50-mile radius. All or portions of four "recreation demand study regions" are within the 50-mile radius (e.g., regions 3, 4, 5, and 6). Based on equal area allocation, an estimated 5,887 campsites exist in this area. Applying four persons per campsite results in an estimate of 23,548 campers in the region.

(c) Seasonal Residents

It is believed that a large shift in the summer seasonal and seasonal resident populations occurs in the coastal communities from Newburyport (5 miles) to the Rockport-Gloucester area (12 miles). It is estimated that 5,140 of the 19,229 "vacant-seasonal and migratory units" reported in the 1970 U.S. Census of Housing for Essex, Middlesex, Suffolk, Norfolk, Plymouth, and Worcester Counties were within the 10- to 50-mile radius of the site. About 67 percent or 3,419 of these estimated units are in Essex County. A large concentration of seasonal residences exists in the Plum Island beach community in Newburyport. It is believed that the majority of seasonal units in Essex County exist along the coast between Plum Island and the Rockport-Gloucester area. The total seasonal resident population for the 10- to 50-mile area of Massachusetts is estimated at 39,064 persons.

(d) General Employment Patterns

Shifts in the daily work force population occur in this same area. The major shifts are believed related to workers commuting to the major urban areas noted above. The largest shift would be related to work force population commuting into the Boston-Route 128 area. Total employment in Essex, Middlesex, Norfolk, Plymouth, Suffolk, and Worcester counties was 1,175,766 in 1940 and 1,803,919 workers in 1970.

g. Estimate of Peak Population

Table 2.1-18 presents a summary of the total peak summer transient population within 10 miles of the site. As indicated in Table 2.1-18, the peak summer weekend day transient population, including seasonal residents, overnight visitors, and daily transients, is estimated at about 84,366 for the 5-mile radius and approximately 32,622 for the 5- to 10-mile area. Figure 2.1-25 shows the distribution of this estimated summer peak weekend transient population. Figure 2.1-26 is a similar figure showing an estimate and distribution of the summer weekday transient population. This estimate assumes that both lot and street-type beach area parking would be at 43 percent capacity as estimated for the weekend condition including the estimate of the manufacturing work force (based on the maximum number of vehicles observed in lots within the 5-mile radius on a weekday on 7/20/79). The estimate reflects highest weekday count of 46 weekdays between June and September for which data was included. The summer weekday transient population is estimated at 57,553 for the 0- to 5-mile area and 81,041 within 10 miles.

A number of other facilities exist in the vicinity of the site. Table 2.1-13, Table 2.1-14, and Table 2.1-15 provide information including estimated populations for medical-related facilities (e.g., hospitals and nursing homes), schools, and other facilities. Estimated peak populations associated with such facilities were not included in the peak summer population estimate indicated in Table 2.1-18 and Figure 2.1-26 and Figure 2.1-27 since it is assumed that these people are included in other categories (e.g., permanent population) or the facilities (e.g., schools) are not operating during summer weekends.

2.1.3.4 Low Population Zone (LPZ)

The low population zone for the Seabrook Station site is defined as a circle with a radius of 1.25 miles. The center of the circle is located among site structures, approximately 250 feet southwest of the reactor containment.

Three considerations apply in the selection of the LPZ in accordance with 10 CFR Part 100. The first is that the low population zone "...contains residents, the total number and density of which are such that there is a reasonable probability that appropriate protective measures could be taken on their behalf in the event of a serious accident." This requirement is satisfied by the specified LPZ. The second consideration of 10 CFR Part 100 requires that the specified whole body and thyroid dose limits be met. This requirement is also satisfied by the specified low population zone. The third consideration is that the population center distance must be at least one and one-third times the distance from the reactor to the outer boundary of the LPZ. It is this consideration which sets the size of the low population zone at a radius of 1.25 miles since the closest distance to an assumed population center, namely the Hampton Beach area, is 1.67 miles (see Subsection 2.1.3.5).

Figure 2.1-28 is a map which shows the low population zone. The major roadway which traverses the low population zone is U.S. Route 1 which is a north-south road to the west of the site. Portions of several feeder roadways to U.S. Route 1 are also included in the low population zone. There are no roadways which permit traveling through the low population zone in an east-west direction.

There is one railway which traverses the low population zone in a north-south direction. This is a spur owned by the Boston and Maine Railroad. The line terminates 6 miles to the south and is used infrequently.

There is one school within the low population zone boundary, the Seabrook Elementary School, located on Walton Road south of the site near the outer edge of the LPZ. The 1978 enrollment of this school was approximately 740. A list of all schools within a distance of 10 miles from the site is shown on Table 2.1-14. There are no other institutions, such as hospitals or prisons, within 5 miles of the site. The closest hospital to the site is the Amesbury Hospital, located about 5½ miles to the southwest.

The Bailey Division of USM Corporation is inside the low population zone at about 1 mile west-southwest of the site. About 1,000 people are employed there. Several commercial establishments, such as shops, retail stores, and restaurants, are located along the section of U.S. Route 1 which passes through the low population zone. Two shopping centers are located within the low population zone along Route 1. One is about 1 mile to the west, and the other is ¼ mile to the southwest.

The major beaches in the area are outside the low population zone. They are located east of Route 1A in Salisbury, Massachusetts and Seabrook and Hampton, New Hampshire. The seasonal homes and lodging facilities are also primarily located along the coast outside the LPZ. A portion of Hampton Harbor and sections of several tidal brooks and rivers are located within the LPZ. None of these is a major shipping route. However, these waters are used for recreation, principally during the summer. Population estimates for the beach areas are discussed in Subsection 2.1.3.3.

The other major recreational facility in the area is the greyhound racing track in Seabrook. This facility is outside the low population zone 2 miles west of the site.

Table 2.1-19 presents permanent population estimates for the years 1983 and 2025 and for 10-year periods from 1980 through 2020 for the low population zone. Figure 2.1-29 and Figure 2.1-30 show the projected resident population within the low population zone for the years 1983 and 2025, respectively. Transient population data for the low population zone are presented in Table 2.1-20. These population estimates for the permanent population were derived in the same manner as discussed in Subsection 2.1.3.1.

2.1.3.5 Population Center

The closest municipality with a current population of 25,000 or more people is Portsmouth, New Hampshire. This city is located approximately 12 miles north-northeast of the site. Available projections (Reference 2) for this city indicate that the population will increase at a relatively slow rate.

Amesbury, Massachusetts, 4 miles south-southwest of the site, had a 1970 population of about 11,000. The estimated 1985 population for Amesbury is 19,000 (Reference 3). Continued growth of Amesbury could result in that municipality becoming a population center during the life of the station. Newburyport, Massachusetts with a 1970 population of about 16,000 is located about 6 miles south-southwest of the site. Its growth is projected to be less than that of Amesbury with an estimated population of about 17,000 in 1985 (Reference 3).

The transient population in the vicinity of the site is sufficiently large that the Atomic Licensing Appeal Board, in the course of construction permit proceedings, directed that the beach area to the east of the site be considered the population center (Reference 7). The Board ruled that Route 1A to the east of the site serves as the real boundary of the populated areas. The area between the site and Route 1A to the east is largely marshland and provides a buffer between the site and the roadway. It is concluded that the population center should continue to be bounded by Route 1A and the population center distance should remain at 1.67 miles.

2.1.3.6 Population Density

Subsections 2.1.3.1 and 2.1.3.2 describe the methods used for estimating population distribution within 50 miles of the site. Figure 2.1-31 is a plot of the cumulative resident population to a distance of 50 miles for 1983, the projected year of plant startup. For comparison purposes, the population resulting from a uniform density of 500 persons per square mile is also shown on Figure 2.1-31. Table 2.1-21 indicates the cumulative population density out to 50 miles from the site for the permanent resident population. The maximum population density for 1983 is 532 persons per square mile.

Figure 2.1-32 shows the projected resident population to a distance of 50 miles from the site for the year 2025, the approximate end of the useful life of the facility. These values are compared with the population resulting from a uniform distribution of 1,000 persons per square mile. The maximum cumulative density of year 2025 is projected to be 1,155 persons per square mile as shown in Table 2.1-21.

2.1.4 References

1. U.S. Census Bureau, Census Bureau Display Program for the Fifth Count Summary Tapes of the 1970 Census (for towns within 5 miles of the site).

2. New Hampshire Office of Comprehensive Planning, "Interim Revisions New Hampshire Population Projections for Towns and Cities to the Year 2000," August 1977.

3. Massachusetts Department of Public Health-Office of State Health Planning, "Population Projections Massachusetts, City and Town Age-Sex Data, 1980 and 1985," August 1978.

4. Massachusetts Department of Public Health-Office of State Health Planning, "Population Projections Methodology and Handbook for Users," August 30, 1978.

5. Bureau of Economic Analysis, Regional Economic Analysis Division, U.S. Department of Commerce, "Population, Personal Income and Earnings By State; Projections to 2000," October 1977.

6. SPO Statistical Reports, Population Projection Series PPS-2, Maine Municipal Population Projections 1977, 1980, 1982, August 1977.

7. Atomic Licensing Appeal Board, ALAB-422, July 26, 1977.

8. 1977 New Hampshire Outdoor Recreation Plan. N.H. Department of Resources and Economic Development, Office of Comprehensive Planning.

2.2 NEARBY INDUSTRIAL, TRANSPORTATION AND MILITARY FACILITIES

This section provides information on the locations and extent of nearby, industrial, transportation and military facilities.

2.2.1 Locations and Routes

The locations of significant facilities within 5 miles of the site have been included in the figures at the end of this section.

2.2.1.1 Industrial Facilities

The area within 5 miles of the Seabrook plant site has been surveyed to identify any significant manufacturing plants, chemical plants, mining operations, and petroleum production or storage facilities. Emphasis has been placed upon identifying industrial facilities that handle, store, use, or produce hazardous materials[1] in quantities sufficient to present potential hazards to the operation or maintenance of the Seabrook Station. A total of eight facilities that used significant quantities of hazardous materials were identified within 5 miles of the Seabrook site (Reference 85).

Information on eight facilities that required evaluation is summarized in Table 2.2-1. The locations of these eight facilities are shown in Figure 2.2-1.

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[1] Chemicals that were identified for needing further evaluation were screened using the protocol described in R.G. 1.78, Appendix A.

2.2.1.2 Transportation Facilities

The locations of transportation facilities within 5 miles of the Seabrook site center are depicted in the following four figures. The transportation facilities include highways, airports, navigable waterways, rail, and pipelines.

a. Figure 2.2-2 indicates the locations of both the significant highways, and the single rail line within the 5-mile radius.

b. Figure 2.2-3 shows the location of the Hampton Airport with respect to the Seabrook site. This airport is the only airport within the 5-mile radius.

c. Figure 2.2-4, titled "Water Transportation Routes," shows Hampton Harbor and the navigable waters offshore.

d. Figure 2.2-5 depicts the general location of natural gas pipelines within the 5-mile radius of the site center.

2.2.1.3 Military Facilities

There are no military facilities within 5 miles of the Seabrook site center (References 2 through 8).

2.2.2 Descriptions

Nearby industrial, transportation and military facilities are described in this section. Facilities and activities within 5 miles of the site are described to the level of detail which includes primary function and activity indicators. In addition, descriptions of selected major facilities just outside the 5 miles are included.

2.2.2.1 Description of Facilities

a. Industrial Facilities

Industrial facilities are described in Subsection 2.2.2.2.

b. Transportation Facilities

1. Railroads

Presently, there is a single rail line, the Guilford Rail System, Portsmouth-Hampton Branch, which serves the areas within 5 miles of the plant site (Figure 2.2-2). This branch provides service to areas south of Portsmouth, New Hampshire. While through-connections were once possible to points south, the rail bridge across the Merrimack River (approximately 6 miles south of the plant site) is no longer serviceable and no service to Massachusetts is possible. The closest approach of the rail line to the Containment Building is approximately 2,100 feet. The closest point of useable track is 3.1 miles to the north in Hampton, New Hampshire (Reference 80). There are no plans to rehabilitate the track south of this point.

All Guilford Rail System customers are located north of the Seabrook site. No rail traffic associated with them passes through the site area. Service to these customers is provided on a five-day-per-week basis, but total activity is limited to a few rail cars per run.

2. Highways

Figure 2.2-2 shows the highways in the vicinity of the Seabrook site. The major highways in the area include:

(a) U.S. Route 1 passes 1 mile west of the site at its closest point. This route is a major local artery providing access to the local towns between Newburyport and Portsmouth. It is two-lane highway which carries an average daily traffic volume of 8,700 vehicles at the New Hampshire-Massachusetts state line (Reference 9). As it passes the site, Route 1 is a two-lane roadway with a paved median.

(b) I-95, the New Hampshire Turnpike, passes 1.6 miles to the west of the site. This highway is the major surface transportation between Boston and points northeast, including Portsmouth and Maine. With average daily traffic of 36,000 vehicles, it is the busiest highway in New Hampshire. The turnpike consists of four travel lanes in each direction.

(c) Route 1A is the local arterial that provides access to the coastal areas, located 1.7 miles east of the site. As it passes near the site, it is a four-lane highway. The Route 1A bridge from Seabrook to Hampton, however, is two lanes. Average daily traffic counts on Route 1A indicate volumes of approximately 7,700 vehicles.

(d) Route 51, the Hampton-Exeter Expressway, passes 2.0 miles to the northeast of the site. It is the only major eastwest road providing high-speed connection from the Hampton Beach area to Route 95 and Exeter, NH.

In addition to these major routes, there are numerous additional roadways in the site area. These include:

- Route 151 (running north-south) - Closest approach is 4 miles north of the site. Major intersections are with Routes 1 and 101D.
- Route 101 (running east-west) - Closest approach is 8 miles north of the site (Portsmouth area/Epping Road). Major intersections are with Routes 51 and I-95.
- Route 88 (running north-south) - Closest approach is 1½ miles north of the site (Exeter-Hampton Falls Road). Major intersection is with Route 1.
- Route 84 (running east-west) - Closest approach is 1½ miles northwest of the site. Major intersections are with Routes 1 and 150.
- Route 27 (running east-west) - Closest approach is 3 miles north of the site (High Street/Hampton Road). Major intersections are with Routes 51, 1, 1A and I-95.

- Route 111 (running east-west) – Closest approach is 5 miles north of the site (Atlantic Avenue/N. Hampton Road). Major intersections are with Routes 1A, 1 and 51.

- Route 101E (running east-west) – Closest approach is 2½ miles north of the site (Winnicunnet Road). Major intersections are with Routes 1 and 1A.

- Route 107 (running east-west) – Closest approach is 1½ miles west of the site (New Zealand Road/Horse Hill Road). Major intersections are with Routes 1 and I-95.

- Route 150 (running north-south) – Closest approach is 4 miles west of the site (Market Street/Amesbury Road).

- Route 286 (running east-west) – Closest approach is 2 miles south of the site (Old Beach Road/Forest Street). Major intersections are with Routes 1, 1A and I-95.

- Route 110 (running east-west) – Closest approach is 4 miles south of the site (Elm Street). Major intersections are with Routes 1, 1A and I-95.

Likewise, there is a network of local streets which serves the area within 5 miles. The major portions only of the street system are shown in Figure 2.2-2.

The Maine Emergency Management Agency (MEMA) is the only regional government agency that collects information on the quantities and highway transportation routes of hazardous materials. Neither New Hampshire nor Massachusetts compiles such data. MEMA requires facilities to report the routes taken for the delivery of these hazardous materials from the point of origin to destination. The latest MEMA report has been evaluated in Reference 85.

Transportation of hazardous materials is currently regulated by 49 CFR Parts 100 to 199 (Reference 23). These regulations define regulated hazardous materials and specify acceptable means for transporting them. Both New Hampshire and Massachusetts have deemed the federal regulations adequate for protecting the public. The Massachusetts Department of Public Works has adopted the regulations for intrastate application as well as for interstate commerce. New Hampshire has adopted the Federal Regulation 49 CFR which applies to transportation of hazardous materials by highway (Reference 76). I-95 is the most heavily traveled roadway in the area. It is likely that the vast majority of potentially hazardous material is transported through the area on this route. It is expected that only deliveries to local businesses require trucks carrying hazardous cargo to use other roadways in the area.

A study in 1988 of transported hazardous chemicals provided no changes from the original evaluation for rail transport. However, the 1988 study of Highway Transportation of Hazardous Chemicals identified large volume chlorine shipments. Approximately ten 16-ton chlorine cargo tank truck shipments may be transported past the site. An evaluation of the chlorine hazard was conducted to determine the probability of a chlorine truck accident (Reference 74). The probability of such an event was determined to meet the regulatory objective (References 1 and 75) and not pose any undue risk.

A 2008 evaluation (Reference 82) identified a wide range of chemicals as being transported along Interstate 95. These chemicals were evaluated using the guidance in RG 1.78. This evaluation determined that the analysis in the 1988 study still bounds all of the chemicals identified. Therefore, the recent evaluation has determined that the transportation of chemicals on the highways does not pose any undue risk to control room habitability.

A 2012, 2015 and 2018 evaluation (Reference 83, Reference 84 and Reference 85) identified a wide range of chemicals as being transported along Interstate 95. These chemicals were evaluated using the guidance in RG 1.78. This evaluation has determined that the transportation of chemicals on the highways still does not pose any undue risk to control room habitability.

3. Airports and Waterways

Airports, airfields and waterways are described in Subsections 2.2.2.5 and 2.2.2.4, respectively.

c. Military Installations

There are no military installations, missile bases, firing ranges or training within 5 miles of the Seabrook plant site (References 2 through 8). The nearest activity which might be considered military is the U.S. Coast Guard Merrimack River Station in Newburyport, Massachusetts. This facility is a small boat station located approximately 6 miles south-southwest of the site. The station engages in no military activities. Its primary responsibilities are search and rescue and pollution control. During peak activity in the summer months, the station operates three boats. The largest is the 44-foot cutter assigned to the station year-round. It is supplemented by a 41-foot and a 21-foot boat during summer months. Current staff of the station is 26 persons. This number may fluctuate slightly. The Coast Guard vessels inspect the Hampton Harbor area on a regular basis. Approximately once per week, and sometimes more frequently, a cutter will enter Hampton Harbor (Reference 3).

The two nearest major military installations are Pease Air Force Base (Newington, NH) and the Portsmouth Naval Shipyard (Kittery, ME). Both are located near Portsmouth, New Hampshire, approximately 12 miles north-northeast of the Seabrook site. The Portsmouth Naval Shipyard is engaged solely in the repair and overhaul of submarines (Reference 4). Staff levels provide for a full-time staff of approximately 100 to 175 Navy personnel assigned to the shipyard. In addition there is an average of four to five submarines at a time assigned to the shipyard. During the time the submarines are assigned to the shipyard, they are not armed. Armaments are off-loaded at other military installations 50 miles or more from the site.

The closest end of the Pease Air Force Base runway is 11.5 miles north of the site. Landing patterns bring military aircraft close to the site on a regular basis. These aircraft movements are discussed in more detail in Subsection 2.2.2.5. There are no operations associated with the air base, other than the aircraft movements, which warrant attention as potential safety considerations (Reference 5).

There are no Army bases or missile bases within a 5-mile radius of the site (Reference 6). Fort Devens, Massachusetts is the nearest major Army facility, more than 40 miles from Seabrook. The nearest missile base is a Nike site in Beverly, Massachusetts, more than 20 miles from Seabrook.

2.2.2.2 Description of Products and Materials

Only eight significant industrial facilities were identified within 5 miles of the Seabrook site that use hazardous chemicals requiring evaluation. The eight facility evaluations are summarized below. The complete evaluation is contained in Reference 85. The descriptive characteristics include Standard Industrial Classification (SIC) number, which identifies the type of business activity, product produced by each commercial facility and a brief description of the product line or activity of each facility.

Figure 2.2-1 shows the location of the eight facilities which may have sufficient quantities of hazardous chemicals to produce a potential hazard to the Seabrook Station control room habitability.

a. The Henkel Corporation

The Henkel Corporation has one facility located on Batchelder Road in Seabrook, 2 miles from Seabrook Station. All chemicals were evaluated and found not to be a hazard to the control room.

b. US Foods

US Foods is located on London Lane in Seabrook, 2 miles from Seabrook Station. All chemicals at this location were evaluated and found not to be a hazard to the control room.

c. Foss Manufacturing

The Foss Corporation is located on Merrill Industrial Drive in Hampton, 2.8 miles from Seabrook Station. All chemicals at this location were evaluated and found not to be a hazard to the control room.

d. Mackenzie Fuels

Mackenzie Fuels is located on London Lane in Seabrook, 2.2 miles from Seabrook Station. All chemicals at this location were evaluated and found not to be a hazard to the control room.

e. Brazonics, Inc.

Brazonics, Inc. is located on Tide Mill Road in Hampton, 2.5 miles from Seabrook Station. Brazonics manufactures aluminum brazed assemblies including enclosures, chassis, coldplates and heat exchangers for defense, aerospace and commercial industrial markets. All chemicals at this location were evaluated and found not to be a hazard to the control room.

f. Eastern Propane Gas, Inc.

Eastern Propane Gas, Inc. is located on Lafayette Road in Hampton, 1.6 miles from Seabrook Station. All chemicals at this location were evaluated and found not to be a hazard to the control room.

g. Aero Dynamics, Inc.

Aero Dynamics, Inc. is located on Batchelder Road in Seabrook, 2 miles from Seabrook Station. All chemicals at this location were evaluated and found not to be a hazard to the control room.

h. Giant Lift Equipment, Co.

Giant Lift Equipment, Co. is located on Lafayette Road in North Hampton, 7.4 miles from Seabrook Station. All chemicals at this location were evaluated and found not to be a hazard to the control room. Included, although outside of 5 mile radius, due to large quantity.

2.2.2.3 Pipelines

There are no major pipelines that pass within 5 miles of the Seabrook site. Neither is there any major natural gas, fuel oil or other petroleum product storage facilities. The nearest major pipeline is the jointly owned, 30-inch diameter natural gas pipeline of Portland Natural Gas Transmission System and Maritimes & Northeast Pipeline. The pipeline travels through Maine and New Hampshire, where it connects with the North American pipeline grid at Dracut, Massachusetts. It is about 10 miles from the plant at its closest point.

Within the 5-mile radius, there is local natural gas service. The New Hampshire towns, which include Seabrook, Hampton, South Hampton, North Hampton, Hampton Falls and Kensington, are served by Unitil Company. The two Massachusetts towns within 5 miles, Amesbury and Salisbury, are served by Key Span Energy Delivery New England. The areas which are served by natural gas lines beneath the street system are shown in Figure 2.2-5. The local distribution system is comprised of pipes varying from approximately one inch in diameter to six inches in diameter. These pipes are buried beneath the frost line under or adjacent to the local street system.

The major gas feeder pipeline to the Seabrook area is an 8-inch diameter, high-pressure line that extends from the west to a point adjacent to NH Route 107, midway between U. S. Route I-95 and U. S. Route 1, where it is about 1.25 miles from the plant (Reference 80). The pipeline pressure is 125 psi, but Unitil may increase the pressure to meet customer demand. The potential impacts on plant operations from the increase in high-pressure to 200-psi have been evaluated (Reference 81). The evaluation showed there is no impact to the operation of the plant from the accidental release of natural gas from the high-pressure line. The distance between the point of a postulated gas explosion to the plant is greater than the safe separation distance estimated in accordance with Regulatory Guide 1.91. Also, the gas concentration buildup in the Control Room (CR) would be well below the minimum needed to affect CR habitability.

Other local feeder lines operate at intermediate pressures of up to 99 psi. The majority of the system, however, is lower pressure with a rating of less than 60 psi. The system is served by two types of valves. There are automatic pressure control valves at those locations where pressure is stepped-down for local service. There are manual shut-down valves at nearly all branches. The manual shut-down valves enable the isolation of individual street lines as necessary.

The Unitil Gas Company system rings three of four sides of the Seabrook site. A four-inch line serves customers on Route 1, southwest of the site. A three-inch gas pipe connects the Route 1 pipe with the gas system along Route 1A at the waterfront. The connection is a three-inch pipe beneath Walton Road, Washington Street and South Main Street. At its nearest point, which is at the corner of Walton Road and Washington Street, the gas pipe is approximately 1 mile southwest of the site center.

A two-inch gas line passes beneath the mouth of Hampton Harbor along the Route 1A bridge that connects Seabrook with Hampton. The pipe lies directly on the channel bottom, and shifts slightly with tidal movements. Just north of the bridge, the power plant cooling intake tunnels pass beneath this gas line. This intersection is the point at which a component of the power plant system is in closest proximity to a gas pipe.

2.2.2.4 Waterways

No major shipping channels exist within 5 miles of the power plant and its intake system. Some shipping takes place in the open navigable waters several miles east of Hampton. The nearest potential route to the Seabrook site is a straight bearing between the Portsmouth, New Hampshire Harbor and the eastern shore of Cape Ann.

Both the U. S. Coast Guard (USCG) and U. S. Army Corps of Engineers (Corps) collect information on the transport of hazardous materials on waterways. Records at the USCG Marine Safety Office, Portsmouth, NH, indicate that about 300 vessels arrive and are inspected at Portsmouth each year (Reference 80). Most ships pass by Cape Ann either arriving or leaving Portsmouth. The minimum distance from the plant to this shipping lane is over 10 miles. The distance to the shipping lane from the Circulating Water System offshore intake is almost 8 miles.

Of the 300 ships per year, only about 4 to 6 are chemical ships, although other ships may also carry some chemicals. No shipment of chlorine or ammonia has been noted in recent years. Tankers carrying LPG arrive from the east and do not travel between Portsmouth and Cape Ann.

Submarines leaving the naval shipyard in Portsmouth use a submarine transit lane to reach the open seas (Reference 30). This lane, marked "A," together with the nearest straight bearing route, in navigable waters, tangent to the Seabrook site is shown in Figure 2.2-4.

Hampton Harbor lies a little more than a mile east of the Seabrook site. Access to the harbor is attained by a narrow, shallow channel shown on Figure 2.2-6. Existing charts show controlling dimensions of 5½ feet depth by 150 feet width (Reference 31). The harbor master, however, indicates an actual channel depth of three to four feet at low tide (Reference 32). As a result, the harbor serves only small boats. During the summer months, a total of approximately 500 small craft use the harbor. The largest active boats in the harbor are the party fishing boats, which are about 60 feet long. No cargo-carrying craft currently use the harbor.

Facilities in the harbor include the Seabrook Town Pier, the Hampton State Fish Pier and the Hampton Beach Marina. Both gas and diesel fuel are available at the marina. The harbor master estimated a maximum capacity of 5,000 gallons for each of two fuel tanks. Since the harbor prohibits marine tanker access, these tanks must be filled by tanker truck delivery.

2.2.2.5 Airports, Airfields and Airways

a. Local Airfields

There are two landing facilities within 5 miles of the Seabrook plant site. The first is the Hampton Airport in Hampton, New Hampshire. The Hampton Airport is a privately owned General Aviation facility. The airport is located just northeast of the junction of Routes 1 and 151, in the Fogg Corner section of Hampton. The airport is approximately 2 miles north of Hampton Center. It has a turf runway which is oriented roughly north-south. The runway is 2,050 feet long with a width of 300 feet. The southern end of the runway is approximately 4¼ miles north-northeast of the site. An extension of the runway centerline passes 1.2 miles east of the site.

In September 1976, there were 20 aircraft based at Hampton Airport. All of these aircraft were single engine planes. Eight of these were smaller craft with seating for three or less passengers. The remaining 12 aircraft had seating for four or more passengers. In 1976, there were an estimated 3,600 local aircraft operations and 2,600 itinerant operations. The airport estimates that it had 3,000 hours of flight training in 1977 and estimates an annual growth rate of 10 percent to 20 percent for local flights and instruction. Approximately 98 percent of the itinerant operations were made by single engine craft, the remainder by small-two engine airplanes. The total operations estimate of 6,200 in 1976 is identical to estimates made in 1970. Neither airframe nor power plant repairs are offered at the airport. Engine oil for piston aircraft and 80 octane fuel are sold at the airport.

The normal take-off and landing patterns for the airport are left hand patterns, i.e., an aircraft taking off on the south runway would make a left turn after gaining altitude, and an aircraft making a landing approach on the south runway would approach from the west on the base leg, and make a left turn off the base leg onto the final approach. Figure 2.2-3 shows typical landing patterns at Hampton Airport. Typical take-off patterns would be the mirror images of the landing patterns shown. It should be noted that because Hampton Airport is a small, privately owned, general aviation facility, there are no specified landing or take-off patterns for the airport. Much of the activity at Hampton Airport is involved with instruction, and on a busy day in the summer, there might be as many as 50 take-offs and 50 landings. The total peak month estimate is 800 operations.

Most aircraft using the south runway for a take-off will turn before they reach the area of the site. The base leg for aircraft approaching the south runway for a landing is normally north of the site.

There are no plans for expansion of Hampton Airport (References 33 through 37).

A total of three accidents and one forced landing associated with the Hampton Airport have been recorded since January 1973 (Reference 38). In 1976, an aircraft faltered during take-off and fell into the trees at the end of the runway. The aircraft was damaged, but no bodily injuries were sustained. In May 1978, an aircraft failed and successfully completed a forced landing on Hampton Beach. Neither damage nor injury resulted. In October 1978, there was an on-ground collision of two aircraft at Hampton Airport. A wing was torn off one of the aircraft, but no bodily injuries were sustained. In November of 1978, an aircraft from Hampton crashed shortly after take-off as a result of a power failure. The pilot attempted to land in the parking lot of the Hampton Court Shopping Center, about a ½ mile northwest of the airport, just off Route 1. The engine stalled and the aircraft collapsed its landing gear upon impact. The front passenger was killed in the accident.

The second landing facility within a 5-mile radius is the Wheelabrator-Frye corporate helipad. The helipad serves only the corporate offices just off the New Hampshire Turnpike in Hampton. The helipad is approximately 3½ miles north-northwest of the plant site. The facility is a 60' by 80' concrete pad which is equipped with lights for night usage. No public record of the use of the helipad exists. Route estimates provided by Wheelabrator-Frye indicate maximum activity at two or three operations per day. The New Hampshire Aeronautics Commission estimates an average activity of three or four operations per week. There is no record of an aircraft accident associated with this facility.

There are two small private airports in Newburyport, just beyond the 5-mile radius, the Plum Island Airport and the Pleasant View Airpark. The Plum Island Airport (7.2 miles south) is by far the busier of the two. It has two runways. The first is a 2,700 feet by 60 feet asphalt runway, the second is a 2,300 feet by 100 feet turf runway. FAA estimated 8,000 annual local operations and 5,500 annual itinerant operations in 1977, with a total peak month of 2,400. The peak month reflects the highly seasonal use of the airport. Many of the local flights are training sessions and short beach view rides for summer tourists. The airport has 26 based general aviation aircraft of which one is a seaplane (Reference 39).

The Pleasant View Airpark (7.0 miles south-southwest) is a 4,000-foot turf runway. FAA has no air traffic records for the airpark and indicates no based aircraft (Reference 40).

Outside the 5-mile radius, there are two major civilian airports and one air force base that were surveyed. Grenier Field, the Manchester, New Hampshire municipal airport, is the closer of the major civilian airports. It is located approximately 30 miles west of the site. Total annual operations of approximately 116,000 at this airport are well below the 1,000 d^2 (900,000) assessment criteria (Reference 41).

Logan Airport, in Boston, is the major regional airport. It is approximately 38 miles south-southwest of the Seabrook site. Approximate annual operations of 399,000 are well below the 1,000 d^2 criteria of 1,444,000 (Reference 42).

b. Pease Air Force Base Description

On January 5, 1989, the Secretary of Defense announced the closure of Pease AFB pursuant to the Base Closure and Realignment Act. According to the Draft Environmental Impact Statement prepared by the Air Force, the proposed disposition of the property involves transfer to the Pease Development Authority (PDA) for reuse as an international hub with commercial trade, manufacturing and aviation-related activities in adjacent areas. The projected mix of aircraft includes the Air National Guard 157th Air Refueling Squadron (KC-135s) and commercial operations (Reference 78).

The original FSAR description of Pease is retained for historical purposes. The aircraft hazard analysis is more conservative since FB-111A aircraft are no longer based at Pease. As previously identified to the NRC, "the aircraft hazard at Pease will be significantly reduced beginning in the summer of 1990 due to the transfer of all FB-111A aircraft to another facility and the pending closure of Pease Air Force Base" (Reference 79).

Pease Air Force Base (PAFB) is located in the town of Newington, New Hampshire, adjacent to and northwest of the city of Portsmouth, approximately 12 miles north of the site (Figure 2.2-7). Presently, the remaining aviation unit stationed at PAFB is the 157th Air National Guard Refueling Group (KC-135s).

There is one airstrip at PAFB, designated runway 34-16 (34 on the south end, 16 on the north end). The 34 end is about 11.5 miles north of the site, and the extended runway centerline passes approximately 7.5 miles northeast of the site at the closest (tangent) point, approximately 9 miles from the end of the runway. The airfield coordinates are 43° 05' north latitude and 70° 49' west longitude. Runway 34 is oriented on a bearing of 345° (based on true north), and the runway is 11,320 feet long by 300 feet wide.

Pease AFB is equipped with extensive navigational aid radio equipment, including VHF Omni Range (VOR) radio, TACAN (Tactical Air Navigation) radio and Instrument Landing System (ILS). The ILS is not available for runway 16. The base also has Surveillance and Precision Approach Radar (PAR) equipment.

The principal reason for considering the potential of airplane accidents involving Pease aircraft at the Seabrook site is because of the various instrument approaches and the radar traffic patterns utilized by Pease AFB when runway 34 is in use, which can bring aircraft into the vicinity of the site (Figure 2.2-7, Figure 2.2-8, Figure 2.2-9, Figure 2.2-10, Figure 2.2-11, Figure 2.2-12). Of these approaches, only the Distance Measuring Equipment 3 (DME-3), High Tactical Air Navigation (HI-TACAN 34), and radar pattern would bring aircraft near the site. Also, only aircraft weighing more than 12,500 pounds are of interest, since the critical structures are shown by analysis to be capable of withstanding the impact of an aircraft weighing at least this much. The containment has been shown by analysis to be capable of withstanding the impact of an FB-111A weighing as much as 81,800 pounds.

Although the risk of accidents is no greater in controlled airspace than while in flight cross country (Reference 43), an extensive investigation has been made of air traffic at Pease Air Force Base, accident rates of aircraft operating from Pease, and of previous methods of calculating aircraft crash probabilities.

As a result of this investigation, it has been determined that the method used by the NRC, Directorate of Licensing, in its evaluation of the Boardman Nuclear Plant (Reference 43) is applicable to the Seabrook site.

1. Aircraft Traffic at PAFB

Radar Approach Control (RAPCON) at PAFB maintains a record of all aircraft traffic requesting services. The total traffic count for 1979 was 60,612. A breakdown of this count by month and type of service is shown in Table 2.2-4. Of the 60,612 contacts, only those associated with Instrument Flight Rules (IFR) arrivals and some overflights would be a potential hazard to Seabrook. IFR departures do not present a hazard to the site because Standard Instrument Departures (SIDs) are normally used by aircraft departing PAFB. Aircraft using these SIDs are turned either east or northwest several miles north of the site (Reference 44), because of mission requirements and the fact that the Boston control zone is to the south of PAFB. Stage II radar is usually a request for spacing by Visual Flight Rules (VFR) traffic in the landing pattern. Again, this is not a problem because the VFR pattern is inside a 5-mile radius of PAFB. IFR satellite applies to small aircraft landing and departing at Skyhaven airport or other small airports within the Pease terminal area. VFR service applies to VFR aircraft originating or terminating within the PAFB terminal control area. These are usually small civilian and military aircraft (Reference 45). Overflight radar advisories are usually made by any aircraft, at an altitude of less than 5,000 feet, that is transitioning through the Pease terminal control area. In general, large aircraft (greater than 12,500 pounds) are not at altitudes this low. Also, it should be realized that the Pease terminal control area applies to approximately a 40-nautical mile radius on the average (Figure 2.2-13) from PAFB. Personnel at RAPCON estimate that at most the number of aircraft overflights (greater than 12,500 pounds) are on the order of 100-200 per year (Reference 45).

2. Number of Flights (Ni)

The number of FB-111A overflights at Seabrook (within approximately 5 miles of the site) is estimated to be 2,352 flights per year. This number was computed by counting the actual number of FB-111A IFR arrivals (2,090) during a six month period (September, November, December 1979, January, February and March 1980). Table 2.2-5 depicts the monthly IFR arrival count for FB-111A and other types of aircraft. The number of arrivals over the six-month period was then doubled to determine the expected annual number of flights (2.0x2090 = 4180). This value was then in turn multiplied by 0.75 which is the percent of the time (Reference 46) that runway 34 is in use during the year (0.75x4180 = 3135). Of the landings on runway 34, approximately 25 percent approach PAFB from the northeast facilitating a right turn to base for landing. Conversation with the PAFB Safety Officer (Reference 47) indicated that an effort was being made to have more of the aircraft approach from the northeast. This approach is desired because it avoids a large amount of commercial air traffic. In addition when the downwind to base to final approach is made over water, population centers are avoided, thus minimizing noise complaints. Based upon the above, aircraft that approach from the northeast are eliminated by multiplying the above total (3135) by 0.75 (0.75x3135 = 2352). This procedure was followed for other aircraft of interest with the results listed in Table 2.2-6.

A comparison between the number of flights passing near the plant, as given in the 1974 Seabrook PSAR, with those numbers used in this analysis indicates a large decrease in the traffic at PAFB. After careful checking of the 1974 data, however, it was determined that the 1974 traffic count included take-offs as well as landings. As stated earlier, SIDs take departing aircraft away from the site and, therefore, take-offs should not be included in the count. Using a 1979 monthly take-off rate of 180 per month for FB-111A aircraft (Reference 48) and backfitting the 1974 data, we calculate an annual number of IFR arrivals of approximately 2,700. This indicates a slight decrease of FB-111A traffic from 1974 to 1980 which is consistent with the trend in annual hours flown (Table 2.2-7).

3. Accident Rate (Ri)

In previous aircraft crash probability studies for air carriers, the accident rate was based on fatal accidents because nonfatal accidents were probably not severe enough to cause harm to a nuclear site. USAF aircraft accidents are classified as "minor" or "major," with a subcategory of "aircraft destroyed." It is felt that the category of interest for probability calculations should be the number of aircraft destroyed.

Norton Air Force Base has provided data describing the mode of flight of all FB-111, C-130 and KC-135 aircraft destroyed. They have also provided a table of hours flown by type aircraft (see Table 2.2-7 and Table 2.2-8).

Accidents that occurred on the runway, or close to the runway, should not be considered in the accident rate used for the Seabrook site. The only accidents used should be accidents that could affect the site during the aircraft transit time near the site. Therefore, only those accidents associated with the in-flight mode were used in obtaining the relevant accident rate.

The resulting accident rate for the C-130 is, therefore, 2.2×10^{-6} acc./hr., based on 11 accidents in 5,050,788 hours; for the KC-135, the accident rate is 2.48×10^{-6} acc./hr., based on ten accidents in 4,034,916 hours.

For the FB-111A, three accidents were eliminated from the data base. Two of these accidents involved high-speed low-level tactical training, and the third was a night refueling mission resulting in the destruction of two aircraft. The resulting accident rate, based upon the remaining four accidents in 165,818 hours, is 2.41×10^{-5} acc./hr. For other military aircraft an accident rate of 5×10^{-6} acc./hr. has been assumed.

4. Weight of FB-111A

An analysis of FB-111A flight plans over a four-month period (Table 2.2-9), results in an annual estimate of twelve FB-111A aircraft at gross weights greater than 81,800 pounds arriving at the high fix (Warni, Figure 2.2-8). Warni is a fix 34 nautical miles northwest of PAFB, and the navigator is required to enter the aircraft weight at the fix in the flight plan. The average weight at Warni during the four-month period was 68,000 pounds.

c. Commercial Aircraft

Two federal airways, V-3 and J-55, and the Saybrook 3 standard terminal arrival into Kennedy Airport, may bring aircraft within 2 miles of the Seabrook site. V-3 may be used by aircraft departing Boston for Europe, and is a function of the winds enroute. Aircraft are usually transitioning to the high altitude jet routes when on V-3. J-55 is a jet route used by aircraft above 18,000 feet (Figure 2.2-14). Saybrook 3 would be used by aircraft inbound from Europe with an altitude transition to 18,000 feet at the Kennebunk VOR.

To determine the annual number of aircraft that may come within 2 miles of the site, an analysis was made of the Pease Sector peak day traffic count as recorded by the Boston Center. FAA area specialists determined (Reference 49) that 224 flights of a possible 4,866 could come within 2 miles of Seabrook. See Table 2.2-10 and Table 2.2-11 for aircraft breakdown by type and weight on either V-3, J-55, or Saybrook 3. The number of large aircraft (greater than 12,500 pounds) flying over the site on this peak traffic count day was estimated as 128. An in-flight accident rate of 3.89×10^{-7} (Reference 50), an average airspeed of 500 mph and a glide ratio of 17 is assumed for all commercial aircraft.

Subsection 3.5.1.6 details the aircraft hazards assigned to the Seabrook site as a result of the aircraft operations described above.

2.2.2.6 Projections of Growth

a. Industrial Facilities

The immediate region surrounding the Seabrook plant site reflects some relatively strong growth trends in comparison to other New England regions. In 1977 for instance, Rockingham County, New Hampshire, which includes the towns of Seabrook, Kensington and the Hamptons, attracted nine new industries. In addition, nine existing industries expanded their operations in the area. These statistics reflect the trend toward general annual growth (Reference 51).

Industrial growth in Amesbury, Massachusetts is also expected to be strong. Cargocaire Engineering has recently completed two facilities near the new industrial park in the southeast corner of Amesbury. The most significant growth in the near future will take place in the new industrial park area on Monroe Street, 4 to 5 miles southwest of the Seabrook site. A new road in this area was completed in 1980 and eleven lots for industrial development have been created. As of October 1980, one new industrial firm was about to locate in Amesbury's industrial park, and the town is seeking others to build and locate there as well.

As of the fall of 1980, there were no immediate plans for new industry or expansion in Salisbury, Massachusetts. Expansion of the small industrial park on Faranas Drive is planned, but growth is fairly slow. Only two new industries located in Salisbury in the past two years.

In Seabrook, growth in the past several years has culminated in completion of three new industrial facilities since 1978. None of the three, however, is a potential handler of hazardous materials.

As of the fall of 1980, no specific plans have been announced for new industrial development.

The Hampton Building Inspector's office knew of no plans for development in the Hamptons (Reference 30). In addition, officials at Southeast New Hampshire Regional Planning Commission know of no other specific plans for significant building programs or expansions in the Town of Kensington, the only other town within 5 miles of the Seabrook site.

b. <u>Transportation Facilities</u>

1. <u>Railroads</u>

The Guilford Rail System has no plans to rehabilitate the Portsmouth to Seabrook line beyond Hampton, NH, leaving the closest point to the plant as 3.1 miles. The Northern New England Passenger Rail Authority instituted passenger service in 2002 from Portland, Maine to Boston, Massachusetts. Even in the unlikely scenario that passenger service trains haul hazardous materials, the closest stopping point will be Exeter, NH, which is beyond 5 miles from the plant (Reference 80).

2. <u>Highways</u>

Neither Massachusetts nor New Hampshire is planning significant highway improvements within 5 miles of the Seabrook site center (References 53, 54, and 80).

3. Airports

In 1976, the New England Regional Council provided a grant to the State of New Hampshire specifically for evaluation of the need and feasibility of a Seacoast Area Airport in New Hampshire. The study was conducted by the State Office of Comprehensive Planning with the New Hampshire Aeronautics Commission as technical advisor (Reference 55). The report concluded that a need exists for a new general aviation airport. However, as of the end of 1979, both the New Hampshire Office of Comprehensive Planning and the New Hampshire Aeronautics Commission have indicated (References 56 and 57) that there are no plans or proposals to begin to develop a major seacoast airport in the site vicinity, and that there exists little interest in such expansion for the foreseeable future. No new studies of need for such a facility have been undertaken or proposed as of December 1979 (References 56 and 57).

On July 5, 1977, New Hampshire RSA Chapter 422 was amended to provide that no new airport can be established without the approval of the community in which the airport is proposed to be located. Such an approval process has not been initiated for any of the candidate airport sites.

No major plans for expansion of air operations at Pease Air Force Base were identified.

4. Waterways

USCG and Corps data indicate the hazardous materials transported by waterway are cyclic, with tonnage varying from year to year. Over a recent ten-year period, however, there has been an insignificant increase in total tonnage.

5. Pipelines

Natural gas pipeline construction is increasing in New England (Reference 80). The largest major pipeline project nearest Seabrook is the Maritimes & Northeast Pipeline providing natural gas service for a 525-megawatt power plant project in Newington, NH. This project is about 14 miles from the plant. Locally, pipelines are expected to expand as needed to meet customer demand.

6. Military Facilities

There are no known plans for developing military facilities within 5 miles of the Seabrook site center (References 2, 6, 7, and 8).

2.2.3 Evaluation of Potential Accidents

2.2.3.1 Determination of Design Basis Events

Evaluations of hazards in the vicinity of the site, due to potential accidents from nearby industrial, transportation and military installations, indicate that most such accidents cannot affect the safe operation of the plant, and that the probability of those accidents which may affect safe operation is acceptably low, of the order of 10^{-7} per year or less. Accordingly, it is not necessary to define any Design Basis Events relating to these hazards. (Hazards from aircraft flights in the site vicinity are evaluated in Subsection 3.5.1.6).

a. Explosions

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c. Toxic Chemicals

1. Offsite hazards

No toxic chemicals are transported past the site on the Boston and Maine rail line, and no nearby facility stores or uses any significant amount of toxic chemicals. Information on possible shipments of toxic chemicals by road was not generally available, but the lack of local users makes it extremely unlikely that such chemicals would be transported along U.S. Route 1A (1 mile west of the site), since this highway is used mainly for local traffic.

A 1988 study of highway transportation of hazardous chemicals identified several large volume chlorine shipments. An evaluation of the probability of an accident with the truck transported hazard concluded that the probability meets regulatory guidelines (References 1 and 74). A list of the hazardous material to include all toxic chemicals located offsite is provided in Table 2.2-1.

2. Onsite hazards

There are a number of chemicals stored onsite which are classified as potentially toxic according to USNRC Regulatory Guide 1.78, or "dangerous" according to statements in a book by I.N. Sax (Reference 72). A review and evaluation were conducted to identify onsite chemicals with a re-evaluation in 1988 and again in 2008 (and subsequently annually as part of the Control Room Habitability Program). The studies were used to assess the potential effects to control room habitability in the postulated event of a chemical release.

A determination of whether an onsite chemical constitutes a hazard to control room habitability is based on guidelines in Regulatory Guide 1.78. Onsite surveys, inventories and review of information transmitted to state and local authorities in accordance with SARA, Title III were conducted. All chemicals meeting the guidelines of Regulatory Guide 1.78 and present in weights greater than 100 pounds were identified (Reference 82).

The characteristics of hazardous chemicals considered important as a potential hazard to control room habitability were their physical state, quantity and location relative to the control room intakes. Potentially hazardous chemicals were evaluated with regard to their volatility, toxicity, type storage container, quantity and location. The chemicals were evaluated and categorized as either posing no hazard or providing a potential hazard to control room habitability. There were six chemicals present on site that were considered for further evaluation for their potential threat to control room habitability (Reference 82).

The evaluation's conclusion was based on considerations of the guidelines in Regulatory Guide 1.78 and survey information that identified hazardous chemicals being stored in containers of limited quantities, in a nonvolatile state (liquid or solution), or presented no plausible mechanism to enter the control room air intakes because of storage location.

Chemical release concentrations were calculated using the computer code HABIT, Version 1.1. Two types of accident scenarios were considered: "maximum concentration" and "maximum concentration-duration". For a maximum concentration accident, the total content of the largest, single container of the hazardous chemical is instantaneously released. This is called an instantaneous or "burst" release. For a maximum concentration-duration accident, the total content of the largest single container of the hazardous chemicals is continuously released through the largest safety relief valve or similar connection on the container. This is called a continuous or "leak" release.

The six chemicals present onsite that had the potential to provide a hazard to the control room habitability were Propane, Hydrogen (trailers), Sodium hypochlorite (NaOCl), Carbon Dioxide (Refrigerated Liquid), Nitrogen (Gas) and Sulfur Hexafluoride (SF6). These chemicals were evaluated following the guidance of Regulatory Guide 1.78, Rev. 1; using the computer code HABIT, Version 1.1 to estimate the atmospheric dilution and concentration of a released chemical and its resultant buildup in the Control Room; and comparing the resultant Control Room concentration to the IDLH or other values as the toxicity limit to determine in Control Room Habitability is maintained. It was concluded that none of these chemicals would affect maintaining Control Room Habitability if released under postulated accident scenarios (Reference 82).

d. <u>Fires</u>

No industrial facility in the site vicinity is capable of producing an offsite fire capable of damaging the site. The major propane spill postulated above presents the greatest offsite hazard. It was shown above that the probability of an onsite flammable concentration of propane is acceptably low. An offsite deflagration of the propane cloud cannot result in unacceptable damage because the safety-related buildings are designed to withstand onsite fires of much longer duration and much higher heat fluxes. Smoke detectors at the control room intakes ensure that the habitability of the control room is maintained. Procedures used to mitigate the consequences of onsite fires are discussed in Subsection 9.5.1.

e. Collisions with Intake Structures

The three intake structures of the cooling water tunnel are located about mid-depth in 58' of water, well away from any navigational channel. Various navigational hazards (shallow water and rocks) located between Hampton Harbor entrance and the intake structures prevent any vessel with draft greater than 5' from passing over the intake structures, effectively precluding impact with the structures. If such an impact would occur and damage the reinforced concrete intake structure, the cooling towers would provide adequate backup cooling.

f. Liquid Spills

The depth at which the intake structures are located, and the substantial distances between intakes and navigational channels, tend to mitigate the consequences of any corrosive, cryogenic or coagulant spill, should it occur. However, if such a spill should occur, the cooling towers would provide adequate backup cooling.

2.2.3.2 Effects of Design Basis Events

No design basis events relating to nearby transportation, industrial or military facilities have been identified.

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2.3 METEOROLOGY

2.3.1 Regional Climatology

2.3.1.1 General Climate (Reference 1)

The Seabrook site is located along the coast of New Hampshire about 2 miles inland from the open Atlantic Ocean. The site topography is generally flat and causes no special climatic phenomena.

New Hampshire lies in the prevailing westerlies, the band of winds aloft that blow from west to east. A large number of air mass fronts and storm systems pass through New Hampshire each year. There are three distinct types of air masses that affect the site area.

a. Cold, dry air originating in sub-arctic North America

b. Warm, moist air from the Gulf of Mexico or the subtropical Atlantic

c. Cool, damp air moving in from the North Atlantic.

As the prevailing flow aloft over New Hampshire is usually offshore, the first two types of air masses influence the site area more than the third. The climate of the site is thus continental in character, but with an important maritime influence.

The prevailing surface wind comes from a westerly direction, predominantly northwesterly during the winter and southwesterly in the summer. In spring and summer a sea breeze is usually established along coastal New Hampshire, often penetrating inland, well past the site.

Winter temperatures at the site are modified because of the proximity of the ocean water, which is relatively warm compared to winter air temperatures. For this reason, a good proportion of winter storm precipitation falls in the form of rain or wet snow. As an onshore breeze is often present on summer days, lower summer maximum temperatures are observed along the New Hampshire coast than are observed farther inland. Relative humidity is generally moderate at the site, and is lowest in late winter or early spring and highest in late summer or early fall.

Precipitation is uniformly distributed throughout the year. Low pressure, or frontal, storm systems are the principal year-round moisture producers. New Hampshire is subjected not only to storms that track across the continental United States, but also to intense winter storms, "northeasters," that move northeastward along the U.S. east coast. During the winter months northeasters can produce heavy rain or snowfall, and occasionally bring ice storm conditions to the area. During the summer, thunderstorms produce locally heavy rainfall amounts.

Occasionally during the summer or fall months, a storm of tropical origin will affect New Hampshire, but only a very few will retain near or full hurricane force. The site, therefore, may be affected by a hurricane, including associated heavy rainfall, high winds and high tides.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Bases

a. Regional Climatological Data Stations

Figure 2.3-1 shows the locations of the Seabrook site and weather stations in the general area from which climatological data were obtained. The general location and type of data available from these weather stations are as follows:

Portland International Jetport National Weather Service Office (Portland NWS) This station is located about 59 miles north-northeast of the site just inland from the Atlantic Ocean and is a primary source of regional meteorological data for the site. The Portland NWS collects complete meteorological data on a continuous basis.

Boston Logan International Airport National Weather Service Office (Boston NWS) This station is located about 38 miles south-southwest of site on a landfill that extends into Boston Harbor, which is part of the Atlantic Ocean. It is a primary source of regional meteorological data on a continuous basis.

Concord Municipal Airport National Weather Service Office (Concord NWS This station is located 40 miles west-northwest of the site. The Concord NWS collects complete meteorological data on a continuous basis.

Pease Air Force Base Air Weather Service Station (Pease AFB) This station is located about 13 miles north-northeast of the site in Portsmouth.

Instrumentation information regarding the above offsite NWS and military weather stations is presented in Table 2.3-1.

Data from the following cooperative weather stations was also used:

Portsmouth, New Hampshire This station is located about 13 miles north-northeast of the site and is maintained by the Department of Public Works, a cooperative weather observer.

Rockport Massachusetts National Weather Service Climatological Station This station is located about 27 miles southeast of the site. This station collects daily maximum and minimum temperature and precipitation data.

Sanford Maine National Weather Service Climatological Station This station is located approximately 35 miles north of the site. Daily maximum and minimum temperature and precipitation data are recorded at this station.

Greenland New Hampshire National Weather Service Climatological Station This station is located about 7 miles north of the site and collects daily maximum and minimum temperature and precipitation data.

b. Regional Severe Weather Climatology

1. Hurricanes

Atlantic hurricanes are most common during late summer and early fall. During the period 1871-1977, approximately 43 tropical cyclones passed within 100 nautical miles (115 statute miles) of the site. Of these, 22 storms were classified as hurricanes, and only 3 retained full hurricane state within 100 nautical miles of the site (Reference 2).

Tropical storms or hurricanes that reach the New England area usually pass northward west of the site or on a northeast track south of the site. Since, to date, the only hurricanes or tropical storms to reach the Seabrook area have had to travel a substantial distance overland, the potential impact of such storms is significantly reduced. Potential impact is usually confined to the effects of high tides and heavy rainfall (Reference 1).

2. Tornadoes and Waterspouts

Tornadoes have occurred in all the New England States. The mean annual number of tornadoes per 10,000 square miles for the period 1953-1976 in New Hampshire, Maine and Massachusetts are 2.5, 0.8 and 5.2, respectively (Reference 3).

A National Severe Storms Forecast Center (NSSFC) listing of tornadoes within a 50-nautical mile radius of the site indicates that 69 tornadoes occurred during the period 1950 through 1977, with a mean path area of 0.124 square miles (Reference 4).

Thom (Reference 5) has developed a procedure for estimating the probability of a tornado striking any point from an analysis of mean path length and width and the frequency of tornado occurrence in the area. Applying Thom's procedure to the NSSFC data gives an annual probability of a tornado striking any point within 50 nautical miles (57.6 miles) of the site of less than 7.8×10^{-5} with a mean recurrence interval of greater than 12,900 years. The calculation excluded the water area within the area of interest.

In spite of the low probability of a tornado occurrence, seismic Category I structures at the Seabrook site, except for the refueling water tank spray additive tank enclosure and cooling tower, are designed to withstand the "Standard Tornado" as described in NRC Regulatory Guide 1.76, Rev. 0 (Reference 6). This design basis tornado has the following characteristics:

(a) A maximum wind speed of 360 miles per hour

(b) A rotational speed of 290 miles per hour

(c) A maximum translational speed of 70 miles per hour

(d) A minimum translational speed of 5 miles per hour

(e) A radius of maximum rotational speed of 150 feet

(f) A pressure drop of 3.0 pounds per square inch

(g) A rate of pressure drop of 2.0 pounds per square inch per second.

Regulatory Guide, Rev. 1 (Reference 55) is followed for design basis tornado characteristics for potential uplift of Category I Electrical Tunnel manway removable concrete plugs and slabs. This design basis tornado has the following characteristics:

(a) A maximum wind speed of 200 miles per hour

(b) A rotational speed of 160 miles per hour

(c) A maximum translational speed of 40 miles per hour

(d) A minimum translational speed of 5 miles per hour

(e) A radius of maximum rotational speed of 150 feet

(f) A pressure drop of 0.9 pounds per square inch

(g) A rate of pressure drop of 0.4 pounds per square inch per second.

In an analysis of waterspout occurrences using Storm Data Reports (1959-1973) and ship log reports (1850-1940), a total of 14 waterspouts was reported off the coast between Boston and Portsmouth of which 3 were considered to have caused coastal damage (Reference 7). A waterspout coming ashore and striking the site would not have a destructive effect greater than that of a tornado. This is based on the wind speed of a waterspout not being greater than the design basis tornado of Regulatory Guide 1.76. With exactly the same wind speeds, it is concluded that a waterspout would be less destructive than a tornado as it would contain less solid debris than a tornado that had been traveling overland.

3. Thunderstorms, Lightning and Hail

Table 2.3-2 shows the mean number of days with thunderstorms for various weather stations in the general Seabrook area. Thunderstorms have occurred during every month of the year, with the maximum during the summer. Pease AFB data can be considered most representative of the Seabrook site, showing a thunderstorm frequency of about 19 per year with a maximum monthly mean of about 5 in July (Reference 11).

Using the thunderstorm frequencies shown in Table 2.3-2 for Pease AFB and statistics relating to thunderstorm occurrence and to the probability of cloud-to-ground lightning as presented by Viemeister (Reference 12), estimates of the frequency of occurrence of cloud-to-ground lightning were derived for the site on a seasonal and annual basis for objects extending to heights of 50, 100, 200 and 500 feet above grade. These results are provided in Table 2.3-3.

Marshall (Reference 13) presents an alternative methodology for estimating lightning strike frequencies which includes consideration of the attractive area of structures. Marshall's method consists of determining the number of lightning flashes to earth per year per km^2 and then defining an area over which the structure can be expected to attract a lightning strike. Assuming that there are 0.135 flashes to earth per thunderstorm days per km^2 near the Seabrook site (Reference 13) and that the Seabrook site experiences 19 thunderstorm days per year (Pease AFB data, Table 2.3-2), there are approximately 2.57 flashes to earth per year per km^2 around the Seabrook site area. If the length of a structure is L, its width W, and its height H, Marshall defines the total attractive area A of that structure for lightning flashes with a current magnitude of 50 percent of all lightning flashes as:

$$A = LW + 4H (L + W) + 12.57 H^2$$

The following building complex dimensions were used to conservatively estimate the attractive area for the Unit #1 building complex:

$$L = 200m, W = 120m$$

Defined roughly by a rectangle outlined by the Turbine Building, Administrative Building, Fuel Storage Building, and containment structure.

$$H = 56m$$

Defined by the height of the primary vent stack.

$$A = 0.135 \text{ km}^2$$

Consequently, the lightning strike frequency computed using Marshall's methodology is given as 0.35 flashes/year, for the Unit #1 building complex.

Table 2.3-4 lists the total number of days with hail over a 40-year period for Boston, Portland and Concord. The data indicate that, on the average, the site should expect less than one day per year with hail (Reference 14). Hailstorms in the Seabrook area are seldom severe, although large hail has been reported. During the 13-year period between 1955 and 1967, an average of 0.2, 0.6 and 1.3 storms per year with hailstones 1.5 inches in diameter or larger have been reported for New Hampshire, Maine and Massachusetts, respectively (Reference 15).

4. Strong Winds

Table 2.3-5 lists the fastest mile wind speeds recorded at Boston, Portland and Concord. The data indicate that wind speeds over 40 mph can occur during any month of the year. During the winter these speeds are normally caused by northeasters that move up along the coast. During the warmer months, high winds are normally associated with thunderstorms and squall lines that pass through the area. Hurricanes could produce high wind speeds during the late summer and early fall.

Thom (Reference 16) plotted isotachs of annual extreme-mile wind speeds at 30 feet above ground for several recurrence intervals across the United States. The annual extreme-mile wind speed derived from Reference 16 for the Seabrook site indicates a sustained 95 mph wind speed can be expected with a 100-year recurrence interval. Other studies (References 17, 21), which also plotted isotachs for fastest mile of wind at 30 feet above ground across the United States, indicate a fastest mile of wind for a 100-year probable period of recurrence of 110 mph and 100 mph for the Seabrook site. These studies (References 16, 17, 21) were published in 1968, 1961 and 1972, respectively.

The more conservative value of 110 mph was used as the 100-year period of occurrence design wind velocity for seismic Category I structures at 30 feet above ground. The vertical wind velocity profile and the appropriate gust factor used for seismic Category I structure wind loading analyses are discussed in Subsection 3.3.1.

More recently published extreme wind speed-probability studies (References 50, 51) provide more reliable information since they are based on statistical analyses of the longer available data bases and Monte Carlo simulations of hurricanes. Table 2.3-6 shows the fastest-mile extreme wind speed derived from References 50 and 51 for various recurrence intervals at 10-meters and 30-meters above grade for the Seabrook site.

5. Snowload

The American National Standards Institute, Inc., (ANSI) gives the 100-year recurrence interval snow load on the ground in the Seabrook area as 42 pounds per square foot (Reference 21). The maximum 24-hour precipitation amount observed in the site during the snow season (November through April) is 5.4 inches of water, as shown in Table 2.3-23. From this value, a conservative 48-hour probable maximum snowfall is defined as having twice the water content of the maximum 24-hour storm, or 10.8 inches. As required by Regulatory Guide 1.70 (Reference 22), the Probable Maximum Winter Precipitation was determined, which resulted in a 48-hour precipitation of 16.1 inches (Reference 23). Assuming this amount of precipitation fell on top of the 100-year recurrence interval snowpack of 42 psf, as given by ANSI, it would result in a compacted snow load of 125.7 psf. This is considered an "unusual" load condition as described in Chapter 3. Roof loading for safety-related structures due to precipitation, including ice, snow and rain, are discussed in Subsection 2.4.2.3.

The February 6-8, 1978 snowstorm which struck New England was one of the most intense, persistent, severe winter storms on record (Reference 24). The highest melted precipitation associated with the storm was 4.55 inches reported at Pembroke, Massachusetts (Reference 18). The New England climatologist, Robert E. Lautzenheiser, had previously stated that the February 23-28, 1969 snowstorm was probably the worst storm in 100 years. The highest melted precipitation associated with that storm was 4.62 inches reported at Rockport, Massachusetts. The 4.62 inch value is equivalent to a snowfall load of 24 psf. When this is combined with the 100-year probable maximum snowpack of 42 psf, it results in a total snow load on the ground of 66 psf.

6. Ice Storms

Freezing precipitation, or glaze ice, does occur in the Seabrook area. Data for freezing rain at Portsmouth (Reference 25) are presented in Table 2.3-7. Mapped data for the period 1928 to 1937 indicate that the site averages 2-3 ice storms per year. For the nine-year period of study, about 12 storms occurred resulting in ice with a thickness of 0.25 inch or more, of which about 6 storms had ice of 0.5 inch or more (Reference 26). More recent mapped data for the period of 1950 to 1969 (Reference 27), indicates that the site averages about 8 ice storms per year.

7. High Air Pollution Potential and Mixing Heights

The Seabrook site is not in an area of frequent air pollution episodes or alerts. A study of synoptic weather map analysis for 1936 through 1975 shows high pressure stagnation conditions lasting four days or more over the site occurring 12 times with an average of 4.4 stagnation days per case (Reference 28).

Holzworth (Reference 29) analyzed five years of data to determine occurrences in the United States of episodes of meteorological conditions unfavorable for atmospheric dispersion. Holzworth indicated episodes of high air pollution potential as periods with low mixing depth and light winds. A summary of the Holzworth data as it applies to the site appears in Table 2.3-8. The data indicate that prolonged periods with a combination of low wind speed and low mixing height are uncommon in the site area.

Holzworth (Reference 29) also plotted isopleths of mean seasonal and annual morning and afternoon mixing heights across the United States from the same five years of data. For the Seabrook site, the seasonal and annual values of the mean daily mixing heights occurred as follows:

Mean Daily Mixing Heights

Season	Morning	Afternoon
Spring	710 m	1400 m
Summer	450 m	1400 m
Autumn	590 m	1100 m
Winter	700 m	900 m
Annual	600 m	1200 m

The above data represent estimates of the average depth of vigorous vertical mixing, which give an indication of the vertical depth of atmosphere available for mixing and dispersion of effluents.

8. Ultimate Heat Sink

Data collected at Pease AFB for the 25-year period (1956-1974 and 1976-1981) were used to evaluate the performance of the Ultimate Heat Sink with respect to maximum evaporation and drift loss, water cooling capacity, and basin water freezing during extreme winter weather.

Maximum evaporation and drift loss will occur during periods of large differences between the ambient dry bulb and wet bulb temperatures with accompanying high wet bulb temperatures. A seven-day (168 hours) averaging period is appropriate for analysis of water inventory. The maximum 168-hour average wet bulb temperature calculated from the Pease AFB data base was found to be 73.93°F and was accompanied by a concurrent dry bulb average temperature of 77.87°F. This condition occurred during the seven-day period ending on July 18, 1972 at 1500 LST. To assure that evaporative water losses would not be underpredicted, they were computed utilizing the seven-day average wet bulb temperature of 74°F while conservatively assuming a concurrent average dry bulb temperature of 85°F (see Subsection 9.2.5).

Minimum heat transfer to the atmosphere occurs during periods of high wet bulb temperature. Regulatory Guide 1.27, Revision 2, suggests that the worst-case meteorological conditions for the parameters of concern be used. In the post-LOCA plus seismic event case where the tower is used, primary component cooling water temperature undergoes a four-hour transient rise to a peak, then decreases. This case is the limiting parameter for the tower and requires the analysis of four hour average wet bulb temperatures. Four-hour average wet bulb temperatures were computed on a running average basis from the Pease AFB data base and occurrence frequencies were tabulated. The results of this analysis for average wet bulb temperatures greater than 65°F occurring during the "summer" period (beginning June 1 and ending September 15) are presented in Table 2.3-9. The frequency of occurrence of four-hour average wet bulb temperatures greater than or equal to 75°F during the "summer" period of the data base may be seen to be 1,275 occurrences. This corresponds to a probability of 2.012×10^{-2} of experiencing a four-hour average wet bulb temperature equalling or exceeding 75°F during any given four-hour period of the summer. This probability is used in evaluating the acceptability of cooling tower performance (see Subsection 9.2.5).

Additional meteorological information was used in establishing operating restrictions on the cooling tower as described in Subsection 9.2.5. This information consisted of frequency distributions of nine-hour average wet bulb temperatures for the second half of the month of July occurring during the hours of 7:00 PM to 9:00 AM (corresponding to ending hours of nine-hour averages from 3:00 AM to 9:00 AM). These frequencies are presented in Table 2.3-10.

Basin water freezing, as well as measures to mitigate this wintertime meteorological effect, were evaluated with respect to the occurrence of extreme low dry bulb temperatures (see Subsection 9.2.5). The meteorological parameters of most concern in this analysis are dry bulb temperature and wind speed. The number and lengths of contiguous periods of extreme loss (<15°F) 24-hour average dry bulb temperatures were identified and occurrence frequencies computed. These data are presented in Table 2.3-11. A conservative wind speed assumption of 20 mph was used in conjunction with this analysis.

2.3.2 Local Meteorology

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

Monthly and annual summaries of meteorological parameters from long-term data stations representative of the area are presented in this section. Summaries of onsite meteorological data collected at Seabrook from November 1971 through March 1973 are also provided in this Section and in Appendix 2A. A new onsite meteorological tower has been erected at the same location as the old tower and became fully operational in April 1979. Data summaries from this new tower for the time period April 1979 through March 1980 are presented in Appendix 2B. Severe weather data (snowfall, tornadoes and wind velocities 75 to 125 mph) and extremely severe weather data (wind velocities greater than or equal to 125 mph) were used to determine an estimated frequency of loss of offsite power due to weather as input into the Station Blackout analysis (see Section 8.4.2).

a. Wind

Wind roses for the four seasons and 12-month period (November 1971-October 1972) of collected onsite data are provided in Figure 2.3-2, Figure 2.3-3, Figure 2.3-4, Figure 2.3-5 and Figure 2.3-6, respectively. The data indicate that westerly through northwesterly winds predominate during most of the year. During the summer months, southwesterly through west-northwesterly, and east-southeasterly through south-southeasterly winds are prevalent. Wind direction persistence summaries for 22.5 and 45.0 degree sectors are presented in Appendix 2A.

Seasonal and 12-month period wind roses collected onsite from the new onsite meteorological tower (April 1979-March 1980) are provided in Appendix 2B. Wind direction persistence summaries for this same period are also provided in Appendix 2B.

b. Temperature

Table 2.3-12, Table 2.3-13, Table 2.3-14, Table 2.3-15, Table 2.3-16 and Table 2.3-17 present long-term mean and extreme temperature values for a number of stations in the Seabrook area. Portsmouth data can be considered representative of long-term Seabrook temperatures. Monthly onsite mean and extreme temperature values for the time period April 1979 through March 1980 are presented in Appendix 2B.

Extremes of temperature are uncommon due to the proximity of the site to the Atlantic Ocean. During the winter, arctic air masses passing through New England can produce low minimum temperatures, but the frequency and persistence of such extreme values along the coast is less than for stations located farther inland. During the spring and summer a seabreeze usually moderates temperatures from reaching high extremes at the site.

Detailed analyses have determined that the highest hourly temperature recorded during the period 1957 through 1981 at Pease AFB (Portsmouth, NH) was 101°F on July 1, 1964 (hour 13). The hottest contiguous 24-hour period containing this temperature extended from June 30 (hour 15) through July 1 (hour 14). The hourly temperature progression for this period is provided in Table 2.3-18. Hourly temperature data associated with the five hottest and five coldest 24-hour average temperatures recorded at Pease AFB in the period 1957 through 1981 are given in Table 2.3-19.

The 100-year return period maximum dry bulb temperature value has a probability of occurring or being exceeded of 1.0% (i.e., 1/100) each year.

Twenty years of temperature data recorded at Pease Air Force Base (AFB), Portsmouth, New Hampshire, from 1982–2001 were used in the analysis (Reference 52). Pease AFB is the closest first-order National Weather Services site to Seabrook Station, being located approximately 13 miles north-northwest of the site. Since Pease AFB is located further inland than Seabrook Station, use of Pease AFB meteorological data provides more conservative extremes since it is less affected by the moderating influence of the Atlantic Ocean.

Using the method and instructions of ASHRAE 2005 (Reference 53) and the extreme annual maximum and minimum dry bulb temperature values and their associated standard deviation for Pease AFB, the 100-year return period maximum and minimum hourly dry bulb temperature values for Pease AFB were determined (Reference 54). These values are listed below.

100-Year Return Period
Maximum and Minimum
Hourly Dry Bulb Temperature Values

Description	Value
Maximum	100.9ºF
Minimum	-16.8ºF

Since the design of certain equipment is dependent upon the maximum and minimum temperatures averaged over time periods greater than one hour, 100-year return period extreme temperatures for 24-hour averaging periods was determined (Reference 30). These values are listed below:

100-Year
Return Period Temperature (ºF)

Averaging Period	Maximum	Minimum
24-hour	89	-16

c. Atmospheric Water Vapor

Long-term mean monthly relative humidity statistics at Pease AFB are provided in Table 2.3-20. Onsite dew point statistics for the period April 1979 through March 1980 are provided in Appendix 2B.

Joint frequency distributions of the onsite moisture deficit have been prepared for each stability category and wind direction.

Those data from April 1972 through March 1973 are presented in detail in Appendix 2A.

Outdoor relative humidity extremes of 10 percent minimum and 100 percent maximum were used in the design of the HVAC systems for all safety-related buildings. The basis for the selection of this humidity range is the assumption that relative humidities at or near 100 percent occur during fog, dew formation and precipitation that are frequently observed in this climate. Relative humidities less that 10 percent are not observed under the climatic conditions affecting the site.

d. Precipitation

On the average, the Seabrook area has about 129 days per year with measurable (0.01 inch or more) precipitation, as indicated in Table 2.3-21. Table 2.3-22, that shows mean monthly and annual precipitation amounts, indicates monthly precipitation is equally distributed over the year, with mean monthly amounts generally between 2.7 to 4.6 inches. The site can expect an annual precipitation of about 43 inches.

Summer rainfall is caused primarily by thunderstorms and convective shower activity. Precipitation during the rest of the year generally results from the passage of low pressure systems. During the colder months of the year, intense coastal storms or northeasters move north-eastward along the New England coast, usually affecting coastal locations with heavy rain or snow and on occasion, ice storm conditions. Occasionally during the summer or fall, a storm of tropical origin will cause substantial rainfall and high winds in the vicinity of the site.

Precipitation extremes for area stations are presented in Table 2.3-23, Table 2.3-24, Table 2.3-25 and Table 2.3-26. Based on the Portsmouth data, a maximum monthly precipitation amount of about 14 inches and a maximum 24-hour precipitation amount of about 7 inches could be expected at the site.

While periods of prolonged drought are not common, dry spells do occasionally occur. March 1915 and October 1924 were particularly dry, as indicated in Table 2.3-26.

Snow falls in the site area as early as November and as late as April. Mean snowfall statistics for the area, Table 2.3-27, indicate that the site can expect an annual snowfall of about 72 inches. Maximum snowfall data are presented in Table 2.3-28 and Table 2.3-29, which suggest a maximum 24-hour snowfall of about 22 inches and a maximum monthly snowfall of about 54 inches, based on Portsmouth data.

The ground is normally covered with snow from late December until well into March, although it may remain bare for several weeks during this period in a milder winter. A continuous snow cover of at least one inch lasts 30 to 45 days in a usual winter, but continued for 87 days in the snowy winter of 1955-1956. The average maximum snow depth is about 18-24 inches (Reference 25).

e. Fog

The proximity of the ocean is an important factor in fog occurrence at the site. During the spring and summer months, fog forms offshore as warm, moist air flows over the relatively cold ocean water. With any persistent eastern component in the wind direction, the fog that often lies just offshore during the warmer months can reach the Seabrook site. This situation is supported during the summer by local heating and a resulting seabreeze.

Table 2.3-30 provides information on the mean number of days with heavy fog at surrounding stations. Based on Pease AFB data, Table 2.3-31, all months of the year have a fairly consistent frequency of occurrence of fog. Although fog at Pease AFB occurs about 15 percent of the time, it is dense enough to restrict visibility to 1 mile or less only about 3.5 percent of the time (Reference 25). Table 2.3-32 lists the mean number of hours with visibility less than 0.5 miles.

Statistics on fog persistence at Portland are presented in Table 2.3-33 for the 10-year period (1968-1977). This table indicates that durations of periods of fog lasting 48 hours or longer can occur several times a year.

f. Atmospheric Stability

Joint frequency distributions of Pasquill stability class by the temperature difference (delta T) method are presented in Appendix 2A and Appendix 2B. Summaries of atmospheric stability persistence are also provided in both Appendices. The onsite data from the new meteorological tower indicate that from April 1979 through March 1980 unstable, neutral, and stable conditions occurred as follows:

Frequency of Stability Classes

Stability Classification	150'-43' Delta-T	209'-43' Delta-T
Unstable (A,B,C)	21.1 percent	12.7 percent
Neutral (D)	41.5 percent	43.3 percent
Stable (E,F,G)	37.3 percent	44.0 percent

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

A map is presented in Figure 2.3-7 which shows the topography within a 5-mile radius of the site. Maximum elevation with distance is plotted in Figure 2.3-8 for each of 16 sectors radiating from the plant site. The heights shown in these cross sections are for the highest representative terrain at that distance in the sector, and not necessarily the exact height at the precise bearing and distance shown.

The immediate site area is tidal marsh with short grass, reeds and tidal channels. Short trees begin at the edge of the marsh as the terrain becomes slightly irregular. A few short ridges and hills occur within the first 5 miles of the site.

A map showing detailed topographic features within a 50-mile radius of the site is presented in Figure 2.3-1. The first hills and ridges of the White Mountains of New Hampshire occur 20-25 miles northwest, west and southwest of the site. Hilly terrain with peaks between 200 and 500 feet are found 25 to 40 miles from the site.

The plant is not expected to cause any significant influence on the local meteorology as cooling towers or spray ponds are not planned for normal operations.

2.3.3 Onsite Meteorological Measurement Program

2.3.3.1 Pre-Construction Program

An instrumented meteorological data tower was erected at the Seabrook site and operated from November 1971 until June 1974 for plant construction licensing purposes.

The tower was 150 feet high with a base at approximately 10 feet MSL, and was located near the south edge of the Browns River, about 700 feet east of the railroad. The location of the tower relative to the site is shown in Figure 1.2-1.

There were no trees or other vertical obstructions in the immediate vicinity of the tower site. The nearest significant growth was 25-35 foot trees about 500 feet to the west and south-west of the tower. There was no significant vegetation between the tower site and Hampton Harbor. Grass was planted under the tower out to a radius of 50 feet to assure conservative delta T data.

The tower was instrumented as shown in Table 2.3-34. After one year of data accumulation, the original Aerovane wind system at 30 feet was replaced with a Bendix 3-cup anemometer and vane system. Another Bendix wind system was installed at the 130-foot level at the same time.

Wind data were recorded on Bendix Model 141-2 dual strip chart recorders. The temperature systems used Rosemont precision resistance bridges and recorded on an Esterline-Angus multi-channel recorder. One channel of the recorder was used to print a reference value of 0 volts from which all traces were calibrated.

The temperature sensors were installed in aspirated shields on the tower. The vertical temperature difference (delta T) was measured between 30 and 130 ft. The system was scaled for a range of from -10°F to +18°F, for a full span scale of 28°F. The Rosemount platinum resistance sensors and bridge system had an accuracy of 0.1 percent of span or ±0.02 ohms, whichever was greater; the maximum possible system error therefore was ±0.09°F. The recorder accuracy was +0.25 percent of scale, or +0.07°F. As a result, the maximum delta T system error could be ±0.16°F, with a probable system error of ±0.11°F.

All equipment was checked for normal operation before installation on the tower. At that time, the delta T system was calibrated to a 0.0°F value by means of a simultaneous ice bath of both sensors. All laboratory tests were made with each sensor permanently connected to the cable to be used with the sensor on the tower.

The dew point sensor was installed on the tower in March 1972. Recorded dew point data had been verified by bi-weekly multiple sling psychrometer readings taken at the 30-foot level on the tower.

Occasional minor adjustments to the recorded dew point data had been made to maintain the data within an accuracy of ±0.5°C.

Data recovery rates for individual tower parameters are given in Table 2.3-35. This table shows that the Seabrook meteorological program satisfied the 90 percent data recovery specified in Regulatory Guide 1.23 (Reference 32).

In addition to bimonthly meteorological strip chart review, every three months recorded temperatures were checked against tower value obtained with ASTM precision thermometers. Wind systems were checked for trouble-free operation every three months. Wind direction and speed transmitters were removed from the tower and given a complete laboratory check at least every six months to assure they were working within the manufacturer's specifications.

Processing of the onsite meteorological strip charts was as follows. For hourly data values, a mean value for the 30 minutes preceding the hour was determined directly from the strip charts. This value was transferred to a punched card by means of a Gerber semi-automatic analog-to-digital converter. The punched cards were checked by computer for consistent values from one hour to the next. After all checks were verified, a punched card was prepared that contained the date, time and hourly values for all the parameters measured on the tower. These cards were used to prepare the data summaries in Appendix 2A.

2.3.3.2 Preoperational Program

A new 210-foot high instrumented meteorological data tower was erected at the same location as the old tower to collect data for plant operating licensing purposes. This new tower became operational in April 1979.

The meteorological tower was instrumented for wind measurements at heights of 13.1 meters (43 feet), and 209 feet above the base. The tower is located at an elevation of approximately 10 feet MSL, and as such, the low-level wind and temperature sensors were approximately 53 feet MSL. Since plant grade is 20 feet MSL, the low-level sensors were located at an elevation of approximately 10m above plant grade rather than 10m AGL. The difference in values measured at 33 feet (10m) AGL versus 43 feet AGL on the meteorological tower should not be significant. Wind speed and direction were observed by Climatronics F460 wind systems which had a starting speed of less than 1.0 miles per hour. Wind direction and speed were recorded on Esterline-Angus Model LllS2S strip chart recorders.

The ambient temperature difference was measured on the tower between 150 and 43 feet and between 209 and 43 feet. These data were obtained by Rosemount platinum temperature sensors and precision resistance bridges and recorded on an Esterline-Angus Model E1124E multi-channel recorder. Ambient temperature was also measured by this system for the 43-foot level. The temperature and delta T sensors were installed in Teledyne Geotech aspirated shields.

Dew point was initially measured at the 43-foot level on the tower by a General Eastern Model 1200 APS dew point system. The General Eastern dew point system was replaced in May 1981 with a Climatronics lithium chloride dew point system. Dew point data was also recorded on the Esterline-Angus E1124E multi-channel recorder.

A heated tipping bucket precipitation gauge and an Eppley pyranometer were also installed.

A digital recording system was the primary data collection mechanism for the Seabrook Meteorological System. A MODCOMP minicomputer scanned the wind parameters at 1-second intervals and all other parameters at 5-second intervals. The data were compiled as 15-minute averages. Four 15-minute averages per hour were recorded. The first 15-minute average for each hour was used to represent that hour's data in analytical computer programs. The analog strip charts were used as a backup source of data and for quality control analysis.

Table 2.3-36 presents the equipment components, performance specifications, and system accuracies for both the analog and digital data systems. Presented values are summaries from manufacturer's specification sheets.

All equipment was checked and calibrated before installation. The delta T and temperature systems were calibrated by means of constant-temperature baths.

All equipment calibration was performed with the sensor connected to the cable to be used with the sensor on the tower.

Preventive maintenance and complete calibration checks of the temperature, delta T and dew point systems were performed per proposed Technical Specification requirements. Wind transmitters were removed from the tower every six months and tested in a low-speed wind tunnel for normal operation and a starting speed of less than one mile per hour.

The sensors and data processing procedures met the requirements for time averaged values as specified in NRC Regulatory Guide 1.23.

2.3.3.3 Operational Program

a. Primary Meteorological Monitoring System

The 210-foot meteorological tower structure used for plant operating licensing purposes continues to be used during plant operation. The monitored parameters include those required by plant Technical Requirements and those used to perform dose projections for both routine and accidental atmospheric releases.

Wind speed and direction are monitored with ultrasonic anemometers on booms located at 43 feet and 209 feet above ground level. The ambient temperature difference between the 150- and 43-foot levels and the 209- and 43-foot levels are measured with temperature probes housed in motor-aspirated temperature shields. Ambient temperature is also measured at the 43-foot level. A heated tipping bucket precipitation gauge and a pyranometer collect data near the base of the tower.

The Main Plant Computer System (MPCS) monitors the meteorological tower parameters. The wind speed, wind direction, and delta-temperature signals are sampled, averaged, and recorded. These data are available for on-demand display on MPCS terminals located in the control room, Technical Support Center (TSC), and Emergency Operations Facility (EOF).

All parameters are also recorded on a datalogger located in the Instrument Building near the tower's base. The datalogger is used as a backup source of data and for quality control analysis. Backup power is supplied from the station's Train A diesel generator to the equipment at the meteorological tower and the main plant computer system. Table 2.3-37 presents the equipment ranges and performance requirements. A daily channel check of the wind and delta-temperature instrumentation is performed in accordance with station Technical Requirements to demonstrate channel functionality. Corrective action is initiated if any of the meteorological instrumentation is determined to be malfunctioning.

The equipment preventative maintenance and calibration activities conducted during the preoperational monitoring program continue during the operational monitoring program. These activities occur at a frequency compatible with the station's Technical Requirements.

b. Backup Meteorological Monitoring System

A backup to the Primary Meteorological Monitoring System instrumentation is provided by an independent tower located approximately 200 feet southeast of the primary meteorological tower. The backup tower has wind speed and wind direction sensors mounted on a crossarm located at Elevation 53 ft MSL, the same elevation as the lower sensors on the primary tower. System accuracy is adequate to support the use of these data in the Radiological Emergency Plan.

The following are calculated and displayed in the main control room: average wind speed, average wind direction, wind direction standard deviation and atmospheric stability (determined as a function of wind speed, wind direction standard deviation, and time-of-day.) In addition, these four parameters are provided as input into the MPCS. The backup meteorological monitoring system is provided with power sources that are different than those used on the primary meteorological monitoring system. No one failure of the power sources or instrumentation associated with the primary or backup meteorological monitoring systems will prevent the availability in the control room of the meteorological information required to support the Radiological Emergency Plan.

Preventative maintenance and calibration are performed at the same frequency as the primary meteorological monitoring system.

2.3.4 Short-Term (Accident) Diffusion Estimates

2.3.4.1 Objective

Conservative and realistic estimates of atmospheric diffusion at the site boundary and the outer boundary of the low population zone (LPZ) were culated for appropriate time periods using meteorological data collected onsite during the time period April 1979 through March 1980.

2.3.4.2 Calculations

Gaussian diffusion models were used to compute estimates of the local atmospheric dilution factors for the exclusion area boundary and the low population zone using hourly meteorological data collected at the Seabrook site. Two sets of dilution factors (CHI/Q values) were calculated: (1) a concentration dilution factor for evaluating ground level concentrations of noble gases, tritium, carbon 14 and nonelemental iodines, and (2) an effective gamma dilution factor for evaluating gamma dose rates. Cumulative probability distributions were prepared for both types of dilution factors to determine the appropriate dilution factors for a number of time intervals ranging from 1 to 720 hours.

a. Concentration CHI/Q Equations

Hourly concentration dilution factors were computed for ground-level releases using both the plume centerline model and the sector average model. Plume centerline values are for estimating short-term atmospheric dispersion (up to 8 hours) and sector average values are for dispersion during longer periods of time. The formulae and assumptions are as follows:

1. Plume Centerline Model

During neutral and stable atmospheric conditions (i.e., for Pasquill stabilities D, E, F and G) and low wind speeds (less than 6 mps, atmospheric dispersion is computed by the following equations (Reference 34):

$$CHI/Q = \frac{\xi_M}{\pi \bar{u} M \sigma_y \sigma_z} \tag{1}$$

$$CHI/Q = \frac{\xi_G}{\pi \bar{u} \Sigma_y \Sigma_z} \tag{2}$$

and the lesser value is used. In these equations,

CHI/Q = the relative concentration (sec/m^3)

$\bar{u}$ = the wind speed at 13.1 meters above grade (m/sec)

σ_y = the lateral plume standard deviation (m)

σ_z = the vertical plume standard deviation (m)

Σ_y = the lateral plume standard deviation corrected for building-wake effects (see Eq. 3)

Σ_z = the vertical plume standard deviation corrected for building-wake effects (see Eq. 4)

M = the meander factor (a function of atmospheric stability and wind speed), and

ξ_G, ξ_M = correction factors that account for increases in ground-level concentration due to multiple eddy reflections from the ground and from the stable atmospheric layers aloft (see Eq. 6).

The adjusted standard deviations Σ_y and Σ_z, which account for the additional dispersion of the effluent plume within the wake caused by buildings adjacent to the release point, are defined as

$$\Sigma_y = \left(\sigma_y^{\,2} + 0.5\, A/\pi\right)^{1/2} \tag{3}$$

and

$$\Sigma_z = \left(\sigma_z^{\,2} + 0.5\ \mathrm{h_B}^2 / \pi\right)^{1/2} \tag{4}$$

where

h_B = the height of the building causing the additional dispersion (m), and

A = the building's smallest vertical cross-sectional area (m^2).

The maximum values of Σ_y and Σ_z are restricted by the conditions:

$$\left(\Sigma_y\right)_{\max} = \sqrt{3}\sigma_y$$

and

$$\left(\Sigma_z\right)_{\max} = \sqrt{3}\sigma_z$$

Dependence of the meander factor on wind speed and atmospheric stability is shown in Figure 2.3-10 for distances up to 800 meters. Beyond this distance use is made of the adjusted meander factor defined as:

$$M' = \left(M - 1\right)\left[\frac{\sigma_y(800)}{\sigma_y}\right] + 1 \tag{5}$$

where σ_y (800) is the lateral standard deviation of the plume at 800 meters. Note that for unstable conditions (stabilities A, B and C) and for wind speeds greater than 6 mps (independent of stability) the meander factor is equal to unity. For these conditions, atmospheric dispersion is based entirely on Eq. (2).

Definition of the reflection correction factors ξ_G and ξ_M was based on the plume trapping equation in the U.S. EPA workbook on atmospheric dispersion (Reference 35). The EPA equation was reduced to the form

$$\xi_G = \sum_{j=-n}^{n} e^{-(\gamma j)^2} \tag{6}$$

and

$$\xi_M = \sum_{j=-n}^{n} e^{-(\beta j)^2} \tag{7}$$

where

$$\gamma = \frac{\sqrt{2}\,L}{\Sigma_z} \tag{8}$$

and

$$\beta = \frac{\sqrt{2}\,L}{\sigma_z} \tag{8A}$$

L is the height of the reflection layer and 2n is the total number of reflections. The value of n was conservatively set equal to 6; n = 3 or 4 is normally sufficient to include the important reflections.

In the case of large plume standard deviations, where multiple reflections occur and uniform vertical mixing has taken place, the equations reduce to simpler forms. For instance,

$$\underset{\Sigma_z \to \text{large}}{\text{limit}}(\xi_G) = \int_{-\infty}^{\infty} e^{-\tau 2q2}\,dq = \frac{\Sigma_z}{L}\sqrt{\pi/2} \tag{8B}$$

and is achieved for Σ_z greater than approximately 2L. In most cases, where Σ_z << L, ξ_G and ξ_M are equal to unity.

2. Sector-Average Model

Atmospheric dispersion during intervals greater than 8 hours was based on the sector average model,

$$(\text{CHI/Q})_{sa} = \frac{2.032\,\xi_G}{X\bar{u}\Sigma_z} \tag{8C}$$

where X is the distance from the release point to the receptor. Note that when Eq. (8B) is substituted into Eq. (8C) the latter reduces to

$$(\text{CHI/Q})_{sa} = \frac{2.55}{X\bar{u}L} \tag{8D}$$

which is the familiar form of the sector-average dilution equation with uniform vertical mixing.

b. Effective Gamma CHI/Q Equations

Hourly effective gamma dilution factors were computed for ground level releases according to a sector average finite cloud model. The assumptions and formula used are described in Subsection 2.3.5.2.

c. Dilution Factors

Both concentration CHI/Q values and effective gamma CHI/Q values were computed by the above models for each sequential hour of measured meteorological data.

Evaluation of each hourly dilution factor was based on the average wind speed and the vertical temperature gradient indicated in the meteorological data.

A limited mixing layer depth L of about 900 meters was determined by calculating the average of the mean annual morning mixing height and the mean annual afternoon mixing height for the Seabrook site area (Reference 29).

Values for σ_y and σ_z were computed by applying parabolic interpolation (on a log-log basis) to tabular data of these parameters versus distance. The data were extracted from the Pasquill-Gifford curves for atmospheric stabilities A through G (Reference 34). σ_z values were restricted to a maximum of 1000 meters.

Building-wake effects were computed using a building cross-sectional area of 2090 sq. meters and a building height (h_B) of 54.8 meters.

The hourly dilution factors obtained as described and the corresponding direction in which the wind was blowing during each hour were then stored in sector-dependent arrays for sequential processing. This involved the averaging of selected hourly CHI/Q values over successive, overlapping time intervals of 1, 2, 8, 24, 96 or 720 hours, (the last five intervals correspond to the time periods: 1 to 2 hours, 2 to 8 hours, 8 to 24 hours, 1 to 4 days, and 4 to 30 days, as specified in Regulatory Guide 1.4).

For each selected interval size, the processings began with the first hourly CHI/Q value on record, and was then repeated for the same interval size starting with each subsequent hour of CHI/Q data. In the averaging process, the only CHI/Q values within a given time interval that were considered in evaluating the mean dilution factor for the interval were those for the specific wind direction being analyzed. Missing data were handled by imposing the condition that at least half of the entries within an averaging interval corresponded to valid observations. Missing data points were not included in the averaging.

As an illustrative example, consider a 4-hour interval and the following sequence of hourly wind directions: WWWWSMWMMWSSSSS. In this sequence, W is for the west sector, S is for the south sector and M represents missing data. Assuming that each hourly dilution factor is equal to unity, and limiting the total number of valid observations per averaging interval to at least 2 as described above, the sequence of average dilution factors for the west sector are as follows: 4/4, 3/4, 2/3, 2/3, 1/2, blank, 2/2, 1/2, 1/3, 1/4, 0 and 0.

The average dilution factors, computed as described, were subsequently classified for each twenty-two ½ degree sector into groups, and corresponding cumulative frequency distributions were prepared.

The CHI/Q value which is exceeded 0.5 percent of the total time was then determined for each sector, and the maximum value chosen as the maximum sector conservative CHI/Q.

Overall site accident dilution factors were also computed for each of the time intervals of interest. These parameters were determined by first developing arrays of sector-dependent CHI/Qs (averaged over selected time intervals) and then forming an equivalent sector-independent array consisting of the maximum CHI/Qs at equivalent locations in the sector-dependent arrays. These maximum CHI/Qs were then used to form an overall-site cumulative distribution, from which the values at desired percentile points were determined.

The CHI/Q value which was exceeded no more than 5 percent of the total time was then determined and classified as the overall-site conservative CHI/Q.

Sector-dependent concentration CHI/Q values and effective gamma CHI/Q values for the various time intervals are presented in Table 2.3-38 for the exclusion radius (914.4 meters) and in Table 2.3-39 for the outer boundary of the low population zone (2012 meters). Cumulative frequency distributions for the overall site and for the maximum sector CHI/Q values are provided in Table 2.3-40, Table 2.3-41, Table 2.3-42, Table 2.3-43, Table 2.3-44, Table 2.3-45, Table 2.3-46 and Table 2.3-47, and summaries of CHI/Q values for appropriate percentiles are given in Table 2.3-48 and Table 2.3-49.

Note: The short-term accident CHI/Q values for the low population zone, exclusion area boundary, and control room were recalculated as part of a dose reanalysis project for implementing alternative source term (AST). The revised CHI/Q values used for AST are based on guidance provided in Regulatory Guide 1.145, NUREG/CR-2858, NUREG/CR-6331, and Regulatory Guide 1.194. Appendix 2Q discusses the determination of CHI/Qs for offsite locations and Appendix 2R discusses the determination of CHI/Qs for the control room.

2.3.5 Long-Term (Routine) Diffusion Estimates

2.3.5.1 Objective

Realistic estimates of annual average atmospheric transport and diffusion characteristics to a distance of 50 miles from the plant were calculated using meteorological data collected onsite during the period April 1979 through March 1980.

2.3.5.2 Calculations

a. Ground Level Concentration (CHI/Q) Deposition (D/Q) Values

Estimates of relative ground level concentration (CHI/Q), both undepleted by ground deposition and depleted by deposition, and deposition values (D/Q) have been calculated for all points of interest. Table 2.3-50, Table 2.3-51, Table 2.3-52, Table 2.3-53, Table 2.3-54, Table 2.3-55, Table 2.3-56 and Table 2.3-57 indicate the calculated CHI/Q and D/Q values. These calculations were performed in accordance with the model described below.

b. Diffusion Model

1. Background

Atmospheric dilution factors (CHI/Q) and deposition rates (D/Q) were calculated using a dispersion model which makes use of the following:

- hourly meteorological data
- straight-line trajectory with sector-averaged Gaussian dispersion
- fumigation and trapping
- part-time ground-level and part-time elevated releases (mixed mode release model)
- momentum plume rise
- terrain elevation
- depletion in transit, and
- multiple eddy reflections from both ground and stable inversion layers aloft.

The method of analysis involves computation of the following parameters on an hourly basis:

(CHI/Q) the nondepleted dilution factor for evaluating ground level concentrations of noble gases, tritium, carbon 14 and nonelemental iodines,

$(CHI/Q)_D$ the depleted dilution factor for evaluating ground level concentrations of elemental radioiodines and other particulates,

$(CHI/Q)_\gamma$ an effective gamma dilution factor for evaluating gamma dose rates from a sector-averaged finite cloud (multiple-energy undepleted source), and

(D/Q) the deposition factor for evaluating dry deposition of elemental radioiodines and other particulates.

Average dilution and deposition factors were determined from:

$$\left(\bar{F}\right)_\ell = \frac{1}{N} \sum_{j=1}^{m} \left(F\right)_{\ell j} \qquad (9)$$

where F is any one of the four factors listed above, l is the sector identification number, m is the number of hourly values computed for the sector, and N is the total number of values for all sectors.

The fundamental equations used were based on Regulatory Guide 1.111 (Reference 36), and are described below.

2. Nondepleted Dilution Factors

(a) Diffusion Model

Atmospheric transport and diffusion was based on the straight-line flow model with Gaussian diffusion as presented by Sagendorf (Reference 37). The equation for this model, for use in computing sector-averaged hourly dilution factors, is:

$$(CHI/Q) = \frac{2.032}{X\bar{u}} \left\{ \frac{E_\tau}{\Sigma_z} \xi_G + \frac{(1-E_\tau)}{\sigma_z} \xi_E \right\} \qquad (10)$$

where,

X = the distance from the release point to the receptor (m),

$\bar{u}$ = the hourly average wind speed (m/sec),

E_τ = the entrainment coefficient (equal to unity for ground-level releases and to zero for elevated releases),

σ_z = the vertical plume standard deviation at distance X for the atmospheric stability prevailing during the hour of interest (m),

Σ_z = the vertical plume standard deviation corrected for building wake effects (m),

ξ_G = the reflection correction for ground level releases, and

ξ_E = the vertical attenuation and reflection correction for elevated releases.

Note that Eq. 10 applies during normal atmospheric conditions. The effects of fumigation and trapping caused by seabreezes and onshore gradient flows are described later. The terms within the brackets represent, respectively, the contributions to the dilution factor from the entrained and the nonentrained portions of the release during the hour of interest. The effluent is considered to occur as an elevated release ($1-E_\tau$) x 100 percent of the time (1 hour in this case) and as a ground-level release (E_τ) x 100 percent of the time. Details on the definitions of the various parameters are given in the sections that follow.

(b) Entrainment

As outlined in Regulatory Guide 1.111, effluents are considered as ground-level releases ($E_\tau = 1.0$), elevated releases ($E_\tau = 0.0$), or mixed mode releases ($0.0 < E_\tau < 1.0$) depending on (a) the elevation (h_S) of the release point above grade relative to the height (h_B) of adjacent solid structures and (b) the effluent-exit velocity (w_o) relative to the speed of the prevailing wind ($\bar{u}$) during the time period of interest. The various cases are as follows:

for $h_S \le h_B$

$E_\tau = 1.0$

for $h_S \ge 2h_B$

$E_\tau = 0.0$

for $h_B < h_S < 2h_B$

$$E_\tau = 1.0 \text{ when } W_o/\bar{u} \le 1 \quad (11)$$

$$E_\tau = 2.58 - 1.58\,(W_o/\bar{u}) \text{ when } 1 \le W_o/\bar{u} \le 1.5 \quad (12)$$

$$E_\tau = 0.3 - 0.06\,(W_o/\bar{u}) \text{ when } 1.5 \le W_o/\bar{u} \le 5 \quad (13)$$

$^{E}\tau = 0.0$ when $W_o / \bar{u} \geq 5$ (14)

(c) Vertical Standard Deviations and Building Wake

Values of σ_z, the plume vertical standard deviation, were computed by applying parabolic interpolation (on a log-log basis) to tabular data of σ_z versus distance. These data were extracted from Pasquill-Gifford curves in Regulatory Guide 1.111 for atmospheric stabilities A through G.

For ground-level releases, consideration was also given to the additional dispersion of the effluent plume within the wake caused by buildings adjacent to the release point. In such cases, use was made of an adjusted vertical standard deviation defined as:

$$\Sigma_z = (\sigma_z^2 + 0.5\, h_B^2/\pi)^{1/2} \tag{15}$$

The maximum value of Σ_z was restricted by the condition

$$(\Sigma_Z)_{max} = \sqrt{3}\,\sigma_z \tag{16}$$

(d) Vertical Dispersion and Reflection

The ξ_G and ξ_E parameters in Eq. (10) represent the exponential decrease in ground-level concentrations with increasing plume height, and the increase in concentration from multiple eddy reflections from the ground and stable atmospheric layers aloft. Definition of these parameters was based on the plume trapping equation in the U.S. EPA workbook on atmospheric dispersion (Reference 35):

$$\xi_G = \sum_{j=-n}^{n} e^{-(\beta' j)^2} \tag{17}$$

and

$$\xi_E = \sum_{j=-n}^{n} e^{-(\alpha+\beta j)^2} \tag{18}$$

where

$$\alpha = \frac{h_e}{\sigma_Z \sqrt{2}} \tag{19}$$

$$\beta = \frac{\sqrt{2}\, L}{\sigma_z} \tag{20}$$

and

$$\beta' = \frac{\sqrt{2}\ L}{\Sigma_z} \tag{21}$$

h_e is the effective plume height above ground, L is the height of the reflection layer and 2n is the total number of reflections. Parameters h_e and L are discussed later.

In the case of large plume standard deviations, where multiple reflections occur and uniform vertical mixing has taken place, the equations reduce to simpler forms. For instance,

$$\underset{\sigma_z \to \text{large}}{\text{limit}} \left(\xi_E\right) = \int_{-\infty}^{\infty} e^{-(\alpha + \beta j)^2}\ dj = \frac{1}{\beta}\sqrt{\pi} \tag{22}$$

$$= \frac{\sigma_z}{L}\sqrt{\frac{\pi}{2}} \tag{23}$$

and is achieved for σ_z greater than approximately 2L.

Similarly, for ground level releases,

$$\underset{\Sigma_z \to \text{large}}{\text{limit}} \left(\xi_G\right) = \frac{\Sigma_z}{L}\sqrt{\frac{\pi}{2}} \tag{24}$$

When Eqs. (23) and (24) are substituted into Eq. (10), the latter reduces to

$$\text{(CHI/Q)} = \frac{2.032}{X\bar{u}L}\sqrt{\frac{\pi}{2}} = \frac{2.55}{X\bar{u}L} \tag{25}$$

which is the familiar form of the sector-averaged dilution equation with uniform vertical mixing.

In this work, n was conservatively set equal to 6; n = 3 or 4 is normally sufficient to include the important reflections.

(e) Effective Release Height

The effective release height, h_e, was determined in accordance with Regulatory Guide 1.111 from:

$$h_e = h_s + h_{pr} - h_t - c \qquad (26)$$

where

c = the downwash correction for low relative exit velocity (see below),

h_{pr} = the rise of the plume above the release point according to Sagendorf (Reference 37)(m),

h_s = the physical height of the release point (m), and

h_t = the maximum terrain height (above the release point grade elevation) between the release point and the receptor ($h_t > 0$)(m).

The downwash correction factor is defined as (Gifford, Reference 38)

$$c = 3(1.5 - W_o/\bar{u})d \text{ when } W_o/\bar{u} < 1.5 \qquad (27)$$

and

$$c = 0 \text{ when } W_o/\bar{u} \geq 1.5 \qquad (28)$$

where

d = the inside diameter of the release vent (e.g., stack) (m),

W_o = the vertical exit velocity of the plume (m/sec)

$\bar{u}$ = the mean wind speed (m/sec)

(f) Plume Rise

Only nonbuoyant plumes were assumed to emanate from the vents of the power station. The momentum jet equations reported by Briggs (Reference 39) and Sagendorf (Reference 37) were utilized as outlined below.

For neutral or unstable conditions the transitional rise was computed by the equation

$$h_{pr} = 1.44 \left(\frac{W_o}{\bar{u}}\right)^{\frac{2}{3}} \left(\frac{X}{d}\right)^{\frac{1}{3}} d \qquad (29)$$

and

$$h_{pr} = 3\left(\frac{W_o}{\bar{u}}\right) d \qquad (30)$$

and the lesser, more conservative value was used.

For stable conditions, the results from Eqs. (29) and (30) were compared with the results from the following two equations

$$h_{pr} = 4\left(\frac{F_m}{S}\right)^{1/4} \qquad (31)$$

$$h_{pr} = 1.5\left(\frac{F_m}{\bar{u}}\right)^{\frac{1}{3}} S^{-\frac{1}{6}} \qquad (32)$$

and the smallest value of hpr was used. In the last two equations, F_m is the momentum flux parameter and S is a stability parameter. They are defined as

$$F_m = W_o^2\left(\frac{d}{2}\right)^2$$
$$S = \frac{g}{T}\frac{\partial\theta}{\partial z} \qquad (33)$$

where

g = the acceleration of gravity (m/sec^2),

T = the ambient air temperature (deg K), and

$\frac{\partial\theta}{\partial z}$ = the vertical potential temperature gradient (deg K/m) defined as (Reference 39):

$$\frac{\partial\theta}{\partial z} = \frac{\partial T}{\partial z} + 0.0098 \qquad (34)$$

(g) <u>Mixing Depths</u>

Vertical diffusion of the plume can be inhibited by the existence of a stable atmospheric layer (an elevated inversion) aloft. The rate of vertical mixing is reduced in such cases and the stable layer can be considered as an effective lid on vertical transport of pollutants.

Although there are many hours in the year characterized by unlimited mixing depths, in this analysis it was conservatively assumed that all hours were characterized by the average of the mean morning and mid-afternoon mixing depths for the year.

The effect of plume trapping is included in the terms in Eq. (10).

3. Seabreeze and Onshore Gradient Flow

(a) General

Formulation of a thermal internal boundary layer (TIBL) under conditions of seabreeze or onshore gradient flow can lead to ground-level concentrations which are higher than those calculated for a simple straight-line model. Figure 2.3-11 illustrates the importance of the TIBL on the behavior of effluents.

During an onshore wind, the cool and stable marine air is heated from below by the land surface and becomes unstable in the lower levels. When the plume begins to intercept the top of the TIBL the material in the plume is mixed rapidly downward in the unstable air within the TIBL. This rapid downward mixing is referred to as fumigation (References 40, 41 and 42).

For releases which occur at ground level, the material is trapped within the TIBL. This lid limits the mixing volume to a greater extent than the average mixing depth for the area around the site and can also result in higher ground-level concentrations.

(b) Criteria for and Occurrence of TIBL Formation

The following criteria have been established.

(1) TIBLs Can Occur Only during Spring and Summer

The examination is restricted to those periods when the land-water temperature difference can result in the formation of a TIBL. A seabreeze season extending from April through September is appropriate for the Seabrook site.

(2) TIBLs Can Occur Only during Daytime

The examination is restricted to those times when there is sufficient solar intensity to generate a TIBL. TIBLs were conservatively assumed to occur between 0800 EST and 1800 EST during the seabreeze season.

(3) The Wind Direction Must Be Onshore

The overwater fetch must be sufficiently long to stabilize the air mass. Based on observations reported in the literature (Reference 43 and 44), an overwater fetch of five to ten miles will result in a marine inversion several hundred feet deep. Wind trajectories from the northeast clockwise through the south-southeast would have a sufficient overwater fetch to result in a stable air mass moving over the Seabrook site.

(4) The Wind Speed Must Be in an Appropriate Range

Too low a wind speed will not a support a TIBL. If the wind speed is too great, mechanical turbulence will overcome any thermal effects and a TIBL will not be formed. A range of wind speeds between about 4.5 and 22 miles/hour characterizes the conditions of interest. This range is consistent with the onsite data and with the fundamental nature of the seabreeze.

(5) Solar Radiation Must Be Sufficiently Strong

Since it is the heating of the land which causes the development of a TIBL, the intensity of solar radiation is an important parameter. A minimum value of 0.35 Langleys/minute has been assumed for solar radiation. This magnitude of solar intensity occurs early and late on bright days. This value compares with peaks of about 1.2-1.3 Langleys/minute at mid-day during clear summer days.

As shown in Table 2.3-58, TIBLs were estimated to form during approximately 100 days and lasted on average 5.3 hours per day during the period April 1979 - September 1979. The highest monthly TIBL frequency, 133 hours per month, occurred during June.

(c) Criteria for and Occurrence of Sea Breeze Conditions

Estimated frequency of sea breeze conditions for the Seabrook site during the same time period are also provided in Table 2.3-58. A sea breeze condition was defined as an hour where a localized daytime onshore flow occurred simultaneously with an opposing larger scale (but weaker) inland geostrophic wind directed offshore. Unlike gradient onshore flows, localized sea breeze onshore flows result in the development of sea breeze fronts/convergence zones and recirculation cells.

A localized sea breeze onshore flow would develop under the same conditions under which the TIBL forms; i.e., strong solar radiation and daytime land-surface temperatures rising above the ocean-surface temperatures. In order to differentiate between a true localized sea breeze and a gradient onshore flow, Worcester NWS wind data was used to determine if a larger scale offshore geostrophic wind existed further inland. Data from Worcester (located approximately 70 miles SW of the Seabrook site) were chosen because the observations are obtained on an elevated plateau free from localized terrain effects and are taken far enough inland (45 miles from the coast) to preclude influence by any sea breeze fronts.

The criteria used to determine frequency of sea breeze conditions at the Seabrook site are as follows:

(1) Seabreezes Occur Only under the Same Conditions Conductive to TIBL Formation

The criteria used above to identify TIBL formation are also used to identify seabreeze conditions.

(2) There is an Opposing Offshore Pressure Gradient

Winds observed simultaneously at Worcester are offshore (from the S clockwise to NNE).

As shown in Table 2.3-58, sea breeze conditions were estimated to occur during approximately 89 days and lasted an average of 4 hours per day during the period April 1979 - September 1979. The highest monthly seabreeze frequency, 92 hours per month, occurred during July.

(d) TIBL Geometry

No onsite measurements correlating the shape of the TIBL with the governing parameters are available. In the absence of site measurements, the following relation was selected (Reference 40, 41 and 43).

$$h_{TIBL}\ (X) = 8.8 \left(\frac{X}{\bar{u}\Delta\theta} \right)^{1/2} \qquad (35)$$

where

$h_{TIBL}(X)$ = the depth of the boundary layer above the ground surface (m),

X = the distance from the shore along the wind trajectory (m),

$\bar{u}$= the hourly average wind speed (m/sec), and

$\Delta\theta$ = the potential temperature difference between the top and bottom of the marine inversion.

The TIBL was assumed to follow the terrain. The maximum depth of the TIBL was limited to the climatological mixing height. A reasonable value of 4°C was assumed for $\Delta\theta$ in Eq. (35).

(e) Ground-Level Concentrations

A model was developed to calculate ground-level concentrations from reactor effluents during conditions of seabreeze and onshore gradient flow. The calculation is based on the straight-line air flow equation (Eq. 10), with the following conditions:

- Pasquill stability class F is assumed above the TIBL (effects both dispersion and plume rise for releases above the TIBL).
- Tower-indicated stabilities, classes A through D (with E, F and G defaulting to D), are assumed below the TIBL whenever the TIBL is above the levels of measurements; stability class B is assumed for TIBLs lower than the levels of tower measurements.
- Effluents released beneath the TIBL are trapped; mixing depth (L) in Eqs. (20) and (22) is replaced by the height of the TIBL; the plume is not permitted to penetrate the TIBL even if the plume rise equation predicts so.

- Effluents released above the TIBL travel downwind until the plume and TIBL intersect (see below); beneath the TIBL, σ_z and Σ_z are set equal to twice the TIBL height to insure Eqs. (17) and (18) predict uniform mixing (applies also to the entrained portion of the elevated release); a correction factor is applied for partial mixing within the mixing zone.

Intersection between the plume and the TIBL is defined to occur when the turbulence just begins to disturb the lower portion of the plume, that is,

when

$$h_{TIBL}(X') = (h_s + h_{pr} - c) - 2.15\ \sigma_z \tag{36}$$

where

σ_z = for Pasquill stability class F. Total envelopment of the plume by the TIBL occurs

when

$$h_{TIBL}(X'') = (h_s + h_{pr} - c) + 2.15\ \sigma_z \tag{37}$$

Before the plume-TIBL intercept, the standard elevated plume equation applies. After total envelopment, uniform vertical mixing is attained. For the intermediate zone, a correction factor is applied to the second term in Eq. (10) to account for partial mixing. This factor is equal to (Reference 42).

$$\frac{1}{\sqrt{2\pi}} \int_{-\infty}^{P} \exp\left(-0.5\ p^2\right)\ dp \tag{38}$$

where

$$P = (h_{TIBL} - h_e)/\ \sigma_z \tag{39}$$

h_e = the effective plume height defined by Eq. (26) and h_{TIBL} is the height of the TIBL at the receptor.

For the 527 hours of TIBL formation estimated to occur during the period April 1979 - March 1980, the lowest TIBL height predicted to occur over the plant primary vent stacks is 93 meters above ground level (AGL). Since the primary vent stack release height is approximately 56 meters AGL, all releases during TIBL formation occur below the TIBL. The closest the effective stack height ever approaches the TIBL during all hours of TIBL formation is 28 meters. As such, the effect the TIBL has on the annual average atmospheric dispersion estimates is to limit the vertical mixing depth during hours of TIBL formation.

TIBL terrain correction factors (defined as ratios of annual average relative concentrations (CHI/Q) and deposition rates calculated considering trapping versus annual average CHI/Q and deposition rates calculated without considering trapping) are presented in Table 2.3-59, Table 2.3-60, Table 2.3-61 and Table 2.3-62. These annual average TIBL terrain correction factors were calculated for primary vent stack releases and were compiled using April 1979 - March 1980 onsite meteorology. Distances beyond 20 miles are not presented because the annual average transport and diffusion model assumes the TIBL does not extend beyond 20 miles. In addition, because one of the criterion for TIBL formation is an onshore flow, only the SW clockwise through NNW downwind sectors show TIBL terrain correction factors other than one.

Table 2.3-59 shows that TIBL terrain correction factors for the undepleted CHI/Q average approximately 1.03 for the affected downwind sectors and range from a minimum value of 0.97 (0.25 miles W) to a maximum value of 1.19 (0.75 miles NW). TIBL terrain correction factors for the depleted CHI/Q, deposition rates, and effective gamma CHI/Q show similar patterns as indicated in Table 2.3-60, Table 2.3-61 and Table 2.3-62. Note that a few of the TIBL terrain correction factors are slightly less than one due to the stipulation that all E, F and G stability classes measured below the TIBL during TIBL occurrences default to D stability.

4. <u>Depleted Dilution Factors and Deposition Rates</u>

Only dry deposition of elemental radioiodines and other particulates and attendant plume depletion were considered for all releases. The effects of wet deposition and radioactive decay in transit were not included. The models employed are described below.

(a) Depleted CHI/Q

Due to impaction of particles on surfaces, such as walls, vegetation and ground, depletion of plumes containing radioiodine and other particulates occurs in the downwind direction. Ground-level concentrations corrected for depletion in transit were computed for each hour according to the formula

$$(\mathrm{CHI/Q})_{\mathrm{D}} = \frac{2.032}{\mathrm{Xu}} \left\{ \frac{\mathrm{E}\tau \mathrm{D_G}}{\Sigma_z} \xi_G + \frac{(1-\mathrm{E}_\tau)\mathrm{D_E}}{\sigma_z} \xi_{\mathrm{E}} \right\} \tag{40}$$

where D_G and D_E are the depletion factors for ground-level and elevated releases and the remaining parameters are as defined in Subsection 2.3.5.2b.2.

(b) Deposition Rates

The depletion model employed was based on a deposition velocity v_d defined as

$$v_d = \frac{\text{rate of deposition}}{\text{concentration near the surface}} \tag{41}$$

Combining Eqs. (40) and (41), one obtains the deposition rate equation

$$(\mathrm{D/Q}) = v_d\,(\mathrm{CHI/Q})_{\mathrm{D}} \tag{42}$$

The relationship of average wind speed and areal grass density to deposition velocity was examined in detail by Pelletier and Zimbrick (Reference 45) and is as follows:

$$\frac{v_d}{\rho}\left(\mathrm{cm^3/g-sec}\right) = 19.4\,\bar{u}(\mathrm{m/sec}) \tag{43}$$

where

ρ = the areal grass density (dry weight, g/cm^2)

(c) Depletion Factors

The depletion factors D_G and D_E which account for dry deposition in transit, were computed according to (Reference 46)

$$D_G = \exp\left(-\delta \int_o^X \frac{\mathrm{dX}}{\Sigma_z}\right) \tag{44}$$

$$D_G = \exp\left(-\delta \int_o^X \frac{dX}{\sigma_z \exp\left(h^2 / 2\sigma_z^2\right)}\right) \qquad (45)$$

where

$$\delta = \sqrt{\frac{2}{\pi}}\ \frac{\nu d}{\bar{u}} \qquad (46)$$

and

$$h = h_s + h_{pr} - 0.5h_t - c \qquad (47)$$

Note that

$$h = h_e + 0.5h_t \qquad (48)$$

and indicates the conservative use of an average terrain height between release and receptor points for depletion calculations.

Equations (44) and (45) were used to compute, by numerical integration, tables of depletion factors for all atmospheric stabilities and a number of effective plume heights. Depletion factors for use in Eq. (40) were then obtained by applying parabolic interpolation to the tabular data for the prevailing stability and effective h.

Depletion during trapping and fumigation conditions was calculated using the conservative assumption that the TIBL does not exist. For releases above the TIBL, an elevated plume was assumed if the receptor point was before the totally enveloped zone. Depletion based on a ground-level plume was assumed for receptors within this zone.

5. Gamma Dilution Factors

Evaluation of long-term external gamma doses from routine atmospheric releases is normally based on extensive simplifying assumptions that are essential in reducing the complexity of the problem. An evaluation model that is of interest to the nuclear power industry is described by D.H. Slade (Reference 46, Subsection 7-5.2.5). The model, which is characterized by a finite sector-averaged plume with Gaussian activity distribution in the vertical plane, is of interest because it provides a means of establishing effluent emission limits and making dosage estimates that are more realistic than those based on uniform semi-infinite clouds. The model has been included in Regulatory Guide 1.109 (Reference 47) as guidance toward evaluating compliance with 10 CFR Part 50, Appendix I.

This section presents the method by which the finite-cloud dose rate equation in Regulatory Guide 1.109 has been reduced to that for a semi-infinite cloud via the definition of an effective gamma dilution factor.

(a) Finite-Cloud Gamma Dose Rate Equation

According to Regulatory Guide 1.109, gamma air doses from elevated releases (and ground-level releases as well) may be computed using the equation

$$D^{\gamma}_{\text{finite}} = \frac{260}{X\Delta\phi} \sum_{n,s,k,i} \frac{Q_i^{'} f_{ns} \mu_a^k \bar{E}_K \; \bar{I} A_{ki}}{\bar{u}_n} \tag{49}$$

where

A_{ki} = the photon yield for gamma-ray photons in energy group k from the decay of radionuclide i, (photons/disintegration),

$D^{\cdots}_{\text{finite}}$ = the annual total gamma air dose at a distance X in the sector at angle ϕ (mrad/year),

X = the distance from the release point to the receptor (m),

$\bar{E}_k$ = the energy of the kth photon energy group (MeV/photon),

f_{ns} = the fraction of the time that stability class s and wind speed n occur for sector φ, (dimensionless),

$\bar{I}$ = the result of the numerical integration accounting for the distribution of radioactivity according to meteorological conditions of wind speed and atmospheric stability. In addition, I is a function of the photon energy E_k and is $= {}_1 + K_2$ as formulated in Slade (Reference 46); K is the air buildup factor,

$Q_i^{'}$ = the release rate of radionuclide i (Ci/yr),

$\bar{u}_n$ = the mean wind speed of wind speed class n (m/sec),

$\Delta\phi$ = the sector width over which atmospheric conditions are averaged (radians), and

μ_a^k = the air energy absorption coefficient for the kth photon energy group (m^{-1}).

From Eq. (49) it is possible to define an energy dependent gamma dilution factor for use in computing hourly air dose rates as follows:

$$(\text{CHI/Q})_k^\gamma = \frac{2\mu_a^k\left(\bar{I}_1 + K\bar{I}_2\right)}{\sqrt{\pi}\,\bar{u}X\Delta\phi} \tag{50}$$

As shown in Reference 48, the integrals I_1 and I_2 in this equation reduce to the following limits when the size of the finite cloud becomes large:

$$\bar{I}_1 = \frac{1}{\sqrt{2}\mu^k\sigma_z}\exp\left[\frac{-h_e^2}{2\sigma_z^2}\right] \tag{51}$$

and

$$\bar{I}_2 = \bar{I}_1 \tag{52}$$

μ^k is the air total linear attenuation coefficient for the kth photon energy group. For this condition, since

$$K = \left(\mu^k - \mu_a^k\right)/\,\mu_a^k \tag{53}$$

Eq. (50) reduces to

$$(\text{CHI/Q}) = \frac{2.032}{X\bar{u}\sigma_z}\exp\left(-h_e^2/2\sigma_z^2\right) \tag{54}$$

and is dependent of gamma energy. Eq. (54) is the basic equation for sector-averaged semi-infinite clouds.

To reduce Eq. (49) to a form similar to that for a semi-infinite cloud, it is necessary to define a few more parameters as follows:

$$\bar{A}_K \sum_i = \frac{Q_i'\ A_{ki}}{\sum_i Q_i'} \tag{55}$$

$$\overline{E}_k = \frac{\sum_i E_{ki}\ Q_i^{'}\ A_{ki}}{\sum_i Q_i^{'}\ A_{ki}} \tag{56}$$

$$\overline{CHI/Q}^{\gamma} = \frac{\sum_k E_{ki}\ Q_i^{'} A_{ki}}{\sum_k E_k A_{ki}} \tag{57}$$

and (using Eq. (9) without the sector identification subscript)

$$\overline{\mathrm{CHI/Q}}^{\gamma} = \frac{1}{N} \sum_{j=1}^{m} \left\{ (\mathrm{CHI/Q})^{\gamma} \right\} j \tag{58}$$

In these equations, E_{ki} is the actual energy of a gamma ray in group k emitted by isotope i. The other parameters are as previously defined.

Substitution of these equations in Eq. (49), and replacing f_{ns} by () for the hourly factors, results in:

$$D^{\gamma}_{finite} = 3.17\, x 10^4 \left(\overline{CHI/Q} \right)^{\gamma} \sum_i Q_i^{'}\ (DF)_i^{\gamma} \tag{59}$$

where

$$(DF)_i^{\gamma} = \sum_K \frac{260\ \sqrt{\pi}}{(2)\ (3.17) x 10^4}\ A_{ki}\ E_{ki} \tag{60}$$

is the gamma air dose factor for radionuclide i as used in Regulatory Guide 1.109. 3.17x104 is the number of pCI per Ci divided by the number of seconds per year. Note that Eq. (59) is similar in form to the dose equation for semi-infinite clouds.

(b) Application

Equations (50), (55), (56), (57) and (58) were used in computing effective gamma dilution factors for evaluating air gamma doses from finite clouds for both ground-level and elevated releases.

Equation (50) was computed on an hourly basis for 10 to 15 gamma energy groups covering the range 0.01 to 4 MeV. Energy-averaged factors were then computed by Eq. (57) for each hour of interest. The integrals 1 and 2 were computed by numerical integration using the model described in References 47 and 48. Data for the total linear energy absorption and attenuation coefficients were taken from Reference 49.

Gamma dilution factors during seabreeze conditions were calculated by ignoring the presence of the TIBL and applying the following modification to account for the increased concentrations near the ground due to the presence of the TIBL:

- For ground-level releases (trapping conditions) Σ_z was limited to the maximum value of 0.47 h_{TIBL}, where h_{TIBL} is the height of the h_{TIBL} at the receptor location.
- For trapped elevated releases, the plume effective height and standard deviation were adjusted to ensure that the entire plume is within the TIBL while maintaining the ground-level concentration:

$$\sigma z \rightarrow \sigma z^{\gamma} = \frac{h_{TIBL}}{\left(h_e / \sigma_z + 2.15\right)} \tag{61}$$

and

$$h_e \rightarrow h_e^{\gamma} = \frac{\sigma_z^{\ \gamma}}{\sigma_z} \tag{62}$$

- For releases above the TIBL that become entrained, E_z was set equal to 0.47 H_{TIBL}
- For releases above the TIBL that are not entrained, σ_z was retained for distances less than the plume-TIBL intercept and was set to 0.47 H_{TIBL} for receptors beyond that point; receptors within the fumigation zone were assumed to be exposed to both an elevated plume and a fully entrained plume, the contribution from each plume being based on Eq. (38).

6. Model Applicability

The Gaussian dispersion model, (straight-line trajectory airflow model described in Regulatory Guide 1.111) used for the CHI/Q and D/Q calculations has been widely accepted by the scientific and engineering community, including industrial users and regulatory agencies. The results of calculations are generally considered to represent reasonable, and often conservative, estimates of pollutant concentrations.

For the Seabrook site, the feature which has been specifically evaluated with respect to atmospheric dispersion is the effect of the land-water interaction. The model used, as well as similar models, to calculate the ground level concentrations during conditions of seabreeze and onshore gradient flow have undergone validation analysis at other sites.

A smoke tracer experiment was conducted at a coastal nuclear power station site as reported in Reference 40. The application of the model described above provided generally good agreement with observation of the location of the fumigation point.

The results of related field experiments are reported in References 41 and 42. These experiments show fairly successful application of similar models in predicting concentrations from SO_2 sources at Great Lakes locations.

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2.4 HYDROLOGIC ENGINEERING

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

Seabrook Station site is located in the northern part of Seabrook, New Hampshire, approximately one mile from the western shore of Hampton Harbor. Hampton Harbor is situated at the confluence of Hampton River, Browns River and Blackwater River, and is located on the coast of New Hampshire, about 1.5 miles north of the Massachusetts state line and 13 miles south of Portsmouth Harbor. The towns of Hampton, Hampton Falls, and Seabrook abut Hampton Harbor on the north, west, and south respectively. The villages of Hampton Beach, north of the harbor entrance, and Seabrook Beach, south of the entrance, border the navigable waters of the harbor.

The entrance to Hampton Harbor is crossed by highway Route 1A. New Hampshire Route 236 crosses the Blackwater River about two miles south of the harbor entrance on a fixed bridge, and the Boston and Maine Railroad crosses Mill Creek, Browns River and Hampton Falls River about two miles west of the harbor entrance on small bridges. The rivers are navigable up to these bridges.

The station site is situated on a point of land the terminus of which is called "The Rocks," located between the Browns River and Hunts Island Creek. Adjoining the site is a broad, flat marsh zone in the north, east and south, identified as Hampton Flats, with an elevation of approximately +4 feet MSL.

The normal high tide water level at Hampton Harbor estuary is approximately +4.6 feet MSL while site grade is +20 feet MSL; therefore, the estuary will accept the surface drainage of the plant site. The natural drainage features of the area surrounding the site have been left unchanged.

The station structures are located at finished grade elevation of +20 feet MSL. Some of the existing ground elevations of the site beyond the plant limits and bordering on the salt marsh are below this elevation and could be exposed to flooding from wave runup produced by the hypothetical storm surge. These locations of the plant site adjoining the salt marsh (northeast, east, southeast and south sides) are protected by a riprap revetment or a seawall at the edges of the embankment. Figure 2.4-1 shows the topography, the arrangement of the plant site, and the protective revetment and sea walls.

2.4.1.2 Hydrosphere

The New Hampshire Coastal Area is a 47,000 acre, triangular-shaped drainage basin at the eastern end of Rockingham County in the extreme southeastern corner of New Hampshire. It includes all of the drainage entering the Atlantic Ocean between Odiornes Point in Rye (the south entrance point to the Piscataqua River) and the southern end of Seabrook Beach at the Massachusetts state line, 16 miles to the south. It is with the Seabrook area, particularly Hampton Harbor, that this report is primarily concerned. The major hydrologic features of the region are depicted in Figure 2.4-2, Figure 2.4-6 and Figure 2.4-7 (2 sheets).

Geographically, Hampton Harbor is located about 13 miles south of Portsmouth Harbor, 8 miles south of Rye Harbor, 1.5 miles north of the Massachusetts state line and 5 miles north of Newburyport Harbor, at the mouth of the Merrimack River.

Hampton Harbor itself is a shallow lagoon of about 596.8 acres behind two barrier beaches: Hampton Beach to the north of the harbor entrance, and Seabrook Beach to the south of the harbor entrance. The harbor is roughly 1.2 miles wide by 1.5 miles long. It is situated at the confluence of several rivers. These are shallow tidal streams emptying into Hampton Harbor from the Blackwater River, Hampton Falls River and Taylor River drainage basins.

The mean tidal range of Hampton Harbor itself is about 8.6 feet, varying from 4 feet below to 4.6 feet above MSL. Since the harbor is very shallow, only 5-6 feet of water remains in the deeper channels at low tide and only 2-3 feet of water covers most of the area. The volume in the intertidal zone or the tidal prism of Hampton Harbor is 224 million cubic feet.

Within the entire Hampton Harbor estuary, the volume in the tidal prism (between MLW and MHW) is approximately 470 million cubic feet. The maximum average tidal velocity through the harbor entrance is about 1.7 fps.

The Hampton River is tidal for two miles to the northwestward, where it is fed largely by the Taylor and Hampton Falls Rivers. The Taylor River has a total length of 10 miles and a total fall of only 75 feet. This river has a safe yield of between 1 and 10 million gallons per day within a length of 1 mile above the Hampton Falls River. The Hampton Falls River has a total length of nearly seven miles and a total fall of nearly 120 feet. The lower of two series of small falls has been developed by three small dams near the village of Hampton Falls, about two miles upstream from the mouth of the river. The impounded water bodies, Dodge Ponds, have a total surface area of roughly 20 acres.

A third tributary of the Hampton River is the 8-mile long Tide Mill Creek which drains the south-central part of North Hampton and the eastern part of Hampton. It flows southward through extensive marshes into the Hampton River about one mile north of Hampton Harbor.

The Blackwater River terminates in a 4-mile long tidal inlet which extends two miles southward from Hampton Harbor to the Massachusetts state line. In addition, Browns River, Hunts Island Creek and Mill Creek flow into the confluence from the west. Although there is a discernible watershed, it is small and the attendant fresh water run-off is not particularly significant. Thus, the several streams and their branches primarily serve a tidal stream directing the semidiurnal inward and outward flow of saline water.

Estimates of the areal extent of the salt marsh and Hampton Harbor are found in several sources. The variance in the estimation is high, probably reflecting the difference in interpretation of what constitutes marshland and harbor. Whether or not planimetry was used on Geological Survey maps, Coast and Geodetic Survey charts or aerial photographs may also cause differences in values of areal extent.

The New England River Basins Commission (1971) estimates the total acreage of marsh and open water as 5700 acres of which some 4990 acres are tidal marshes. It further states that there are several hundred acres of intertidal flats fringing the mouths of the tidal streams and the harbor.

Normandeau Associates, Inc., (1971) estimates the tidal marsh area to be approximately 3800 acres as obtained from planimetry of a U.S.G.S. Topographical map.

The Corps of Engineers (1972) estimates the tidal marsh to be about 8 square miles (5120 acres) in extent.

The New England - New York Interagency Committee (1955) estimates the open water harbor area at high tide to be 300 acres in extent.

The New Hampshire Fish and Game Department (1964) estimates the salt marsh to be 2784 acres in extent, although a check of their figures indicates 3085 acres of tidal marsh and 23 acres of dunes and flats.

Finally, Ebasco Services, Inc., (1969) estimates the water surface area in the estuary arms at mean low water to be 24 million square feet (550.9 acres). This result was arrived at by planimetry of aerial photographs.

The plant site is located between the Browns River and Hunts Island Creek, both of which are less than 3 river miles long. These two rivers are mainly contributed to by the estuarine tide from Hampton Harbor and carry very little surface run-off.

Refer to Subsection 2.4.13.2 for information on groundwater use in the area.

2.4.2 Floods

2.4.2.1 Flood History

Land areas lying in tidal zones of estuaries are subject to flooding from sea water moving landward, particularly when the tide levels are raised by storm surges, wind setups and wind-generated waves. Records indicate that the highest levels of tidal flooding along the New Hampshire coast have been caused by coastal storms, commonly referred to as "northeasters." Somewhat lesser flooding has also been produced by hurricanes.

Nearly all of the severe northeasters have occurred during the winter or early spring months, from November through April, whereas the more damaging hurricanes have been experienced in late summer, most frequently in August and September. The observed tide elevations at Boston, Massachusetts and Portland, Maine, during five northeasters and three hurricanes, and estimated average levels of flooding along the New Hampshire shore (Reference 1) are tabulated in Table 2.4-1.

Tidal flooding damage along New Hampshire's 18 miles of shorefront has usually been moderate in the past. In Hurricane Carol, on August 31, 1954, the estimated damages, excluding wind damage, amounted to about $20,000. In the record storm of December 29, 1959 the damages to shore properties, excluding boats, fish traps, and similar items amounted to an estimated $60,000. In the two storms of January 20, 1961 and November 30, 1963 the reported damages, other than boats blown ashore, were minor amounts and resulted mainly from flooding of low-lying roads.

The northeaster of February 19-20, 1972 was said to be the worst of this century up to that date in the New Hampshire coastal area. Tidal elevations for Boston, Massachusetts and Portland, Maine were below record heights; however, exceptionally high winds, breakers and surf combined to cause major coastal damage estimated between $500,000 and $5 million by the National Oceanic and Atmospheric Administration (Reference 2). The predicted astronomical tide height for Hampton Harbor was 9.6 feet MLW. The surge height at Hampton Harbor is unknown.

The northeast storms of January and February 1978 are recorded as the worst storms of record for the New Hampshire coastal area. Tide elevations during the February storm reached unusually high levels as a result of exceptionally high winds measured in excess of 100 MPH offshore of the site and monthly spring tide. Wind and wave damage along the New Hampshire sea coast was substantial and resulted in the designation of several communities as natural disaster areas. The predicted astronomical tide maximum was 10.3 feet MLW combined with a surge height of 2.5 feet, resulting in a tidal elevation of +12.8 feet MLW.

2.4.2.2 Flood Design Considerations

The design bases for flood protection of safety-related facilities, systems and equipment of the Seabrook plant, are derived from the hypothetical flooding conditions which the site could possibly be subjected to during its design life, concluding with the highest and most critical flood level resulting from any of the several different probable maximum events. These design bases meet the intent of Regulatory Guide 1.59 (Rev. 2). The categories of hydrologic events, individually or in combination, which have been considered are summarized below; the details are presented in succeeding subsections:

a. Stream flooding from Probable Maximum Precipitation (PMP)

b. Open coast surge flooding

c. Wave height

d. Combination of surge and stream flooding

e. Tsunami induced flooding

f. Seiche flooding

g. Flooding caused by dam failures

Conclusions upon which flood protection design are based were derived from the analyses summarized below:

a. Stream Flooding From Probable Maximum Precipitation (PMP)

The PMP for the local 47.4 square mile Hampton drainage basin over a 24-hour period is estimated as 23.8 inches. The run-off model for the PMP as a lone causation of stream flooding is deemed insignificant in influencing plant design criteria since the storm flooding resulting from the combined occurrence of a probable maximum hurricane (PMH) and a standard project storm (SPS) is much greater than that of a PMP-induced flood.

b. Open Coast Surge Flooding

Surge elevations were computed for both the probable maximum hurricane (PMH) and the probable maximum northeaster (PMN), to determine which event should be used for design open coast surge flooding. Maximum open coast still-water surge elevation at Hampton Harbor resulting from the combined PMH-SPS event is estimated as 14.6 feet MSL. Maximum open coast still-water surge elevation resulting from the PMN is predicted as 12.7 feet MSL. Thus, the critical flood elevations at the site result from the PMH occurring simultaneously with the SPS.

The surge level in the harbor, however, is not expected to be the same as the open coast surge, since an enclosed bay open to the ocean by a narrow inlet is expected to experience a tidal amplitude range lower than that at the open coast due to the frictional effect of the narrow inlet. This frictional effect is expected to be especially more pronounced during the peak harbor surge because of the short period of time it lasts. The effect in the case of Hampton Harbor will be further increased by the eleven piers of the highway bridge that span the inlet.

c. Wave Height

The largest supported wave height at the Seabrook perimeter is predicted as 7.9 feet. This takes into account the influence of the elevation of the salt marshes in front of the site.

d. Concurrent Flooding Events

The joint occurrence of PMP and PMH has an extremely low probability. Instead, the standard project storm (SPS) is a more reasonable event to be considered for possible coincidence with a PMH storm surge. The critical phasing of the PMH open coast storm surge with the standard project flood (SPF) discharge results in a probable maximum stillwater level at the site, with an allowance for cross-wind setup within the harbor, of 14.5 feet MSL.

e. Wave Runup and Overtopping

During the occurrence of the design basis PMH-SPF event, wave runup resulting in overtopping of the protective revetment and seawall is anticipated. The duration of the actual overtopping conditions is limited to a brief period of time, approximately one to two hours, occurring at the time of peak water levels and maximum wave attack. Wave overtopping, which would primarily occur on the southeast side of the site, will result in ponding of water on the plant grade and run-off via the northeast side of the site. The maximum depth of ponding, occurring at the time of peak overtopping, is estimated at an elevation not exceeding 21 feet MSL. This value includes the effect of local intense SPS precipitation on the plant grade and allows for available drainage capacity in the yard drainage system. The maximum wave height which would be supported in this depth of water is nominally 0.6 feet.

Assuming that the maximum supported wave arrives at any safety-related structure undiminished in height, the maximum runup elevation on a smooth vertical wall would be 21.8 feet MSL (see Subsection 2.4.5.3). Wave runup and overtopping will, however, not affect any safety-related structures, since all such structures are provided with appropriate flood protection.

f. Tsunami Flooding

Such activity is extremely rare on the U.S. Atlantic coast and would result in only minor wave action inside the harbor.

g. Ice Flooding

No specific ice flooding design criteria are proposed for safety- related facilities, since the facilities are located at an elevation which makes them invulnerable to any local ice activity. Local flooding due to ice formation is considerably below maximum surge conditions.

h. Potential Dam Failures

There are no dams, area reservoirs, plant cooling water canals or plant cooling water reservoirs, planned for the Seabrook site. In view of the topography and characteristics of the local streams, it seems unlikely that reservoirs of any substantial size will be built.

2.4.2.3 Effects of Local Intense Precipitation

Tabulated below is the local probable maximum precipitation for the Seabrook Station site covering a 10-square mile area as determined from Hydro-meteorological Report No. 33 (Reference 3); the time distribution of this PMP for 6-hour durations was obtained using Plate 11 of EM 1110-2-1411 (Reference 8).

Time Period (hours)	Rainfall Accumulation (inches)
1	8.6
2	12.0
3	15.2
4	17.9
5	20.4
6	22.7
12	25.2
24	27.3

The above table is used as the basis for the run-off analysis. Tabulated time incremental distribution of the PMP for the drainage area of 47.4 square miles is depicted in Figure 2.4-3.

The underground storm drainage system capacity is calculated to be about 100 cfs (at the threshold of ponding). No credit is taken for the underground storm drainage system in the site design basis flooding analyses.

Water is assumed ponded on the entire site ($2x10^6$ square feet) when the site drainage system capacity is exceeded or when the drains are blocked. This ponding is conveyed off site via overland flow along the site perimeter.

Site grading, particularly in the vicinity of the catch basins, was established as 0.5 percent minimum to encourage run-off. Elevation of the catch basin gratings are at, or above, 19 ft MSL.

Site drainage was investigated for the PMP. The one-hour PMP for the site is 8.6 inches. Applying this precipitation over the $2x10^6$ ft^2 site area and ignoring all precipitation losses, the average flow rate off the site is 398 cfs. Assuming no credit for the storm drainage system, this local intense precipitation would pond on the site until the road perimeter elevation of 20.5 feet MSL were exceeded. Once elevation 20.5 feet MSL is exceeded, water would flow over the roadway and proceed to flow offsite over the flood protection structures. The length of roadway around the perimeter of the plant to the south, east, and north is about 3,800 feet. For the analysis, 2,000 feet of roadway was credited as being available for overflow. The depth over the roadway crown can be determined from the weir equation:

$$Q = CLH^{3/2}$$

A conservative weir coefficient, C, of 2.8 is applicable. Solving for H, the height necessary to pass 398 cfs, yields 0.2 feet. Therefore, when added to the roadway crown elevation, the maximum water surface elevation for the PMP-induced PMF is 20.7 feet MSL. Complete blockage of the storm drainage system is considered unlikely and, therefore, a portion of the 398 cfs would be conveyed offsite through the drainage system which has a capacity of about 100 cfs (at the threshold of ponding). Therefore, the maximum water surface elevation of 20.7 feet MSL is conservative. All entrances and openings in safety-related facilities, except the Fuel Storage Building and the Primary Auxiliary Building, were therefore designed to be at least 1 foot above the plant grade of 20 ft MSL. The Fuel Storage Building, with the truck bay door sill at elevation 20 feet 6 inches, is provided with internal curbs 1 foot high (nominal). The entrance vestibule into the Equipment Vault section of the Primary Auxiliary Building is at elevation 20 feet 8 inches. The floor of this vestibule is sloped up 4 inches so that the high pint in the floor is 1 foot above the plant grade of 20 ft MSL.

Any potential flooding from a 50 percent PMP accumulation from the standard project storm would, obviously, result in considerably less ponding.

The following design considerations were used to ensure that roofs of safety-related buildings had the capability to dispose of local intense rainfall, up to and including the severity of the PMP, without the design basis loading of the roofs being exceeded:

a. All safety-related buildings (except the containment enclosure) are designed with relatively flat roofs having inclinations of approximately 1 percent.

b. Interior roof drains are provided of sufficient number and capacity to discharge the accumulated amount of rainfall without exceeding the roof loading corresponding to a snow load of 66 psf on the ground. The design of the roof drains is based on at least 3 inches per hour of rainfall. The design of the roof drains on buildings without parapets is based on at least 6½ inches per hour of rainfall.

c. Plant dewatering flow into the roof drain system is negligible (maximum of ≈ 10-12gpm) compared to the design flow.

d. In anticipation of interior roof drains possibly becoming clogged and causing impoundment of water, the parapets around the perimeters of safety-related building roofs have been designed low enough (in most cases, 9 inches higher than the roof crown), or with scuppers, to allow overflowing before the design roof loading is exceeded.

e. The openings in the roofs, such as ventilators, hatches, etc., are located above the probable maximum water level on the roofs.

To determine the most favorable approach to roof drain design, variables such as roof size, the size and number of roof drains and rate of precipitation were correlated and studied. Results indicated that the PMP can cause water accumulation on the roofs not exceeding two inches above the crown on those roofs with drains designed for 3 inches per hour rainfall.

Establishment of roof design values must account for load magnitude and probability in order to develop both a safe as well as functional design. Roof loadings are classified as "normal" or "unusual," as developed in Chapter 3.

The "normal" or basic snow loading on the roof will be based on a snow loading of 66 psf on the ground, as established from the snow load data given in Section 2.3.

An alternative "normal" load condition was evaluated as follows (References 4 and 5): assuming a maximum ice build-up on roofs of 4 inches (21 psf) concurrent with 8 inches (42 psf) of impounded rainfall up to the average 12-inch height of parapet of most buildings, (the typical parapet varies from 9 inches at the roof crown to 15 inches at the roof low point), the total roof loading becomes 63 psf, which is less than the 66 psf determined in Section 2.3.

For structures where roof features do not allow the height of the parapet to be low enough to limit the weight of any impounded water to less than 66 psf, the basic "normal" roof load, that portion of the roof which could exceed the basic loading was designed for the most severe combination of ice, snow and/or impounded water.

Unusual load cases occurring under the most extreme postulated conditions result in an "unusual" snow load of 126 psf, as established from the snow load data given in Section 2.3. All safety-related structures are designed to withstand this force in the "unusual" load combinations given in Table 3.8-16.

Based on the foregoing analysis, it is concluded that snow and ice accumulation on roofs, or impounded rainwater, will not exceed the design basis for safety-related buildings.

2.4.3 Probable Maximum Flood (PMF) on Streams and Rivers

Maximum flood levels in the site area will occur from the effects of northeasters or hurricanes. However, the effects of local stream and river flooding have been considered, and are developed and presented in the following sections.

2.4.3.1 Probable Maximum Precipitation (PMP)

The PMP represents the critical depth-duration-area rainfall relations which would result if conditions during an actual storm in the region were increased to represent the most critical meteorological conditions considered probable. The critical meteorological conditions are based on an analysis of air-mass properties (effective precipitable water, depth of inflow layer, temperatures, winds, etc.), synoptic situations prevailing during the recorded storms in the region, topographical features, season of occurrence, and location.

This analysis is available for the United States east of the 105th meridian in Reference 3. From Figure 2.4-4, the all-season PMP for a 200-square mile drainage basin with a 24-hour duration was 20.5 inches. This is translated into PMP values for other durations and drainage areas by use of Figure 2.4-5. The PMP for the Hampton River drainage basin (area = 47.4 square miles) and a 24-hour storm is 23.8 inches.

Storm configuration and orographic effects are irrelevant in this analysis. The coastal topography in the general Seabrook site area is relatively flat, thereby eliminating potential orographic effects. Snow melt is insignificant, and of no concern to the site.

2.4.3.2 Precipitation Losses

A precipitation loss of 0.05 inch per hour plus a 1.0-inch initial loss has been used for this area. These values were conservatively selected from tabulations of accepted values presented in References 6 and 7.

2.4.3.3 Run-off Model

To develop a PMF from rainfall, a run-off model is required. The model in common usage is the unit hydrograph derived from stream gaging records. Since there are no gaging stations or records available for any of the streams tributary to Hampton Harbor, it was necessary to synthetically consider the effects of stream flooding.

The drainage basin of Hampton Harbor and its major tributary streams is outlined in Figure 2.4-6, taken from a portion of the United States Geological Survey topographic map of "Exeter, New Hampshire - Massachusetts." The total drainage area covers about 47.4 square miles. The topographic details of this drainage basin are shown in Figure 2.4-7.

The run-off model for PMP as a lone causation of flooding at the Seabrook site was considered, and is insignificant in affecting design criteria for the safety-related plant facilities, because the flooding from the combined PMH SPS event is much greater than a PMP-induced flood (PMH-SPS flooding is treated in Subsection 2.4.5). This is illustrated by the following analysis which is presented here to infer the insignificance of a PMP flood as a sole cause of site flooding.

A run-off model for the entire basin PMP was made according to the method described in "Design of Small Dams" (U.S. Department of the Interior, 1973). The method is based on precipitation data from "Hydrometeorological Report No. 33" for PMP values. Soil losses were obtained from tabulations presented in References 6 and 7; the values are 0.05 inch per hour plus 1.0 inch initial loss.

The basin was divided in two parts; 12.2 square miles of submerged area where no precipitation losses were considered and where precipitation was directly transformed in cubic feet per second of inflow to the bay according to the time distribution of rainfall obtained from the analysis of the PMP data for the total (47.4 sq. mi.) area; and 35.2 square miles of natural drainage with a single concentration time computed by taking the maximum length of the individual basins (L) and the corresponding difference in elevation along their channels (H) between the most distant point of the watershed and the outlet to the submerged area (Hampton Harbor Bay). According to the formula of the California Department of Highways, the time of concentration for the natural drainage basin was computed:

$$T_c = (11.9\ L^3/H)^{0.385}$$

Where L is in miles and H is in feet

This is a generally accepted formula whose coefficients were obtained from regression analysis of a considerable amount of data for watersheds in the U.S. by the Soil Conservation Service.

The run-off model combines the PMP values for different storm durations into one single hydrograph made on the basis of individual triangular hydrographs for the time intervals of the resulting run-off distribution (rainfall minus losses). The individual triangular hydrographs were obtained utilizing the incremental run-off distribution and the time of concentration.

2.4.3.4 Probable Maximum Flood Flow

Run-off response of the watersheds was divided, as previously indicated, into two areas, the submerged area (12.2 sq. mi.) and the natural area (35.2 sq. mi.). The corresponding hydrographs are shown in Figure 2.4-8 and the combined hydrograph in Figure 2.4-9. The latter represents the inflow into Hampton Harbor from the start of the rainfall period.

2.4.3.5 Water Level Determinations

To compute the flood levels due to the PMF hydrograph, a routing procedure was applied to account for tidal variations inside the bay and outflow through the Hampton Harbor inlet which will be the obliged drainage for the flood waters. The method used for routing the open coast surge into the bay as described in Subsection 2.4.5 was modified to allow for the introduction of a run-off hydrograph:

The orifice flow into the bay is written as:

$$Q = \pm C\ A_I \sqrt{2g|h_\circ - h|}$$

where

Q = discharge in cubic feet per second (cfs)

C = discharge coefficient, having a value between 0.55 and 0.65 ($C = 0.65$ was used in the computations).

A_I = inlet flow area (feet^2) which is a function of the open coast tidal elevation above MLW given by the equation:

$A_I = 3000 + h_o (600 + 37.3\ h_o)$

h_o = open coast tidal elevation (feet, MLW)

g = gravitational constant (32.2 feet per second^2)

h = average surface elevation of bay above MLW (feet)

The (±) sign depends on whether h_o-h is positive or negative. This indicates that flow through the inlet reverses in direction when the bay level is higher than the open coast level. The inflow from the PMF can be added to the tidal flow to find the total flow into the bay:

$$Q = \pm C\ A_I \sqrt{2g|h_\circ - h|} + Q_h(t)$$

where $Q_h(t)$ is the flood flow in cfs.

The equation governing the bay elevation is a simple continuity equation of the form:

$$\frac{dh}{dt}A_b = \pm C\ A_I\sqrt{2g\left|h_\circ - h\right|} + Q_h(t)$$

or

$$\frac{dh}{dt} = \frac{1}{A_b}\left(\pm C\,A_I\sqrt{2g\left|h_\circ - h\right|}\ + Q_h\ (t)\right) = F(h,t)$$

where A_b, is the flooded bay area in feet^2, which is given as a function of the elevation in the bay above MLW by Figure 2.4-10.

This equation can be solved by means of a fourth order accuracy Runge-Kutta numerical integration method with h(o) = h(t=o) = 0, h_n = h(n, Δt), where Δt is a given time interval. The solution is:

$$h_{n+1} = h_n + \frac{1}{6}\left(b_1 + 2b_2 + 2b_3 + b_4\right)$$

where

$b_1 = \Delta t\ .\ F(h_n, t)$

$b_2 = \Delta t\ .\ F(h_n + ½b_1, t + ½\ \Delta\ t)$

$b_3 = \Delta t\ .\ F(h_n + ½b_2, t + ½\ \Delta\ t)$

$b_4 = \Delta t\ .\ F(h_n + b_3, t + \Delta\ t)$

The flood hydrograph and the astronomical tide should be added so that the highest level produced inside the bay is the maximum possible combination of flood and tide. Because the maximum tidal levels occur at roughly 6 hours from the beginning of the tidal rise and the peak flood discharge occurs roughly 6 hours after the initiation of rainfall, the critical condition will result from starting the flood hydrograph simultaneously with the flood (rising) tide (Figure 2.4-11). This will effectively result in the greatest possible volume of flood water inside the bay and therefore cause the highest flood levels at the plant site. Results of the computations for t = 0.2 hours appear in Figure 2.4-12. The projected increase in water level under critical conditions would be 2.4 feet to a maximum stillwater elevation of 13.0 feet MLW (8.9 feet MSL). This is well below the maximum levels projected for a probable maximum storm surge as indicated in Subsection 2.4.5.

2.4.3.6 <u>Coincident Wind Wave Activity</u>

The effects of wind wave activity are considered in Subsection 2.4.5, Probable Maximum Surge and Seiche Flooding. The stillwater surface increase of 2.4 feet associated with the PMF does not produce the maximum controlling water levels. Therefore, wind wave activity has not been considered coincident with the PMF.

2.4.4 Potential Dam Failures

There are two small artificial ponds in the vicinity of the site. Dodge Pond near Hampton Falls is about 1.2 miles northeast of the site, and a similarly-sized body of water is located on the Taylor River 2 miles north-northeast of the site. The small size of these ponds makes them of no concern to the safety-related plant facilities.

There are no other existing or planned reservoirs in the vicinity of the plant site or on any of the streams tributary to Hampton Harbor. These tributaries are generally classified as coastal streams by the Corps of Engineers, and it is reasonable to assume that no reservoirs will be built within the expected plant life. Also, there are no cooling water canals or cooling water reservoirs planned for the Seabrook plant.

2.4.4.1 Reservoir Description

See Subsection 2.4.4.

2.4.4.2 Dam Failure Permutations

See Subsection 2.4.4.

2.4.4.3 Unsteady Flow Analysis of Potential Dam Failures

See Subsection 2.4.4.

2.4.4.4 Water Level at Plant Site

See Subsection 2.4.4.

2.4.5 Probable Maximum Surge and Seiche Flooding

2.4.5.1 Probable Maximum Winds and Associated Meteorological Parameters

The probable maximum stillwater levels at Seabrook Station will result from simultaneous routing of the PMH open coast surge and the SPF discharge into Hampton Harbor. The PMH storm parameters and the SPS parameters are discussed in Subsection 2.4.5.2b.

2.4.5.2 Surge and Seiche Water Levels

a. <u>Surge and Seiche History</u>

Hampton Harbor will experience surge flooding caused by storm activity off-shore in the Atlantic Ocean. The extent of this flooding will depend on the severity and path of the storm.

The major cause of storm damage along the New Hampshire coast from tidal flooding and wave attack is the northeaster coastal storm, not hurricanes. A summary of peak experienced heights of past tides (astronomical tide combined with surge) from U.S. Coast and Geodetic Survey gages at Boston, Massachusetts, and Portland, Maine, are given in Table 2.4-1 (Reference 1).

A comprehensive summary of North Atlantic Ocean tropical cyclones lists all storms that have occurred during the years 1901 to 1963 (Reference 9). Also given is the incidence of tropical cyclones in various sections of the eastern and southern United States. For the coastline from Connecticut to Maine, a total of 57 tropical cyclones has affected the area, and of these, 11 have been of hurricane wind intensity (winds 74 mph or over). Furthermore, of the 11 hurricanes, 9 have tracked inland in the New England area while the remaining two had storm centers that remained offshore or moved inland in another area.

Coastal storms in New England, commonly referred to as northeasters, are more frequent in occurrence than hurricanes. These extra-tropical storms occur along the northern part of the Atlantic coast. In 1957-58 alone, some 18 storms were noted. Northeasters occur in the late fall, winter or early spring with nearly all of the more destructive northeasters occurring from November through April (Reference 10). Northeasters producing strong winds and high surges along the New England coast are well developed and mature extra-tropical lows. The gross features of the more common northeaster consist of a single center of low pressure associated with one cold and one warm front.

A chronological account of severe northeasters is listed in Reference 10, and dates back to 1717. A storm of February 23, 1723 produced the highest stillwater tide level ever recorded at Boston. This level of +15.4 feet MLW (+10.7 feet MSL) has not been repeated. At Hampton, New Hampshire, considerable berm overtopping and inland flooding was experienced as a result of this storm. A northeaster of April 14, 1851 produced a +15.1 feet MLW (+10.4 feet MSL) tide at Boston and a storm of December 26, 1909 produced a +15.0 feet MLW (+10.3 feet MSL) tide at Boston and a +12.7 feet MLW (+8.8 feet MSL) tide further upcoast at Newburyport, Massachusetts.

Daily observations of tide elevations over 47 years, from Coast and Geodetic Survey gages at Portland, Maine (1914-1959) and Boston, Massachusetts (1922-1960) have identified 51 northeast storms which have produced "high surge" (high surge is arbitrarily defined as 2.5 feet at Portland and 2.9 feet at Boston and is in excess of the astronomical tide). The highest surge during this period of record was 5.1 feet at Boston and 4.3 feet at Portland (Reference 10).

A tidal study comparing tidal differences along the New Hampshire coast concluded tides at Portsmouth Navy Yard to be typical of Hampton Harbor. The frequency of occurrence for tides 1, 2, 3 and 3.5 feet above mean high water (MHW) was determined from daily Portsmouth observations for the years 1927 to 1934, 1941, and 1943 to 1957 (Reference 11). Results are given in Table 2.4-2. The summary for all tides which exceeded MHW by at least 2 feet is given in Table 2.4-3 for the 17.7 year period of record.

b. Surge and Seiche Sources

The PMH surge at the entrance to Hampton Harbor was calculated by use of the bathystropic storm tide theory with the most severe hurricane parameters from References 12 and 13. The PMH as defined in Reference 12 is, "A hypothetical hurricane having that combination of characteristics which will make it the most severe that can possibly occur in the particular region involved. The hurricane should approach the point under study along a critical path and at optimum rate of movement." This analysis postulates the occurrence of such a storm which has a critical combination of parameters and criteria. The open coast surge at the entrance to Hampton Harbor was analytically routed through this inlet entrance into the inner bay using a hydraulic open channel flow-type analysis. The routing of the open coast storm surge was phased with the SPF discharge to produce critical flood levels within the harbor.

In general, when a hurricane crosses the Continental Shelf and moves onshore, severe damage can occur to shore structures as a result of flooding and wave action, unless these structures are properly designed. The maximum water level is a function of mean low water depth, the astronomical tide, and the rise in water level due to several factors. These factors include the hurricane atmospheric pressure reduction, initial surge, wind stress component perpendicular to the bottom contours (onshore wind component), and wind stress component parallel to the bottom contours which result from the Coriolis force. These forces deflect the flow to the right in the northern hemisphere (alongshore wind component). The current parallel to the bottom contours is also known as the bathystropic flow. Initial surge occurs long before winds have arrived, resulting in a slow general rise in sea level. This surge is determined by the speed of the hurricane in relation to the speed of free-gravity long waves for specific depths.

1. Astronomical Tide

All tidal data have been derived from U.S. Coast and Geodetic Survey publications (NOAA) (Reference 14). Astronomical tides are of the mixed-semidiurnal type with two highs and two lows occurring daily. The closest station for tidal data available is Hampton Harbor, while tidal harmonic constants are available for the reference station at Portland, Maine. Hampton Harbor tidal parameters are given in Table 2.4-4.

During the period from 1927 to 1970 there has been a total rise in sea level elevation, as related to the land, of about 0.3 feet in the Portsmouth, New Hampshire area. The trend from 1940 to 1970 has been a rise of approximately 0.0002 feet per year (Reference 15).

2. Probable Maximum Hurricane Parameters

Selection of the basic parameters to define the Probable Maximum Hurricane (PMH) was made from Reference 12, along with other selected criteria to establish the most critical combination. Those parameters and criteria are as follows:

(a) CPI (Po)

A central pressure index (CPI) value of 27.42 inches of mercury was selected from Table 1 of Reference 12. (Values listed in Table 1 for Latitude 42.83 degrees North, the approximate latitude at the mid-point of the traverse line, defined as the line along which surge calculations are performed as shown in Figure 2.4-13.)

(b) Asymptotic Pressure (Pn)

An asymptotic pressure value of 30.42 inches of mercury was selected from the PMH envelope curve from Figure 6 of Reference 12.

(c) Radius of Maximum Winds (R)

A mean radius of maximum winds (RM) value of 30 nautical miles and a large radius of maximum winds (RL) value of 56 nautical miles were selected from Table 1 of Reference 12.

(d) Forward Translation Speed (V_t)

A moderate translational speed (MT) value of 37 knots and a high translational speed (HT) value of 52 knots were selected from Table 1 of Reference 12.

(e) Maximum Wind Speed (U_{Max})

Maximum wind speeds were calculated using equations 2 and 3 of Reference 12 for combinations of radius of maximum winds and translational speeds. The following maximum wind speeds were used:

(1) 125.00 miles per hour for RM = 30 nautical miles and MT = 37 knots.

(2) 120.40 miles per hour for RL = 56 nautical miles and MT = 37 knots.

(3) 133.65 miles per hour for RM = 30 nautical miles and HT = 52 knots.

(4) 129.04 miles per hour for RL = 56 nautical miles and HT = 52 knots.

(f) PMH Path

The hurricane path for maximum open coast surge is a critical factor which in combination with PMH parameters determines the duration and magnitude of storm wind intensity over the critical fetch and the resulting peak hurricane surge elevation. The path resulting in peak hurricane surge will approach the site from approximately normal to the offshore bottom contours (parallel to the traverse line); therefore, the hurricane path selected was from 105 degrees or travelling toward the site in a direction N 75° W (true north), Figure 2.4-13. The center of the hurricanes was passed south of the plant site by a distance that resulted in the maximum winds passing directly over the site area, while the surge calculations were calculated along a traverse line intersecting the site, bearing N 75° W, Figure 2.4-13. Since this traverse is approximately normal to the offshore bottom contours, maximum surge heights are calculated.

Included in Reference 9 are plots of tropical cyclone paths during the years 1871 to 1963. Inspection of these plots shows no hurricane path from the southeast, as used in this analysis. In all cases, the hurricanes that reach New England follow a path that is roughly parallel to the New England coastline. This is due to the general Northern Hemisphere air flow patterns or steering winds. The general movement of air over most of the tropics is from the east; while in higher latitudes it is usually from the west. Consequently, a tropical cyclone moves initially westward, drifting slightly northward under the influence of Coriolis forces. As the storm moves into the higher temperature latitudes, the prevailing westerly winds dominate and the storm changes direction or recurves and moves eastward. The point of recurvature is defined as the geographic location of this change in storm direction from westward to eastward. A summary in Reference 9 gives the locations of points of recurvature for all tropical cyclones from 1901 to 1963. In all cases but one, the point of recurvature was below Latitude N 40°. The one exception was a tropical cyclone that was formed at Longitude W 45° and tracked to Nova Scotia and recurved on out to sea.

The general westerly steering winds make it possible for a hurricane to come overland and track directly off shore over the Seabrook site. Reference 9 shows three instances where this did occur in 1872, 1876, and 1878.

It is concluded that the PMH path used to calculate maximum surge flooding at Seabrook Station has an extremely low probability of occurrence. Conversely, a hurricane moving overland directly over the site and out to sea to produce the lowest tide conditions, has a reasonable frequency of occurrence for design purposes.

(g) Hurricane Rainfall from SPS

The amount of rainfall from any given storm is a function of several factors: surrounding air masses, path, terrain, as it affects orographic lifting with attendant torrential and widespread intense rainfall, and other more complex phenomena within the storm itself. Examination of rainfall records associated with the passage of intense hurricanes over or near the northeastern seaboard indicates that rainfall distribution in those storms has been relatively light along the coast, with heavier amounts noted inland due to rising topography. Coincident with the PMH, precipitation and flooding resulting from the SPS were considered within Hampton Harbor. The precipitation was considered to be 50 percent of that for the PMP in accordance with standard practice. The method presented in Subsection 2.4.3 was used for computation of the SPF discharge hydrograph.

(h) Astronomical Tide

The predicted high spring astronomical tide cycle for the entrance to Hampton Harbor is shown in Figure 2.4-11. The maximum value of this tide is +10.6 feet MLW, which was used in the computation of the open coast surge and was assumed to occur coincidental with the arrival of the peak open coast surge.

A detailed examination of 20 years of tidal predictions for the Hampton Harbor area (Reference 34) indicates that a high spring tide of 10.6 feet MLW is a liberal estimate of the required 10 percent exceedence high tide. A summary of the highest predicted tides for Hampton Harbor over a 25-year period is presented in Table 2.4-5.

(i) Initial Surge

The AEC requested that an initial surge of 0.9 feet be used for Hampton Harbor as attributed to tidal anomaly evaluated on the basis of variations between observed and predicted tide. At this site such variations can only be attributed to atmospheric factors, i.e., atmospheric pressure patterns in the adjacent area and wind effects which could produce a change in sea elevation. Since predictions involve sea level effects of the most extreme evaluation of all atmospheric phenomena occurring coincidentally with the high spring tide, there is no apparent basis for adding a value for "initial rise" to the astronomical tide. However, since there is insufficient data to conclusively document whether or not an initial surge value attributed to nonpressure and non-wind effects should be included, a value of 0.9 feet initial surge was used in this study as requested.

(j) Bottom Friction Coefficient

The value of the bottom friction coefficient used in the analysis was 0.003. This value is identical with that specified by the Coastal Engineering Research Center (by verbal communication) for surge computations along the Atlantic Coast.

(k) Wind Stress Coefficient

The value of the wind stress coefficient was selected to provide a conservative estimate of PMH surge using the verified Dames & Moore computer surge model (Reference 17), which indicated that the proper value of the wind stress coefficient would be calculated from the following:

Constant part of coefficient = 1.0×10^{-6}

Constant multiplier part of coefficient = 1.4×10^{-6}

The corresponding values used in the computer analysis, Runs 1 through 5 (Table 2.4-6), for the Seabrook site were 1.1×10^{-6} for the constant part and 1.6×10^{-6} for the constant multiplier part. The results of computer analysis, Run 6 (Table 2.4-6), were based on wind stress coefficients of 1.1×10^{-6} (constant part) and 2.5×10^{-6} (constant multiplier part), which are extremely high values. As the values of these coefficients used for Run 6 of the Seabrook open coast surge analysis are greater than necessary, the magnitude of the wind stress coefficient is greater and, consequently, the computed surge is greater than could possibly occur.

3. Northeaster Parameters

In general, a northeaster develops in the Gulf of Mexico and South Atlantic regions, moves north to northeast, and reaches maximum intensity in the New England area. Their forward speeds range from 6 to 43 knots averaging 22 knots. Fetch lengths range from 300 to 1400 nautical miles while observed wind speeds of 55 to 65 knots are not unusual in a mature northeaster, but winds speeds of 70 knots are rare. Because of their forward speed and wind fetch characteristics, the resulting northeast storm surges generally occur within a 12-hour to 3-day interval in the New England area (Reference 10). The storms possessing the lowest central pressure, the strongest maximum pressure gradient, and the longest fetch length have the highest surge-producing potential.

A comparison of hurricanes and northeasters in northern New England shows that although hurricanes have higher wind speeds, they move with a faster forward speed, have a smaller area of strong winds, and shorter fetch lengths than northeasters. Therefore, the surge-producing effects of hurricanes last a shorter length of time with the short period of past records showing that northeasters have produced higher surges north of Cape Cod.

4. Open Coast Surge

Open coast surge elevations were computed for the probable maximum hurricane (PMH) and for the probable maximum northeaster (PMN) to verify which event should be used for the design open coast surge flooding. Hurricane surge elevations at the open coast were computed using a bathystropic storm tide theory essentially the same as that described in Reference 16, the only difference being in the treatment of the bottom friction effect. A computer program written by Dames & Moore was used in the calculation procedure (Reference 18). Input to the program consisted of the hurricane parameters and criteria previously discussed in this report, the wind field of the hurricane determined using procedures in Reference 12, and the offshore bottom profile to a depth of 810 feet.

The offshore bottom profile along the traverse line used in the computations is shown in Figure 2.4-14. The profile extends from the shoreline west of the Hampton Harbor entrance to deep water. The open coast surge is taken at the entrance to Hampton Harbor.

Four combinations of radius of maximum winds and forward translational velocity of the PMH were considered in calculating the open coast surge. The maximum winds of the analyzed storms were assumed to occur on a radial line located 115 degrees clockwise from the translational velocity vector of the hurricane (see Reference 12). The paths of the hurricanes were taken parallel to the traverse line and displaced to the south by a distance to allow the maximum hurricane winds to travel along the traverse line. Results of the maximum open coast surge for the four PMH storms (six computer runs) considered are shown in Table 2.4-6. The surge elevations shown include the maximum spring tide of +10.6 feet MLW.

The results shown in Table 2.4-6 indicate that the maximum open coast surge occurs for the large radius, high speed storm, which is not significantly different from the results of the other three PMH storms. Therefore, it is not immediately obvious which PMH will yield the maximum water level within Hampton Harbor.

Runs 1 through 4 are based on no initial surge while Runs 5 and 6 are more conservative repetitions of Run 4, including an initial surge of 0.9 feet. The maximum open coast surge level for Run 6 is +18.6 feet MLW (+14.5 feet MSL).

A probable maximum northeaster (PMN) surge elevation was considered for the Seabrook site since past recorded tide elevations along the New England coast have higher recorded elevations for northeasters than for hurricanes.

In Figures 26 and 27 of Reference 10 are curves representing the estimated probability of extreme high tide height at Boston, Massachusetts and at Portland, Maine based on northeasterly storm data for 1922 to 1960 at Boston and for 1914 to 1959 at Portland. These figures are reproduced as Figure 2.4-15. Tide elevations were extrapolated from these figures for a return period of 10,000 years, since this was the return interval assumed for the PMH occurrence. The Boston tide elevation is MLW +16.7 (+12.0 feet MSL), while the Portland tide elevation is +15.8 feet MLW (+11.3 feet MSL). These tide elevations are based on maximum annual observed tides which include the combined effects of storm surge and astronomical tides. Past records indicate that tide elevations at Hampton Harbor are sometimes equivalent to tides at Boston, but are usually less. During the occurrence of the PMN, the tide at Hampton Harbor should be approximately one foot less than that at Boston since the high astronomical tide at Boston is +11.7 feet MLW, compared to +10.6 feet MLW at Hampton Harbor. Therefore, the PMN surge elevation at Hampton Harbor would be less than the PMH surge elevation of +18.6 feet MLW (+14.5 feet MSL) at Hampton Harbor.

Although a northeaster surge is longer in duration than that of a hurricane, the peak storm surge duration conditions appear to be similar (Figure 24 of Reference 10). It is concluded, that using the PMH storm track of N75°W, even though a hurricane is very unlikely to be headed in this direction off the New England coast, it will produce a higher storm surge resulting in higher tide elevations within Hampton Harbor than the PMN storm surge. Therefore, the PMH open coast storm surge was used in conjunction with the SPF discharge to perform the surge analysis within Hampton Harbor.

5. Hampton Harbor Surge

Probable maximum stillwater levels in the harbor are produced by the combined occurrence of the PMH surge and the SPF discharge. The principle component of the combined event, the PMH, will be considered first. The method of routing the open coast surge into the bay, possible erosion of Hampton and Seabrook beaches during the PMH, overtopping of the flooded shore berm and the method of combining the various components of the harbor surge will be discussed.

The coastline of New Hampshire experiences loss or deterioration of its recreational beaches through gradual erosion and storm wave attacks, but due to increased shore-front development and use, federal and state protection measures have been carried out. Hampton Beach has remained relatively stable since the stabilization of Hampton Harbor inlet by jetties in 1935, due to beach fills placed since jetty construction. Erosion does progress from the north end of Hampton Beach in a southerly direction, with the beach fill material migrating south temporarily nourishing downcoast areas. Beach fill protection is based on a natural berm elevation of 15 feet above MLW that exists at Hampton Beach, and a minimum width of 150 feet above MHW. Gradual loss of the beach exposes protective structures to more severe wave attack and, in the past, wave overtopping and flooding has occurred on the shoreline roads and improvements. This has caused erosion of bluffs in some places.

The present protection measures in the vicinity of Hampton Harbor (see Figure 2.4-7) are for a design tide of +12 feet MLW (+7.9 feet MSL), and are not intended to provide complete protection in case of hurricanes or exceptional storms of infrequent occurrence. However, elevations along the Hampton Beach shore-front development range from +16.5 feet MLW (+12.4 feet MSL) to +20 feet MLW (+15.9 feet MSL), while elevations along the Seabrook Beach shore-front development from the jetty to about 4500 feet south, range from +17 feet MLW to +20 or more feet MLW. For a reach of about 3500 feet south of this stretch to State Highway 286, shore front development ranges from about +13 feet MLW (+8.9 feet MSL) to +17 feet MLW (+12.9 feet MSL).

The shore-front development at Hampton Beach is protected by riprap revetment, concrete-encased steel sheet pile bulkhead, and gravity seawalls. The steel sheet pile was driven to a top elevation of +19 feet MLW. Landward of this shore protection is a dense concentration of businesses and cottages extending 1000 feet and more across the berm width. To a somewhat lesser extent, the entire section of Seabrook Beach also has shorefront cottages extending from roughly 100 to 200 feet from the high water line to 1000 to 2000 feet landward across the barrier berm. Riprap and concrete seawalls and other protective structures have been built to front most of the shorefront cottages along the northern 2000 feet of Seabrook Beach (Reference 11).

Wave action in advance of actual storm passage attacks coastal beaches in varying degrees depending on both offshore and onshore beach slopes, the proximity (or existence) of dunes, type of underlying material, wave characteristics and other factors. The horizontal extent of beach erosion can vary; in major hurricanes some 10 to 20 feet horizontal loss of beach has been observed. In long duration northeast storms, the repeated occurrence of sequential abnormally high tides and wave action has caused horizontal erosion of beachfront as much as 50 to 100 feet.

Maximum wave heights generated by the PMH and breaking on the shore front were calculated using the methods outlined by Weggel (Reference 20). Breaking waves no higher than 16 feet are anticipated during the brief period of peak surge. Furthermore, these maximum waves will break several hundred feet from the berm crest. Within this distance, successively smaller breaking waves can be expected.

It is concluded that some erosion of beach material will occur along with damage to shorefront structures. However, the duration of the peak PMH storm surge is such that insufficient time is available for breaching the wide berms, and even if such a breach did occur, passage of the storm would have occurred thereby causing the surge to recede, eliminating the possibility of storm flooding. Berm overflow can be expected near State Highway 286 due to lower berm elevations in that area, but calculations to determine the extent of resultant increased flooding indicated its insignificance.

The main means of flow for the PMH storm surge into the harbor is through the inlet, with overtopping of the developed shoreline berms being insignificant in adding to the bay surge elevation obtained from the inlet inflow. From the surge results previously presented, it seems clear that either the large-radius, moderate-speed storm (exemplified by Run 2) or the large-radius high-speed storm (exemplified by Run 4) will cause maximum harbor basin surge. The surge elevation of Run 4 is slightly larger than for Run 2, but since the surge of Run 2 is due to a slower moving PMH, it might result in a higher harbor surge elevation.

The open coast storm surge hydrographs from Runs 2 and 4 are shown in Figure 2.4-16. These figures were constructed from the time-dependent open coast surge results in the following manner: a constant amount equal to the maximum high spring astronomical tide was added to all water depths in the analysis; therefore, the open coast surge, in excess of tide, was obtained by subtracting the constant value of astronomical tide from the computed surge outputs. The time-dependent tidal cycle shown in Figure 2.4-11 was then added to the above results, with phasing so that the maximum tide occurs at the same instant as the peak open coast surge.

The open coast surge hydrographs shown for Runs 2 and 4 are a function of time relative to the occurrence of mean low water at the harbor entrance. Since the two hydrographs are essentially identical, for the design case it is reasonable to use the results of the large radius, high-speed storm of Run 6 which is similar to Run 4 and includes the sea level anomaly and the initial surge of 0.9 feet.

The surge analysis within Hampton Harbor was performed by assuming the surface area of the flooded bay, A_b, is a function of average water level rise above MLW within the bay, and that the average water level rise within the harbor is a function of the total flow, Q, into the harbor, where:

$$Q = Q_I + Q_o + Q_{SPF} = A_b \frac{dh}{dt}$$

Q_I = open coast PMH surge discharge through the harbor inlet

Q_o = open coast PMH surge flow over the flooded portions of the south berm

Q_{SPF} = SPF discharge

h = average surface elevation of bay above MLW, feet

The method presented in Subsection 2.4.3 was used for computation of the SPF hydrograph. The precipitation was considered to be 50 percent of that for the PMP.

A flood hydrograph developed for the 47.4 square mile drainage area (divided between 35.2 square miles of natural drainage and 12.2 square miles of submerged area) appears in Figure 2.4-17. The precipitation which falls on the submerged area is assumed to be immediately incorporated into the bay waters, while that which falls on the dry area will run-off with an average time of concentration of two hours. The volume of flow into the bay was conservatively allowed to increase over an additional two-hour period to its base level.

Further assuming that the flow area of the inlet, A_I, is given in terms of the open coast surge height above MLW, h_o, and a constant representing the MLW flow area, then the orifice flow into the bay is written as:

$$Q_I = \pm CA_I \sqrt{2g\left|h_o - h\right|}$$

The terms have been previously defined in Subsection 2.4.3.

Boundaries used to describe the flooded bay area, A_b, were Hampton and Seabrook Beaches on the east, State Highway 286 on the south, and contours shown on U.S.G.S. Quadrangle sheets for the west and north sides. Actually, about 3500 feet of the south end of Seabrook Beach and 3500 feet of the east end of Highway 286 along the described boundary would be overtopped during the PMH occurrence, since this stretch has elevations of +13 feet MLW (+8.9 feet MSL) to +17 feet MLW (+12.9 feet MSL). This developed shoreline area with a maximum overtopping condition of 5.6 feet was insignificant in raising the bay surge elevation calculated from inflow through the entrance. The relationship between the water elevation above MLW and the flooded bay area is shown in Figure 2.4-10.

The inlet area, A_I, was comprised of a constant value plus a change due to change in elevation of the surge, and was represented by the following relationship:

$$A_I = 3000 + h_o\,(600 + 37.3\,h_o),\ \text{ft}^2$$

The time-dependent variation of the open coast surge, h_o, shown in Figure 2.4-18 for Run 6, including the astronomical tide and sea level anomaly, completes the necessary input required for solution of the surge response within Hampton Harbor. The equation governing the bay surge is of the form:

$$\frac{dh}{dt}\ F(ht)$$

$$F(h,t) = \left[\pm CA_I \sqrt{2g\left|h_o - h\right|} + Q_o + Q_{SPF}\right]\frac{1}{A_b}$$

The above equation was solved by means of a fourth order accuracy Runge Kutta numerical integration method with $h_{t=o} = 0$. Let $h_n = h(n\ \Delta t)$, where Δt is a given time increment, and it may be shown that

$$h_{n+1} = h_n + \frac{1}{6}(b_1 + 2b_2 + 2b_3 + b_4)$$

where,

$$b_1 = \Delta t\ F(h_n, t)$$

$$b_2 = \Delta t\ F(t + \tfrac{1}{2}\Delta t, h_n + \tfrac{1}{2} b_1)$$

$$b_3 = \Delta t\ F(t + \tfrac{1}{2}\Delta t, h_n + \tfrac{1}{2} b_2)$$

$$b_4 = \Delta t\ F(t + \Delta t, h_n + b_3).$$

Table 2.4-7, Table 2.4-8 and Table 2.4-9 show the results of routing the open coast hydrograph of Run 4 into Hampton Harbor without the addition of the SPF, for values of C = 0.55, 0.60, and 0.65. The peak bay surge occurs for the largest discharge coefficient (C = 0.65); however, the surge results are not significantly different for the discharge coefficient, C = 0.60.

The PMH open coast surge from Run 6 (Figure 2.4-18) was routed into Hampton Harbor using a discharge coefficient, C = 0.65. The critical phasing of the PMH open coast surge with the SPF event was determined by trial and error method. The results are presented in Figure 2.4-18, which show the surge hydrograph for Hampton Harbor due to the combined PMH surge and SPF discharge event. The maximum still water level within the harbor is +19.0 feet MLW (+14.9 feet MSL).

As the PMH winds approach the bay, water level changes will occur due to wind drag on the water surface. The site is roughly 1.3 nautical miles opposite the harbor inlet on the western boundary of Hampton Harbor. The wind setup will occur due to the component of wind which lies approximately along the traverse line. Reference to Run 6 computer results shows that the maximum wind component value is about 125.5 mph and occurs some 20 nautical miles aft of the storm center; therefore, the peak winds along the traverse lag the storm passage by about 0.4 hours. Since the peak open coast surge corresponds roughly with the coastal arrival of the storm center, and the peak bay surge lags the peak open coast surge by about 0.4 hours, the maximum wind setup will coincide with the maximum bay surge.

The wind setup, ΔS, is calculated from the relationship (Reference 19):

$$\Delta S = d_t \sqrt{\frac{1.0122 U U_x W}{d_t^2} + 1} - 1$$

where:

ΔS	=	the wind setup in feet at the western boundary of Hampton Harbor.
U	=	the magnitude of the wind speed coincident with the given water depth and reduced for frictional coastal effects.
U_x	=	the wind component along the traverse line at the bay's center, mph.
d_t	=	the bay's total average water depth, feet.
W	=	the distance across the bay (1.3 nautical miles at the site location and for bay surge in excess of about +15 feet MLW).

The value for d_t was obtained by adding or subtracting the average topographic elevation along the fetch to the results of the combined PMH-SPF routing. The total stillwater elevation hydrograph at the plant site, including storm surge, wind setup and astronomical tide is shown in Figure 2.4-18. The maximum total stillwater elevation is +19.7 feet MLW (+15.6 feet MSL) at the Seabrook site.

The original design stillwater level analysis routed a maximum PMH open coast stillwater level of +18.6 feet MLW (includes an astronomical tide of +10.6 feet MLW) concurrent with a peak SPF discharge of 72,800 cubic feet per second into Hampton Harbor to the Seabrook site. The routing model used a computer incremental time step of 0.25 hours. The maximum stillwater level calculated inside Hampton Harbor was +19.0 feet MLW, and when including the additional effect of hurricane winds blowing across Hampton Harbor, the maximum stillwater level calculated for the Seabrook site was +19.7 feet MLW. When a computer incremental time step of 0.05 hours was used, the maximum stillwater levels calculated within Hampton Harbor and at the Seabrook site were +18.7 and +19.4 feet MLW, respectively. This analysis included a maximum open cost stillwater level of +18.7 feet MLW. Therefore, the stillwater levels used in the flooding analysis of Seabrook are conservative.

The PMF discharge was routed into Hampton Harbor concurrent with a maximum open coast stillwater level of +18.7 feet MLW. The PMF discharge hydrograph used in this analysis was that shown in Figure 2.4-9 with a peak discharge of 136,500 cfs. When using a computer incremental time step of 0.05 hours, a maximum stillwater level within Hampton Harbor of +18.75 feet MLW was calculated. When the PMF discharge hydrograph was replaced in the analysis with a discharge hydrograph having a peak discharge of 29,250 cfs (less than that for the SPF), a maximum stillwater level within Hampton Harbor of +18.70 feet MLW was calculated.

These values of +18.70 feet and +18.75 feet MLW are less than the maximum stillwater level within Hampton Harbor of +19.0 feet MLW used in the original flood analysis, and result in less than the +19.7 feet MLW that was used at the Seabrook site in the original analysis.

Sensitivity tests have shown that discharge rates versus duration in the SPF and PMF range are insignificant in raising Hampton Harbor water levels during the PMH occurrence due to the area-height relationship of the harbor and the harbor's rapid response to the PMH open coast water levels flowing through the harbor entrance and over the beach dune system at water levels above about 12 feet MLW.

2.4.5.3 Wave Action

The wave characteristics within Hampton Harbor during the PMH event will vary continuously, in response to the changing water levels and wind conditions over the bay. The wave heights and wave periods generated are dependent upon the local wind speed, wind duration, water depth and effective fetch within the bay during most of the PMH event.

Observation of the Run 6 computer results indicates a total effective distance of the wind field along a line coincident with the traverse line of about 700 nautical miles. Since the PMH moves at a speed of about 50 knots, the total wind field passes a point on the traverse line in about 14 hours. Using Figures 1-8 of Reference 19 results in a maximum deepwater wave height of about 90 feet for the hurricane's maximum winds with a 14-hour duration.

As deepwater waves of this magnitude travel toward shore, they undergo the effects of refraction and shoaling, reducing the wave heights. Results of the wave generation and transformation calculations indicate that breaking waves exist at the inlet to Hampton Harbor during the times of maximum open coast surge, even when considering the time lag of maximum wave heights from the peak open coast surge arrival due to wave generation and travel time.

These deepwater waves will break as they approach the Hampton Harbor Inlet due to:

a. A large outcrop of rocks, exposed during MLW, about 2500 feet offshore of the harbor entrance

b. Water depths of only -4 feet MLW offshore of the harbor entrance

c. Interference of the north jetty with a crest elevation of +12 feet MLW.

Hurricane waves generated in deep water and approaching Hampton Harbor will be diffracted through the harbor entrance, and therefore, can approach the site with maximum intensity only through a narrow window centered along direction S75°E. The consideration of this window assumes that the north harbor entrance jetty is largely damaged and ignores any effects from the highway span and bridge piers. Waves approaching along the critical directions defined by this window, while suffering some decrease in height from diffraction, can also be regenerated within the harbor as they approach the site. Deepwater waves approaching the site from other directions will effectively be screened out.

The time history of wind speed and direction over Hampton Harbor obtained from the computer printout of the surge computation is given in Table 2.4-10. The wind speeds have been reduced from their full over-water values by a factor of 0.89 to account for the friction effect at the shoreline as given in Reference 12. The effective fetch lengths and water depths for Hampton Harbor corresponding to their respective wind directions are also given in Table 2.4-10. The water depths given are based on the hydrograph of stillwater levels in the bay (see Figure 2.4-18) for the combined PMH-SPF event and the average topography along the fetch direction.

The maximum and significant wave heights and periods generated in Hampton Harbor were computed by the method given in Section 3.6 of Reference 36, and are presented in Table 2.4-10. Maximum wave heights presented are the 99 percentile value for nonbreaking waves and the maximum supportable wave height for the given water depth and bottom slope for breaking waves.

The maximum and significant heights and periods associated with the waves regenerated in Hampton Harbor were estimated from the height of the maximum supported deepwater wave at the harbor entrance and its height following diffraction into the bay. The diffracted wave was regenerated over the effective fetch from the entrance to the site to determine its period. The results are summarized in Table 2.4-11. In the wave regeneration analysis no credit was taken for the limiting effects of the Hampton Harbor entrance jetties and bridge piers on the incident wave heights and periods. The bridge clearance of 7.3 feet, at maximum stillwater level, would chop off the portion of the wave crest higher than +26.3 feet MLW, while the bridge piers would prevent passage of the total incident wave energy by reflection.

The portions of the plant grade subject to critical wave attack are shown in Figure 2.4-21. The eight sections shown in the figure have been selected on the bases of the surrounding topography and the location of the safety-related plant structures. The topography along these sections is presented in Figure 2.4-22. The usual wave hindcasting assumption, that the wind generated waves travel in the downwind direction, was made to determine the principal direction of wave approach to the site. However, when maximum supported waves at a particular section could occur by allowing waves to be generated with their full height while traveling at angles up to 45 degrees with the wind direction, this was allowed. Furthermore, when wave overtopping of the plant grade at a particular section could occur by permitting waves to approach the section obliquely at angles up to 60 degrees with the normal to the seawall or revetment this was permitted. The maximum period associated with these waves is the larger of the generated or regenerated wave period. In this manner, a complete time history of wave attack on the eight sections was developed. The intention was to insure that estimates of the wave runup and overtopping elevations would be higher than expected.

The results of the analysis are shown in Figure 2.4-19. Curve A gives the wind direction and the principal direction of wave generation over the bay. Curve B gives the maximum wave period. Curve C gives the stillwater level at the site. Curves D1 through D8 give the maximum breaking wave height and the duration of wave attack at each of the respective sections. The solid portions of the curve indicate that overtopping conditions exist, and the dashed portions that non-overtopping conditions exist. The maximum breaking wave heights are based upon the maximum toe depth, rather than the average toe depth at the wave protection structure, with an additional allowance of one foot for possible scour. The maximum wave period was used in the calculation, since this results in the critical breaking wave height, runup and overtopping conditions.

Runup calculations based on the design wave conditions and coincident stillwater levels indicate that wave overtopping of the plant grade, primarily on the southeast side of the site, is possible during the one to two hours of peak water levels and wave action. On the northeast side, wave exposure is limited, first due to the low water levels during the early stages of the combined PMH-SPF event, and then because this side is in the lee during the period of maximum wave attack. The result is that the protective structures on the northeast flank are not overtopped and will permit water to drain from the plant grade by overflow.

Estimates of the rate of water overtopping the southeast flank were based on the spatial and temporal variation of wave-conditions along the protective revetment and seawall, using the curves provided in Subsection 3.27.2 of Reference 19. The effect of concurrent local intense SPS precipitation was included. The major assumptions required for the analysis were:

a. There is a three-hour lag between the peak SPS precipitation rate and the occurrence of maximum stillwater level at the site.

b. The overtopped revetment/seawall can be divided into a number of discrete sections.

c. The quantity of water delivered to the plant grade from the combined effect of wave overtopping and SPS precipitation that exceeds the available capacity of the yard drainage system, will drain by flowing over the site perimeter flood protection structures (seawall, revetments and retaining wall).

In addition, assumptions regarding the plant grade area and perimeter were made to maximize the inflow quantities and minimize the drainage quantities to produce a conservatively high estimate of the depth of water ponding on the plant grade.

The inflows to the site are shown on Figure 2.4-23.

Since the design stillwater level when using the PMF/PMH event, as described in Subsection 2.4.5.2b.5 does not exceed +19.7 feet MLW (+15.6 feet MSL), wave overtopping rates provided above are still the upper design values. These values, however, have been increased by 70 percent to account for wind effects.

When using the Regulatory Guide 1.59 maximum open coast stillwater level of +17.8 feet MLW, the maximum stillwater level at the Seabrook site is +18.9 feet MLW (+14.8 feet MSL). With this lower design stillwater level there is less wave overtopping.

The flooding analysis conservatively calculated the quantity of wave overtopping waters used in assessing maximum water levels on the site. The analysis maximized the potential overtopping including the allowance for bottom scour, the use of maximum supportable waves, ignoring any reduction effects due to wave approach angles, and increasing wave overtopping quantities a full 70 percent for potential wind effects.

Almost all of the wave overtopping occurs along the vertical seawall section. This overtopping is a direct result of waves breaking against the vertical seawall structure and, therefore, the wave runup and overtopping is heavily concentrated immediately behind and adjacent to the seawall. The calculated wave overtopping quantities are based on the continuous occurrence of maximum supportable waves. In actuality, there will be periods of lower wave activity resulting in reduced rates of wave overtopping and intervals of time with no wave overtopping. During peak overtopping conditions, using the maximum wave height and a 5-second period, the seawall is overtopped about 30 percent of the time.

The seawall and adjacent site flood protection structures are elevated to 20.0 feet MSL, while the crown of the site's perimeter roadway is elevated to 20.5 feet MSL. The crown of this road is located approximately 100 feet from the face of the seawall. The stillwater level site ponding within the perimeter roadway due to the PMP is assumed to be not exceeding 21 feet MSL, again taking no credit for the storm drainage system. A portion of the wave overtopping waters will spread to either side of the seawall and be directed back over the adjacent non-overtopped rock riprap shore barrier due to the 0.5 foot gradient between the perimeter roadway crown and the top of the flood protection barrier, and be directed back across the vertical seawall as wave overtopping is intermittent along the seawall.

Figure 2.4-20 shows the five major potential flow pathways for wave overtopping waters to exit the site. Flow pathways 1, 2 and 3 are assumed bounded to the north by the site perimeter roadway. Flow pathways 4 and 5 are bounded at both their upstream and downstream limits by the site perimeter roadway system with a crown elevation of 20.5 feet MSL.

An analysis was performed to estimate maximum water levels throughout the site, and the relative percentages of flow conveyed by the five previously identified flow pathways. Wave overtopping water was allowed to spread laterally into flow pathways 1 and 3 from 2 and north through 4 and 5 if the necessary hydraulic head was developed. Water was also allowed to return off-site over the vertical seawall, Pathway 2, during times when the seawall was not being overtopped.

The discharge capacity of each flow path was determined using standard hydraulic methods. The discharges in flow pathways 1, 3, 4 and 5 were determined using surface flow and weir flow over the flood protective structures. The discharge in flow pathway 2, back over the seawall, was determined using a combination of weir and orifice flow. To make these calculations, it was assumed that the area between the seawall and the roadway had a uniform head. This is a conservative approach in that no credit is taken for enhanced lateral flow away from the seawall area due to the hydraulic gradients which surely exist during overtopping.

Table 2.4-13 shows the discharge capacity of each flow path for various heads behind the seawall. As can be seen, a water level of about 21.0 feet MSL behind the seawall provides sufficient hydraulic gradient to produce flows which are approximately equivalent to the wave overtopping rate shown in Figure 2.4-23, taking no credit for site drainage. The water elevation within the site perimeter roadway calculated by this method is just less than 21.0 feet MSL, again taking no credit for the underground storm drainage system. In actuality, there are over 50 catch basins within the roadway area. It is extremely doubtful that all of these catch basins would be blocked and therefore it is concluded that the water level within the perimeter roadway is less than 21.0 feet MSL.

During the time of maximum wave overtopping, the wind is blowing in a northwesterly direction (N62°W). This wind had already been used to increase overtopping water quantities by 70 percent. The site buildings will cause the wind flow pattern to flow up and over, as well as, around the buildings creating a stagnant area and/or areas of reversal in flow patterns with diminished velocities near ground level and along the buildings. The wind effects on overtopping water should have a minimal net result on any large-scale transport of water especially since the water is so shallow.

Wind effects should be more in the form of wind-blown spray from site ponding water which would negligibly affect the site flooding levels. The percent of overtopping waters flowing through the site to the north and east sides of the plant, flow pathways 4 and 5, have been shown to be minimal in Table 2.4-13.

The maximum wave height which would be supported in the water ponding on the plant grade is 0.6 feet and would have the form of a moving hydraulic jump, or bore. These waves could be impulsively generated by the overtopping waves. However, actual transmission of incident waves, or wind wave generation on the grade is considered negligible when compared to the impulsively generated waves. As this bore-like wave propagates across the plant grade it will lose energy due to friction and percolation. Assuming that the wave arrives at the safety-related structure undiminished in height, an estimate of the maximum wave runup is produced which is higher than would actually occur. The resulting maximum wave runup elevation on a smooth vertical wall is 21.8 feet MSL. The location of walls of safety-related structures subject to the maximum wave runup are shown on Figure 2.4-21.

The maximum wave runup elevation on the vertical retaining wall at the north side of the site (see Section 1, Figure 2.4-21) is 19.6 feet MSL. The maximum runup was determined by assuming that a 2-foot wave due to wave agitation could reach the retaining wall at the time of peak stillwater level, even though the retaining wall is in the lee for wave generation. Similarly, the maximum runup elevation on the 1 on 1.9 to 1 on 2.5 slope revetment on the northeast side is 18.0 feet MSL.

It is concluded that a maximum sustainable site ponding level at Seabrook Station for the combined PMF/PMH event is less than 21.0 feet MSL.

2.4.5.4 Resonance

Resonance in harbors becomes a problem when the natural period of oscillations of the harbor is equal to the period of the incoming waves. The natural period of Hampton Harbor during peak surge conditions is approximately 30 minutes. This figure is based on an approximated basin 8000 feet long by 21,000 feet wide, by 8.7 feet deep, and the equation for the natural period of an open-ended basin. The significant period of the incident waves is considerably less than 30 minutes; therefore, resonance will not be a problem at the site.

2.4.5.5 Protective Structures

To ensure the protection of safety-related facilities during peak PMH surge activity, four designs of protective structures have been provided along the portions of the site perimeter which will be exposed to wave action (see Figure 2.4-21). To the south and southeast, wave protection is provided by a stone revetment having a slope of 1.5 horizontal to 1 vertical. Along a portion of the southeast perimeter a vertical seawall is provided. On the east and northeast a compacted structural fill (tunnel cuttings) slope is utilized. Finally, along portions of the north side of the site a sheet pile retaining wall is provided.

Flood protective structures along the perimeter of the site have been designed for the wave conditions listed in Table 2.4-12. The stone revetment design for the south side (designated Revetment A, Figure 2.4-21) is governed by the maximum wave condition at Section 6 of Figure 2.4-21. The design consists of a uniform layer rip-rap structure with a slope of 1 on 1.5. The Revetment A design, defined in Table 2.4-14 and Figure 2.4-24, is placed along the site perimeter as shown on Figure 2.4-21.

The design of the stone revetment along the southeast side (Revetment C) is governed by the maximum wave conditions at Section 4 of Figure 2.4-21. This design also consists of a uniform layer riprap structure with a slope of 1.5 horizontal to 1 vertical. The Revetment C design, defined in Table 2.4-14 and Figure 2.4-24, is placed along the site perimeter as shown on Figure 2.4-21.

The design of the riprap revetment is based on methods given in Section 3.27 of Reference 19.

The controlling wave conditions for design of the vertical seawall on the southeast side of the site are defined by the maximum wave activity for Section 5 of Figure 2.4-21 and are given in Table 2.4-12. The seawall is designed to withstand the wave forces given in Table 2.4-15 and on Figure 2.4-25. These wave forces, when resisted by the passive earth pressure acting against the opposite side of the vertical seawall, do not produce the governing design load. The vertical seawall is founded directly on bedrock which provides adequate protection against toe scour.

The only site area subjected to significant overtopping during the hypothetical probable maximum flood is along the vertical seawall on the southeastern side of the site. It is not anticipated that this overtopping will cause significant erosion because of its short duration. However, the area behind the vertical seawall and adjacent to the two Class I electrical manholes (#13/14 and #15/16) and their associated duct banks will be paved to prevent erosion of the fill material due to wave overtopping. To accomplish this, the area between the vertical seawall and the edge of the south plant road will be paved. This paving will extend approximately 20 ft beyond the vertical seawall's extremities to the east and west. The area covered will be approximately 81.5 ft x 490 ft (4440 sq. yds.), sufficient to ensure the integrity of both the vertical seawall and the electrical manholes and duct banks.

The sheet pile retaining wall on the north side of the site is designed for the wave conditions existing along Section 1 and given in Table 2.4-12 and for the wave forces given in Table 2.4-16 and on Figure 2.4-25.

The design of the vertical seawall and retaining wall are based on the wave forces computed by the methods given in Reference 19 for breaking, broken and nonbreaking waves.

The design of the east and northeast sections of Revetment B (designated on Figure 2.4-21) consists of compacted tunnel cuttings. For erosion protection, Revetment B will be faced with quarrystone as defined in Table 2.4-14 and on Figure 2.4-24. The east section of Revetment B will have a slope of 1 on 3. The northeast section of Revetment B will have a nominal slope of 1 on 2.5 that varies from 1 on 1.9 to 1 on 2.5. This variation in revetment slope will allow Rocks Road to have a 12-foot width without infringing on the marsh. Wave action against the fill revetments during peak PMH surge would be minimal and would have no adverse impact on any plant safety-related structures, systems, or components.

2.4.6 Probable Maximum Tsunami Flooding

Tsunami activity related to underwater geo-seismic activity is extremely rare on the U.S. Atlantic coastline. The only recorded activity on the northeastern U.S. coast resulted from the Grand Banks Earthquake of 1929. Although localized flood damage was experienced in the south end of Newfoundland, the southward propagation of the tsunami was insignificant. Tide elevation changes of about +0.9 feet were recorded at Ocean City, Maryland and Atlantic City, New Jersey.

A comprehensive listing of all global tsunami activity dating back to 425 B.C. (Reference 21) shows Atlantic Ocean disturbances to be rather localized. The Caribbean and Portuguese coastline appear to be the most active tsunami generating zones, but past history shows negligible effects on the U.S. Atlantic coastline from tsunamis originating from these areas. This includes the most severe offshore Atlantic disturbance that caused heavy destruction to the Portuguese coast in 1755.

Resonance in Hampton Harbor is discussed in Subsection 2.4.5.4 where it was pointed out that the natural period of the approximated basin was 30 minutes. It is conceivable that a tsunami with a period of 30 minutes could strike the Hampton Harbor area and excite the harbor to resonance, but the resultant seiche would be minor in comparison with the PMH storm surge criteria that will govern plant design.

It is concluded, that the Seabrook Station site will experience no significant tsunami effects. The maximum suspected tsunami would result in only minor wave action which is insignificant compared to the maximum expected hurricane storm wave effects as discussed in Subsection 2.4.5, Probable Maximum Surge and Seiche Flooding.

2.4.6.1 Probable Maximum Tsunami

See Subsection 2.4.6.

2.4.6.2 Historical Tsunami Record

See Subsection 2.4.6.

2.4.6.3 Source Tsunami Wave Height

See Subsection 2.4.6.

2.4.6.4 Tsunami Height Offshore

See Subsection 2.4.6.

2.4.6.5 Hydrography and Harbor or Breakwater Influences on Tsunami

See Subsection 2.4.6.

2.4.6.6 Effects on Safety-Related Facilities

As was stated in Subsection 2.4.6, no significant tsunami effects are expected on safety-related facilities. No design criteria for tsunami effects are therefore being considered.

2.4.7 Ice Flooding

There can be significant ice formation in Hampton Harbor and its tributary streams during severe winter conditions. Ice formations of over six inches in thickness have been reported in the Hampton section of the harbor, and formations over a foot thick have occurred in the Seabrook section. However, local flooding conditions due to ice formation are not significant to plant operation, since the major increase in water level due to maximum surge conditions is much greater than the effect of local stream flow. Since maximum surge water levels would exist only during peak storm periods, there would be no concurrent ice flood condition at maximum design conditions. Safety-related facilities are located at an elevation which makes them invulnerable to any local ice activity. Ice blockage of cooling water sources is not possible. The inlet structure is about 7000 feet offshore and draws water from below mid-depth in over 50 feet of water (mean low water). The open ocean in this region does not freeze, and the prevailing offshore winds of the winter will tend to disperse seaward any ice which escapes from the harbor. Consequently, the safety-related facilities need no specific ice design criteria.

2.4.8 Cooling Water Canals and Reservoirs

2.4.8.1 Canals

There are no cooling water canals.

2.4.8.2 Reservoirs

Seabrook Station cooling water is supplied by the Atlantic Ocean as the primary source. A standby service water cooling tower and 7-day makeup reservoir are provided as part of the ultimate heat sink (see Subsection 9.2.5).

2.4.9 Channel Diversions

Seabrook Station draws cooling water from the Atlantic Ocean approximately 7000 feet east of Hampton Beach; therefore, upstream diversions or rerouting will not affect the cooling water supply.

2.4.10 Flood Protection Requirements

None of the safety-related facilities at Seabrook Station are susceptible to flooding as discussed in Subsections 2.4.2.3 and 2.4.5.

2.4.11 Low Water Considerations

2.4.11.1 Low Flow in Rivers and Streams

Low flow from rivers and streams tributary to Hampton Harbor will not affect the water level or the availability of plant makeup water. Sufficient water to meet plant needs is available from the Atlantic Ocean. The controlling low water depth is given in Subsection 2.4.11.2.

2.4.11.2 Low Water Resulting from Surges, Seiches or Tsunamis

Low water levels at the site within Hampton Harbor will occur when the PMH winds acting on the bay area are such to produce a drawdown effect on the bay's western boundary. When the above occurs in conjunction with conditions of low astronomical tide, minimum water depths near the site will prevail.

The minimum low astronomic tide for Hampton Harbor is -2.2 feet MLW (-6.3 feet MSL). At extreme low tide, Hampton Harbor consists of the conjunction of two rivers, the Hampton and Blackwater. The Hampton River runs roughly south near the site and in the vicinity has a depth of from 5 to 18 feet at Mean Low Water with an average of about 7 feet. Therefore, at low astronomical tide, the depth of the river would be about 5 feet. The average width of the river is about 600 feet.

The predicted lowest tide given on U.S.C. & G.S. nautical charts for the Hampton Harbor area is -3.5 feet MLW (-7.6 feet MSL). The treatment of extreme low tide or drawdown due to hurricanes is scarcely found in engineering literature, and therefore it is difficult to treat hurricane drawdown effects with sound theoretical background. In this analysis, it was assumed the PMH approached the site such that its maximum winds were essentially in an easterly direction, and successive approximations of setdown in water surface elevation at shore were made using hourly average wind speeds of from 110 mph at shore to 114 mph some 15-20 miles offshore. Based on this analysis, a setdown of 3.0 feet would occur coincidental with the low astronomical tide, resulting in an extreme low tide occurrence at the site of -5.2 feet MLW (-9.3 feet MSL).

Additional lowering of the minimum stillwater level due to wave action is concluded to be not appreciable. This is due to the plant site location at the head of the fetch for an offshore wind direction. Also, Hampton Harbor will be effectively screened from deep water swell because the inlet entrance will be reduced in cross section by the extreme low water condition. Furthermore, the strong offshore winds will dissipate what little wave energy does pass through.

2.4.11.3 Historical Low Water

The controlling low water condition would occur from the effects of northeaster or hurricane winds as previously discussed in Subsection 2.4.11.2. The predicted lowest tide given in U.S.C. & G.S. nautical charts for the Hampton Harbor area is -3.5 feet MLW (-7.6 feet MSL), while the minimum low astronomical tide for Hampton Harbor is -2.2 feet MLW (-6.3 feet MSL). There have been gaging stations in Hampton Harbor to record historical elevations of low tide.

2.4.11.4 Future Control

There is no future control of low water levels anticipated for Hampton Harbor.

2.4.11.5 Plant Requirements

The service water pumps provide a minimum safety-related cooling water flow of 9800 gpm during a loss of reactor coolant accident with a loss of offsite power, and a total flow of 21,000 gpm for normal full power operation. The service water pumps take suction from a common bay of the Service Water Pumphouse. The service water bay is supplied with cooling water from the Atlantic Ocean through the circulating water intake tunnel and two lines from the circulating water intake transition structure. Two additional lines connect the service water pump bay with the discharge tunnel, which would be utilized for the cooling water supply during the anti-fouling heat treatment of the intake tunnel (see Subsection 9.2.1). For design details of the Service Water Pumphouse see Figure 1.2-46, Figure 1.2-47 and Figure 1.2-48.

During operation other than anti-fouling heat treatment, the service water effluent is discharged directly into the condenser discharge lines leading to the discharge tunnel. During anti-fouling heat treatment of the intake tunnel, cooling towers will be used to keep the service water temperature at or below 80°F. Blowdown from the Service Water System is discharged into the intake tunnel. The discharge tunnel of the Circulating Water System ends with a submerged multi-port jet diffuser whose discharge ports are spaced sufficiently far apart to prevent interference of adjacent discharge plumes, and are oriented for best heat dispersion (see Subsection 10.4.5).

The design bases for effluent submergence, mixing and dispersion were established from field data collected at the site as well as physical and analytical modeling of the submerged multi-port diffuser discharge. The service water flow is diluted by a factor greater than 1:18 in the nominal circulating water flow. The combined service and circulating waterflow receives additional immediate dilution greater than a factor of 1:7 at the sea surface resulting in a service water dilution factor on the sea surface greater than 1:126. The discharge effluent undergoes additional dilution after reaching the sea surface which is a function of wind speed and direction and natural ocean currents.

2.4.11.6 Heat Sink Dependability Requirements

The service water pumps are supplied with cooling water from the Atlantic Ocean through one tunnel and two independent supply lines. Both supply lines are located at an elevation low enough to ensure an adequate supply of cooling water and adequate pump submergence during extreme low water level of the Atlantic Ocean.

However, in the unlikely event that the Atlantic Ocean supply source is not available, the Service Water System can supply sufficient cooling water to components performing safeguards functions during a loss-of-coolant accident (see Subsection 9.2.5).

The fire protection system is supplied with water from two onsite 500,000-gallon storage tanks and is, therefore, not dependent on the level of ocean water for an adequate water supply (see Subsection 9.5.1).

2.4.12 Environmental Acceptance of Effluents

Estimated activities of liquid radioactive releases for each liquid system associated with normal operation and anticipated operational occurrences, as well as the assumptions made regarding dilution of these releases and the estimated doses received by the general public as a result of station releases, are discussed in Subsection 11.2.3. As indicated in these sections, there is a very low probability that accidental liquid discharges from the station could contaminate any existing well supplies in the area, since groundwater movement is toward neighboring tidewater bodies and away from populated inland areas.

Hydrothermal model studies were conducted by Alden Research Laboratories to predict the effects of the heated condenser water discharge from Seabrook Station.

A submerged multi-port discharge concept as well as a submerged single-port discharge scheme was investigated in a previous hydrothermal model study (Reference 22). The submerged jet discharge concept induces rapid dilution of the heated effluent with the receiving water. In the near-field region of the discharge, the effluent is diluted by at least a factor 10 at less than 1000 feet from the point of discharge. This factor is constantly increasing with greater distance from the discharge zone into the far-field region. The rapid initial dilution and mixing of the effluent tends to reduce the effect of radioactivity on benthic marine organisms, minimizes radio-ecological concentration processes in the free-swimming fish of the area, minimizes the extent of the high ΔT zone of the near-field region and reduces the potential for recirculation of effluent from the discharge to the submerged inlet.

The possibility for recirculation of effluents on a localized basis is minimal. Due to the initial rapid dilution of the discharge, any effluent reaching the inlet over 3,000 feet away is highly diluted. Also, the heated effluent rises to the surface during initial mixing with receiving waters to form a stratified surface layer. The location of the inlet structure over 3,000 feet from the discharge and the submergence of the inlet being below mid-depth in about 60 feet of water preclude any potential for direct recirculation of undiluted effluent. If it exists at all, the only possibility for recirculation is of highly diluted and well dispersed effluent; however, this could occur only during times of onshore currents.

The natural offshore drift of the receiving waters and the inherent momentum of the diluted effluent plume at the surface tends to carry the activity in liquid discharges away from the discharge point, thus preventing any localized concentration in the ocean along the coast or in Hampton Harbor and reducing the potential for recirculation.

2.4.13 Groundwater

Seabrook Station is located in what is termed by Meinzer (Reference 23) as the Northeastern Drift Province. Principal groundwater supplies in the area come from glacial drift. The average annual temperature in the area is 50°F. Mean annual precipitation is about 43 inches and annual loss to evaporation from water bodies is approximately 25 inches. Seepage into the groundwater body is highly variable owing to the variations in the permeability of the surficial deposits.

The hydrologic boundaries of the site are Hampton Harbor, the local drainage courses and impervious subsurface materials.

2.4.13.1 Description and Onsite Use

a. Regional Aquifers, Formations, Sources and Sinks

The study area comprises the drainage basins of Hampton River, Browns River, Blackwater River and Hampton Harbor. It includes the towns of Hampton, Hampton Falls, Kensington and Seabrook in New Hampshire and Salisbury, Massachusetts. Throughout the area, groundwater is found in the bedrock and in overlying glacial and recent deposits. The seaward limit of the fresh groundwater body does not extend greatly beyond the tidewater margins of Hampton Harbor. Infiltration of precipitation is retarded in places by the impermeable marine sediments which overlie much of the area. The shallow unconsolidated surficial deposits overlying bedrock are the principal aquifers in the area. These consist of beach deposits, swamp deposits and glacial drift. The latter includes till, ice contact, marine and outwash deposits. Groundwater in the underlying bedrock is limited to fractures which become less frequent at increasing depths. The effective depth for fractures to transmit water is about 300 feet.

1. Aquifers and Formations

The largest quantities of groundwater are obtained from course-grained sediments in the ice contact deposits which consist primarily of stratified sand and gravel. These are the coarsest in texture of all the local deposits and average about 50 feet in thickness. As shown in Figure 2.4-26, their areal extent is small, except in the vicinity of Hampton and Salisbury. These deposits are a source of public water supply for the towns of Seabrook, Salisbury and Hampton.

Lesser amounts of groundwater, adequate for meeting the needs of homes, farms and small industries are available from the outwash deposits. Well yields from them generally do not exceed 100 gpm (Reference 24). In the study area, the outwash is mostly made up of fine sand, commonly less than 25 feet thick, and is a source for small domestic supplies.

Some small wells are also developed in the till and in beach sands. The till which is an assorted mixture of rock particles in a matrix of clay and silt, generally yields only a few gpm to a well in this area (Reference 25). Groundwater development from permeable beach sands in the Hampton and Seabrook Beach areas is limited. Freshwater occurs there as a thin lens, in many places only a few feet thick, which is floating on saline water. Recharge to the lens is from infiltrating precipitation which originates in the beach areas. These till and beach sand deposits are not considered an important source of water for the region.

Impermeable marine deposits largely consisting of silt and clay are widely distributed in the area. They are not a source of well supplies but locally confine groundwater in ice contact deposits, till or bedrock (Reference 24).

Bedrock which underlies the unconsolidated materials is composed of the Newburyport quartz diorite and the metamorphosed sediments of the Merrimack group. There is little apparent difference in the water-bearing properties of the different types of rock. Most bedrock wells yield less than 10 gpm from depths up to 300 feet (Reference 26).

Swamp deposits almost wholly occupy the tidal marshes and contain brackish or salty water. These deposits are impermeable and are not sources of water supplies.

2. Sources and Sinks

The groundwater body in the area occurs under water table conditions, except in some places where it is confined by marine sediments. It is principally sustained by infiltrating precipitation, which in the region averages about 43 inches per year. The infiltration capacities of soils in the area vary considerably and, where the soil is composed of marine clays, groundwater recharge is greatly retarded.

The regional water table approximates the configuration of the topography, and frequently occurs within 10 feet of the ground surface. Groundwater movement is limited to drainage areas where streams intersect the water table and in areas where streams are tributary to tidewater. Because these drainages are relatively small, groundwater flow paths from points of recharge to discharge generally do not exceed one mile (Reference 24).

Recharge to aquifers in the region is accomplished by the infiltration of precipitation. The places immediately underlaid by ice-contact deposits and by outwash and shore deposits (see Figure 2.4-26) are the principal recharge areas. These deposits are sufficiently permeable to absorb water readily. They commonly form terraces and plains whose flat surfaces retard surface run-off, and thereby afford ample storage space to accommodate the additional water (Reference 27).

Many places immediately underlain by till also serve as recharge areas, but here the rate of recharge is comparatively small. Not only is the till less permeable than the outwash and ice-contact deposits, but it locally forms hills whose slopes shed water rapidly.

Recharge occurs intermittently, and usually follows a seasonal pattern. During the growing season, most of the precipitation that enters the soil is retained there to satisfy soil-moisture requirements, and recharge therefore is small. During the rest of the year when plants are dormant, the soil-moisture requirement usually is small, and recharge is great whenever there is much rain or snowmelt. The peak usually accompanies snowmelt during the spring season.

Groundwater is discharged naturally through springs, by seepage to streams and other bodies of surface water, and by evapotranspiration. It is discharged artificially through wells and artificial drains. Discharge to streams, called groundwater run-off, usually is greatest soon after periods of dry weather and sustains the flow of the streams when there is little or no surface run-off. Discharge by evapotranspiration is greatest during the growing season.

Under natural conditions, the principal discharge areas in the Seacoast Region are stream channels, the swamps, and the coastline. The water table normally slopes toward the streams, and groundwater enters them wherever they flow on permeable material. Groundwater is discharged in swamps and other low areas by seepage whenever the water table is high enough to intersect the land surface, and by evaporation and transpiration at times when the water table is only a short distance below the land surface. Along the coastline, some of the groundwater evaporates and some of it seeps directly into the ocean.

Changes in groundwater storage take place as a result of changes in the ratio between recharge and discharge. In general, periods when recharge is greater than natural discharge occur in late fall, winter, and early spring while evapotranspiration is ineffective. During late spring, summer, and early fall, however, the amount of groundwater in storage declines. The decline occurs because most of the rainfall that infiltrates into the soil is evaporated or transpired by plants and does not reach the zone of saturation. Recharge and natural discharge continue, though at a reduced rate.

Changes in groundwater storage are reflected by fluctuations in groundwater levels; these levels rise when recharge exceeds discharge and decline when discharge exceeds recharge.

In general, the greater the permeability of a deposit, the smaller the water-level fluctuations. In till, for example, fluctuations ranging from 10 to 20 feet are not unusual, especially in wells located on hills or slopes. During periods of recharge, the low permeability of the till prevents rapid lateral percolation of groundwater to areas of discharge, and the water level rises considerably. However, during periods of little recharge, the groundwater continues to drain and discharge slowly; thus, the water level declines. In contrast, fluctuations of only a few feet are common in wells in ice-contact deposits. These deposits are sufficiently permeable to transmit groundwater laterally at rates approximating those of recharge, and large rises in water levels ordinarily do not occur (Reference 27).

b. Local Aquifers, Formations, Sources, and Sinks

No major aquifers underlie the site or its vicinity. Locally, the most productive aquifers are in the outwash deposits which are widely distributed just west and southwest of the site (Figure 2.4-26). The outwash, however, is made up mostly of predominantly fine, silty sand of low permeability. In the site area, it is up to 35 feet thick, and, generally, overlies marine sediments.

Local occurrences of coarser grained glacial and/or recent deposits are evident both to the northwest and under the tidal marshes east of the site (Figure 2.4-27). These deposits, however, contain either brackish or salty water, or would be subject to salt water intrusion under pumping conditions because of their proximity to salt water bodies.

On the site property, bedrock occurs at or near the surface, becoming deeper under the tidal marshes to the south and north where it is as much as 70 feet or more below sea level. On the site, the bedrock forms a partially buried ridge trending in an approximately easterly direction. It is overlain by a sandy textured, but well compacted, till up to 62 feet thick. A sequence of marine and recent marsh deposits normally rests on the till along or just north of the Browns River near the northern site boundary and also in adjoining areas to the south (Figure 2.4-26).

West of the site, thin outwash deposits overlie either till or marine silts and clays. To the east, toward Hampton Beach, medium to fine sands, 50 feet or more in thickness, occur just below ground level on recent marsh deposits (Figure 2.4-27). The sands, which appear permeable, are essentially saturated with salt water. They are outwash or older shore deposits with beach sands overlying them in the Hampton Beach area.

In the site area, the water table is found at depths no greater than 17 feet, and generally less than 10 feet. West of the site area in the sandy outwash material it is usually within 5 feet of the ground surface.

Predominant groundwater movement is toward the tidal areas; however, local flow lines are modified by variations in permeability of water-bearing materials and by topography. Plots of available water table levels in the plant area are shown on Figure 2.4-28 and Figure 2.4-29. The contours in Figure 2.4-29 are based on water level readings taken in the B, D and E series borings, described in Subsection 2.5.4.3 and located on Figure 2.5-17. Rate of groundwater movement is expected to range from a few feet to several tens of feet per year. Based on available information, the average permeability of both the till and bedrock is less than 10 gpd per ft^2 (gallons per day per square foot). Permeability of the marine deposits is less than 1 gpd per ft^2.

c. Utilization of Groundwater by the Plant

Groundwater will be used during operation of Seabrook Station for potable, sanitary and nonsafety-related purposes. The total estimated demand is 110×10^6 gallons per year or about 200 gallons per minute.

The town of Seabrook supplies 50,000 gallons per day or 35 gallons of groundwater per minute to Seabrook Station from the Seabrook water supply system. Additional demand will be met by a series of bedrock wells at two well fields located approximately 2000 and 3000 feet to the west and north of the site, respectively. Well locations are shown in Figure 2.4-30 and specific well data in Table 2.4-17.

Groundwater storage on site includes two 500,000-gallon fire tanks.

2.4.13.2 Sources

a. Groundwater Use

1. Present Regional Use

Most water supplies in the area are dependent on groundwater sources. Public supplies in the towns of Seabrook and Salisbury are taken from wells which tap aquifers in ice contact deposits. These wells yield from about 300 to 700 gpm, and range from 22 to 54 feet deep (Reference 24). The town of Seabrook at the present uses five wells for its public water supply, and all of these are located at least two miles from the site. Most homes, as well as commercial and industrial users in Seabrook, are supplied by the town's municipal water system (Reference 24). The Salisbury Water Company uses four wells to supply water to most homes and industries in Salisbury, Massachusetts.

Other wells supplying mostly domestic and farm needs are scattered throughout the area, including the towns of Hampton Falls and Kensington, which are both without public water supply systems. In the site vicinity, a few private wells supply homes to the north of Seabrook Station.

2. Tabulation of Existing Users

Figure 2.4-31 shows the location of all known active wells in the region (Reference 28). Data for each of these wells and for many test borings are presented in Table 2.4-18 and Table 2.4-19. The information provided in these tabulations includes names of owners, location, year completed, depth, diameter, type, geologic characteristics, water level and type of use.

Figure 2.4-30 and Table 2.4-17 provide information on wells in the immediate vicinity of the site. The two nearest well fields are located approximately 2000 and 3000 feet to the west and north of the site, respectively.

3. Town of Seabrook Municipal Water System

The town of Seabrook is served by its own municipal water works system, whose source is groundwater wells. The basic system, first put into use in 1956 with two wells, now consists of five active high-yield groundwater wells, each with a pump and pumphouse. Present storage capacity is provided by a 720,000-gallon storage standpipe and a 1,000,000-gallon tank.

The system, with approximately twenty miles of 6", 8", 10" and 12" diameter distribution pipe, is outstanding in size and service in comparison to the small population of the town and to the water systems of adjacent towns.

The groundwater drawn in Seabrook is of good quality, as it generally is throughout the whole southeastern New Hampshire region (Table 2.4-20).

The water consumption rate in Seabrook has been steadily increasing over the past decade. Table 2.4-21 contains figures on pumpage for Seabrook and now shows an average increase of about 20,000,000 gallons per year.

4. Town of Salisbury, Massachusetts, Water Supply System

The town of Salisbury at present is served by the privately owned Salisbury Water Supply Company which draws its supply from four wells in the northwestern corner of Salisbury (Figure 2.4-31). The four wells draw from 400 gpm to 700 gpm to supply the town's residential and industrial users. A 300,000- and a 325,000-gallon standpipe are also in use.

5. Projected Future Use

The demand for water in this region is expected to grow at an accelerating rate over the projection period (1980-2020). This increase in water use can be attributed to the shifting industrial trends and increasing suburbanization of New Hampshire. More supply wells and inter-municipal distribution systems are anticipated to satisfy the region's increased demand for water.

Table 2.4-22 presents the water use projections through the year 2020 for towns in Rockingham County and Salisbury, Massachusetts, through 1990. It is expected that both surface and groundwater sources will be developed to provide the required supply.

6. Groundwater Levels

The pattern of water-level fluctuations in the region is irregular, reflecting variations in precipitation and temperature. This is illustrated in Figure 2.4-32 which correlates the hydrographs of selected wells in southeastern New Hampshire with monthly precipitation records (Reference 31).

The water table in the site area is mostly in till or bedrock at depths no greater than 17 feet, and usually less than 10 feet below the ground surface. In the outwash deposits west of the site (Figure 2.4-26), it occurs mostly within 5 feet of the surface. Some partially confined groundwater is found at depth in bedrock fractures. Evidence of this was found along the edge of tidal marshes, where fresh groundwater with a chloride content ranging from 38 to 144 ppm was encountered in bedrock borings under sufficient hydrostatic head to cause flowing conditions (Reference 24).

b. Flow Directions and Gradients

In southeastern New Hampshire, groundwater generally moves from the inter-stream areas, where much of the recharge takes place, toward nearby streams or other bodies of surface water into which some of the groundwater is discharged. During warm weather, some groundwater also is discharged directly to the atmosphere by evaporation and transpiration in areas such as swamps or marshes where the water table is at or near the surface. Under the hydraulic gradients that exist in nature, the rate of groundwater movement is very slow. In the aquifers of the report area, groundwater moves at rates that range from a few inches per year to a few feet per day.

Groundwater movement in the site area is toward adjoining tidal areas, and essentially normal to the water table contours shown on Figure 2.4-28. Local modifications in flow lines are the result of variations in permeability of water-bearing materials and of topography. Per NEI 07-07 a fate and transport study was performed indicating concurrence with the original flow gradients excepting that near-surface groundwater is redirected eastward by the south sea wall (Reference 37). Rates of groundwater movement at the site do not exceed 100 feet per year (Reference 24). This is based on a water table gradient of 0.06 feet per foot, as observed during high water table conditions, and an average permeability of 5 and 4 Meinzer units (gallons per day per square foot at prevailing groundwater temperatures) for the till and bedrock, respectively. The low permeability of the till and bedrock is substantiated by the lack or relatively small response in water levels to tidal fluctuations, as observed in several borings located along the edge of the tidal marshes. Table 2.4-23 lists the range and mean values of field permeabilities of glacial and bedrock materials. These were determined by falling head and packer tests made in the test borings on the site area. The listed values for the outwash material are representative for the finer sands more commonly found to the west of the site, whereas, the coarser outwash and beach sands to the east (Figure 2.4-26) appear to be much more permeable, and values of 1,000 gpd per square foot or more are probably not uncommon.

Local areas of pumping for both plant and nonplant use may result in localized areas of reversibility of groundwater flow in the vicinity of the pumping wells. However, the general movement of groundwater in the area will follow the natural groundwater movement which is toward adjoining tidal areas and essentially normal to the water table contours shown in Figure 2.4-28. Widespread reversal in the direction of groundwater flow due to overpumping may lead to saltwater intrusion and degradation of the aquifer. Pumping from Unit 2 containment building and a French drain well just east of the Unit 1 containment has resulted in a local cone of depression, stopping the eastward progress of groundwater in the immediate area, but not promoting seawater intrusion (Reference 38). Optimum distribution of pumping and monitoring of water quality will protect against adverse effects associated with reversal of groundwater flow in the vicinity of Seabrook Station. The plant dewatering system pumps at a flowrate to ensure seawater intrusion does not occur.

c. Recharge Area within the Influence of the Site

Under natural conditions, nearly all recharge to aquifers in southeastern New Hampshire is accomplished by the infiltration of precipitation within the area. The principal recharge areas are the places immediately underlain by ice-contact deposits and by outwash and shore deposits. These deposits are sufficiently permeable to absorb water readily. They commonly form terraces and plains whose flat surfaces retard surface run-off and, thereby, afford ample opportunity for infiltration. They, generally, also provide sufficient storage space to accommodate the additional water.

Many places immediately underlain by till also serve as recharge areas, but here the rate of recharge is comparatively small. Not only is the till less permeable than the outwash and the ice-contact deposits, but it commonly forms hills whose slopes shed water rapidly. Furthermore, because till generally is thin, it may at some places become so fully saturated during prolonged periods of wet weather that potential recharge is rejected.

The site is primarily underlain by well compacted till up to 62 feet thick and, therefore, it is not an important recharge area (Reference 24).

2.4.13.3 Accident Effects

There is no evidence to indicate that accidental liquid radioactive material released at the site could contaminate any existing well supplies in the area, since groundwater is moving toward neighboring tidewater bodies and away from populated inland areas. Moreover, public supply wells are located in areas beyond reasonable limits of groundwater travel from the site area.

Liquid radioactive material released on the site could conceivably reach nearby tidewater bodies. Groundwater movement in the site area is toward adjoining tidal areas and essentially normal to the water table contours shown in Figure 2.4-28. Local modifications in flow lines are the results of variations in permeability of water-bearing materials and of topography. The maximum rate of groundwater movement will occur under conditions of maximum permeability and maximum water table gradient. Table 2.4-23 lists the range and mean values of permeability determined in the various soil samples taken in the vicinity of the site.

Assuming the release of a contaminant near the southern boundary of the site, the maximum rate of travel to the marsh via a groundwater path can be determined. Borings near the southern boundary of the site or just south of the site on the marsh show that the soils are primarily silty sands. These soils correspond to the till and marine (silty phase) deposits in Table 2.4-23 for which the maximum permeability is 25 gpd/ft^2.

Assuming a water table gradient of 0.06 feet per foot, as observed during high water table conditions, and a porosity of 0.3, the maximum rate of groundwater movement along a flow path moving southward from the southern portion of the site is 0.7 ft/day. The shortest distance from a site location at which a radioactive liquid spill could hypothetically occur to the marsh is about 200 feet. Therefore, it will require at least 290 days for a liquid radioactive release at the site to reach the marsh. Furthermore, a part of such contamination would be absorbed on clay or silt particles in the till and marine deposits.

The nearest point of body-contact water activity to the site is in the marsh and estuary of Hampton Harbor. Once a liquid radioactive release had entered the marsh, it would reach Hampton Harbor during a normal tidal cycle. Therefore, as above, it would require at least 290 days for a liquid radioactive release at the site to reach the nearest point of body-contact water activity. The release would be greatly diluted before reaching Hampton Harbor.

It is unlikely that any wells will be located east of the site in the future because the groundwater underlying the marsh is brackish. Also, the Seabrook municipal water system is well developed and serves nearly 100 percent of the town's residents. Any future users will be served by this system which draws its water from wells far to the west of the site or from alternative sources located elsewhere. The Hampton Beach area is served by the town of Hampton municipal water system, which draws water from wells far to the north of the site. The nearest public wells to the site in the town of Salisbury are far to the south and would not be influenced by an accidental liquid discharge at Seabrook Station. The location of the plant dewatering system wells helps to ensure that no contaminated liquids migrate beyond the plant buildings in the event of any accidental liquid discharge from adjacent systems.

2.4.13.4 Monitoring or Safeguard Requirements

Tritium contamination was identified in the containment annulus in 1999. The source of this contamination was identified to be from the spent fuel pool. Twenty-four monitoring wells have been installed to track any migration of contamination in groundwater. Samples are collected and analyzed for tritium and gamma emitters although no gamma activity has ever been identified. Tritium contamination has been identified in wells proximate to the Fuel Storage Building, but has not migrated from that area. Dewatering wells as described in Subsection 3.4.1.2 maintain the tritium plume in the area of the Fuel Storage Building.

The natural movement of groundwater in the site area is away from the public and private wells in the region.

The two closest well fields to the Seabrook Station lie approximately 2000 feet west and approximately 3000 feet north of the site. The monitoring program will include monitoring of representative wells from these fields.

2.4.13.5 Design Bases for Subsurface Hydrostatic Loading

As stated in Subsection 2.4.13.2, the groundwater table in the site area is mostly in till or bedrock at depths no greater than 17 feet and usually less than 10 feet below the original ground surface. By assuming groundwater at elevation +20.0 feet MSL (the finished plant grade), the most severe case for hydrostatic loading is considered for design of structures.

All subsurface portions of safety-related structures, systems, and components have been designed for hydrostatic pressure and uplift due to the assumed groundwater level at elevation +20.0 feet MSL, plus 0.6 feet of ponding on site. The design water surface elevation for hydrostatic pressure and uplift is therefore 20.6 feet MSL. Flood loading is further discussed in Section 3.4.

There are no wells which are used for safety-related purposes.

During construction, dewatering was provided for foundations of partially completed safety-related structures to prevent both overall and local uplift.

The methods employed perforated pipes, channels, a combination of perforated pipes and channels, or pumping into an adjacent excavation.

Perforated pipes, when used, were arranged in a grid system and embedded in gravel within the leveling course of fill concrete below the waterproofing membranes. Water was directed to submerged sump pumps which pumped it to a collection system.

Where channels were used, a gridwork of small grooves was cut or preformed in the upper surface of the leveling course of fill concrete immediately below the waterproofing membrane. With this method, water was also directed to submerged sump pumps which pumped it to a collection system.

In some instances, the excavation for a safety-related structure was located near another excavation which was deeper. As a result of pumping to dewater this latter excavation, the groundwater table was lowered below the excavation located at the higher elevation. Water collected in this manner was also pumped to a collection system.

Thus, by these methods, the groundwater level was maintained below the bottom of the structural mat, thereby resulting in no subsurface hydrostatic loading. Each individual dewatering system was operated until construction of the associated structure reached a point where its dead weight offset the uplift due to the design hydrostatic pressure, including an appropriate margin of safety. A portion of the dewatering wells and grid system piping are now used as part of the plant dewatering system described in Section 3.4.1.2.

2.4.14 Technical Specification and Emergency Operation Requirements

The worst of any condition discussed in Section 2.4 does not create an adverse hydrologically related event on safety-related facilities. Plant grade elevation is above the highest postulated water flood level except at the southeast side of the site, as determined by superimposing the PMH maximum wave action on the combined SPS and PMH maximum still water level.

As indicated in Subsection 2.4.5.3, the slight amount of wave overtopping of the revetment at the southeast side of the site will not affect any safety-related structure. Consequently, there are no Technical Specifications related to wave attack or flooding.

2.4.15 References

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2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

Geology and Seismology Summary

Situated in the Seaboard Lowland section of the New England Physiographic Province, the site is located near the north edge of the town of Seabrook, Rockingham County, New Hampshire, 2 miles to the west of Hampton and Seabrook Beaches and the Atlantic Ocean. The physiographic configuration of the site area is characterized by broad open areas of level tidal marshes which are dissected by numerous meandering tidal creeks and linear man-made drainage ditches, interrupted locally by wooded "islands" or peninsulas which rise to elevations of 20 to 30 feet above sea level. The site is located on a peninsula in the marsh, composed of quartz diorite and included quartzitic bedrock locally overlain, prior to construction, by a thin veneer of glacial and postglacial soils. The groundwater table conforms with topography, normally lying 5 to 10 feet below ground surface. No major groundwater aquifers are inferred to underlie the site.

The bedrock basement within 200 miles of the site ranges in geological age from Late Precambrian to Upper Mesozoic, and consists predominantly of hard, crystalline metamorphic and igneous rock types. Mildly metamorphosed to unmetamorphosed, well-consolidated sedimentary and volcanic bedrock types of Carboniferous and Triassic age occur locally in basin structures in the crystalline basement in the Connecticut River Valley, the Narragansett and Boston basins, and in other apparently isolated basins within the Gulf of Maine; loosely-consolidated Coastal Plain sediments of Upper Mesozoic and Cenozoic age blanket the crystalline basement rocks and successor basins in wide areas on the Continental Shelf and in scattered patches near shore within the Gulf of Maine. The entire area is widely covered by a thin veneer of loose, unconsolidated sediments of Quaternary age, derived from continental glaciation and postglacial deposition.

All faulting in the region is assigned either to Early or Middle Paleozoic geosynclinal orogenic episodes, to Late Paleozoic oblique continental collisions, or to Mesozoic extensional tectonics associated with continental separation and the last opening of the Atlantic Ocean. There are no known or inferred tectonic faults displacing Quaternary deposits or postglacial and Recent sediments, nor has any tectonic activity been reported or inferred to have occurred in the region in the past 100 million years since late in the Mesozoic era, except in the limited form of successive broad isostatic uplifts and, the past 2 million years, as successive periods of crustal depression and rebound from continental glaciation and deglaciation.

Bedrock formations in the site area include metasedimentary quartzites, phyllites, and schists, and metavolcanic gneisses of the Merrimack Group of probable Early Paleozoic age, intruded by dioritic rocks of the Newburyport and Exeter plutons of possible Early to Middle Paleozoic age.

All metamorphic and igneous country rocks have been invaded by thin, predominantly northeast-striking, steeply-dipping mafic dikes of both Early Triassic and substantially older Paleozoic ages.

All safety-related site structures are founded on sound bedrock, on concrete fill extending to sound bedrock, or on controlled backfill extending to sound bedrock. A large portion of the site, including Unit I, is founded on a gneissoid phase of the Newburyport quartz diorite intrusive, a hard, durable crystalline igneous rock consisting of medium to coarse-grained quartz diorite matrix intimately enclosing inclusions of dark gray, fine-grained diorite. A small portion of the site, including much of Unit II, is founded on Merrimack Group metaquartzite and granulite which occurs as a large relict inclusion welded into the enclosing Newburyport igneous mass along a broad, transitional-intrusive contact zone. The physical, chemical, and mechanical qualities of the rock in the Merrimack Group metamorphic inclusion are comparable for site foundation purposes to those of the Newburyport igneous rock. Northeasterly-trending mafic dikes transect both igneous and metamorphic country rocks at widely-spaced intervals. The seismic field data are indicative of sound bedrock with a high in situ compressional wave velocity of 18,000 ft/sec and a shear wave velocity of 9,000 ft/sec.

Several dozen discontinuous faults have been identified at the site by detailed geologic mapping in construction excavations. Faulting in the Newburyport igneous rocks exhibits two styles: normal faults of small displacement which dip around 30°-40° to the northwest; and normal faults of small displacement which dip around 50°-60° to the east-northeast. Faulting in the Merrimack Group metasedimentary rocks is essentially characterized by several high-angle reverse or strike-slip faults which strike west-northwest parallel to bedding and dip variably nearly vertical. No through-going faults have been found at the site. All fault sets either die out at one or both ends within the excavated area or are transected by younger mafic dikes. Detailed observations of the bedrock surface and overlying stratified soils have revealed no evidence of postglacial offsets.

The largest earthquake intensity which has affected the site area in historic times is Intensity VII (MM), as estimated theoretically or by isoseismal lines constructed for two offshore earthquakes which occurred in 1727 and 1755, and for which epicentral intensities are estimated at VII (MM) and VIII (MM), respectively.

The 1727, Intensity VII (FIM), event and the 1755, Intensity VIII (MM), event are located offshore of Cape Ann, Massachusetts, about 14 and 30 miles, respectively, to the southeast of the site. These two 18th Century earthquakes, plus two Intensity VII (MM) events which occurred in 1940, about 66 miles to the north of the site in the Ossipee Mountains, New Hampshire, are the largest historical earthquakes reported in New England. The Cape Ann earthquakes are correlated with a localized tectonic structure consisting of a cylindrical mafic intrusive enclosed in a complex of post-metamorphic transcurrent and thrust faults. The Ossipee Mountains earthquakes are correlated with a localized tectonic structure consisting of a cylindrical mafic intrusive lying tangent to a boundary fault of a post-metamorphic block fault complex.

The largest earthquake in the coastal area in which the site is located is the offshore Intensity VIII(MM) event of November 18, 1755. An epicentral Intensity VIII event adjacent to the site is considered to be the maximum earthquake potential, although it is inconceivable that an Intensity VIII(MM) earthquake could occur on the crystalline bedrock at this site. In fact, an Intensity VIII on bedrock is a rarity from much larger earthquakes than the Cape Ann earthquake. An Intensity VIII on the adjacent tidal marsh and beach materials would be VI or less on the site bedrock. Because the Safe Shutdown Earthquake is a nearby event, the ground motion associated with it would be relatively rich in high frequencies (allowed for in the response spectra). The horizontal ground acceleration associated with the Safe Shutdown Earthquake is selected as 0.25g. The design response spectra are provided based on an earthquake of 10 to 15 seconds duration with a recommended zero period peak horizontal ground acceleration of 0.25g.

Technical Investigations Summary

The geologic, geophysical and seismological investigations described in Subsections 2.5.1, 2.5.2, 2.5.3, 2.5.4.1 and 2.5.4.4 were carried out under the direction of Weston Geophysical Corporation.

The text and figures for Regional Geology and Tectonics (Subsection 2.5.1.1; Figure 2.5-1, Figure 2.5-2, Figure 2.5-3, Figure 2.5-4 and Figure 2.5-5, inclusive), and for the site vicinity and site (Subsections 2.5.1.2 to 2.5.1.2b.6; Figure 2.5-6, Figure 2.5-7, Figure 2.5-8, Figure 2.5-9, Figure 2.5-10, to Figure 2.5-11, Figure 2.5-12, Figure 2.5-13, Figure 2.5-14, Figure 2.5-15, Figure 2.5-16 and Figure 2.5-17 inclusive) were prepared by John R. Rand, geologic consultant to Weston Geophysical, as were geologic-tectonic subjects discussed in Subsections 2.5.2, 2.5.3 and 2.5.4.1. John Rand also conducted geologic investigations of the site and cooling water tunnels area, of the Scotland Road and "Portsmouth" faults, and logged approximately 24,400 feet of bedrock cores in 205 borings drilled at the site, in the area of the cooling water tunnels, and at the Scotland Road and "Portsmouth" fault investigations.

The seismological data presented in Subsection 2.5.2 were prepared by the Weston Geophysical staff. The geophysical investigations consisting of seismic refraction survey and in situ velocity measurements as described in Subsection 2.5.4.4 were also conducted by Weston Geophysical.

Subsection 2.5.1.2b.6 and Figure 2.5-19, Figure 2.5-20, Figure 2.5-21, Figure 2.5-22, Figure 2.5-23, Figure 2.5-24, Figure 2.5-25, Figure 2.5-26, Figure 2.5-27, Figure 2.5-28, Figure 2.5-29, Figure 2.5-30, Figure 2.5-31, Figure 2.5-32, Figure 2.5-33 represent a summary of the results of geological investigations made at the site itself during construction. This information was collected and prepared by Francis X. Bellini of Yankee Atomic Electric Company, Anthony J. Stewart (while he was with Yankee), and by David H. Corkum (while he worked for United Engineers and Constructors).

2.5.1 Basic Geologic and Seismic Information

This Subsection is presented in two parts: the geology of the entire region, followed by descriptions of the geology in the site vicinity and at the site.

2.5.1.1 Regional Geology

The region is defined by a 200-mile radius from the site.

a. Regional Physiography and Geomorphology

1. Introduction

The site is situated in the central part of the Seaboard Lowland section of the New England physiographic province (Fenneman, 1938), about 30 miles southeast of the New England Upland section of the province, and about 2 miles west of the shoreline where the Seaboard Lowland meets ocean waters of the Gulf of Maine. The physiographic provinces and sections which lie within 200 miles of the site are shown on Figure 2.5-1, and include:

PROVINCE	SECTION
New England	Seaboard Lowland
	New England Upland
	Connecticut Valley Lowland
	White Mountain Section
	Green Mountain Section
	Taconic Section
	Hudson Highlands
Coastal Plain	Atlantic Ocean
	Gulf of Maine
Piedmont	Piedmont Lowlands
Valley and Ridge	Middle Section
	Hudson Valley Section
Appalachian Plateaus	Catskill Section
	Southern New York Section
	Mohawk Section
Adirondack	-
St. Lawrence Lowlands	Champlain Section

2. New England.Province (Site Province)

The Seaboard Lowland section in the area in which the site is located is a 40-mile wide, northeast-trending zone which is bordered on the northwest by the New England Upland section, and bordered on the southeast, beneath the waters of the Gulf of Maine, by the Coastal Plain. The southeasterly half of the Seaboard Lowland in the general site area is also submerged beneath the ocean.

The physiographic fabric of the land area in the New England region within 200 miles of the site is characterized by a series of sub-parallel belts, elongate to the northeast, of lowlands, uplands, and mountain ranges or groups. These northeast-trending physiographic belts largely reflect regional variations in the structure or lithology of the underlying bedrock, which ranges in age from Precambrian to Mesozoic. These differences are further accentuated by differential weathering and erosion. The topography has been rounded or subdued by the scouring action of continental glaciation which moved over the region intermittently during the Pleistocene epoch.

The Seaboard Lowland, a northeast-trending belt, ranges on land from about 25 to 75 miles in width. The Lowland is characterized by subdued, gently rolling topography which gradually rises from sea level to about 500 feet in elevation at its boundary with the New England Upland. Local relief in the Lowland, within 200 miles of the site, rarely exceeds 200 feet. Local monadnocks on the Lowland are commonly supported by granitic bedrock types. The bedrock geology of the Lowland includes an Early Paleozoic foldbelt of metasedimentary and metavolcanic rocks intruded by granites through southeastern New Hampshire and southwestern Maine; and a crystalline basement complex of Late Precambrian age in eastern Massachusetts and Rhode Island. In the lower elevations, glacio-marine clay-silt deposits blanket the terrane, locally underlying or inter-layering with granular ice contact and outwash deposits.

The New England Upland is a maturely dissected plateau ranging in elevation from about 500 to 2,000 feet, underlain largely by Silurian and Devonian eugeosynclinal metasedimentary rocks which were folded, recrystallized, and consolidated in a broad northeast-striking foldbelt during the Acadian Orogeny (Devonian). Monadnocks rising above the Upland terrane are generally composed of metamorphic bedrock of Acadian age; some of the more prominent of these, however, are supported by discordant intrusive bodies of Middle and Late Mesozoic age scattered along a zone trending north-northwest across New Hampshire into southern Quebec, from southwestern Maine and southern New Hampshire. In southwestern New Hampshire and west-central Massachusetts, the New England Upland is largely supported by north-trending, Lower Paleozoic granitic domes of the Bronson Hill anticlinorium and by Precambrian rocks of the Berkshire Uplands.

The Connecticut Valley Lowland, a distinctive low-elevation physiographic and geologic element, trends northward into the New England Upland for about 100 miles through central Connecticut and west-central Massachusetts. The valley, formed by crustal rifting in Early Mesozoic time, contains relatively easily eroded sandstones and shales of Triassic and Jurassic age, locally inter-layered with resistant diabase flows which form prominent ridges.

The White Mountain physiographic section forms the central core of the New England province, with elevations rising from about 2,000 feet above sea level to a maximum on Mt. Washington of 6,288 feet. Lithologically, the White Mountain section represents a mountain group, with both plutonic rocks of Paleozoic and Mesozoic age, and Paleozoic metamorphic rocks of high metamorphic grade supporting high elevation peaks. Mt. Washington and adjacent peaks of the Presidential Range are supported by Lower Devonian quartzite and schist, while numerous lesser peaks within 20 miles to the southwest of Mt. Washington are supported by granite and volcanic rocks of Middle Mesozoic age. Northeasterly from Mt. Washington, prominent peaks are held up by either plutonic or metasedimentary rocks. Structurally, the White Mountain section is a throughgoing, northeast-trending core of Ordovician and older rocks in the Boundary Mountain anticlinorium, bounded on the northwest and southeast flanks by metamorphic and igneous rocks of Devonian age.

In the western-most part of the New England province, the narrow belt of the Green Mountain section ranges in elevation from about 1,000 to 3,000 feet, and reflects closely the continuous north-trending fabric of fairly open anticlinal folding and westerly directed thrust faulting of crystalline Precambrian basement masses and overlying Lower Paleozoic miogeosynclinal metasedimentary rocks.

The Taconic section is characterized by a mountainous terrain supported by quartzite, schist, and phyllite metamorphic rocks, with a prominent valley on the east underlain by relatively unresistant marble bedrock.

The north-trending alignment of the section reflects the underlying bedrock fold and fault structure which developed during Taconic and Acadian orogenies (Paleozoic time) by westerly directed crustal compression.

The Hudson Highlands section, a narrow southwestward extension of the upland terrane of the New England province, is underlain mainly by Precambrian crystalline rocks related to those of the Green Mountain and Berkshire Uplands. The section is characterized by elevations ranging to about 1,200 feet, cut by deep, structurally controlled valleys trending parallel to the section. The section boundaries with the Middle section of the Valley and Ridge province to the northwest, and with Juro-Triassic sedimentary rocks of the Piedmont Lowlands to the southeast are fault controlled and abrupt.

3. <u>Coastal Plain Province</u>

The Atlantic Coastal Plain extends from the Gulf of Maine through southeastern New Jersey, and forms the Continental Shelf beneath the Atlantic Ocean. The province is a low-elevation region comprised of loosely consolidated sediments of Cretaceous and Cenozoic age resting on basement rocks which constitute the on-strike extensions of the Precambrian, Paleozoic, and Mesozoic terranes of the upland areas. Beneath Long Island, Cape Cod, and the offshore southern islands, Coastal Plain sediments underlie locally thick deposits derived from Pleistocene glaciations. The Coastal Plain section is characterized by a series of seaward-dipping sedimentary formations which thicken toward the continental slope.

Within the Gulf of Maine, Coastal Plain sediments occur as submerged patches or thinly-veneered blankets resting on Mesozoic to Precambrian crystalline basement rocks. The closest approach to the site of Coastal Plain deposits is on Jeffrey's Ledge, about 20 miles to the east of the site.

4. Piedmont Province

The Piedmont Lowlands in the site region are underlain by relatively nonresistant Triassic and Jurassic shales and sandstones with inter-layered resistant diabase flows. The Lowlands are bounded on the northeast and north by a prominent escarpment of the Palisades diabase sill, on the northwest by the Ramapo fault and other border faults of Mesozoic rifting derivation, and on the southeast by the overlap of Coastal Plain sediments of Cretaceous age. The Palisades are the outstanding feature of the section, forming the west bank of the Hudson River from Nyack, New York, southward. Here, the Hudson River follows the contact of the Triassic shales with the underlying crystalline basement rocks. Southwestward, the Precambrian and Early Paleozoic basement of metamorphic rocks and igneous intrusives is cut by other Triassic sediment-filled basins.

5. Valley and Ridge Province

The Hudson Valley section is a lowland resulting from erosion along an outcrop belt of relatively nonresistant shales and slates, lying between the more resistant sedimentary rocks of the Catskill Mountains and Helderberg escarpment to the west, and the harder metamorphic rocks of the Taconic Mountains to the east. Most of the section has both low elevation and relief, and is underlain primarily by Ordovician shales which have been exposed both by recent glacial action and by earlier large-scale erosion which stripped off the Silurian and Devonian limestones. The northern part of the valley is largely blanketed by Late Pleistocene deposits of glacial outwash, deltas, and glacial lake clays. To the south, the valley narrows gradually and becomes gorge-like between abrupt uplands of hard metamorphic rocks near Poughkeepsie, New York.

The Middle section in the site region is characterized by a more typical northeasterly elongate topographic pattern of valleys and ridges resulting from differential erosion of folded sedimentary rocks, with resistant sandstones commonly supporting ridges. At its southeastern margin, the Middle section is a lowland underlain by Early Paleozoic limestone and shale, bounded by the abrupt slopes of the Reading Prong-Hudson Highlands.

6. Appalachian Plateau Province

The Catskill Mountain section lies west of the Hudson Valley and extends as a salient into the Appalachian Plateau. This area of somewhat mountainous relief consists of a maturely dissected, slightly higher plateau which reaches an elevation of approximately 4,000 feet. The underlying bedrock, sedimentary formations of Middle and Upper Paleozoic age, is more deformed than those of the uplands to the west. The mountains owe their prominent relief to a resistant coarse sandstone and conglomerate caprock (Catskill Formation). The area has been glaciated, and glacial deposits abound in the deep and prominent steep-sided valleys.

The southern New York section (in the northern part of the Appalachian Plateaus in the site region) was formed by dissection of the uplifted flat-lying sandstones and shales of the Devonian Catskill Delta. Relief is moderate to high. Westward beyond the site region the upland surface is represented by flat-topped divides. Drainage is generally to the southwest.

The Mohawk section, a lowland resulting from erosion along an outcrop belt, lies between the Adirondacks and the Helderberg escarpment. The belt is commonly of low elevation and relief, underlain by relatively soft or nonresistant Ordovician shales which have been exposed by early large-scale erosion, which stripped away the overlying Silurian and Devonian sandstones, and by Pleistocene glacial action. The Mohawk Valley is largely blanketed by deposits of Late Pleistocene outwash, deltas, and lake clays.

7. Adirondack Province

The Adirondack Province is a mountainous glaciated uplift region in which peaks are commonly well rounded by erosion. Many peaks reach elevations of about 4,000 feet, with two peaks over 5,000 feet. The province merges into the plains of the St. Lawrence River valley to the north and west, and into the Mohawk River valley to the south. Eastward into the block-faulted terrane of the Champlain Lowlands the slope is more abrupt.

The Adirondack Mountains are supported by crystalline Precambrian metamorphic and igneous rocks, and are transected by many prominent northeasterly trending lineaments which commonly reflect shear zones or major faults. These lineaments frequently control drainage and the landforms. Many lakes follow geologic contacts, or are in valleys along weak rock units. Young glacial deposits clog the normal radial drainage, and lower areas are dotted with lakes, ponds, and swamps.

8. <u>St. Lawrence Lowlands Province</u>

The northwestern physiographic province in the site region includes the St. Lawrence River Valley, the low hills south of the river valley, and the Lake Champlain Valley. The underlying rocks, Cambrian and Ordovician sandstones, dolomites, and limestones, dip gently away from the Adirondacks. Relief is approximately 100 feet. Streams draining the northern and eastern slopes of the Adirondacks flow across the province. The shoreline of Lake Champlain is largely controlled by north-south and east-west faults which have broken the Paleozoic sandstones and carbonates into large blocks. Bedrock of the St. Lawrence Valley is blanketed by fine-grained, glacio-marine and glacio-lacustrine sediments of Late Pleistocene age.

9. Physiographic Development

The development of the physiographic features characterizing the site region was initiated at the close of the Mesozoic era. Following peneplanation, the region was elevated and subjected to subareal weathering, erosion, and dissection of the peneplain surface. Sediments transported from the landmass during this time were carried seaward to form the Coastal Plain sedimentary deposits. Crystalline basement rocks underlying the elevated landmass in New England were deeply weathered, and in the site area schistose metamorphic bedrock underwent more extensive and deep weathering than the adjacent intrusive plutonic rocks.

Following the long period of Cenozoic weathering and degradation of the landmass, successive advances of continental glaciation occurred during the Pleistocene epoch. The ice sheets removed the residual soils and loose weathered bedrock surface, and on withdrawal/melting deposited a ground moraine of generally stony till on the scoured bedrock surface. Locally, the morainal deposits are overlain by ice-contact and outwash deposits. Depression of the landmass by the weight of thick glacial ice, combined with a rise of sea level due to the melting of that ice sheet, resulted in submergence of wide areas of the lowlands. Rock flour released from the melting ice was deposited on the undulating surface of the submerged lowlands and valleys as a blanket of marine clay-silt, or as lake deposits along the major river valleys. Crustal rebound, following the removal of the last glacial ice, elevated the upper surface of the marine clay-silt blanket and lake water-plane deposits above sea level by as much as several hundreds of feet. The reworking of glacial deposits in Recent time has produced sand beaches along the coastline. In low-lying areas adjacent to the coast, tidal marshes have been built on the surface of earlier outwash and glacio-marine clay-silt deposits.

b. Regional Surficial Geology

Igneous bedrock crops out at the site and at the entrance to Hampton Harbor, 2 miles to the east, while the adjacent metamorphic schist bedrock commonly forms bedrock valleys deeply buried beneath deposits of till, outwash and glacio-marine clay-silt. Between the site and the coastline, a broad surface of tidal marsh overlies old sandy beach deposits, sandy outwash and fine-grained marine clay-silt deposits. These unconsolidated sediments locally exceed 140 feet in thickness.

The distribution of surficial deposits on the land area within a 5-mile radius of the site is shown on Figure 2.5-6 (after Bradley, 1958). The deposits are all of Late Wisconsinan age (about 25,000 to 13,000 years old) or younger and consist of Wisconsinan lodgment till overlain progressively by postglacial ice-contact, marine clay-silt and sandy beach deposits, and by recent organic swamp or marsh accumulations and sandy beach deposits. All seismic Category I site facilities are designed to be built on bedrock foundations.

The surficial deposits in the site region are glacially derived and veneer the landmass and much of the Gulf of Maine. They were deposited primarily by the Late Wisconsinan continental ice sheet and the meltwaters of the receding ice. The upland and mountain areas are characterized by a thin veneer of glacial till with interspersed bedrock exposures. Ice-contact and outwash sands and gravel were deposited locally along valleys and are sometimes associated with clay-silt deposits, 9,000 to 10,000 years old. The Seaboard Lowland is characterized by wide areas of glacio-marine clay-silt (rock flour) and by extensive deposits of ice-contact and outwash sands overlying till. Seismic reflection surveys in offshore areas indicate that till, ice-contact, outwash, and glacio-marine clay-silt deposits are also distributed throughout the northern marine sector. The southern terminus of the last major glacial advance is defined on Cape Cod and along the southern New England coast and islands by east-west elongate deposits of terminal moraine tills. To the south of the glaciated region, the continental shelf is blanketed by a veneer of Recent clastic sediments, with local occurrences of deep channel fillings on an irregular pre-Pleistocene erosion surface.

No geologic, seismic, or manmade hazards of safety significance to the site are known or inferred to relate to regional or local surficial geologic features. There are no areas near the site that are currently undergoing intense erosion.

c. Regional Bedrock Geology

1. Introduction

The site is situated on Early Paleozoic intrusive igneous rocks of the Newburyport pluton in the Coastal Anticlinorium tectonic province. The regional bedrock geology surrounding the site is shown on Figure 2.5-2. A schematic regional geologic profile depicting major bedrock and structural elements is shown on Figure 2.5-3. Regional tectonic elements, faults and major crustal basement terranes are shown on Figure 2.5-4. Regional tectonic provinces are delineated by an overlay on the tectonic base on Figure 2.5-5.

2. Bedrock Geology

The bedrock basement, within 200 miles of the site, is comprised of five distinctive crustal blocks which range in geological age of formation from Late Precambrian to Middle Paleozoic, and consist predominantly of hard, crystalline metamorphic and igneous rocks of orogenic origin (Figure 2.5-2, Figure 2.5-3, and Figure 2.5-4).

Unmetamorphosed sandstones, shales, and carbonate rocks of Lower Paleozoic miogeosynclinal origin overlie the Precambrian (Grenvillian) crystalline basement in the area to the west of New England, and are in turn overlain through a broad region by Middle Paleozoic sandstones and shales of platform and deltaic origins. Mildly metamorphosed to unmetamorphosed, well consolidated, Late Paleozoic conglomerates, sandstones, shales, and local coal seams of continental origin occur locally in faulted intermontane basins on Late Precambrian (Avalonian) crystalline basement to the south of the site in the Boston, Narragansett, and outlier basins; possibly in isolated basins within the Gulf of Maine area; and in broad basins and localized fault slices in southwestern New Brunswick and eastern Maine. Unmetamorphosed conglomerates, sandstones, shales, and volcanic rocks of Triassic and Jurassic ages and of continental origin occur in rift basins in the Early Paleozoic crystalline basement in southeastern New Jersey, central Connecticut and Massachusetts, in the Bay of Fundy area, and probably in scattered locations within the Gulf of Maine and beneath the Continental Shelf.

The youngest crystalline rocks in New England are alkaline ring complexes of Permian to Middle Cretaceous ages, the White Mountain Plutonic Series, which discordantly intrude both Early Paleozoic and Middle Paleozoic crystalline basement to the north and east of the site, in a zone which trends south-southeasterly through New Hampshire and southwestern Maine to offshore northeastern Massachusetts. Isolated plutons with White Mountain Plutonic Series affinities also occur in these basement rocks in southeastern Vermont, southwest-central Maine, and near the Maine border in southeastern Quebec. Alkaline plutons of Middle Cretaceous age, the Monteregian Hills Plutonic Series, intrude both Early Paleozoic and Late Precambrian (Grenvillian) basement rocks in and adjacent to the block-faulted embayment near Montreal in southern Quebec, and at two or more isolated localities in northwestern Vermont and northeastern New York. Numerous small mafic dikes of both Triassic and Middle Cretaceous ages occur widely in the region, and are particularly abundant in southeastern Quebec, Vermont, New Hampshire, western Maine, and through eastern Massachusetts into central Connecticut. Mafic dikes of both Early Triassic and unknown premesozoic ages occur at the site and on Cape Ann, Massachusetts, to the south of the site.

Bedrock formations younger than Middle Cretaceous are rarely found on land areas in the region to the north of eastern New Jersey and southernmost New England. Offshore to the south of New England, the Continental Shelf is comprised of loosely consolidated Cretaceous and Tertiary Coastal Plain sediments overlying Mesozoic or older basement rocks. Patches of Tertiary Coastal Plain sediments occur on Triassic and older basement rocks in the Gulf of Maine. The entire 200-mile region is widely covered by a thin veneer of loose, unconsolidated sediments of Quaternary age, derived from Pleistocene continental glaciation and post-glacial deposition.

3. Bedrock Structure

The fabric of bedrock structure in much of New England is characterized essentially by a series of elongate belts of folded and faulted metamorphic rocks and included plutonic masses of Early to Middle Paleozoic age. The more westerly of these foldbelts strikes as a group of discrete anticlinoria and synclinoria from southwestern Connecticut northerly through Massachusetts and Vermont into southeastern Quebec. The eastern foldbelts trend north through eastern Massachusetts from southern Connecticut, and swing gradually to the northeast through New Hampshire into Maine. The New England Paleozoic foldbelts are terminated to the southeast along a fundamental fault boundary. The basement in the land area of eastern Connecticut, Rhode Island, and southeastern Massachusetts, to the southeast of the fault boundary, is characterized by a Late Precambrian (Avalonian) granitic platform terrane which does not exhibit evidence of the Paleozoic orogenic deformations which created the New England foldbelts.

To the west of the highly deformed foldbelt terrane of New England and its southwesterly projection through southeastern New York and New Jersey, essentially flat-lying Paleozoic sedimentary rocks rest unconformably on Late Precambrian (Grenvillian) basement. These sedimentary formations generally exhibit only localized, minor deformation in the form of warping or open folding and, in the southern Quebec embayment and around the edges of the Adirondack uplift, block faulting.

Numerous major fault structures of great lateral extent are found throughout the region of the New England foldbelt terrane, commonly trending with the northeasterly pattern of the folded bedrock structure. Faulting of bedrock in the region ranges in age of development from Early Paleozoic to Upper Mesozoic. Early and Middle Paleozoic faulting is commonly characterized by low-angle thrusts and high-angle reverse faults developed during successive episodes of orogenic compression. Late Paleozoic faulting is characterized by a complex of thrust faulting with a right-lateral strike-slip component along the boundary zone between the foldbelt terrane and the Precambrian platform of southeastern Massachusetts, and by high-angle, right-lateral, strike-slip displacements along the northeast-trending fault system in southern Maine and southwestern New Brunswick. Mesozoic faulting is commonly seen as high-angle, normal displacements related to post-orogenic crustal uplift and tensional stresses associated with the last opening of the Atlantic Ocean.

There are no known or inferred tectonic faults displacing Quaternary glacial deposits or post-glacial and recent sediments, nor has any tectonic fault activity been shown to have occurred in the region in the past 100 million years since late in the Mesozoic era.

d. Regional Tectonics

1. Introduction

The major tectonic elements of the site region are defined on Figure 2.5-4. On Figure 2.5-5, boundary lines are overlaid on the tectonic elements map to partition the region into major tectonic provinces having unique geologic origins or exhibiting distinctive structural or geophysical characteristics. These provinces were formed by fundamental tectonic episodes which occurred at times in the geologic past ranging from about 100 million years ago to more than 500 million years ago, in response to stress regimes which are not active today. Some of the provinces have undergone major deformational effects from two or more different stress regimes; some have experienced only minor or localized tectonic modification in the course of as much as 1 billion years.

As shown on Figure 2.5-5, the region within 200 miles of the site is divisible into 12 major tectonic provinces. The division is defined first, by the geographic distribution of five fundamental crustal blocks which, as discussed in Subsection 2.5.1.1e, were separately created by orogenic episodes approximately 1,100, 600, 480, 450, and 360 million years ago. Each of these five basic blocks is characterized by geologic, lithologic, and structural features which are unique to it, and which terminate abruptly, commonly along faulted boundaries, against the neighboring crustal blocks.

The basic province divisions are then modified to delineate areas or regions in which portions of the basic crustal blocks or overlying platform deposits have been substantially altered and deformed by subsequent major tectonic forces. These superimposed tectonic provinces geographically delineate the areas in which portions of the crustal blocks have been extensively broken by post-consolidation transcurrent faulting, or by deep-seated block faulting associated with repeated crustal uplift or subsidence, or where sedimentary formations above the crust have been faulted and/or folded.

In some of the major tectonic provinces, it is possible, as shown on Figure 2.5-5, to define provincial subdivisions on the basis of specialized subregional structural or historical geologic features. These specialized geologic features may either have been intimately associated with the overall historical and structural development of a given crustal block or province itself, or have been developed in response to localized stress regimes.

Each province appears to have within it a reasonable degree of consistency of specific structural or geophysical features impressed upon it by ancient compressional or tensional stress regimes. Although the provinces, as defined on Figure 2.5-5, are reflective of ancient stress regimes, they are, per se, only indirectly reflective of modern, relatively low-magnitude crustal stresses. Of greater importance than broad provinces in influencing the present-day stress field are specific zones of crustal weakness, or localized discontinuities in rock density, rigidity or geometry on which stress tends to concentrate.

The site region is partitioned into the following 11 tectonic provinces, with geologic subprovinces as shown:

(a) Coastal Anticlinorium - Site Province

(b) Thrust Complex

(c) Southeastern New England Platform

- Long Island Shelf

(d) Long Island Platform

(e) Merrimack Synclinorium

- White Mountain Plutonic Series

(f) Western New England Foldbelt

- Middlebury Synclinorium
- Green Mountain Anticlinorium
- Connecticut Valley Synclinorium
- Bronson Hill Anticlinorium

(g) Western Quebec Seismic Zone

- Monteregian Hills Plutonic Series

(h) Adirondack Uplift

(i) Appalachian Plateau

(j) Valley and Ridge

(k) New York Recess

East Coast Gravity High

2. Coastal Anticlinorium (Site Province)

The site is located on crystalline igneous rocks of the Newburyport pluton in the southern part of the Coastal Anticlinorium tectonic province. Rocks of the site province are characterized by sequences of metavolcanic and metasedimentary formations interpreted to be of Cambro-Ordovician age (Billings, 1956; Brookins and Hussey, 1978; Osberg, 1974), overlain by quartzites, slates, phyllites, and calc-silicate rocks which have previously been considered of Siluro-Devonian age (Billings, 1956; Hussey, 1962; Doyle, 1967; Novotny, 1969), but which recently have been interpreted to be of Early Paleozoic age or older (Aleinikoff and Zartman, 1978; Lyons, Boudette and Aleinikoff, 1979).

The province has been subjected to both pre-Silurian and locally Acadian (Devonian) orogenic folding, intrusive activity and metamorphic deformations, all followed by post-metamorphic left-lateral kink-banding (Hussey, 1978) and right-lateral strike-slip faulting (Wones and Stewart, 1976) of Late Paleozoic ages. Intermittently from Late Paleozoic to Middle Mesozoic time, the older rocks of the province were intruded by a few isolated, discordant plutons of the White Mountain Plutonic Series. In Early Triassic time, the southern part of the province from the site area north to Portland, Maine, was extensively intruded by predominantly northeast-trending mafic dikes. Foliation fabric in the province commonly strikes quite uniformly to the northeast, parallel to post-metamorphic major faults.

3. Thrust Complex

The Coastal Anticlinorium is bounded to the southeast, in northeastern Massachusetts, by an arcuate thrust fault complex of Late Paleozoic age which encloses Early Paleozoic metamorphic rocks and Middle Paleozoic volcanoclastic rocks of Coastal Anticlinorium affinities, Early Paleozoic intrusive rocks and, locally, blocks and slices of Late Precambrian (Avalonian) lithologies of the southeastern New England Platform (Cameron and Naylor, 1976; Nelson, 1976; Schutts et al., 1976). The most intense deformation in this complex occurs in northeastern Massachusetts, from 7 to 30 miles to the south of the site, and reflects a collision boundary between the crustal plate of the Avalonian platform to the south and that of the Coastal Anticlinorium to the north.

The frequency of faulting in the Thrust Complex diminishes along the zone as it curves to the southwest and trends into eastern Connecticut. The fault zone is characterized in northeastern Massachusetts by numerous, closely-spaced, east-northeast-striking, moderately north-dipping thrust faults which display an aspect of right-lateral strike-slip movement (Skehan, 1968; Alvord et al., 1976; Dennen, 1976). The two major faults of the complex are the Clinton-Newbury fault, which delimits the northern boundary, and the Bloody Bluff fault, which delimits the southern boundary of Coastal Anticlinorium lithologies. The southern boundary of the Complex is defined by the North Border fault of the Boston basin. Aeromagnetic data (Boston Edison Company, Pilgrim II, PSAR, 1976) suggest that the fault complex converges to the east of Cape Ann, Massachusetts.

Based on distinctive differences in metamorphism and in the hydrothermal alterations and fundamental chemistry of rocks on either side of the Bloody Bluff fault, the Bloody Bluff fault is interpreted to represent the greatest of all faults in the Complex, and to mark the northern boundary of the Avalonian terrain (Cameron and Naylor, 1976; Schutts et al., 1976; Nelson, 1976). Radiometric dating of cataclastic rocks in the Clinton-Newbury fault zone, at the northern boundary of the Complex, indicates a Middle Permian age of last faulting (Public Service Company of New Hampshire, Seabrook PSAR, 1974).

Magnetic surveys (Boston Edison Company, Pilgrim II PSAR, 1976) indicate the presence of a cylindrical mafic pluton in the fault complex just offshore to the north of Cape Ann, about 15 miles southeast of the site.

On its projection to the southwest, the Thrust Complex narrows and loses its ground-surface identity on a relatively simple, low-angle thrust zone, the Lake Char fault, trending south just to the west of the Connecticut-Rhode Island border. The upper plate of this thrust consists of metamorphic rocks of Coastal Anticlinorium affinities which lie as a thin-skin cover over older Avalonian rocks (Dixon and Lundgren, 1968; Pease and Fahey, 1978).

4. <u>Southeastern New England Platform</u>

The south of the North Border fault of the Boston Basin, the Southeastern New England Platform consists largely of Late Precambrian-Early Paleozoic (Avalonian) granitic basement, with supracrustal basins containing continental sedimentary rocks (with minor volcanic members) which range in age from older Paleozoic in the Boston Basin to Carboniferous in the Narragansett and neighboring basins in Rhode Island and southeastern Massachusetts. The Platform is relatively little deformed and does not show evidence of Acadian orogenic deformation. In the Boston Basin, the sedimentary rocks have been folded and thrust faulted from the south, with apparently thin-skinned tectonic deformation (Billings, 1976). In the southwestern part of the Narragansett Basin, in southeastern Rhode Island, deformation of the Carboniferous sedimentary rocks includes folding, metamorphism, and two episodes of east-west thrusting. In eastern Connecticut, the Precambrian rocks of the Southeastern Platform underlie a thin cover of pre-Silurian rocks beneath the Lake Char and Honey Hill fault surfaces. Most of the Platform rocks have been affected by an Alleghanian thermal or metamorphic event, locally including granitic plutonism. The Platform has not, however, been deformed internally by throughgoing crustal fault structures, and there is no evidence within the Platform of deformational effects derived from or associated with the intense Acadian orogeny which folded, metamorphosed, and faulted the terranes to the north and west during Middle Devonian time (Cameron and Naylor, 1976; Nelson, 1976; Schutts et al., 1976). Based on paleomagnetic and radiometric research, Schutts et al., (1976) propose that the Southeastern Platform was part of a crustal block which was separate from North America during the Devonian.

The fundamental crustal boundary along the northwestern edge of the Southeastern Platform is interpreted to trend southwesterly on the projection of the Bloody Bluff fault, past the north side of the Willimantic Dome toward the southern end of the Juro-Triassic Connecticut Valley basin. Approximately 16.5 miles northeast of New Haven, Connecticut, geologic, magnetic, and gravity data indicate a strong northwest-trending structural fabric which parallels the Connecticut River, and transects the southwesterly projection of the Platform boundary.

Offshore to the south, in the area of the Long Island Shelf (Schlee, 1977), the Platform basement slopes to the south and is blanketed by a seaward thickening wedge of loosely consolidated Coastal Plain sediments of Cretaceous and Tertiary ages. Sheridan (1974) interprets the basement of the Southeastern Platform to extend roughly 100 kilometers south of the southern New England shoreline. Aeromagnetic patterns (U.S. Geological Survey, 1976) suggest that in wide areas of the Long Island Shelf, Mesozoic volcanic rocks immediately underlie the Coastal Plain sediments (Valentine, 1978).

The only mapped fault structure in the Coastal Plain sediments in the site region is the New Shoreham fault (McMaster, 1971). Detailed seismic surveys by Weston Geophysical Corporation (New England Power Company, NEP 1 and 2 PSAR, 1978) reveal clearly that Cretaceous and presumed Tertiary sediments have been deformed along the zone, and that the underlying "basement" reflector is offset, down to the east, by as much as 130 feet. Although these geophysical surveys were not able to discern whether deformation of the sediments was related to tectonic faulting or merely to differential settlement of the sedimentary column across a buried topographic escarpment, they were able to demonstrate that sediment deformation along the feature occurred more than 120,000 years ago, and possibly as much as 20 million years ago.

5. Long Island Platform

The northern boundary of the Long Island Platform (Klitgord and Schouten, 1977; Schlee, 1977) is defined by sharp offsets in the continental crust along a zone of block faulting. The southern boundary (beyond the southern edge of Figure 2.5-5) is defined by the "east coast magnetic anomaly" (Taylor et al., 1968). The Platform itself is considered to be a series of graben and horsts whose axes are parallel to the Baltimore Canyon trough and the Georges Bank trough (Klitgord and Schouten, 1977). Sheridan (1974) has interpreted the younger basement beneath Coastal Plain sediments in the Long Island Platform area to be Jurassic evaporite, carbonate, and terrigenous deposits, more than 20,000 feet thick, overlying Triassic sedimentary rocks in a down-faulted basin in the older-basement crystalline rocks. The location of the inferred northern boundary of the Province as shown on Figure 2.5-5 is only approximate, having been estimated from Sheridan's (1974) very small-scale regional maps.

6. Merrimack Synclinorium

The Coastal Anticlinorium (site province) is bounded to the northwest by the Merrimack Synclinorium, a distinctive crustal block which was consolidated by intense compressional forces during the Acadian orogeny in Early-Middle Devonian time. Bedrock in the Synclinorium is made up predominantly of a great thickness of clastic eugeosynclinal metasedimentary rocks with minor calcareous units and few metavolcanic rocks, widely intruded by granitic masses (Billings, 1956). Subsequent to its consolidation as a crustal block, the Synclinorium was deformed by Late Paleozoic transcurrent faulting and by Permo-Triassic to Middle Cretaceous volcanism and discordant emplacement of plutons of the White Mountain Plutonic Series. The boundary zone between the Merrimack Synclinorium and the Coastal Anticlinorium site province is characterized by a system of northeast-trending, post-metamorphic faults.

The province is characterized by distinctly different major structural features than those of the surrounding provinces, particularly with respect to its transverse, northwest-striking fold trends, northwest-trending elongations of many Devonian granitic plutons, and northwest-trending gravity patterns (Englund, 1976). In central New Hampshire, the province contains a large physiographic-geologic-aeromagnetic anomaly, enclosing the Ossipee, Belknap, and Merrymeeting Lake Mesozoic plutons, which has been interpreted as a collapsed volcanic caldera (Boston Edison Company, Pilgrim II PSAR, 1976). In southwestern Maine, the Lewiston-Pittsfield fault zone experienced substantial post-Acadian transcurrent movement (Dallmeyer and VanBreeman, 1978); similar offsets of metamorphic isograds (Morgan, 1972) on mapped fault zones about 30 miles to the northwest of the Lewiston-Pittsfield structure suggest a similar style of transcurrent crustal deformation in that area. Two Late Paleozoic alkaline plutons of White Mountain Plutonic Series affinities occur in close spatial association with the Lewiston-Pittsfield structure in southwest-central Maine.

The White Mountain Plutonic Series is an elongate north-northwesterly grouping of alkaline, central complex intrusive plugs or stocks which occur on the land area predominantly within a zone trending for 140 miles through southwestern Maine and eastern New Hampshire. Regional aeromagnetic surveys suggest that the southern terminus of this main zone occurs on a cylindrical pluton immediately offshore to the north of Cape Ann, Massachusetts. Isolated intrusives of the Series occur 50 miles to the east of the main zone at Androscoggin Lake and Litchfield, Maine; 40 miles to the north of the zone near Megantic, Quebec; and 50 miles to the west at Mount Ascutney, Vermont.

The age of emplacement of these intrusives ranges from Late Carboniferous to Middle Cretaceous, with the most extensive plutonism (in terms of rock volume emplaced) associated with Early Jurassic volcanic activity in north-central and northern New Hampshire. This plutonism was essentially contemporaneous with the outpouring of mafic lavas into the Connecticut Valley rift basin. The younger intrusives of the Series, about 110 to 120 million years in age, are predominantly clustered in southeast-central New Hampshire and southwestern Maine. The character of the magnetic signature of these younger mafic plutons suggests that the prominent circular magnetic anomaly just north of Cape Ann is also a member of this younger series, emplaced in the offshore projection of the intensely faulted northeastern Massachusetts zone of deformation, or thrust fault complex. In central New Hampshire, the Ossipee pluton occurs as a prominent monadnock of high relief in the northern part of a regionally anomalous topographic lowland. The bedrock geology in this same area is characterized by a pattern of Mesozoic plutons associated with a roughly elliptical distribution of granitic intrusive rocks of Middle Paleozoic age. Known faulting in this area includes a north-trending and northeast-trending silicified zone, two faults offsetting the northeast rim of the Ossipee pluton, and a N75°E fault structure which is tangent to the Ossipee pluton at South Tamworth, New Hampshire.

7. Western New England Foldbelt

The Western New England Foldbelt was formed as a crustal block by westerly directed compressional forces during the Taconic orogeny, culminating about 450 million years ago (Fisher and McLelland, 1975; Bence and Rajamani, 1972). The Foldbelt is made up of a series of four major north-trending components defined, from west to east, as the Middlebury Synclinorium, the Green Mountain Anticlinorium, the Connecticut Valley Synclinorium, and the Bronson Hill Anticlinorium.

Bedrock in the Middlebury Synclinorium consists of miogeosynclinal sandstones and carbonate rocks locally overlain by huge allochthonous masses of eugeosynclinal slates and quartzites, all of Cambrian and Early Ordovician age. In the Green Mountain Anticlinorium, thrust slices and arches of Late Precambrian (Grenvillian) crystalline rocks are overlain on the east by Cambro-Ordovician shelf and eugeosynclinal metasedimentary rocks with local metavolcanic units. In the Connecticut Valley Synclinorium, a Cambro-Ordovician eugeosynclinal basement terrane is overlain by Siluro-Devonian miogeosynclinal and euogeosynclinal metasedimentary rocks and minor metavolcanic members. The Bronson Hill Anticlinorium is characterized by Late Precambrian (Avalonian(?)) and Early Paleozoic plutonic gneiss domes overlain by Ordovician volcanic and clastic rocks of island arc derivation.

Subsequent to its consolidation to a crustal block in Ordovician time, the province was locally metamorphosed and thrust-faulted during the Acadian orogeny; broken by simple, widely-spaced normal faults in Late Paleozoic-Lower Mesozoic time; and intruded by a few discordant ring-complex plutons during Late Mesozoic time.

8. Western Quebec Seismic Zone

The Western Quebec Seismic Zone in the site region is characterized by a central, essentially undeformed sequence of Cambrian-Ordovician sandstones, shales, and limestones and a broad belt of Precambrian Grenville-age rocks, bordered to the north and south by highly-deformed, Grenville-type rocks of the Laurentian and Adirondack Mountains. The zone is broadly defined as a tectonic province on the basis of the distribution of anomalously frequent and occasionally high-intensity earthquake activity.

The zone is marked by numerous high-angle faults, including the Winchester Springs and Gloucester faults in the site region, and the Ottawa-Bonnechere graben beyond the site region to the northwest. Maximum displacement on the faults is approximately 1,700 feet. Faults trend predominantly northwest, and swing to the northeast near Montreal. Associated with the faults are numerous mantle-derived alkaline intrusives, carbonatites, mica-periodotite pipes and diatreme breccias, ranging in age from Precambrian to Cretaceous.

9. Adirondack Uplift

The Adirondack Uplift province is a domical region of exposed high-grade (granulite facies) gneisses, syenite, and anorthosite, overlapped on the east and south by Cambrian and Ordovician platform sedimentary rocks. Radiometric dates of rocks in the province range between 1,020 to 1,100 million years, about 100 million years younger than Grenville Group metasedimentary and metaintrusive rocks of the lowlands to the west, and record the age of magmatic crystallization and granulite metamorphism in the Uplift (King, 1976). The province boundaries are delineated on (1) the northwest by the Highland Boundary fault; (2) the north by the Western Quebec Seismic Zone; (3) the east by the termination of block faulting against the Cambro-Ordovician Taconic block; and (4) the south and west by the apparent termination of exposed pre-Devonian normal fault structures in Ordovician sedimentary rocks. The structural feature of essential significance in defining the tectonic province, and in differentiating it from its neighboring provinces is the character of closely-spaced, north- to northeast-trending block faulting of the crystalline mass.

The crystalline rocks of the Uplift are closely faulted internally, and some of the faults extend through Cambrian-Ordovician sedimentary rocks which lap onto the Uplift on the southwest, south, and east sides. Block faulting in the Mohawk, Hudson, and Champlain valleys, to the south and east of the Uplift, is interpreted to be of Early Silurian(?) age, associated with a doming episode of the Uplift, and subsequent doming may have occurred in Late Silurian through Lower Devonian time (Fisher et al., 1970).

Fault movements younger than Middle Paleozoic have not been reported for areas within the central part of the Uplift. In the eastern part of the province, however, Cady (1969) described east-west cross faults of the Champlain fault system as at least of Mesozoic age, and Fisher et al., (1970) and McHone (1978) report the emplacement there of Mesozoic dikes, some of which are themselves faulted. Burke (1977) has hypothesized that a 2-kilometer uplift of the Adirondacks in Miocene-Pliocene time could have reactivated the "Champlain-Lake George rift system."

Isachsen et al., (1978) reported that releveling surveys suggest that the Adirondack Mountains dome, which formed sometime later than Upper Devonian time, is currently undergoing uplift at the rate of 3-4 mm/year, although investigations specifically to detect recent surface movements on faults within the Uplift have not yet been successful.

10. <u>Appalachian Plateau</u>

The Appalachian Plateau in the site region consists of little-deformed Paleozoic platform sedimentary formations of Cambrian to Devonian age resting upon a south-dipping Precambrian crystalline basement block of Grenvillian age. Several normal faults of pre-Devonian age have been inferred to cut Ordovician and older rocks beneath unfaulted Middle Paleozoic sedimentary cover in the northern part of the province (Isachsen and McKendree, 1977). The boundaries of the province in the site region are defined by topograhic (erosional) escarpments along the north and northeast sides, and by a general zone along the south edge where the more prominent fold and thrust structures of the Valley and Ridge province die out.

11. <u>Valley and Ridge</u>

The Valley and Ridge tectonic province involves Cambro-Ordovician miogeosynclinal sedimentary rocks on the southeast, and Cambrian to Pennsylvanian platform sedimentary rocks to the northwest, all of which have been folded and thrust-faulted toward the northwest. These fold and thrust tectonics are thin-skinned, and are not believed to have involved remobilization of the underlying Grenvillian basement block, ranging to more than 30,000 feet beneath the surface rocks.

The major deformation in the southwestern part of the province lying within the site region is of apparent Late Paleozoic (Alleghanian) age (Drake, 1970).

In the narrow northeastern arm of the province delineated on Figure 2.5-5 along the Hudson River and eastern edge of the Appalachian Plateau (Catskill Mountains), from Albany southward, a chain of fold-and-thrust structures has been variously defined as of Acadian age (Woodward, 1957; Bird and Dewey, 1975; Ratcliffe et al., 1975), or of Alleghanian age (Sanders, 1969). Sanders noted that this fold-thrust zone dies out northward in the area of Clarksville, New York, west of Albany.

In the "Little Mountains" (about 55 miles south of Albany), Sanders (1969) described thrust faults which are not only folded themselves, but which dip westward, apparently passing beneath strata which underlie the gently east-west folded Devonian deposits of the Catskill Mountains. Sanders has also noted vertical folding of Triassic deposits in southeastern Pennsylvania, and suggested that a major compressional deformation may have been imparted to Valley and Ridge rocks after Late Triassic and prior to Late Cretaceous time. Fisher et al., (1970) describe a graben of Silurian-Devonian strata in the Valley and Ridge about 70 miles south-southwest of Albany as probably a fault trough of Triassic age.

Regardless of age, the general style of compressional fold and thrust deformation in this narrow northeastern arm of the province is comparable to that of the classic Alleghanian deformation in the Valley and Ridge farther to the southwest, and it, in turn, is broken here by numerous normal faults (Sanders, 1969; Fisher et al., 1970), unlike structural features reported in neighboring provinces.

12. <u>New York Recess</u>

The province consists of Cambro-Ordovician geosynclinal deposits and included Precambrian thrust slices which were consolidated to a crustal block during the Taconic orogeny; locally deformed and metamorphosed by the Acadian orogeny; compressionally faulted, intruded and thermally altered by the Alleghanian orogeny; broken by normal faulting and intruded by mafic dikes during Triassic continental rifting; and finally, subjected to three episodes of large-scale, left-lateral folding and strike-slip faulting in Late Jurassic time.

As delineated on Figure 2.5-5, the New York Recess tectonic province includes that area of the present Taconic crustal block which has been subjected to major fault deformation at recurring intervals from Late Precambrian to Middle Mesozoic time. No other area of the site region has experienced comparable deformations, nor does any other area exhibit the geometric characteristics of a continental recess, subjected repeatedly to the most extreme strains as a sequence of continental collisions were driven against it intermittently throughout Paleozoic time.

Burke and Dewey (1973) described a mechanism for continental separation at angular junctions over plume-generated "hot-spots." They suggested that "bends in continental margins commonly mark the sites of triple junctions and, further, that these bends ... are inherited from irregularities in the continental margin formed at (earlier) opening..." of ocean basins. They cited the area of Long Island, New York, as a four-armed junction consisting of the Connecticut and Newark graben and two contintent margin flexures.

Rogers (1975) pointed out that salients (bends convex toward the craton) and recesses (bends concave toward the craton) are prominent features along the cratonal margin of the Appalachian Mountain chain, further noting that the recesses are relatively angular bends where structural trends from the two sides intersect, and that few individual structures continue through from one side of a recess to the other.

The term "New York Recess" was first used by Rankin (1976) to describe a triple junction there whose arms have been carried away; he further noted that the area between western Massachusetts and eastern Pennsylvania stood structurally high during most of the early Paleozoic, and coincides with the New York Recess.

The structural history of the New York Recess dates from the time of its formation as a continental "headland" with the opening of Iapetus, a proto-Atlantic Ocean, in Late Precambrian time. Ratcliffe (1971, 1977) has reported repeated movements, both compressional and extensional, on the Ramapo fault system in southeastern New York in Late Precambrian, Ordovician to pre-Middle Devonian, Carboniferous, Triassic, and Jurassic times (the latter two, only west of the Hudson River). Long and Kulp (1962) report a "true" age of the Precambrian rocks in the Hudson Highlands of southeastern New York at 1,150 million years, with a pronounced metamorphic event at about 840 million years and, immediately south of the Ramapo-Canopus fault system, a resetting of ages at about 360 million years, a time for which Mose et al., (1976) reported igenous activity and brittle fracture.

In the region of northeast New Jersey and eastern Pennsylvania, Drake (1970) has reported Alleghanian folding, faulting, and northwestward transport of Precambrian rocks in the Reading Prong. Pronounced Alleghanian metamorphism and igneous activity in southern and southwestern Connecticut, following Acadian and Taconic metamorphic events, has been documented by Clark and Kulp (1968). Juro-Triassic faulting and volcanism, associated with rift development of the Connecticut Valley Basin, are particularly pronounced in the southern part of the basin, in south-central and southwestern Connecticut (de Boer, 1968). The final known compressional deformation in the Newark and Connecticut rift basins and their basement rocks occurred between mid- and final Jurassic time, with three large-scale, left-lateral, strike-slip couples (Sanders, 1977; Dewey, 1977).

e. Regional Geologic History

1. Introduction

The bedrock of the site region ranges in age from Late Precambrian-Grenvillian (about 1,100 million years) to Middle Mesozoic (about 110 million years). This basement complex of hard, predominantly crystalline and competent rock is covered locally in offshore areas by loosely consolidated sediments ranging in age from Upper Mesozoic (about 100 million years) to Upper Cenozoic (about 10 million years). These older basement and platform formations are, in turn, widely covered by a relatively thin veneer of loose, unconsolidated sediments of Late Pleistocene to Recent ages (about 70,000 years to the present).

The major historical tectonic episodes which have created the structural configuration of the site region include five periods of orogenic deformation during which distinctive crustal blocks were created, ultimately to be joined by pre-Mesozoic compressional forces to form the present continental crust of the land area. Subsequent to the orogenic creation of the five major crustal blocks, Late Paleozoic continental collisions and Mesozoic extensional tectonics superimposed brittle-fracture deformations on portions of the several crustal blocks, and locally initiated volcanic activity and alkaline pluton intrusions. For the past 70 million years the region has been subjected tectonically only to broad arching uplifts, followed during the past 2 million years, by successive periods of crustal depression and rebound as continental glaciers moved over the region and retreated.

2. <u>Grenvillian Crustal Consolidation</u>

Upper Precambrian geosynclinal formations now located to the west of the site in western New England, New York, and southern Quebec (Figure 2.5-4) were consolidated to a regionally extensive crustal block of gneisses, schists, marbles, and intrusive igneous rocks during the Grenvillian orogeny, about 1,100 million years ago (King, 1976). The block constitutes the crustal basement beneath Cambrian-to-Permian platform sedimentary formations throughout a broad area in New York, Pennsylvania, Ohio, and southern Ontario, and terminates against an older Precambrian crustal block along the Grenville front, about 600 miles west of the site (King, 1976).

3. <u>Avalonian Crustal Consolidation</u>

Late Precambrian rocks which now lie to the south of the site were consolidated and widely intruded by igneous rocks during the Avalonian orogeny, about 600 million years ago (Cameron and Naylor, 1976). The block constitutes the basement in southeastern New England, made up largely of little-deformed granitic rocks, and is also exposed as a remobilized gneiss dome and an apparent thrust slice, respectively, in central Massachusetts and southeastern New Hampshire (Naylor, 1976).

4. <u>Coastal Anticlinorium Crustal Consolidation (Site Province)</u>

   Early Paleozoic or older geosynclinal and volcaniclastic rocks having island arc affinities now occur discontinuously along the Maine coast, in southeastern New Hampshire, northeastern Massachusetts, and eastern Connecticut. Radiometric dating suggests that these rocks were consolidated as a crustal block by orogenic forces about 480 to 500 million years ago (Brookins and Hussey, 1978), temporarily associated with the Penobscot orogenic event of northern Maine (Hall, 1969; Neuman, 1967). The geographic location of this crustal block at the time of its creation is not known.

5. <u>Taconic Crustal Consolidation</u>

   Cambrian and Early Ordovician geosynclinal rocks which trend northerly through western New England from southeastern Pennsylvania into southeastern Quebec were consolidated into a crustal block by westerly directed compression during the Taconic orogeny, culminating about 450 million years ago (Fisher and McLelland, 1975; Bence and Rajamani, 1972). The Taconic orogeny completed a sequence of events which started in Late Precambrian time with a continental separation and ocean opening along an axis trending north through western New England (Rankin et al., 1977; Dewey and Kidd, 1974). Miogeosynclinal sands and carbonate shelf deposits formed to the west, while eugeosynclinal sands and muds were deposited on oceanic crust easterly toward an oceanic ridge-island arc chain in the area of the present Bronson Hill Anticlinorium. In Early Ordovician time, the diverging drift of crustal plates was reversed and, as the plates converged, the shelf sequence to the west was block-faulted and warped downward; the axial zone was anticlinally elevated; huge masses of eugeosynclinal sediments moved westerly as gravity slides off the anticline down into the depressed former shelf zone; slices of consolidated eugeosynclinal rocks were thrust westerly; Grenvillian basement gneisses were remobilized and thrust in imbricate fashion into the anticlinal zone; and ultramafic masses of oceanic crust were kneaded upward into the overlying water-saturated sediments on the east flank of the anticlinal zone (Bird and Dewey, 1970).

   Along the eastern edge of the orogenic zone, the island arc chain, with its distinctive Late Precambrian (Avalonian(?)) and Ordovician plutonic gneiss basement, converged over a subduction zone against the former eugeosynclinal trough to its west, and the entire Cambro-Ordovician crustal block was consolidated at the end of Taconic time and annexed as part of the North American continent (Robinson, 1978).

6. Acadian Crustal Consolidation

Silurian and Devonian rocks to the southwest, north of the site in Massachusetts, New Hampshire, and Maine were deposited largely as thick sequences of eugeosynclinal sands and muds in an oceanic trench to the east of the newly-formed Taconic crustal block (Billings, 1956). Miogeosynclinal and eugeosynclinal sediments with minor volcanic members were contemporaneously deposited in a narrow north-trending trough on downwarped Taconic basement in west-central Massachusetts through eastern Vermont into southeastern Quebec (Cady, 1969).

The eastern eugeosynclinal/oceanic depositional site was then compressed and uplifted between the Taconic block to the west and a northwesterly advancing continental block to the east (which may not have been the Coastal Antclinorium crustal block). In the salient defined by the wide part of the eugeosynclinal basin in central New Hampshire and southwestern Maine, Englund (1976) has proposed that the large central mass of uplifted sediments flowed under gravitational forces into the southwestern and northeastern gaps between the two continental plates, producing (at depth) recumbent folds with northwest-trending axes. Upon continued plate convergence, rocks of the geosyncline were successively deformed into large-scale nappes and upright folds; with intense metamorphism, faulting, and widespread granitic plutonism, the rocks then were consolidated during the Acadian orogeny to form the terminal crustal block of the region, annexed as part of the North American continent about 360 million years ago.

In the narrow Siluro-Devonian trough on the downwarped Taconic basement which extends northerly through eastern Vermont from central Massachusetts, compressional effects of the Acadian orogeny successively produced uplift, recumbent folding, mafic to calc-alkaline plutonism, gravitational uplift of domes and arches, thrust faults and finally discordant calc-alkaline plutoric activity (Cady, 1969).

7. Late Paleozoic Compressional Tectonics

In addition to the fundamental Precambrian and Early-to-Middle Paleozoic tectonic episodes which created the five basic crustal blocks of the region, a final sequence of compressional events near the close of Paleozoic time superimposed or re-activated deep-seated fault structures in several of the crustal blocks, including the Coastal Anticlinorium, in the coastal regions to the north, south, southwest, and west of the site.

In southeastern Maine and on to the northeast into Maritime Canada, Middle Devonian to Late Carboniferous tectonic history is characterized by rifting followed by southwesterly directed, right-lateral, strike-slip faulting (Belt, 1968; Dewey and Kidd, 1974; Wones and Stewart, 1976) along the general boundary zone of the Siluro-Devonian Merrimack Synclinorium and Cambro-Ordivician Coastal Anticlinorium crustal blocks. The Avalonian basement in southeastern New England, which was not in its present position there in Acadian time (Schutts et al., 1976), was transported into place adjacent to the Cambro-Ordovician crustal block along a major transcurrent fault system, of which the Bloody Bluff fault zone in northeastern Massachusetts is a primary feature (Nelson, 1976).

The close of the Paleozoic in the eastern and southeastern coastal regions is characterized tectonically by (1) the emplacement in southwestern Maine of two alkaline intrusives of White Mountain Plutonic Series affinities 297 to 244 million years ago (K-Ar dating by Stone & Webster Engineering Corporation, Montague PSAR, Appendix 27, 1974); (2) the development in northeastern Massachusetts of a complex of closely spaced thrust faults between the Clinton-Newbury fault and the north edge of the Boston Basin, overlapping the boundary of the Avalonian and Coastal Anticlinorium crustal blocks in mid-Permian time (Public Service Company of New Hampshire, Seabrook PSAR, 1974); (3) the compressional folding and thrust faulting in the Carboniferous rocks of the Boston Basin (Billings, 1976) and Narragansett Basin (Skehan et al., 1975); (4) the emplacement of the Narragansett Pier and Westerly granites in southern Rhode Island (Quinn, 1971); and (5) the numerous determinations of Permian radiometric ages on basement rocks known to have substantially older geologic ages (Zartman et al., 1970).

To the southwest of the site, Late Paleozoic compressional events which deformed the older Taconian and Grenvillian crustal blocks are evidenced by (1) a 255-million year old metamorphic imprint in southern and southwestern Connecticut; (2) the emplacement there of pegmatites and a discordant acid porphyry intrusion having an age of about 250 million years (Clark and Kulp, 1968); (3) right-lateral normal faulting and pseudotachylite development on the Ramapo fault system in northern New Jersey, dated at 259 million years (Ratcliffe, 1977); (4) widespread folding, cleavage development, northwesterly-directed thrust faulting and Grenvillian basement remobilization in and to the northwest of the Reading Prong in eastern-most Pennsylvania (Drake, 1970).

This Late Paleozoic compressional tectonic sequence has been generally defined as the "Allegheny disturbance" by Woodward (1957), and attributed by McKerrow and Zeigler (1972) and Dewey and Kidd (1974) to a collision of northern Africa with Europe and the Canadian Maritime provinces in Late Carboniferous time (Variscan orogeny), followed by the collision of Africa south of the South Atlas fault with North American south of New York during the Alleghanian orogeny in Early Permian time.

f. Mesozoic Extensional Tectonics

Attendant with the last closing of a proto-Atlantic Ocean and the compressional fracturing and metamorphic deformations derived from Late Paleozoic continent-to-continent plate collisions, the site region was uplifted and subject to subaerial erosion. The area of the Siluro-Devonian crustal block in central New Hampshire and southwestern Maine appears to have been particularly elevated and subjected to rapid erosional uncovering, in that K-Ar radiometric dating of micas from Acadian-age rocks in this wide area show Permian ages (Zartman et al., 1970), which Dallmeyer and Van Breeman (1978) have determined to reflect a time of cooling of these rocks, and not an episode of thermal metamorphism.

After the start of the Mesozoic era, a discontinuous chain of rift basins developed in the zones of Alleghanian continental suturing, generally to the southwest and east of the site along the eastern edge of the present continental landmass from Alabama to Nova Scotia. These basins locally accumulated more than 20,000 feet of terrestrial clastic sediments, including coal seams; basin development was accompanied, in Late Triassic and Jurassic times, by extrusions of basalt flows and intrusions of basalt and diabase dikes and sills (Houlik and Laird, 1977; de Boer, 1968). Toward the end of Jurassic time, following three episodes of folding and strike-slip faulting in the Juro-Triassic basins (Sanders, 1977; Dewey, 1977), extensional tectonics in the southern-most part of the site region intensified with the initiation of the opening of the present Atlantic Ocean to the south of the present shoreline. Final opening and separation of North America from northern Africa and Europe was achieved in Middle to Late Cretaceous time (Smith, 1975; Pitman and Talwani, 1972). Successive episodes of alkaline ring complex volcanic-plutonic activity of the White Mountain Plutonic Series coincide with Triassic and Jurassic intrusive activity in the rift basins, and with the Middle Cretaceous final separation of the continental masses, and represent the final important tectonic deformations to have affected the site region (Boston Edison Company, Pilgrim II PSAR, 1976).

g Cenozoic History

At the close of the Mesozoic era, the landmass of the region is postulated to have been roughly comparable, physiographically, with that of today. For the past 70 million years during the Cenozoic era, the region has been subjected tectonically only to broad arching uplifts followed by deep weathering and erosion. The Coastal Plain deposits offshore from the region were derived mainly from the transport of weathered bedrock materials from the land area. The gradually increasing seaward dip of beds in the Coastal Plain deposits provides an indication of the degree of broad gentle warping of the landmass through time.

The last episode in the geologic history of the region was a succession of continental glaciations during Quarternary time (the last 500,000 to 1,000,000 years before present). These several periods of glaciation scoured away the older Cenozoic residual soils of the region down to fresh bedrock and replaced them with deposits of till, ice-contact sands and gravels, sandy outwash deposits, and finally, post-glacial marine and lacustrine clay-silt deposits. Although materials of this type were deposited with each period of glaciation, only the products of the last glacial event, about 25,000 to 10,000 years ago, along with some recent deposits derived from their erosional reworking, are commonly observed on the land surface today throughout the region. Borings and deep excavations occasionally expose some earlier glacial deposits buried in topographic depressions on the bedrock surface, underlying the most recent deposits. No evidence has been reported to suggest that any tectonic fault displacement has occurred in Quarternary deposits in the region, although numerous examples of glacio-tectonic ("ice shove") deformations have been reported. The landmass of the region has also experienced differential upwarping or rebound, as a result of unloading after the melting and removal of the continental ice.

2.5.1.2 Site Geology

a. Site Vicinity Setting (5-8 Miles)

1. Introduction

Site Category I facilities are founded on dioritic igneous rocks and included quartzitic metamorphic rocks, both of which are transected locally by northeast-trending diabase dikes. The dioritic and quartzitic rocks are of Middle Paleozoic age or older. The diabase dikes are of two intrusive episodes, undefined Paleozoic and Early Mesozoic, respectively. The primary published geologic references for bedrock in the site vicinity (Figure 2.5-7) are Billings (1956); Hussey (1962); Novotny (1963, 1969); and Shride (1971, 1976).

The bedrock of the vicinity defined by Figure 2.5-7 consists of hard, crystalline metamorphic and igneous rocks which range in age from possible Precambrian to Early Devonian, locally transected by thin mafic dikes which are commonly of Early Triassic age, with some being of an undetermined Paleozoic age. The igneous rocks at the site are not generally subject to deep weathering effects, and do not contain readily soluble or cavernous lithologies. The bedrock surface in the site vicinity is widely veneered by relatively thin, unconsolidated Late Pleistocene (Wisconsinan) deposits of glacial till, locally overlain by postglacial sandy outwash deposits and marine clay. Recent swamp, marsh, dune, and alluvial deposits are the youngest geological materials in the area (Figure 2.5-6, after Bradley, 1958). All surficial materials have been removed in the area of the Category I facilities in order to found these structures on competent bedrock or concrete backfill. Note: Several references are made in this and subsections following to "former Unit 2" which refers to an abandoned second unit, adjacent to the operating plant. Reference is made to this second unit or its structures for geographic location only.

2. <u>Site Vicinity Surficial Geology</u>

As discussed in Subsection 2.5.1.1b, surficial deposits on the land area within a 5-mile radius of the site (Figure 2.5-6) are all of Late Pleistocene (Wisconsinan) age or younger, and consist of glacial till overlain progressively by post-glacial ice-contact sands and gravels, glaciomarine clay-silt, sandy outwash, and Recent organic marsh accumulations and sandy beach deposits. At the site, all surficial materials have been removed in order to place Category I facilities on or within bedrock foundations.

3. <u>Site Vicinity Lithologies</u>

The Merrimack Group of metasedimentary rocks of the Rye, Kittery, and Eliot Formations (Figure 2.5-7) are predominantly fine-grained, and range from quartzite, schist, and granulite, to slate and phyllite. The upper member of the Rye Formation is a metavolcanic rock consisting predominantly of medium-to coarse-grained mica gneiss, with interbeds of welded tuff, schist, quartzite, and hornblende gneiss or amphibolite. The Merrimack Group, although lacking in fossils, has historically been assigned to an Ordovician-to-Devonian age period. Recent radiometric studies, however, have indicated that these rocks are probably no younger than Ordovician and are possibly of Precambrian age (Aleinikoff and Zartman, 1978; Lyons, Boudette, and Aleinikoff, 1979).

The igneous rocks of the Newburyport pluton include a core of medium-grained quartz monzonite and granodiorite enclosed on the north and west by coarse porphyrtic granodiorite, which, in turn, grades into a zone of gneissoid quartz diorite and intrusion breccia where this phase of the pluton has intruded metasedimentary rocks of the Merrimack Group. Gneissoid quartz diorite and intrusion breccia, with local relict inclusions of metasedimentary rocks, are the predominant bedrock types on which the site is founded. The age of the Newburyport pluton is interpreted to be Devonian (Acadian), but if the Merrimack Group rocks are Ordovician or older, the Newburyport pluton may also be older than Devonian. Billings (1956) interpreted its age to be Ordovician to Precambrian. U-Th-Pb radiometric dates on zircons by Zartman (personal communication, 1979) at around 435-440 million years, and by Naylor (1977) at 445 million years, suggest a Late Ordovician age of intrusion. Zartman has further detected no evidence of an Acadian event (Devonian) in the Newburyport intrusive.

To the south of the Newburyport pluton, the Scotland Road (Clinton-Newbury) fault cuts across the core of the pluton and terminates it against a fine-grained diorite and schist complex of unknown age which is not related texturally or mineralogically to the Newburyport pluton. The diorite-schist complex is intruded locally by elongate bodies of pink quartz monzonite. The southern border of the diorite-schist complex is formed by the Parker River fault, separating the diorite-schist block from the Newbury Volcanic Complex of latest Silurian to Earliest Devonian age. The Newbury Complex was also not affected by regional metamorphism or Acadian granitic intrusive activity.

To the northwest of the site, the Exeter pluton intrudes Merrimack Group rocks in a large, west-dipping mass which is interpreted to be about 3 kilometers thick (Bothner, 1974; Birch, 1979). Numerous smaller bodies of similar rock occur scattered in the area between the site and the main mass. The Exeter pluton is characterized as a massive igneous rock which locally ranges widely in grain size and composition, but which commonly is a medium- to coarse-grained diorite or quartz diorite. It is seen frequently to be transected by thin white aplite dikes. Structural relationships with the country rocks indicate the pluton to be a late syntectonic or post-tectonic intrusive; a preliminary Rb-Sr whole rock radiometric date of 410 million years suggests it to be of Late Silurian or Early Devonian age (Birch, 1979).

Throughout the site vicinity, fine-grained, sub-black mafic dikes intrude both the igneous and the metamorphic country rocks. The dikes commonly strike northeasterly and dip steeply to the northwest. K-Ar radiometric dating by the Applicant of several such dikes at the site and in the site area reveal two distinctly separate episodes of dike emplacement. The younger intrusive event was in Late Permian - Early Triassic time, with six K-Ar dates between 212 to 236 million years. The older event is expressed by two K-Ar determinations of 255 and 295 million years. Since these latter dates are indistinguishable from those obtained on nearby igneous and metamorphic country rocks, and reflect only a broad regional Late Paleozoic crustal cooling episode (Zartman et al., 1970), these dikes may be considerably older than Upper Paleozoic in true age.

4. <u>Site Vicinity Structure</u>

The regional fold structure of the site vicinity is indicated on Figure 2.5-7 by strike-dip symbols on foliation, and by the northeasterly trend of the axes of major anticlines and synclines. The foliation or schistosity in the metamorphic rocks commonly dips steeply to the northwest, and the major folds are slightly overturned to the east. The Rye Anticline plunges southwesterly into the site area, and on the southern nose of the anticline, the schistosity, commonly dips to the south. Foliation in the intrusive gneissoid quartz diorite to the south of the Rye Anticline generally parallels the south-dipping structure in the metasedimentary Kittery and Eliot Formations. Figure 2.5-7 defines this structure schematically in cross section, drawn diagonally across regional structure from northwest to southeast through the site.

5. <u>Site Vicinity Faulting</u>

Numerous small fault displacements may be seen in outcrops in the site vicinity, related to structural adjustments occurring during the folding and igneous intrusive activity of Paleozoic orogenic deformations. Two fault structures (Figure 2.5-8), the "Portsmouth fault" (Novotny, 1963) and the Scotland Road fault (Shride, 1971), were inferred from published reports to be of sufficient lateral continuity to merit detailed physical investigations by the Applicant to evaluate their capability to cause surface faulting.

Detailed reports of these fault investigations are presented in Appendix 2C.

(a) The "Portsmouth Fault"

R. F. Novotny (1963) interpreted the "Portsmouth fault" (Figure 2.5-8) to be a normal fault of unknown displacement which formed the steep west-dipping contact between the Kittery and Rye Formations. He projected the fault to trend for a distance of about 9 miles southerly from New Castle, near Portsmouth, to North Hampton, where it is shown to die out about 1½ miles before intersecting the Newburyport pluton in the site area. No evidence of this inferred fault structure had been detected in the site area.

The interpretation by Novotny of a fault structure in this 9-mile zone relied upon: (1) brecciated and faulted rocks in the Kittery Formation in an exposure on Route 1 in Portsmouth, and brecciated and partly silicified Kittery Formation rocks exposed on Goat Island, New Castle, 9 miles north of the Seabrook site; (2) brecciated and partly silicified Kittery Formation rocks exposed near the east end of Bromley Hill, North Hampton, 6 miles north of the site; (3) an apparently unconformable stratigraphic relationship between the Rye and Kittery

Formations along the 9-mile course of the zone; and (4) the presence of concordant foliated and granulated granitic intrusives in the Rye Formation near the Kittery Formation contact along the trend of the zone. Time of faulting was interpreted by Novotny to be during the Acadian orogeny, and he noted that metamorphic facies in the bedrock formations adjacent to the fault are not displaced by the fault.

Detailed geologic, geophysical, and borings investigations by the Applicant in 1974 (Appendix 2C) have demonstrated that the "Portsmouth fault," as defined by Novotny (1963), does not exist.

(b) The Scotland Road Fault

A. F. Shride (1971) interpreted the Scotland Road fault to be the northeastern projection of the Clinton-Newbury fault of J. W. Skehan (1968). The fault is an apparent north-dipping thrust fault of regional extent on which the hangingwall plate moved from north to south over the footwall block, with some right-lateral displacement (Figure 2.5-8). The time of faulting of the ClintonNewbury system has been interpreted by various workers to relate either to the Acadian orogeny or to post-orogenic adjustments prior to the end of the Paleozoic era. No evidence of surface faulting has been reported or inferred for the structure anywhere along its 60-mile strike length in northeastern Massachusetts.

Detailed geologic, geophysical, and borings investigations by the Applicant in 1973-1974 (Appendix 2C) have demonstrated that the Scotland Road fault trends easterly through Newbury, Massachusetts, quite closely along the trace inferred by Shride (1971) from his field mapping studies. At its closest approach to the site, near Plum Island, Massachusetts, the outcrop trace of the fault is about 6½ miles south of the site. Borings and complimentary geophysical and laboratory studies indicate that in the site vicinity the fault zone dips about 44° to the north and shows no evidence of displacement in the past 199 million years. Radiometric dating of cataclastic fault materials suggests a Middle Permian age for the last fault movement (248 million years ago).

6. Site Vicinity Jointing

Figure 2.5-7 shows by strike-dip symbols the generalized orientation and attitude of the strongest high-angle joints in the area. There are commonly four joint orientations in the area, striking generally north-south, east-west, northeasterly and northwesterly.

In the Newburyport pluton and in the igneous rocks to the south of the pluton, the most obvious or common joint orientation strikes about N30°-45°E and dips steeply to the northwest, while the second most common orientation strikes N30°-50°W and dips steeply to the northeast. In the metamorphic rocks, the strongest or most common joint orientation strikes about N30°-45°E near the north end of the Newburyport pluton, and then, with distance north of the pluton, shifts to about N50°-60°W, approximately normal to the attitude of the schistomity, with very steep dips.

While northwesterly joint strikes are the more common in the metamorphic rocks, and northeasterly strikes the more common in the igneous rocks, the general pattern of jointing suggests a compatible structural relationship with the regional fabric of Paleozoic folding. Field reconnaissance of roadside bedrock exposures, ranging more than 20 miles inland to the west and north of the Seabrook site, has failed to detect any obviously anomalous pattern of jointing which might suggest the presence of some tectonic structure transverse to the Paleozoic folding.

7. Site Vicinity Fluid and Mineral Extraction

Human activities in the vicinity, with respect to the withdrawal or addition of subsurface fluids, are limited to the pumping of near-surface groundwater for public and private water supply systems. Groundwater is pumped relatively more extensively from aquifers in unconsolidated granular ice-contact Pleistocene deposits, and less extensively from interconnecting joint and fracture zones in bedrock wells. Groundwater resources are replenished continually by down-ward percolating rain and snowmelt. No surface subsidence of significance to the site derives from groundwater withdrawal in the region (refer to Subsection 2.4.13).

Minerals extracted in the vicinity consist of Pleistocene sands and gravels excavated for local uses, some crystalline bedrock quarried for construction materials, and bedrock excavated to construct the two site cooling water tunnels beneath the ocean to the east of the site (see Subsection 2.5.1.2b.6. No hazards to the site can be anticipated as a result of fluid or mineral extractions in the area.

8. Bedrock Geology - Circulating Water System

The bedrock geology of the 3.2-mile strip to the east of the site in which two circulating water system tunnels are located is defined by the results of 116 exploratory bedrock borings drilled during the period 1973-1975. Selected boring logs for tunnels investigations are presented in Appendix 2D. Bedrock geology interpreted from results of the relatively widely-spaced borings is shown on Figure 2.5-9. The topography of the bedrock surface in the area of the tunnels, as interpreted both from borings results and extensive seismic surveys, is shown on Figure 2.5-10. Profiles of soils and bedrock types encountered in borings along the courses of the intake and discharge tunnels are shown (vertical exaggeration = 10X) on Figure 2.5-11.

Figure 2.5-12, Figure 2.5-13 and Figure 2.5-14, are provided to show the geology of the tunnels based on the significant data mapped in detail by project geologists during the course of tunnels excavation. Figure 2.5-12 is a geologic plan map of the circulating water system tunnels, showing rock types and mapped faults. Figure 2.5-13 and Figure 2.5-14 show for the intake and discharge tunnels, respectively, geologic profiles (5x vertical exaggeration), schematic plan diagrams, and histograms describing tunnel support requirements (both as-built and preconstruction predictions), water inflow, mole advance rates, and bedrock joint spacing.

Detailed field notes and large-scale geologic map compilations of tunnels geology are stored at the site. A summary report, supplemented with numerous data tables and descriptive figures, is currently being prepared to describe the tunnels geology in detail.

Revisions in the original preconstruction geological interpretations of the tunnels' geology, as generated from detailed mapping within the tunnels, do not suggest modifications of significance to the site. The modifications have merely involved relocating geologic contacts to the east or west of the locations previously estimated by inference from widely spaced borings data combined with the geophysically defined topography of the bedrock surface. Several areas of mixed metasedimentary/igneous rock types were found in the tunnels which the widely spaced borings investigations failed to intersect. Zones of apparently closely jointed or deeply weathered rock seen in borings were found during tunnels excavation consistently to be of less lateral extent and of greater rock competence than interpreted from preconstruction investigations.

(a) Lithologies

The bedrock in the area of the circulating water system tunnels is made up of fundamentally hard and compact crystalline rock types including metamorphic quartzites, schists, and metavolcanics of the Kittery, Eliot and Rye Formations (the Merrimack Group), and igneous diorites of the Newburyport pluton, all intruded locally by relatively thin, tabular diabase dikes.

The quartzite is a medium to dark brownish-gray, finegrained, weakly-foliated rock composed mostly of quartz with minor amounts of biotite, muscovite, sericite, feldspar, amphibole, and garnet. The "metaquartzite" is a medium-dark gray, fine- to medium-grained, quartzose rock which is normally foliated, occasionally resembles a fine-grained diorite, and may also appear to be a transitional rock ("semi-schist") between quartzite and schist. Metavolcanic rocks of the Rye Formation have been identified only in borings at the east end of the intake tunnel, and are comprised predominantly of welded tuff, a light to dark gray rock consisting of angular volcanic rock fragments and some quartz and feldspar crystals in a very fine-grained matrix of volcanic ash, interbedded with quartzite. Diorites of the Newburyport pluton occur in numerous varieties including fine-grained, dark gray diorite; medium to coarse grained, medium gray quartz diorite; locally porphyritic, fine-grained diorite; granodiorite; and "intrusion breccia" of blocks of fine-grained diorite enclosed in, fused to, and partly mixed with a matrix of medium-coarse grained, frequently gneissoid quartz diorite.

The diabase is a dark gray, fine-grained rock which is commonly chilled and aphanitic at contacts and sometimes porphyritic in central portions with phenocrysts of pyroxene and plagioclase. Essential minerals are plagioclase, augite, biotite, chlorite and calcite; disseminated pyrite is a frequent constituent of the dike rocks. Some dikes contain calcite amygdules.

As described on Figure 2.5-11, the bedrock of the tunnels area is widely veneered by a succession of unconsolidated sediments, including bouldery lodgment till, gray glaciomarine clay-silt (glacial rock flour), and outwash or beach sands.

(b) Structure

Internal fold structures and intrusive relationships from borings results in the area of the tunnels appear to be very complex. The borings are too widely spaced and the structure too folded to permit meaningful detailed stratigraphic or lithologic correlations through the areas between borings. Measurements of foliation attitudes in numerous oriented borings in the area suggest tight folding of the metamorphic rocks in the broad area of their contact zone with intrusive diorites, apparently related more to deformation by the forceful intrusion from the south of the Newburyport pluton than to regional orogenic compressional forces directed against the area from the southeast.

As shown by symbols on Figure 2.5-9, orientation measurements of foliation in selected borings show an average easterly strike of foliation attitude in the area, with south dips predominating to the west of Hampton Beach and northerly dips predominating to the east. Inferred foliation attitudes depicted on Figure 2.5-9 generally constitute a rough average of numerous divergent measurements obtained from any given boring, and do not refer to structure at any particular elevation below the bedrock surface.

Most foliation measurements from oriented cores show the layering both in igneous and metamorphic rocks to strike generally east-west, and to dip moderately steeply either to the south or north. Foliation structure tends to turn in the area of diorite intrusion to orient subparallel to the diorite contact. Foliation attitude in individual borings is occasionally seen to vary widely with depth as the boring progresses through metamorphic rock toward underlying diorite. Dip reversals are quite common in the metamorphic rocks, as the boring progresses downward through opposite fold limbs.

Examination of cores from all borings drilled in the area of the tunnel alignments indicates that bedrock foliation is only rarely a significant plane of weakness. Where fresh and not affected by weathering, the rock is well bonded and rarely tends to part along foliation planes in drill cores. Most foliation in the schist, as well as in the quartzite and diorite, takes the form of crude banding, and only rarely are thin zones of laminated and fissile schist or phyllite encountered in borings in the area.

Figure 2.5-9 describes the distribution of igneous rocks of the Newburyport pluton in the tunnels area, as interpreted from bedrock borings. In areas not investigated by borings, elevated portions of the bedrock surface, as defined by seismic surveys, are inferred to be underlain by Newburyport rocks.

The Newburyport rocks in the area of the tunnels appear to have intruded the quartzites and schists of the Kittery Formation in elongate tongues and sheets. The distribution of these intrusive masses suggests that the diorite invaded the Kittery rocks upward from the south, and that fold structures in the Kittery preferentially guided the intrusions along roughly planar paths. The quartzose nature of the Newburyport rocks in the general contact area, including, in some areas, rock types which are difficult macroscopically to discriminate as diorite or quartzite (termed "metaquartzite"), coupled with the apparent absence of diorite in areas where schist bedrock predominates, suggests that at least part of the emplacement of the diorite in the Kittery was accomplished by some form of replacement, or "granitization," of the quartzitic rocks of the Kittery Formation.

The Newburyport rocks in the contact area frequently show a gneissoid (foliated) fabric which commonly dips moderately and steeply toward the south, subparallel to foliation in the adjacent metamorphic rocks. The rock is internally well bonded, and does not show any marked tendency to part along planes of gneissoid foliation.

The youngest rocks in the area are diabase dikes, which were encountered in about 75 percent of all borings put down in the tunnels area. Dikes appear to be the more prevalent in the area of the State Park at Hampton Beach and in the area offshore to the east of the park. Measurements of diabase dike contacts in oriented cores indicate a strong tendency for dikes to strike northeasterly and dip moderately to steeply to the northwest.

(c) Faulting

No throughgoing fault structure has been identified or inferred by the boring investigations in the tunnels' area. The complex, interfingering and essentially unaltered intrusive character of the contact between the Newburyport and Merrimack Group rocks indicates that the contact is not a fault boundary.

Although no single, discrete and continuous fault zone has been detected by boring investigations in the area, numerous features indicative of ancient faulting have been seen in cores from borings throughout the area of the tunnels. Slickensided and frequently chloritecoated joint surfaces having widely divergent attitudes occur throughout the metamorphic rocks in the area. Welded breccias, highly polished joint surfaces, or hydrothermal alteration (bleaching) are noted in more than half of the borings in the area. The distribution of these fault-related features is not confined to, or concentrated in, any given portion of the area.

In only one boring, ADT-16, (discharge tunnel, about 1,000 feet east of the State Park) were welded breccia, highly polished joints, and hydrothermal alteration found together in association with a very narrow fault on which a diabase dike has been displaced. Four additional borings, ADT-16A, ADT-16B, ADT-16C, and ADT-16D were drilled around ADT-16, 50 feet distant from it, to evaluate the significance of the conditions found in ADT-16. These additional borings did not detect a throughgoing fault zone. Measurements of foliation attitudes in oriented cores from these borings indicate that the fold structure in the area is characterized by a series of tight east-northeast-trending anticlines and synclines. Petrographic studies of samples of phyllonite from the narrow fault zone at a 188-foot depth in boring ADT-16, and of diabase, diorite, and semischist ("metaquartzite") in the general fault area, were conducted by Professor Gene Simmons and Dorothy Richter at Massachusetts Institute of Technology. They concluded that "the evidence from thin section textures suggests that the shearing occurred after the major episode of metamorphism and either coincided with, or predated the intense carbonate metasomatism" associated with the country rocks adjacent to the narrow fault structure. Two of the diabase samples studied by the Massachusetts Institute of Technology, one hydro-thermally altered and one relatively fresh, which were taken from below and above the fault zone, were dated (K-Ar) by Krueger Enterprises, Inc., of Cambridge, Massachusetts, at 213 and 221 million years, respectively.

In boring AIT-39 (intake tunnel, about 4,500 feet northeast of the State Park), between depths of approximately 89 feet and 92 feet, the rock is defined as cataclasite, composed of intensely sheared and recrystallized (welded) quartzite. The rock is fresh, compact, and hard, composed of quartz with some sericite or fine-grained muscovite, and is bleached to a tan color. A K-Ar age determination of the fine mica separated from the rock reported 262 ±10 million years, typical of the K-Ar "cooling age" dates common for the country rocks at the site and in the site vicinity. The episode of shearing which produced the cataclasite occurred prior to this Middle Permian date.

The initial question of throughgoing faults at the site related to an east-west thrust fault is shown for the area on the geologic map of New Hampshire (Billings, 1956). Borings investigations at the site and along the tunnel alignments indicated that the fault does not exist. Mapping in the tunnels has demonstrated that the contact between the Newburyport pluton and the Merrimack group metamorphic rocks, earlier depicted as a through-going thrust fault, is an intimately interfingering intrusive contact.

Detailed mapping during construction has identified 104 faults in the tunnel excavations and 61 faults in the site excavations. The system and styles of site faults are described in detail in Subsection 2.5.1.2b.6, with the conclusion that all are considered to be localized deformations. The systems and styles of tunnel faults closely resemble those of the site faults in their orientations, widths, displacements and mineralization, and we conclude that they are also localized, non-throughgoing deformations.

Where possible, faults in the tunnels were correlated from one tunnel to the other during mapping (Figure 2.5-12). Only 15 of the 104 faults were found to occur in both tunnels. This occurs despite the fact that tunnels run parallel at a distance apart of less than 70 feet for almost 10,000 feet of their lengths.

One fault in particular with significant associated weathering, possibly gouge, was viewed with NRC staff during a geologic reconnaissance trip in the tunnels. This fault occurs in the discharge tunnel at station 114+05 where it strikes N33E and dips 79NW and contains what appears to be gouge. The projection of this feature into the intake tunnel would occur at a distance of about 900 feet, near station 120+00 in that tunnel. The only correlative feature in that portion of the intake tunnel occurs at station 120+21 where an 8-inch wide diabase dike strikes across the tunnel nearly parallel with that attitude of the fault in the discharge tunnel. The dike and adjacent metasedimentary rock are thoroughly bleached and weathered, but the dike and country rock are intact and contain no gouge or gouge-like material. The apparent relationship between the fault and the dike at these tunnel locations resembles those observed at similarly northeast-trending faults A-1A and SI-1 in plant site foundation excavations, where diabase intruded along fault planes.

(d) <u>Jointing</u>

In the broad area of the contact zone between the Merrimack Group metamorphic rocks and the Newburyport pluton intrusive rocks along the alignments of the circulating water system tunnels, the metamorphic rocks are characteristically more closely jointed than the adjacent diorite of the Newburyport pluton. Orientation measurements of joints in cores taken from numerous borings in this area commonly show that joints in the metamorphic rocks have widely diverse orientations and attitudes, while those in the adjacent diorite commonly conform to the normal northeast-northwest joint fabric of the region. Joints in the metamorphic rocks in this contact zone are also commonly striated, indicating that some movement took place along the joint surfaces at the time of their formation or at the time of intrusion by the Newburyport pluton.

Since the characteristic of widely divergent jointing is not generally seen in outcrops of metamorphic rocks a few miles to the north of the site area, or within the Newburyport pluton to the south of the site, it appears that this jointing is spatially related to the Newburyport-Merrimack Group contact zone, and is genetically related to dynamic stresses imposed on the metamorphic rocks by the intrusive force of the Newburyport at the time of its emplacement, in Middle Paleozoic time or earlier. The occurrence of intrusion breccia fabric in the Newburyport along the zone of its contact with the metamorphic rocks further attests to the forceful nature of the intrusion of the Newburyport.

Whereas there is an apparent tendency in the Newburyport igneous rocks for closely-spaced, high-angle jointing to diminish in frequency with depth, there is no discernible tendency for closely-spaced, high-angle jointing to diminish with depth in most borings in the metamorphic rocks along the tunnel alignments. Joint surfaces in the Newburyport rocks are normally smooth and planar, but not polished or slippery. Conversely, high-angle joint surfaces in the metamorphic rocks are frequently striated and/or coated with a thin layer of slippery chlorite. Measurements made on joints in oriented cores from borings drilled along the tunnel alignments indicate that low-angle joints are much more numerous than high-angle joints. There is a very general, overall tendency for low-angle joints to dip 30° to 45° into the northwest quadrant, particularly in diorite country rock. In the schist and quartzite, orientations of low-angle joints are less consistent, and numerous oriented borings in schist and quartzite showed low-angle joints within those holes to dip in widely diverse directions.

(e) Weathering

With certain specific exceptions, the bedrock in the area of the tunnel alignments to the east of the site area is not normally subject to deep weathering. The rock is commonly fresh and hard internally, immediately below the bedrock surface, and weathering effects are commonly restricted to thin layers of powdering on highangle joint surfaces. Rusty staining on joints is uncommon. Boring results indicate that, although weathering effects normally decrease with depth below the bedrock surface, minor weathering on joints extends to depths below the elevations of the tunnels.

As shown by stipple patterns on Figure 2.5-10, deep bedrock weathering extends downward to the elevation of the tunnels beneath portions of the marsh and tidal flats area, where the bedrock is subject to bulk softening or decomposition over wide areas. Extreme bedrock weathering to tunnel elevation also occurs in an apparently restricted zone beneath Hampton Harbor. Intermittent thin zones of moderate to extreme weathering on high-angle joints also extend to tunnel elevation at widely scattered locations offshore along the intake tunnel alignment. Borings along the discharge tunnel alignment offshore from the State Park suggest that the bedrock in that area is not deeply weathered.

Three zones in which extreme bedrock weathering extends downward to or below the elevation of the tunnels are near the eastern edge of the marsh off the northwest shore of Commons Island, beneath tidal flats to the northeast of Commons Island, and beneath Hampton Harbor about 800 feet west of the Hampton-Seabrook bridge. The geologic structures which controlled the downward migration of weathering solutions in those areas are not known, and the attitude and orientation of weathered zones cannot be estimated from available data.

Weathering degradation was found in the tunnels to be considerably less than that suggested by the borings investigations, and the broad zones of weathering inferred from the borings were not encountered in that form. As shown on Figure 2.5-12 and Figure 2.5-13, in areas where the bedrock surface is at a low elevation, the underlying bedrock was found in the tunnels to be more closely jointed, permitting the more rapid migration of ground water. These conditions enhance the advance of weathering degradation.

Beneath the northwest shore of Commons Island and the tidal flats in that area, the generally greater weathering effects coincide with the presence there of micaceous metasedimentary rocks. A lense of similar micaceous rock coincides with another zone of inferred deep weathering beneath Hampton Harbor to the west of the Hampton-Seabrook bridge. While softer and more micaceous, the rock at these locations is not severely weathered.

No substantial weathering was found in the tunnels at the earlier-inferred zone of deep weathering immediately to the west of the Hampton-Seabrook bridge. Bedrock weathering encountered in the intake tunnel between Stations 119+70 and 120+25 coincided with the intersection of a hydrothermally altered northeast-striking diabase dike and a moderately dipping, north northwest-striking fault. This is also a location where the bedrock surface is at a relatively low elevation.

(f) Bedrock Surface Topography

Figure 2.5-10 describes the configuration of the bedrock surface in the area of the circulating water system tunnel alignments, as interpreted from refraction and reflection seismic surveys and controlled from place to place by survey data from individual tunnel borings. The elevation of the bedrock surface in the area ranges from a few feet above mean sea level at scattered bedrock outcrops to more than 160 feet below sea level at a point 1.2 miles east of Hampton Beach.

The general configuration of the bedrock surface in the area of the tunnels is characterized by three broad, topographically elevated areas: near the site, the State Park and the Outer Rocks. These three areas are separated by a wide northeast-trending bedrock valley beneath the eastern part of the marsh, and by a series of relatively narrow north-northeast-trending valleys which lie within one-half mile offshore of the State Park. Details of topographic variations interpreted for the bedrock surface along each of the tunnel alignments are shown in profile (vertical exaggeration = 10x) on Figure 2.5-11.

The results of test borings indicate that there is a relationship between bedrock topography and bedrock geology. Igneous bedrock of the Newburyport pluton commonly underlies topographic highs on the bedrock surface, while quartzites and schists of the Merrimack Group metamorphic rocks normally underlie bedrock lows. The metamorphic rocks are significantly more closely jointed than the igneous rocks, and were apparently subjected to deeper decomposition and dissection during periods of pre-Pleistocene subaerial weathering and erosion. Successive periods of continental glaciation during the Pleistocene scoured away the residual soils in the area to produce the present bedrock topography, with closely jointed, sometimes deeply weathered metamorphic rocks underlying the valleys, and widely jointed, normally fresh diorite supporting the ridges.

9. <u>Site Vicinity Radiometric Dating</u>

Numerous potassium-argon (K-Ar) radiometric age determinations have been obtained on bedrock samples from borings and natural outcrops, in connection with investigations to determine minimum ages of fault deformations both at and in the vicinity of the site. Sample locations and age determinations are shown graphically on Figure 2.5-8, and are described in further detail in Table 2.5-1.

A total of 15 determinations of Merrimack Group metamorphic rocks and Newburyport pluton igneous rocks from locations between the Clinton-Newbury Fault and Portsmouth, New Hampshire range from 246 to 308 million years and average about 272 million years. Three samples of diorite-schist complex just to the south of the Clinton-Newbury fault range from 304 to 422, and average about 350 million years. Two samples of older diabase dikes at the site were age-dated 255 and 295 million years, averaging about the same as the country rock ages, at 275 million years. Six younger diabase dikes, including two characterized by bleaching alteration, range in age from 191 to 225 million years, averaging about 214 million years. One faulted diabase dike at the site produced a 236 million-year age in its undeformed portion and a 213 million-year age immediately adjacent to an offsetting fault plane. A single sample of hornblende diorite from a location adjacent to Fault CII-1 near the center of the former Unit 2 at the site produced an (incomprehensible) "age" of 1,181± million years. The youngest age determination obtained in all project investigations is 191 ±9 million years, from a sample of altered but undeformed olivine basalt dike which transects sheared granodiorite of the Newburyport pluton in the Scotland Road fault zone. The youngest age determination obtained in rocks deformed by the Scotland Road fault is 248 ±9 million years, from a sample of cataclasite (brecciated quartz-muscovite schist) with ultramylonite shear structures located about 5 feet above the footwall of the fault, within the most intensely deformed portion of the fault zone.

10. <u>Site Area Groundwater</u>

Refer to Subsections 2.4.13 and 2.5.4.6.

b. Geology at Site

1. Site Physiography

The site is situated in the Seaboard Lowland section of the New England physiographic province (Figure 2.5-1), which is characterized by low, gently undulating topography which rises gradually from the seacoast to 500 feet elevation some 30 miles inland from the coast. The physiographic configuration of the immediate site area is characterized by broad open areas of level tidal marshes, dissected by numerous meandering tidal creeks and man-made linear drainage ditches, and interrupted locally by wooded "islands" or peninsulas which rise to elevations of 20 to 30 feet above sea level. The site is located on a wooded peninsula, enclosed on three sides by tidal marshes, and supported by quartz diorite bedrock to a maximum of about 30 feet above sea level (Figure 2.5-15 and Figure 2.5-16).

The physiography of the area derives primarily from the effects of continental glaciation and postglacial sedimentation. During the Cenozoic era, the area was subjected to subaerial weathering and erosion, whereby the micaceous bedrock formations of the Merrimack Group weathered and were eroded more deeply than the more massive igneous rocks of the Newburyport pluton. Scouring of the weathered residuum at the time of Pleistocene glaciation produced a rolling bedrock topography characterized by ridges and knobs of Newburyport igneous bedrock rising above valleys and depressions in Merrimack metasedimentary rocks. Total local relief on the buried bedrock surface exceeds 200 feet in the general site vicinity.

Following the melting of the last major glacial ice sheet, which deposited a ground moraine of till throughout the area, the area was submerged beneath sea level and a blanket of marine clay-silt was deposited, filling the bedrock valleys and lapping onto hillside slopes. Briefly renewed glacial advance and retreat deposited outwash materials over the older marine clay-silt deposits in the bedrock valleys, further subduing the local topography. In Recent time, following the close of Pleistocene glaciation, outwash sands have been reworked to produce coastal beaches, and tidal marsh development has built a thin blanket of peat over outwash deposits near sea level.

No areas of significance to the site of landsliding, subsidence, uplift or collapse occur in the vicinity. The peat, outwash, and clay-silt deposits, however, which lie on and beneath the tidal marshes adjacent to the site, may be unstable in artificial excavations and during earthquakes. No safety-related facilities of the site are founded on potentially unstable materials.

2. Site Lithologies and Stratigraphy

The site is founded on crystalline igneous rock of the Newburyport pluton, which locally contains well consolidated relict inclusions of crystalline quartzitic units of Merrimack Group rocks (Figure 2.5-16). Throughout the site, north-easterly-trending and steeply-dipping mafic dikes of both Paleozoic and Triassic ages transect the country rocks at intervals of a few feet to more than 200 feet. Prior to excavation for plant construction, the bedrock at the site was locally overlain by glacial lodgment till and postglacial marine clay-silt, outwash sands, and occasional sandy beach deposits.

Bedrock formations in the area of the circulating water system tunnels, from the site easterly for about 3½ miles, have been determined by boring investigations to be similarly comprised of intrusive igneous rocks of the Newburyport pluton, quartzite-schist-gneiss metamorphic rocks of the Merrimack Group, and steeply dipping, northeasterly trending mafic dikes.

The stratigraphic section for the site vicinity (Figure 2.5-7) includes from oldest to youngest: Merrimack Group sedimentary feldspathic schist and metavolcanic gneiss and amphibolite of the Rye Formation; Merrimack Group metasedimentary quartzite, and phyllite of the Kittery and Eliot Formations; gneissoid granodiorite, quartz diorite, fine-grained diorite, and hornblende diorite of the Newburyport pluton; fine- to medium-grained, somewhat porphyritic diabase dikes; unconsolidated sand-silt-cobble glacial till; thinly bedded glacio-marine clay-silt and fine sand; fine-grained outwash sand with occasional boulder erratics; organic humus and tidal marsh peat deposits; and fine-grained beach and dune deposits.

Ages of the Merrimack Group rocks have previously been inferred as Ordovician(?) for the Rye Formation and Silurian(?) for the Kittery Formation (Billings, 1956). Recent radiometric and structural studies in southern New Hampshire (Aleinikoff and Zartman, 1978; Lyons, Boudette and Aleinikoff, 1979) have suggested that the Merrimack Group rocks may all be no younger than Ordovician and possibly as old as Late Precambrian.

The age of the Newburyport pluton, intrusive into the Merrimack Group rocks in the site vicinity, has variously been inferred to be Precambrian or Ordovician (Billings, 1956) based on correlation with the Dedham granodiorite of southeastern Massachusetts, or to be Devonian (Novotny, 1963) based on the petrographic resemblance of its porphyritic grandiorite phase with the Ayer granite, 10 miles to the southwest, also then assumed to be of Devonian age. The age of the Ayer granite, however, may now be inferred from radiometric age determinations to be Early Silurian or older (Peck, 1976; Naylor, 1977). If the Merrimack Group rocks, into which the Newburyport pluton is intrusive, are truly Early Paleozoic or Late Precambrian, the Newburyport pluton may also be significantly older than Silurian or Devonian. Zartman (personal communication, 1979) has noted that there does not seem to be any evidence of Acadian orogenic disturbance in the Newburyport pluton.

There are two ages of diabase dike intrusion in the site vicinity. Six K-Ar radiometric determinations on samples taken both from fresh construction exposures at the site and from borings drilled near the site and to the east in the area of the circulating water system tunnels range from 212 to 236 million years, indicating a Late Permian/Early Triassic emplacement age. Intrusive relationships of some dikes into others indicate that diabase dike emplacement in the area occurred as a series of pulses. Two K-Ar dates on two northeast-trending diabase dikes at the site are 255 and 295 million years, the general time at which the region experienced an episode of crustal cooling. These older dikes may be substantially older than Upper Paleozoic in age.

The ages of the unconsolidated surficial deposits in the site vicinity range from Late Wisconsinan (25,000 to about 13,500 years old) for the basal lodgment till, to immediately postglacial for the glaciomarine clay-silt and outwash sands (about 13,500 to about 11,800 years old), according to studies by numerous workers (Borns, 1973; Kaye and Barghoorn, 1964; Schafer and Hartshorn, 1965). Organic tidal marsh deposits, beaches, and dunes are of Recent age (less than 10,000 years old) and are continuing to develop to the present time.

For detailed discussion and drawings describing the lithologies mapped in bedrock excavations during construction at the site, please refer to Subsection 2.5.1.2b.6 and Appendices 2D and 2F.

3. Site Structural Geology

The internal structural fabric of the bedrock at the site conforms with the attitude of metamorphic folding along the south limb of the southwest-plunging nose of the Rye Anticline (Figure 2.5-7). Steep south-dipping schistosity in the Kittery and Eliot Formations just to the north of the site is mirrored by subparallel, near-vertical foliation, banding and autoliths in the gneissoid Newburyport quartz diorite within the area of site excavations. A large inclusion of Kittery Formation quartzite and granulite, which is enclosed within transitional intrusive contacts by Newburyport rocks, trends easterly through the area of the former Unit 2 at the site, and exhibits very steeply dipping, east-west striking bedding planes (Figure 2.5-16). The contact zone of the Merrimack Group rocks with the Newburyport pluton trends irregularly east-west at a location about 300 feet to the north of the site, and is interpreted from borings investigations to dip steeply to the south, roughly parallel to the adjacent bedrock structure (Figure 2.5-16). Details of the bedrock structure mapped at the site during construction are presented in Subsection 2.5.1.2b.6.

(a) Merrimack Group/Newburyport Pluton Contact Zone

The contact between metamorphic rocks of the Merrimack Group and igneous rocks of the Newburyport pluton is nowhere exposed at ground surface in the site area. Based on his earlier interpretation that the undeformed Newburyport pluton was of Ordovician age or older, and that the folded Merrimack Group rocks adjacent to it were of Silurian age, Billings (1956) was philosophically constrained to define the contact between the two as a thrust fault. Detailed borings investigations conducted by the Applicant at the site, in the site vicinity, and for about 3.2 miles easterly from the site along the course of the circulating water system tunnels have definitively demonstrated that the contact is an intimately interfingering, commonly transitional intrusive boundary of highly irregular shape.

At the site, a large relict inclusion of quartzite and granulite of the Kittery Formation is enclosed within transitional intrusive contacts in the Newburyport rocks (Figure 2.5-16) and exhibits thermal alteration ("baking") effects from the intrusive activity. In the site vicinity, exploratory borings B-20, B-42, and B-11 intersect the contact zone at a location about 300 feet north of the site, and show a fused, transitional, intrusive relationship between the older Merrimack Group metamorphic rock and the younger Newburyport quartz diorite (Profile E-F, Figure 2.5-16). In the zone of the circulating water system tunnels, numerous borings intersected scores of Merrimack/Newburyport contacts displaying interfingering or transitional intrusive relationships between the metamorphic and igneous rocks.

While the actual geologic age of the Newburyport pluton is still not known, its relative age (younger than the Merrimack Group rocks) has been conclusively demonstrated by project investigations. Recent interpretations that the Merrimack Group may be of Early Paleozoic or Late Precambrian age (Aleinikoff and Zartman, 1978; Lyons, Boudette and Aleinikoff 1979) permit renewed speculation that the Newburyport pluton may also be of Early Paleozoic age. Radiometric dating of the Newburyport (Zartman, personal communication, 1979) suggests that its age may be 435-440 ±15 million years. Although specifically sought, no evidence has been found to suggest or infer the presence of Billings' hypothetical thrust fault adjacent to the site.

(b) Bedrock Topography

As shown on Figure 2.5-17, the preconstruction topography of the bedrock surface in the site area was characterized by a central, relatively elevated, east-northeast-trending ridge supported predominantly by Newburyport quartz diorite, bounded on the north and south by lower elevation areas of schist and quartzite of the Merrimack Group formations. The site facilities are constructed on bedrock excavated in the area of the formerly elevated bedrock ridge. The former central ridge displayed an irregular surface, with total bedrock relief within the plant boundaries approaching 45 feet.

Total bedrock relief within the immediate area of the site, as defined by Figure 2.5-17, was about 120 feet, with the lowest elevations occurring several hundred feet to the north on bedrock characterized by well foliated feldspathic schist. As shown also by extensive borings and geophysical surveys to the east of the site in the area of the circulating water system tunnels (Subsection 2.5.1.2a.8, Figure 2.5-10), there is a close relationship in the site area between bedrock elevations and underlying rock types, with relatively resistant igneous rocks of the Newburyport pluton supporting the higher elevations, and with the well foliated, relatively less resistant, micaceous metamorphic rocks of the Merrimack Group degraded to lower elevations.

(c) Fold Structure

The fundamental fold structure in the general vicinity of the site is reflected in the schistosity of the Merrimack Group metamorphic rocks and in the gneissoid banding of the Newburyport igneous rocks at the site. The major fold element in the vicinity is the Rye Anticline, with an axis interpreted from field mapping to plunge to the west-southwest about 1½ miles north of the site. The site area is on the south-dipping southeast limb of the anticline. Along the contact zone with Merrimack Group rocks, Newburyport igneous intrusion was strongly influenced by the existing fold structure in the Merrimack Group rocks. As a result, the igneous-metamorphic contact zone irregularly parallels the fold structure, as do igneous banding and included Merrimack Group relicts within the Newburyport rocks to the south of the contact zone.

Strike-dip symbols on Figure 2.5-16 define the attitude of gneissoid banding and schistosity in the site area, commonly striking east-west and dipping very steeply to the south. Bedding and schistosity in the Kittery Formation inclusion at the former Unit 2 dip essentially vertically. To the north of the site, in the area of the Merrimack/Newburyport contact zone, foliation in both the metamorphic and intrusive rocks dips 50° to 60°, presumably to the south. With the limited exception of localized schistose or phyllonitic bands within the Kittery Formation inclusion at the former Unit 2 Containment, the rocks of the site display no preferred tendency to part or fracture along planes of foliation. There is also no tendency displayed by diabase dikes to follow the country rock foliation, except in specific areas where the dikes were diverted during their emplacement by preexisting fault or phyllonite paths of least resistance. Details of fold structure mapped at the site during project construction are presented in Subsection 2.5.1.2b.6.

(d) Faulting

Although thin-welded breccias and slickenside surfaces were noted in some borings and bedrock exposures at the site prior to construction, no specific patterns of faulting could be inferred from the scattered data at that time. During construction, however, the entire site was excavated to and into the bedrock and was mapped geologically in detail, revealing the patterns of faulting associated with the earlier-observed, isolated fault features. Fault patterns at the site are shown in somewhat generalized form on Figure 2.5-16, and in detail on Figure 2.5-19 and Figure 2.5-20. They are summarized on Table 2.5-2. Detailed discussion of site faulting is presented in Subsection 2.5.1.2b.6.

(e) Jointing

Figure 2.5-16 shows joint attitudes as mapped on natural bedrock exposures in the site area, and generalizes the most prominent jointing encountered at the site during construction excavation. Details of joint systems at the site are discussed in Subsection 2.5.1.2b.6., described on Figure 2.5-21 and summarized in Table 2.5-3.

4. Site-to-Region Tectonic Elements

The site is tectonically comparable to the region in the orientation and abundance of diabase dikes, and in the parallelism of gneissoid banding in Newburyport igneous rocks at the site with fold structure on the southeast limb of the Rye Anticline to the north of the site. The igneous rocks at the site do not exhibit Acadian orogenic deformational effects common to the central New England region, and no throughgoing faults reflective of Paleozoic crustal compression or Mesozoic continental rifting occur at the site. No bedrock deformations due to glacial tectonics or postglacial rebound have been detected at the site.

5. Site Geologic History

The site is founded on crystalline bedrock which consists predominantly of weakly gneissoid quartz diorite of the border phase of the Newburyport pluton. Bedrock in the southern quarter of the site, including the former Unit 2 area, is composed of fine-grained quartzite and granulite of the Kittery Formation, occurring as an undigested relict inclusion or roof pendant intimately enclosed within the quartz diorite matrix. Both igneous and metamorphic country rock types are locally offset by discontinuous small-displacement faults and are transected by through-going, northeast-striking diabase dikes. Prior to construction excavation, the bedrock cropped out in the northern one-third of the site in the area of the plant structures, and was elsewhere largely blanketed by a thin veneer of sandy boulder till, marine clay-silt and outwash sand.

The historical geologic development of the site involved first the sedimentation of somewhat calcareous sands of the Kittery Formation, possibly on a flank of a volcanic island-arc chain in Early Paleozoic or older time. Compressional orogenic forces associated with Early Paleozoic continental convergence then folded and metamorphosed the Merrimack Group sequence of island-arc formations, including the Kittery Formation, into crystalline rocks and created the structural configuration of the Rye Anticline and associated synclinal limbs. In Middle Paleozoic time or earlier, the Newburyport pluton was then emplaced in the pre-existing metamorphic terrain, with its initial border-phase intrusive penetrations at the site guided by south-dipping bedding structure in the Kittery and adjacent formations. Subsequent to consolidation of the Newburyport pluton, but prior to the end of Paleozoic time, the site was subjected to tensional stress and was invaded by numerous northeast-striking diabase dikes.

At the site and throughout the region, within about 100 miles inland from the site, Early to Middle Permian time was characterized by a period of uplift, erosion, and crustal cooling, as evidenced by the widespread condition that bedrock formations known to be geologically older than Permian characteristically produce Permian K-Ar radiometric "ages." K-Ar determinations of Kittery Formation, of Newburyport diorite, and of two diabase dikes at the site range from 255 to 295 million years, reflecting the Permian cooling episode in the geologically older site bedrock lithologies.

The final dynamic tectonism directly to affect bedrock structures at the site was an episode of tensional stress and diabase dike emplacement, again along northeasterly-striking planes of separation, in Early Triassic time. One K-Ar determination of 212 million years on a dike at the site, plus four K-Ar determinations ranging from 213 to 236 million years on dikes sampled in borings at locations just to the north of the site and within 2¼ miles to the east of the site, reflect this intrusive episode. While it is known that successive episodes of volcanism, plutonic intrusion, and mafic dike emplacement affected the region around the site in Jurassic, and again in Middle Cretaceous times, no evidence for deformation at the site during these periods has been detected.

The minor, discontinuous fault offsets recorded at the site are shown by their relationships to each other and to the older and younger dikes to have a minimum age of Early Triassic, older than 212 million years. No direct evidence has been detected at the site of deformation associated with major Late Paleozoic faulting on the Clinton-Newbury fault and other crustal faults of the Northeastern Massachusetts Thrust Fault Complex, lying from 6½ to 30 miles to the south of the site. The widespread occurrence of Permian radiometric "cooling" ages of rocks at the site and in the region to the north of the Clinton-Newbury fault, however, appears indirectly to reflect broad uplift of the site and surrounding region as a result of this major tectonic episode.

During Late Mesozoic and Cenozoic times, throughout the past 100 million years, the region inland from the site was subject to broad crustal uplifts, subaerial weathering and erosion, followed by successive relatively brief episodes of continental glaciation during Pleistocene time, during the past 1 million years. Erosional products from inland areas were deposited as Coastal Plain sedimentary formations in offshore areas. The presence of Cenozoic Coastal Plain sediments at Jeffreys Ledge, only 25 miles to the east of the site, suggests that the site itself may have been veneered with correlative sediments prior to its scouring by successive Pleistocene glacial advances. Only sediments left by or derived from the last Wisconsinan glaciation, about 25,000 to 10,000 years ago, have been found at the site. These have been removed where necessary in order to found the facilities on sound bedrock. The last crustal deformation to have affected the region, including the site, was broad downwarping and subsequent crustal rebound as the Wisconsinan ice sheet advanced over and retreated from the region. No evidence of bedrock deformation related to glacial loading and unloading has been detected at the site.

6. Geology of Site Foundation Excavations

A program of geologic mapping of site foundation excavations was carried out during plant construction. This work consisted of mapping of the entire area of bedrock exposed during site excavation, including detailed mapping of foundation excavations for safety-related structures. A summary map of site bedrock geology is presented in Figure 2.5-19. Map data for Figure 2.5-19 derive from two sources: (1) for areas within excavations for safety-related structures, and a few other areas, information is summarized from maps done at a scale of 1 inch = 4 feet. (2) For all other areas, data were compiled at a scale of 1 inch = 32 feet. In addition to geology, the map shows the outlines of excavation plans for the various plant structures. Abbreviations for the names of the major plant structures with typical elevations for these areas are also shown. Traces of several geologic features on this map show abrupt geometric deflections resulting from the box-like topography of the excavated area. Table 2.5-4 lists the major site excavations and the abbreviations for these used in Figure 2.5-19 and in other tables. Figure 2.5-20 consists of four profiles through the site bedrock. Locations of these profiles are shown on Figure 2.5-19.

Geologic studies at the site focused on bedrock structure, especially on faults. In a terrain of great geologic antiquity and repeated orogenic deformations such as New England, ancient faults in bedrock are ubiquitous. Typically, the situation of a fault in such an area is prima facie evidence of that fault's antiquity and noncapability. Conclusive evidence of the antiquity of the faults present in site bedrock was obtained from detailed geologic mapping of foundation excavations. Evidence that all faults in site excavations are noncapable, as defined in Appendix A to 10 CFR, Part 100, was found in: crosscutting relationships in the bedrock; other bedrock structural evidence; radiometric dating; and undisturbed surficial deposits overlying faults.

Radiometric dating directly supports structural evidence that the last movement on site faults took place more than 200 million years ago. Furthermore, the faults themselves are very short, movement occurred on a very small scale, and strike-slip components of fault movements correspond to motions which would be expected to result from the tensional stress field which accompanied an episode of diabase dike intrusion in Early Triassic time.

(a) Lithologies and Stratigraphy

Four bedrock units are present in site excavations:

(1) The Kittery Formation, a metasedimentary unit, occurs as two xenolithic inclusions. The larger one is in and around the former Unit 2 Containment area and the smaller one is in the pumphouses' excavations. The Kittery belongs to the Merrimack Group, whose age is probably no younger than Ordovician and possibly Precambrian.

(2) The Newburyport Pluton is a complex dioritic intrusive of possible Late Ordovician age or older. This rock unit comprises most of the rock in the excavations.

(3) A hornblende diorite of unknown stratigraphic affinity, but also of very minor volume, occurs along particular horizons within the larger Kittery xenolith.

(4) Diabase dikes, part of regional dike swarms, of both Paleozoic and Early Triassic ages, occur essentially all across the site.

The Kittery metasediment consists of a brownish-gray, impure, micaceous quartzite. It is locally schistose or phyllitic but is generally massive and weakly foliated. The rock consists of quartz, sericite, plagioclase partly altered to sericite, chlorite, and biotite. Schistose zones are more micaceous and chlorotic. A few limited occurrences of bedding were observed. These contained no criteria for top-bottom determination. The beds which are evident are somewhat calcareous. Occasional spindle-shaped calcite concretions are present in the Kittery, paralleling bedding and foliation strike. Some concretions, especially those close to contacts with the Newburyport pluton, are partly or completely silicified. The xenoliths' contacts with the Newburyport are locally gradational, producing mixed varieties of quartz diorite and quartzite with considerable lit-par-lit texture developed.

The Newburyport pluton, as described by Shride (1971), is a syntectonic intrusive complex which consists of a 4-mile-wide central core of quartz monzonite and granodiorite, a 1-mile-wide outer rim of porphyritic granodiorite and, at the northern margin, a mile-wide or narrower apron of gneissoid quartz diorite. Fringing this quartz diorite apron is an "intrusion breccia," the phase of the Newburyport represented in site excavations. This rock, in turn, consists of two intrusive phases. The older phase is a dark gray to black, fine, even-grained diorite which consists of amphibole, plagioclase and minor quartz, biotite and opaques. The younger phase is a light gray, medium-grained quartz diorite composed of quartz, plagioclase which is somewhat sericitized, biotite partly altered to chlorite, and minor apatite and opaques. The two phases occur in about equal amounts in site excavations. The diorite exists as autoliths enclosed by quartz diorite. Quartz diorite also occurs as apophyses in diorite. The apophyses and elongate autoliths give the Newburyport a rough east-west fabric or foliation which is the result of igneous flow. Sharp diorite-quartz diorite contacts are often characterized by a thin (1/10 inch) concentration of biotite in the diorite along the contact. Diorite-quartz diorite contacts are typically sharp but are often permeated. Locally, and on a small scale, mixing of the two phases produces a porphyritic diorite with medium grained plagioclase phenocrysts in the fine-grained diorite. This porphyritic rock and the elongations of autoliths suggest that the diorite was still somewhat plastic when the quartz diorite intruded. A few occurrences of porphyritic quartz diorite also exist.

These have a matrix of typical quartz diorite with phenocrysts of plagioclase up to ¾ inch long. This latter porphyry is similar in texture to the porphyritic granodiorite phase of the Newburyport. Other minor textural variations of the Newburyport at the site include a fine- to medium-grained quartz diorite and a medium-grained granodiorite. With a few isolated exceptions, only the quartz diorite phase of the Newburyport is in contact with the xenolithic inclusions of the Kittery Formation.

The Newburyport at the site contains local minor pegmatites commonly less than six inches thick. These are typically irregular and often pod-like, persisting for only a few tens of feet or less. While most pegmatite contacts are sharp, some gradational contacts with quartz diorite apparently reflect their direct genetic relationship. Minerals in the pegmatites are quartz and plagioclase with minor microcline and biotite and very minor tourmaline. Limited occurrences of epidote veins also exist in the Newburyport. These frequently form the matrix of a fused breccia of dioritic rocks. The veins are tan, massive, and very fine grained. Widths range from hairline to two feet, and lateral persistence is short, similar to that of pegmatites. In one location, pegmatite crosscuts an epidote vein.

Hornblende diorite occurs locally along west-northwesttrending zones in the Kittery metasediments in the former Unit 2 excavations. None is present in the Newburyport igneous rocks. The hornblende diorite's primary mineral is amphibole. It also contains muscovite, sericite after plagioclase and minor apatite, opaques, and chlorite. It is a massive, medium-grained, dark gray, green-tinged rock. This rock occurs in discrete pods which are present beside fault CII-1 and in two places alongside fault CII-3. The hornblende diorite weathers quite readily upon exposure, resulting in a tan to cream rind. It also has a characteristic earthy odor. Contacts with the metasediments are sharp, often slightly open, and lack chill margins. The hornblende diorite is undoubtedly a deep intrusive and it is apparently restricted to the Kittery. Its texture and restricted occurrence suggest it is older than intrusive rocks of the Newburyport pluton. A sheared sample of this rock yielded a rather extreme K-Ar age date of 1181 ±119 million years (Table 2.5-1). Although the accuracy of such an "age" may be questionable, it does not contradict the contention that this rock is ancient.

Diabase dikes are present throughout the site excavations. These dikes are generally aphanitic and dark grey to black. They consist of plagioclase and pyroxene, olivine, serpentine after olivine(?), pyrite, chlorite, and chlorite after biotite. Textures range from ophitic to subophitic. Amygdules which occur infrequently consist of calcite or zeolite. Calcite veins are sometimes associated with the diabase dikes and, in several cases, calcite intrudes and forms the matrix of fused breccia of diabase fragments. Dikes range in thickness from about one inch (nonpersistent splays are thinner) to twelve feet over the site. Composite dikes, observed in site-vicinity borings, also occur at the site. A few exposures of one dike cross cutting another are present in site excavations. The dikes are dilative with only two instances of replacement-permeated contacts, both near faults. Margins of the dikes are normally fused against the host rock, although segments of many dikes have open contacts. Dike contacts are chilled, with chill margins commonly emphasized by a brownish-grey staining. Microscopically, the stained margins appear thoroughly oxidized. This staining may derive from the same source as occurrences of tan bleaching which are associated with dikes. Such bleaching was also encountered in the site-vicinity country rock and has been attributed to Triassic carbonate metasomatism (Subsection 2.5.1.2a.8). Site excavation studies are in agreement with this interpretation on the basis that bleaching on site is limited to dikes or country rock close to dikes.

The dikes in site excavations ordinarily follow fractures which appear to have been tension-induced at the time of dike intrusion. They generally trend N35°-45°E and dip 80°N to 75°S. On the site, the dikes may occasionally follow pre-existing fractures of other orientations in the country rock but can always be traced vertically or laterally into segments with more typical orientations. Dikes are apt to narrow and end abruptly, then reappear two to 20 feet laterally distant with both segments sharing the same orientation. These en echelon offsets are generally left-stepping, although right-stepping dikes also occur. In most of these cases there is no apparent reason for these disjunctions to occur in the massive Newburyport host rock. The presence of the Kittery xenolith may be responsible for some of these disruptions. Others may have resulted from heterogeneity imparted to the Newburyport by the presence of some faults. However, such en echelon arrangements of dikes are not directly caused by movement on faults. Instances of dike offsets due to fault displacement are discussed in Subsection 2.5.1.2b.7(b).

(b) Structure

The Seabrook Station site is located in the Newburyport pluton just south of its northern contact with the Kittery Formation. The principal rock in site foundation excavations is Newburyport diorite and quartz diorite. This igneous rock is fairly massive and homogeneous over the site.

The Kittery quartzite occurs on the site as two large xenoliths. One of these comprises most of the rock in and around the former Unit 2 Containment area. The second occurs in the pumphouses' excavation (Figure 2.5-19). The contacts of the larger xenolith in the former Unit 2 area trend about N80°W and appear to dip steeply. This exposure of metasediment extends in a rough band about 200 feet across and 640 feet long from the north to the south side of the former Unit 2 excavations. In the foundation excavation for the pumphouses, the second, smaller Kittery xenolith trends about N40°W and dips 30°-45°SW. It is some 210 feet long and varies from six to 22 feet wide. This xenolith was observed to pinch out vertically toward the top of the pumphouses' excavation north and south walls.

The contacts between the Kittery and Newburyport range in character from sharp to gradational, veined, permeated, or veined and permeated. Locally there is considerable mixing of the two rock types. The northerly contact of the former Unit 2 area xenolith is mostly gradational, consisting of a mixed zone of quartz diorite and quartzite one to five feet wide. Mixing takes the form of lit-par-lit veining, permeation of the metasediment by quartz diorite, or both, and much of the metasediment gives the appearance of having been baked or partially digested by igneous rock. The contact on the south side of his xenolith is mostly sharp with zones of mixed rock present only locally. Just to the south of this contact is a zone about two to ten feet wide which consists of interfingered lenses of metasediment and apophyses of quartz diorite. These metasedimentary lenses show various stages of alteration.

The Kittery and Newburyport units contain several structural elements including joints, foliation, and faults. Joints in the Newburyport trend mostly northeast while in the Kittery joints trend mostly west-northwest or northwest. The larger Kittery xenolith also contains a fairly strong set of northeast-trending joints. This latter set is especially well developed near the xenolith's contacts. Foliation in both rock units trends approximately eastwest, but this conformity is apparently incidental. This east-west fabric is also present to the south and west along the northwest and west margins of the Newburyport pluton (Shride, 1978).

Diabase dikes at the site cut across the fabric of the country rock, trending N40°-45°E and generally dipping steeply south or occasionally steeply north (Figure 2.5-19). In the excavations, these dikes range from 1 inch to 12 feet in thickness. These dikes are dilative and sometimes show crosscutting relations with each other. Dikes are for the most part tabular, although local segments have various irregularities. Spacing between dikes in site excavations generally ranges from 30 to 300 feet.

(1) <u>Jointing</u>

Joints are the most common geologic structural element of the igneous and metamorphic rocks in site excavations. Figure 2.5-21 and Table 2.5-3 summarize the characteristics of the various joint sets on the site.

In the Kittery xenolith, west-northwest- and northwest-trending joints are prevalent while in the Newburyport, northeast-trending joints are most common. The two most common joint sets in the Newburyport, which strike northeast and dip northwest and southeast, are also locally fairly strong sets in the Kittery especially close to the xenoliths' contacts. Most joints are only slightly weathered or unweathered, closed or tight, planar and smooth. Joints typically are discontinuous, having relatively short strike lengths. Joint frequency decreases with depth in the site excavations which range from elevations +10 to -67(MSL). Mineralization with calcite, quartz, minor chlorite, pyrite, or epidote is sometimes present along joints in the Newburyport. In the Kittery, surfaces are frequently coated with chlorite and minor pyrite.

Slickensides are occasionally present on surfaces described as joints. Where this occurs movement has been observed to be exceedingly small or imperceptible and/or the surfaces themselves have strike lengths of about 20 feet or less.

Jointing in the diabase dikes is prismatic (columnar) in nature. Joint strikes are generally perpendicular or parallel to contacts along any given segment of a dike.

Joint orientations are fairly consistent within both the Kittery and the Newburyport. Figure 2.5-21 shows symbols indicating the orientation of well developed joints in site excavations. Each symbol represents an average of several readings or an individual reading taken from a surface representative of joints in that location. This Figure shows the consistency of jointing patterns within the two rock units in site excavations, and also demonstrates the absence of anomalous trends.

(2) Foliation

Foliation is present in both the Kittery and the Newburyport rocks. The only significant cleavage along foliation in site excavations occurs in schistose or phyllitic zones in the Kittery metasediments. Figure 2.5-22 shows symbols indicating the orientation of foliation. Deviations from the general trends are infrequent and generally occur on a small scale. Foliation in the Kittery trends N80°-90°W and dips steeply both to the north and south. This foliation results largely from the local concentration of micaceous minerals, especially chlorite. Zones with particularly well developed foliation occur intermittently in the larger Kittery xenolith in the former Unit 2 area. The rock in these zones is schistose or phyllitic. They are particularly notable because faulting has occurred along some of them.

Foliation or gneissoid banding in the Newburyport trends about N80°-90°E and dips steeply or is vertical. This foliation is largely defined by the elongation direction of diorite autoliths in the quartz diorite. It results to a lesser extent from aligranent of feldspars and micas in the quartz diorite.

The foliation of the larger Kittery xenolith in the former Unit 2 area is generally conformable with that in the Newburyport. It is also parallel to foliation in the in situ Merrimack Group rocks to the north of the site. This conformity is probably fortuitous; as discussed earlier, the Newburyport pluton possesses an east-west foliation throughout (Shride, 1978).

(3) Faults

A total of 61 faults has been identified and mapped in site foundation excavations. These are shown in Figure 2.5-19. Table 2.5-2 lists fault numbers, locations, orientations, and various descriptive parameters for all mapped faults.

Practically all faulting at the site appears to have occurred along pre-existing joints or foliation planes. Many of the faults are short and none is considered through-going. All except one fault (fault CT-2) terminate with at least one end in the site excavations. In addition to limited lateral extent, some faults show limited vertical extent. Displacements are small, measuring only inches or a few feet for most faults. The scale of faulting suggests it has resulted from minor crustal adjustment. Crosscutting relations and fault motions indicate that the most recent faulting at the site was contemporaneous with diabase dike emplacement and is thus quite ancient.

Site faults were divided into seven sets based primarily on their orientations. The genetic relationship of faults within a given set is evidenced not only by coincidence of orientations but also by their proximity or continuity, similarity of motion sense and slickensides, mineralization and cross-cutting relationships (Table 2.5-2). It is also apparent that fault sets are, in turn, related to one another. This is evidenced by their proximity in space, their motion senses and orientations considered together, and their crosscutting relationships.

Evidence indicates that motion on all these faults is ancient. Some sheared bedrock is hard and fused in such a way as to indicate faulting took place at significant depth below the earth's surface. Several faults contain undisturbed, crystal-lined vugs or surfaces which indicate movement has not occurred since these crystals formed. Five of the seven fault sets include at least two faults which are crosscut by diabase dikes. Intrusion paths of dikes are clearly affected at many faults indicating the existence of these faults prior to the intrusion of the diabase. Initiation of movement along some faults probably resulted from the intrusion, or from cooling and contraction of the dikes themselves. Movement on faults is generally normal and consistent with the tensional stress field which induced dike intrusion. Radiometric dates indicate faulting movements took place before or during intrusion of the diabase dikes. Based on these general facts and on numerous details and observations from site excavations, the principal conclusion of the site excavation mapping study is that none of the faults on the site constitutes any current or potential site safety problem.

a. Set No. 1: NNW-Striking, E-Dipping Faults

Twenty faults with north-northwest strikes and steep east dips are present in the Newburyport in site excavations (Table 2.5-2). These parallel a well developed joint set in the Newburyport. They consist mostly of single discrete fractures in widths of -to ¼ inches. Locally, zones up to three inches across are affected by fracturing. Lengths range from 14 to 260 feet. Although orientations of most of these faults show only minor local variations, several of the faults in this set change orientation near their terminations to strike northeast and dip steeply southeast, paralleling faults of Set No. 3. Northeast-striking, southeast-dipping faults of Set No. 3 (CI-1, CI-1A, and CI-1B) all have significant segments which strike north-northwest and dip east. This association suggests a genetic relation between fault Sets Nos. 1 and 3. Such relations are discussed below.

Slickensides are common and fairly consistent in orientation on faults of Set No. 1. Slickensides generally suggest, as do associated displacements, normal movement down to the east with a small right lateral component. Ratios of strike-slip to dip-slip motions are commonly 0.4 to 0.5 with a few as low as 0.08. Net slips range from less than one to 41 inches.

Mineralization along these faults is common, consisting of calcite with some quartz. These minerals occur in pods up to three inches across or as thin coatings on fracture surfaces. In four cases (EI-1, NI-1, NI-2, and W-5), faults are mineralized with coarse, drusy, prismatic calcite crystals. The undisturbed nature of these crystals, existing as they do within these faults, indicates the lack of movement along these surfaces since the formation of these crystals. Weathering along these faults ranges from slight to severe; the more intensely weathered segments are highly localized.

Faults of this set both offset and are crosscut by diabase dikes and associated dike mineralization. Ten of these faults transect and displace diabase dikes while seven are crosscut by unbroken dikes. Figure 2.5-23 shows dike d5 crosscutting fault EI-1. Examples of offset dikes are shown in Figure 2.5-23 and Figure 2.5-24. Figure 2.5-23 shows dike d5 offset by fault W-6. On the footwall side of the dike, a thin apophysis of diabase intrudes along the fault to the north. This configuration shows that the fault plane existed as a receptive plane of weakness at the time of the dike's intrusion.

Figure 2.5-24 shows faults DI-1 and W-1 cutting dike d5. The dike's intersection with DI-1 was so heavily coated with calcite that measurement of the offset was not possible. Dike d5 does appear to be offset by fault W-1. Immediately adjacent to and within d5, fault W-1 contains two pods of calcite up to three inches across. Dike contacts are offset across these pods with a horizontal separation of about eight to nine inches. Between these pods and along the fault's trend in the dike extends a very thin, closed crack with a very minor coating of calcite. The center portion of the dike thus looked barely disturbed by this faulting.

b. <u>Set No. 2: NE-Striking, NW-Dipping Faults</u>

Ten faults with northeast strikes and moderately shallow northwest dips are present in site excavations (Table 2.5-2). They are parallel to the best developed joint set in the Newburyport igneous rocks. Orientations of these faults are fairly consistent. Significant attitude deviations occur only along two of these faults where, in the Kittery xenoliths, faults CI-2D and SII-1 change strike to north-northwest and dip west.

Most of the faults in this set shear and/or fracture rock in narrow zones up to six inches wide. Lengths are variable, generally ranging from 77 feet to 575 feet.

Slickensides are present on the surfaces of faults CI-2, CI-2C, and CI-2D. Slickensides orientations are fairly consistent and indicate, as do offsets for all faults in this set, normal movement down to the northwest. Net slips are typically several feet in magnitude. Mineralization is absent or consists of only minor quartz.

Three of the faults in this set, CP-1, TII-1, and DII-1, are quite different from the rest. These are narrow, discrete fractures from 10 to 20 feet long. They are closed or tight and often fused. None is slickensided and they are distinguishable from adjacent joints only because of associated offsets.

Weathering along faults in this set is variable. For three short faults, CP-1, TII-1, and DII-1, weathering is slight except for a short section of DII-1, which is severely weathered. The remainder of these faults is moderately weathered with frequent severe weathering and minor, local extreme weathering.

Two of the faults in this set, NI-4 and SII-1, are crosscut by dike d5. Fault NI-4 is displaced by fault T-1 of Set No. 1.

Five faults in this set, CI-2, CI-2A, CI-2B, CI-2C, and CI-2D, referred to hereafter as the "CI-2 group," are intimately related in space with dike d1. Faults in the CI-2 group either end against or slightly offset dike d1 at various locations. The total net slip calculated from measurements of offsets of features in the Newburyport is far greater than that suggested by offsets of dike d1. Figure 2.5-25(a) is a sketch showing dike d1 cut by CI-2 in the EHI area. The fault passes through the dike here as a thin zone of severely weathered rock containing a" or thinner layer of extremely weathered, light gray, silty material (gouge?). There is no apparent offset of dike contacts along the fault. However, the dike's path was clearly disrupted by the presence of the fault during intrusion. Near the fault the dike's contacts are curved and irregular, especially along a six- to seven-foot irregular section where the contact is of replacement-type rather than dilative-type.

Figure 2.5-25(b) shows CI-2C cutting dike d1. This exposure is oriented similarly to the one shown in Figure 2.5-25(a), and shows similar relationships. The intrusion path of the dike has obviously been affected by the fault, and there is a 10-foot section of the dike's southerly contact above the fault which is replacement-type. A displacement of one dike contact can be seen here represented by a few inches of apparent reverse motion. It seems likely, however, that this apparent displacement sense results from the intrusion of an apophysis of diabase which extends up the fault on the hanging wall side. A normal movement sense is indicated by displacement and slicken-sides elsewhere along this fault.

Figure 2.5-25(c) shows the relations between fault CI-2 and dike d1 in the SWT. Here CI-2 ends against dike d1 without offsetting or even penetrating d1. Immediately adjacent to this intersection dike d20 is displaced along a discontinuous shear which occurs right next to, and is undoubtedly related to, CI-2. This offset is essentially clean although the dike did spread against the hanging wall of the fault during intrusion. In this immediate area, dike d20 is crosscut by dike d1.

c. Set No. 3: NE-Striking, SE-Dipping Faults

Ten faults which strike northeast and dip steeply southeast are present in site excavations (Table 2.5-2). These faults parallel a joint set which is well developed in the Newburyport and somewhat less common and prominent in the Kittery. The orientations of most of these faults are fairly consistent. Attitude variations occur along three en echelon faults in this group, CI-1, CI-1A, and CI-1B. These three contain segments which strike N10°-20°W and dip 50°-80°E, reciprocal to the northeast strike variations along faults in Set No. 1.

Fault lengths range from 30 to 390 feet. They generally range from-inch wide discrete fractures to three-inch wide closely fractured zones. The southwesterly end of fault CI-1 is an exception, occurring as a diffuse four-foot wide zone of close joints.

Mineralization of this set with quartz and calcite is common. This mineralization often fuses the faults and ranges from thin coatings to stringers up to three inches across.

Weathering is variable. It ranges from slight to moderate with some localized severe to extreme weathering in a few faults.

Slickensides are usually present. Along with offsets they suggest that movement on these faults is essentially normal, down to the southeast. Two faults, CT-3A and CT-3B, show slickensides which indicate left-lateral strike-slip components slightly higher than their dip-slip components. The motion and attitudes of these two faults are intermediate in direction between faults of this set and faults CT-1 and CT-2, of Set No. 5, described below.

Only three faults in Set No. 3, CI-1, CI-1A, and TW-1, show a crosscutting relationship with a diabase dike. Figure 2.5-26(a) illustrates fault CI-1 where it is crosscut by dike d21 and apparently crosscut by dike d5. In the case of d5, a thin, slightly weathered crack does pass through this dike along the trace of the fault; however, no displacement of dike contacts is visible. It is clear that d5 was deflected along CI-1 during intrusion as evidenced by its change in dip.

Figure 2.5-26(b) shows fault CI-1A and dike d7. CI-1A caused d7 to be deflected during intrusion. The extension of the abandoned path is marked by a thin vuggy crack probably related to dilation. The fault appears to be crosscut by d7 although a thin, slightly weathered crack does exist in the dike along the path of the fault. A displacement of dike d3 by Fault CI-1 is shown in Figure 2.5-26(c). Only the easterly contact shows measurable displacement of dike d5 by fault TW-l (Figure 2.5-28).

d. Set No. 4: NE-Striking, Vertical Faults

Four faults with northeast strikes and vertical or nearly vertical dips occur in site excavations (Table 2.5-2). These faults parallel a joint set in the Newburyport and also parallel the orientation of diabase dikes on the site. This fault set occurs almost exclusively in the Newburyport with only a short segment of fault A-1 in the Kittery. Lengths range from 44 to 400 feet. The width of SI-1 and DI-3 is aboutinch and these are discrete fractures. Fault A-1 is a two- to five-inch wide, closely jointed, weathered zone with considerable mineralization. Fault A-1A consists almost entirely of a zone of fused breccia zero to 18 inches wide. Orientations of these four faults are quite consistent.

Slickensides are absent. Movement sense based on offsets are normal for faults SI-1 (down to the north) and DI-3 (down to the south). Fault A-1 shows apparent left lateral offset while A-lA has no crosscutting relationships visible. Measured displacement ranges from five inches to three feet, although in the case of A-1A no measurable displacement is visible.

Weathering ranges from slight to severe except along portions of DI-3 and A-1A which consist of fused vein-calcite. These portions of these two faults are unweathered or only slightly weathered.

Displacement of the Newburyport is observed or inferred for all four faults. Crosscutting relationships with diabase dikes are visible in three cases. Figure 2.5-27 shows fault SI-1 intruded along its length by dike d4. The portion of d4 within SI-1 is only locally weathered and is not sheared. Figure 2.5-24 shows fault DI-3 extending partway into dike d5. There is displacement along this fault immediately adjacent to d5. However, the fault does not displace the northerly contact of d5 nor does it extend completely through this dike itself. Fault A-1A contains a few pods of fresh unsheared diabase near the northeast corner of the site. Some small angular pieces of diabase also occur in this fault's fused breccia near this same location.

e. <u>Set No. 5: ENE-Striking, S-Dipping Faults</u>

Two faults with east-northeast strike and steep southerly dip occur in the southwest corner of the site excavations (Table 2.5-2). These parallel a weakly developed joint set in the Newburyport. These two faults have fairly consistent attitudes although CT-1 shows local dip-reversal to the east. Fault CT-2 is the only fault in site excavations which has both its ends beyond the limits of those excavations. CT-1's easterly end is within site excavations where it stops against fault PII-1A, as depicted in Figure 2.5-29(b). Site excavations expose a total of 342 feet and 255 feet of CT-1 and CT-2, respectively. CT-1 is a narrow (inch or less) fracture for most of its length while CT-2 is generally a closely fractured zone several inches wide. Slickensides and offsets indicate movement sense is left lateral with some small normal component, down to the southeast. Displacements, computed in terms of net slip, range from one to eight inches.

Calcite mineralization along these faults is common, and weathering is slight to severe.

Fault CT-2 cuts and displaces dike d1 a small amount near the southwesterly end of that dike. Set No. 5 (consisting of just two faults) is one of only two fault sets which do not contain at least one fault which is crosscut by a diabase dike. Fault CT-1 is, however, terminated by fault PII-1A which is, in turn, transected by an unbroken section of dike d1.

f. Set No. 6: WNW-Striking, Steeply Dipping Faults

Nine faults with west-northwest strikes and steep dips occur in site excavations (Table 2.5-2). These occur exclusively within the Kittery xenolith which is present in and around the excavations for the former Unit 2. They parallel the best developed joint set and the foliation in this xenolith, and appear to have developed along phyllitic zones in the metasediment. While strikes are fairly consistent, dips often vary from north to south. Most of these faults consist of zones which are several feet wide and contain numerous chlorite-coated fractures parallel to fault strike plus one or more branches of more intensely sheared rock. Wide zones sometimes narrow to single discrete shears only a few inches wide or less.

Fault lengths range from 89 to 500 feet. All terminate with at least one end in site excavations.

Mineralization consists mostly of chlorite coatings on fracture surfaces with occasional minor pyrite encrusted on surfaces, and minor calcite and quartz. At one location along CII-1 graphite accompanies chlorite.

Weathering is variable along these faults. Rock in the wider portion of fault zones is normally fresh or only slightly weathered with moderate or slight weathering regularly present on associated joints. Severe to extreme weathering is of limited extent and is mostly restricted to the more closely fractured branching shears within zones.

Slickensides directions are typically scattered. This probably results from two movements having occurred along some of these faults as well as from differential movement having occurred along variously oriented surfaces within fault zones. In general, slickensides suggest left-lateral displacement, with a small high-angle reverse component apparent in some cases. Faults CII-1 and CII-2 contain slickensides which suggest left-lateral normal motion. CII-2 also contains a few slickensides (lateral) suggestive of right-lateral motion. Offsets indicate left-lateral displacement for all faults in this set.

Displacements are usually small, with the measured range of 5 to 30 inches typical. Apparent offsets up to 12 feet are, however, present for CII-1 and CII-2. Dike displacements of this magnitude are shown in Figure 2.5-28, and are discussed below.

Figure 2.5-28 shows faults CII-1, CII-2, and TW-1, and diabase dikes d5, d11, d12, and d13. Faults CII-1 and CII-2 here consist of zones from a few inches up to several feet wide often containing or consisting of one or more branching shears. Dikes d11 and d12, which strike northeasterly, are offset in a stepwise manner by both CII-1 and CII-2. The total measured horizontal separation of each dike along each fault is approximately 12 feet. Dike d13, which also strikes northeasterly, is offset along CII-2 alone since it does not extend far enough south to intersect CII-1. Horizontal separation of d13 along CII-2 is also about 12 feet. Dike d5 trends into the area shown in Figure 2.5-28 from the east, having intruded along fault CII-2. Portions of this dike are chilled against dikes d11 and d12. Dike d5 also crosscuts the entire width of CII-2 and all except one discrete fracture plane within CII-1. This one shear offsets d5 an inch or two in a left lateral sense. Immediately to the south of this area d5 strikes northeasterly.

All of the dikes and displaced dike segments here show either some apparent broadening or narrowing against shears or some shape alteration compared with the generally tabular form of these dikes in adjacent areas. Despite the spreading of some dike segments against fault planes, all the visible offsets are clean breaks with only one segment of d11 having any apparent chilled margin against a fault surface. These irregularities, especially of d11 and d12, strongly suggest dike intrusion, continued during or just after faulting. Particularly well documented examples of such dike-fault relations are described in a report of fault investigations at the Shearon Harris Plant Site near New Hill, North Carolina (Carolina Power & Light Co., 1976).

Immediately west of the area shown in Figure 2.5-28, a 10- to 15-foot deposit of glacial till resting on bedrock was mapped in profile. This mapping was done to determine if any displacements were present in this till overlying faults CII-1 and CII-2. The till, which has stratified fabric here, shows no disturbance of any kind at or in the vicinity of these two faults.

Figure 2.5-29(a) shows dike d5 and fault CII-2 at a point east of the area illustrated in Figure 2.5-28. Dike d5 trends from the east into this location striking N45°E. This dike changes strike at and to the west of its intersection with CII-2. The dike is chilled against the fault surface and displays no evidence of offset or faulting here although one short, discontinuous splay from CII-2 does cut across d5. The change of d5's strike here obviously represents the magma's exploitation of a path of least resistance in spite of its contrary orientation.

Figure 2.5-29(b) shows dike d1 crosscutting fault PII-1A. Along the dike's northwest contact there is, for a very short distance, minor irregularity where the contact trends along PII-1A. Here the dike's chill margin is continuous and unbroken, and no fracture extends into the dike. Along d1's southwest contact no irregularity is present at PII-1A although some tight joints do extend six to eight inches into the dike. These joints do not displace the dike's chill margin.

g. Set No. 7: NW-Striking, SW-Dipping Faults

Four faults which strike northwest and dip with moderate steepness to the southwest are present in site excavations (Table 2.5-2). These faults lie almost entirely within the larger Kittery xenolith and parallel a joint set which is fairly to poorly developed in the metasedimentary rock unit on the site. These faults consist mostly of zones several inches wide which are characterized by very close, chlorite-coated joints. These surfaces are wavy and commonly are moderately weathered. Isolated instances of severe or extreme weathering are developed along faults EII-1, EII-1A, and EII-2. Fault EII-3's tight jointing and lack of weathering contrasts with the other three faults in this set.

Strikes of these four faults are fairly consistent. Dips vary over short distances on faults EII-1 and EII-3. Mineralization is similar to that in Set No. 6: surface coatings of chlorite, minor pyrite encrustations, and local minor quartz.

Slickensides were observed at two locations along EII-1 and in one spot on EII-2. Slickensides and offsets suggest normal displacement, southwest side down, with a significant right lateral component for these faults. At two locations along EII-1, a second set of slickensides is developed. These slickenslides are crosscut by the normal right-lateral set and they suggest an earlier reverse motion almost directly updip. Drag in the metasediments reflects both right-and left-lateral motions along faults. Displacements range from about one to eight feet.

Figure 2.5-30(a) depicts the intersection of fault EII-1 and dike d1. The fault occurs in and near the dike as a discrete, severely to extremely weathered fractureto one inch thick. Dike rock adjacent to the fault is severely weathered within one to two feet of the fault plane. This weathered diabase grades progressively into fresh diabase over a distance of about 12 feet on either side of the fault. Country rock adjacent to fault EII-2 and dike d1 is not appreciably weathered. It is not possible to make a precise measurement of the offset of dike contacts along fault CII-1, largely because of the irregularities of the dike's contacts at the fault. It even appears that there may be essentially no offset of d1 along EII-1. There is a measurable offset of a cooling joint within d1. This displacement is two inches in an apparent left-lateral sense, and it is significantly less than the total slip measured elsewhere along this fault.

Figure 2.5-30(b) shows the intersection of fault EII-2 and dike d1. This fault is similar in character to EII-1 where it cuts d1 (Figure 2.5-30(a)), although the pervasive severe weathering along the dike is not present here. The dike's south-easterly contact and a cooling joint are measurably displaced at this exposure, both showing apparent right-lateral offset consistent with slickensides directions.

(4) Pseudo-Offsets of Dikes

A number of pseudo-offsets of diabase dikes along joints are visible in site excavations. Two examples are presented in Figure 2.5-32, and another in Figure 2.5-24. Dikes show what appears to be offset along fractures which are obviously not faults. In these cases the fracture is continuous through the diabase, but the fracture is also so irregular it precludes the occurrence of motion along it. In some cases, as in Figure 2.5-32(b) other features such as dikes and joints crosscut these fractures as possible further evidence that no faulting has occurred along the fracture. Such pseudo-offsets are often characterized by a spreading of the dike against the joint plane. The similarity between these circumstances and the offsets of dikes along faults elsewhere (Figure 2.5-23(c), Figure 2.5-24, Figure 2.5-25, Figure 2.5-26, Figure 2.5-28, and Figure 2.5-30) is sufficient to suggest that such offsets of dikes along faults are at least partly due to deflection of diabase magmas during intrusion.

(5) Fault Ages

Site bedrock shows evidence of essentially two episodes of ancient faulting. One of these is concluded to be Paleozoic in age, the other Early Mesozoic. The evidence of Paleozoic faulting is limited to faults in Sets No. 6 and 7 and possibly Fault CI-2 in Set No. 2. It probably occurred at the time of the intrusion of the Newburyport pluton, over 400 million years ago. A second faulting episode occurred in the Mesozoic. Its effects are somewhat more pervasive and are related to the intrusion of the numerous diabase dikes in the immediate area and the surrounding region, slightly over 200 million years ago.

Potassium-argon age dating was used to confirm the structural and stratigraphic evidence for fault ages. It should be remembered that this method yields only minimum ages for rocks older than about 240 million years, due to a Permian thermal event (crustal cooling) which included much of New England and affected the chemical balance of the K-Ar system at that time (Zartman et al., 1970). Thus, the true age of specimens which date at or above 240 million years may be considerably older than K-Ar results indicate.

Based on the mapping studies and radiometric dates, no movements considered younger than about 212 million years are represented by faulting in site excavations. Thus, none of the faults mapped on the site is a capable fault as defined in Appendix A of 10 CFR, Part 100.

Paleozoic faulting associated with the Newburyport intrusion, or perhaps with some other old compressional event, occurred along all faults of Sets No. 6 (WNW strike, steep dip) and No. 7 (NW strike, SW dip), confined to the larger Kittery xenolith. Motion along faults of Set No. 6 was concentrated along the larger, more schistose or phyllitic horizons in the xenolith. How much of the fracturing in these zones is shear-induced and how much due to cleavage along foliation is not known.

Indeed, one of the more formidable tasks of mapping the metasediments was the detection of evidence of displacement on west-northwest-trending features. Most of these phyllitic zones did show some evidence of movement, and some of the rock showed evidence of cataclasis but without any loss of primary cohesion. Some slickensides and offsets along these faults indicate motion with a dominant reverse component, apparently representative of this Paleozoic motion.

Faults of Set No. 7 (NW strike, SW dip) are similar in character to those of Set No. 6; however, they do not parallel a particularly well developed joint or foliation direction (Table 2.5-2). Slickensides, offsets, and drag indicate some reverse motion on two of these faults, and faulting has resulted in cataclasis without loss of primary cohesion in these faults also.

The earliest general antiquity of the motion on faults in Sets No. 6 and 7 is evidenced by their reverse-(compressional) motion sense. No dynamic compressional tectonism has occurred at the site since Early Mesozoic, as evidenced by the lack of overprint deformations on the slickensides of the 212-226 million-year-old normal faults at the site. Furthermore the retention of primary cohesion of the sheared rock in these zones indicates motion occurred when this rock was buried quite deeply in the crust. Minimum ages are imposed on these faults by the crosscutting of faults CII-2 and PII-1A, respectively, by dikes d5 (K-Ar dated 212 ±9 m.y.) and d1 (K-Ar dated 255 ±13 m.y.). Micas from sheared metasediments in fault CII-1 yielded a K-Ar date of 269 ±11 m.y. This also represents a minimum age for this fault motion. This latter date is similar to another date of 261 ±10 m.y. obtained for an earlier study (GEI, 1975) on sheared metasediments in the site vicinity (Subsection 2.5.1.2a.8(c)), and generally reflects a regional episode of crustal cooling.

One fault in the Newburyport, CI-2 from Set No. 2, shows evidence of an older movement, in the form of a single pocket of fused breccia within the fault. Like the sheared metasediment in Sets No. 6 and 7, this fused breccia is a fresh, hard rock which has lost no cohesion as a result of shearing. A K-Ar date of 246 ±13 million years was obtained from sericite of a breccia sample. Total stratigraphic throw on this fault is considerably more than can be accounted for by displaced diabase dikes along its trend.

Early Mesozoic fault motion for which evidence is visible in site excavations is essentially extensional in character. Normal displacement characterizes almost every offset and slickenside occurrence for faults in Sets No. 1, 2, 3, 4, and 5. Faults of Sets No. 6 and 7, which moved again in the Mesozoic, also show evidence of normal motion. This faulting was contemporaneous with diabase dike emplacement. Five of the seven fault sets include at least one fault which is crosscut by one or more diabase dikes while six out of seven fault sets include faults which offset dikes. These crosscutting relationships furnish a minimum relative age for this faulting. Also, details of the dike-fault relationships such as those shown in Figure 2.5-23, Figure 2.5-24, Figure 2.5-25, Figure 2.5-26, Figure 2.5-27, Figure 2.5-28, Figure 2.5-29 and Figure 2.5-30 clearly indicate the contemporaneous occurrence of faulting and dike emplacement.

Since the dikes on the site are of critical importance in determining the age of movement on faults in the site excavations, radiometric age determinations were performed on five dike samples. Table 2.5-4 shows the results of these age determinations along with four dates of diabase samples from borings in the site area, three others on sheared country rock from site excavations, and one on sheared country rock from a boring in the area. Seven of nine dates from dikes range from 212 to 236 million years. The other two, dike d1 at 255 ±14 million years and dike d3 at 295 ±14 million years, were apparently exposed to the Permian thermal event, and may be even older than their K-Ar dates indicate.

Dike d12 (Figure 2.5-28) yielded two different K-Ar ages. A sample of undisturbed diabase from d12 situated well northeast of fault CII-1 gave an age of 236 million years. A sample of sheared diabase from d12 right along the only shear in CII-1 which displaces d5 yielded an age of 213 million years. This younger age is practically coincident with the age of dike d5, 212 million years.

These dates strongly support the conclusion that the last motion on this fault was contemporaneous with dike emplacement. The structural relations shown in Figure 2.5-28 also demonstrate that the substantial motion on faults CII-1 and CII-2 took place between the time of intrusion of dike d12 and dike d5.

Mesozoic faulting in site excavations occurred exclusively along pre-existing zones of weakness such as joints or older faults. Assuming fault motion resulted from the same NW-SE tensional stress that induced dike emplacement, fault displacement sense should reflect that stress configuration. As discussed earlier, Early Mesozoic faults in site excavations are characterized by normal displacement typical of faulting in a tensional stress field. Probably because motion occurred along pre-existing surfaces rather than on surfaces formed by the tension itself, faults in Sets 1, 5, 6, and 7 have significant lateral motion components. However, it can be demonstrated that these lateral motions are sympathetic with predicted lateral motions for such a stress field.

Figure 2.5-33(a) shows the theoretical failure planes and relative motions along these planes which result from a tensional stress field when the least principal stress, σ3, is directed northwest-southeast. It shows a near-northerlytrending plane with a right-lateral strike-slip component, and an approximately east-west plane with a left-lateral strike-slip component. Figure 2.5-33(b) shows the motions observed on faults in site excavations. The faults' strike-slip component motions conform with the theoretical motions predicted in Figure 2.5-33(a). The north-northwest and northwest fault sets (Nos. 1 and 7) show right-lateral motion while the east-northeast and west-northwest sets (Nos. 5 and 6) display left-lateral motion. Northeast fault sets show no strike-slip component because they trend perpendicular to the least principal stress. It is reiterated here that faulting at the site occurred along pre-existing, favorably oriented planes of weakness (joints, foliation), and not necessarily in strict geometric conformance with the tensional stress under discussion. Nevertheless, the consistency of the strike-slip components of fault motions with such a tensional stress is excellent and it reflects the genetic link between dike emplacement and fault motion indicated by structural relations and age dating.

Relative ages of the various fault sets in foundation excavations were deduced from (1) crosscutting relations among faults themselves, that is, from displacements and terminations of one fault by another; (2) certain geometric considerations, such as splaying or coalescing of a fault of one set into one of another set, or such as the existence of segments of faults having orientations similar to those of faults of a different set; and (3) crosscutting relations among dike and fault combinations.

These data indicated that all faulting in site excavations is either contemporaneous with or older than diabase dikes at the site.

As a further affirmation of the ancient age of the west-northwest fault set (Set No. 6), overlying surficial sediments were mapped at two locations on the site. At the western-most extensions of faults CII-1 and CII-2, just west of the area shown in Figure 2.5-28, a crudely stratified till immediately overlying both faults shows no disturbance, displacement, or disruption of any kind either immediately over or in the vicinity of the faults. At the south side of the site excavations on strike with CII-1 and CII-2A, an exposure of glacial till with overlying marine silt-clay and outwash sand is present. Detailed mapping along a manicured cut in this material showed no offset or disturbance of a number of horizons including the till-marine clay contact, a sandy lens and a color-mottled zone in the marine clay, and the marine clay-outwash sand contact. Similar mapping was carried out prior to construction excavation in trenches dug in the area of the former Unit 2. These trenches crossed the bedrock surface over the traces of fault CII-1, CII-2, CII-3, EII-1, and SII-1. In mapping these trenches, no evidence of any deformation was found along any of the contacts between the surficial units or at the contact of the till and the bedrock surface (Appendix 2L).

7. <u>Site Engineering Geology</u>

(a) <u>Joints, Fractures</u>

Jointing is the most common geologic structural feature of the igneous and metamorphic rock in site foundation excavations. Characteristics of the several joint sets mapped at the site are described in Subsection 2.5.1.2b.7 and summarized in Table 2.5-3. The presence of joints in site bedrock results in no actual or potential adverse effects on plant structures.

Most joints in site excavations are closed or tight. Joint frequency decreases substantially with depth in these excavations, which extend as deep as 63 feet below mean sea level. Spacing was often very close for bedrock between elevations +10 and -10, a horizon which was typically very close to or included the original top of bedrock. Joints were often weathered (seamy) and open in this horizon. Where seamy, broken rock extended below design excavation lines, it was removed so that final-excavated surfaces exposed fresh, intact rock.

Design of foundations for several buildings required the excavation of deep foundations with resultant high vertical walls in bedrock. A number of these walls, especially in their upper sections, exhibited potential instability due to blasting. Where loose or potentially unstable rock underlay any part of the design foundation of structures benched into or adjacent to high vertical, walls, these walls were cut down or back as necessary to expose fresh, intact rock.

For purposes of construction safety rock bolts and/or chain link mesh were installed on many high walls. This rock support is neither intended nor necessary to improve the quality of the bedrock for site-safety purposes. No permanent rock bolting or other systems or rock reinforcing were required or installed in foundation excavations.

(b) Faults, Shears

Faults are described in detail and the question of their age relations is addressed in Subsection 2.5.1.2b.6.

Many of the faults in site excavations are indistinguishable from joints except for associated offsets, mineralization or slickensides. Indeed, all fault movement at the site occurred along pre-existing joints or fractures. These faults thus affect rock quality much as do joints. Some other faults locally contain pockets of severe weathering (rock mostly decomposed). These tend to strike northeast and dip northwest, strike west-northwest and dip steeply, and strike northwest and dip southwest (Fault Sets No. 2, 6, and 7, Table 2.5-2). Extreme weathering (rock totally decomposed) is rare and affects only very narrow (to inch) and short (less than 10 feet) portions of faults.

In all foundation excavations, the bulk of weathered material associated with faults was removed to expose fresh, intact rock. No fault in any of the foundation excavations affects foundation safety or stability.

(c) Foliation, Cleavage, Folds

Foliation is a locally prominent feature in both the Newburyport and Kittery formations in site excavations. This geologic structural element is described in detail in Subsection 2.5.1.2b.6.

In the Newburyport foliation, nowhere is developed in such a way as to control or promote pervasive breakage of the bedrock. Localized zones of closely or very closely spaced, chlorite-coated cleavage planes do occur in the Kittery xenolith in the former Unit 2 area. These commonly have the appearance of phyllitic partings, and may be comparable to thin phyllitic beds present in the Kittery near its type-locality north of the site. As discussed in Subsection 2.5.1.2b.6, faulting in this xenolith has apparently been localized along some of these zones.

As in the case with joints, the presence of foliation and cleavage in the bedrock underlying the site represents no actual or potential hazard to plant structures.

In site excavations only the Kittery xenoliths show evidence of broad regional folding. These xenoliths were detached from the south end of the Rye anticline, the axis of which extends north-northeast for about 14 miles. The attitude of foliation is consistent, striking N80°-90°W and dipping 80°S to 80°N. No small scale folding is discernable in the site excavations. The folded nature of the Kittery poses no hazard to plant structures.

(d) Weathering, Alteration

The bedrock in foundation excavations at the plant site is generally fresh, hard, and unweathered. Weathering is a significant feature only in a 10- to 20-foot zone associated with the top of bedrock, coincident with the close jointing found in this horizon, and locally in a small number of joints or faults. All weathered material associated with the bedrock surface was removed from foundation areas. Weathering persists along joints, faults, and fractures to some degree in all depths of the excavations. Most of this weathering is slight, and the more severe weathering is intermittent along scattered features which neither individually nor collectively represent significant decomposition of the bedrock.

Weathering or alteration of site bedrock is insignificant, and represents no hazard to plant structures.

(e) Cavernous, Unstable Lithologies

No cavernous or unstable materials comprise any portion of the bedrock in site foundation excavation.

(f) Unrelieved Residual Stress

A report on measurement of in-site horizontal stresses in the Newburyport pluton at the plant site is presented in Appendix 2H. These measurements were taken between elevations -2 and -12 (MSL) at depths of 30 to 40 feet below the bedrock surface. The measured stresses are generally comparable in magnitude to those previously measured in the New England region. No instances of rock behavior suggesting the presence of inordinate unrelieved residual stresses were observed during the excavation of rock for deep foundations of plant structures.

2.5.2 Vibratory Ground Motion

2.5.2.1 Seismicity

a. Local and Regional Seismicity

A cumulative seismicity map, inclusive of all pertinent data available as of June 1979, is presented on Figure 2.5-34. Circles with radii of 50, 100 and 200 miles from the site have been superimposed for ease of reference. Thresholds of intensity greater than III(MM) and magnitude greater than 3.0 have been used in this compilation. Figure 2.5-35 includes more recent events. Symbols on this Figure represent seismic events in terms of body wave magnitude (m_{blg}). All events in Table 2.5-5 are plotted in Figure 2.5-35. All events have been plotted according to observed (post-1968) or inferred (pre-1968) m_{blg}. This is done for consistency with parameters used in hazard studies (Weston Geophysical, 1982). All available parameters for each earthquake are listed on Table 2.5-5. The areal coverage of Table 2.5-5 has been expanded from that of the original FSAR to include New Brunswick, Canada, and recent seismic events there. A separate listing of events with dubious origin or coordinates so poorly defined that plotting is unwarranted is given in Table 2.5-6.

Three symbols are used to plot the epicenters on Figure 2.5-34. Events that occurred prior to 1968 are plotted preferentially according to their intensity (octagon), or magnitude (square) when only magnitude is available. Earthquakes that occurred after 1967 are plotted with a single symbol (diamond) to reflect an increased confidence attached to their epicentral and magnitude determination. The size of the symbols is proportional to the size of the events; the proportionality is not strictly linear but attempts to reflect the greater importance of larger events. On Figure 2.5-34, open symbols have been plotted in order to show all the events. On maps which combine geology and seismicity, the epicentral symbols have been filled for sake of clarity, even though some smaller events with identical locations are thereby masked by the larger ones.

As indicated on the seismicity maps, the entire site region has experienced only low to moderate seismic activity, upper-bounded by a few instances of Intensity VIII(MM) level. Within the 50-mile radius circle, a single Intensity VIII(MM) has been reported, and this Cape Ann event is considered to be structure-related in a separate seismotectonic province from that of the site (see Figure 2.5-36 and Figure 2.5-37).

1. Data Base

(a) Sources

The cumulative historical seismicity, as presented here, is taken from Weston Geophysical's earthquake data base. This data base contains a set of parameters for each earthquake, selected on the basis of a comparative review and evaluation of available listings, supplemented by extensive historical research. A parallel compilation was made of all entries contained in major earthquake catalogs and listings. These include the United States Earthquakes Series; The Earthquake History of the United States (1973); the Publications of the Dominion Observatory, and the Seismological Series of the Earth Physics Branch (both of Canada); the Bulletins of the Lamont Doherty Observatory and the New England Seismological Association; as well as listings by Mather and Godfrey (1927), Brigham (1871), Brooks (1960), Pomeroy (1977), etc. The parallel compilation of the above listings facilitated the detection of typographical errors and signaled discrepancies to be investigated. By noting the chronological order of important listings, such as those of Mather and Godfrey, Brigham, Brooks, Eppley, Coffman and von Hake, Smith, Pomeroy, etc., and by returning to quoted (original) references, it was possible, in many cases, to detect misinterpretations carried in these listings. The investigation of historical sources, such as newspapers, scientific bulletins, town histories, private diaries, etc., has contributed important earthquake information which has led to significant revisions of some older historical events. This is illustrated in the Historical Seismicity of New England (Boston Edison Company BESG-7601, 1976), published as part of the PSAR of the Pilgrim II site.

(b) Completeness and Reliability

In considering the cumulative seismicity of a region in terms of an assessment of seismic potential, it is necessary to examine the completeness and reliability of the data set. Because earthquakes are characterized either by their epicentral intensity or their magnitude, and are located by analyzing isoseismal contours and/or instrumental recordings, the spatial and temporal distributions of population and/or seismographic stations influence the number, size, and location of reported events. It is difficult to get a homogenous data set over a long period of time, as both population and networks constantly change; however, as long as proper thresholds and uncertainties are kept in mind, the data set can be normalized.

Even though major catalogs carry entries dating back to more than three centuries for some parts of eastern North America, completeness was only achieved for the early years above a relatively high-intensity threshold, i.e., Intensity VII(MM) or greater. For the region presently under consideration, it is realistic to assume that the seismic history is complete over the last 250 years for events that would be significant in terms of structural design, i.e., with intensities equal to or greater than VII(MM). Smaller events have been included in the earlier years, when they occurred close to settlements.

Figure 2.5-38 and Figure 2.5-39 show that the site region was populated very early; the progressive westward historical migration of the population, both in the eastern United States and Canada, is evident.

This set of older historical data, covering almost three centuries, is long and good enough to provide a valuable insight on the local seismic regime. Although lacking in focal depth information and often affected by large location uncertainty, it is nonetheless indicative of areas where seismic activity has been mostly and continually concentrated. It also permits formulation of a reasonable basis for the estimation of regional upperbounds.

The reliability of early historical data depends on many factors, such as population density, population literacy, and construction practices in the areas around epicenters. A lack of population in the true epicentral area of an event, for example, can lead to that epicenter being mislocated into the populated region where a maximum intensity level was reported. Besides shifting true locations, a lack of an evenly distributed population can also result in underestimated epicentral intensities. The opposite bias can occur in cases where felt reports come only from communities settled along river banks which characteristically experience enhanced ground motion due to the soil column, or where poor construction practices prevailed. In cases of structural damage, one must remember that construction standards were substantially different two centuries ago. Application of the Mercalli scale to reports of fallen chimneys, for example, without consideration of basic construction differences, results in an overestimation of intensity values. The degree of literacy among early settlers varied greatly. Some events could have been lost for lack of a competent recorder; in other cases, the contemporary literary style of the writer makes it difficult to distinguish the objective report from the subjective. For these reasons, historical reports need to be interpreted with care, and cross-correlated whenever possible.

With the beginning of the instrumental era, in the early 1900s, the quality of epicentral locations improved progressively with time, both in the United States and Canada. Yet for the first half of the century, many of the epicentral locations still relied heavily on felt reports.

For much of this era, from the start of the century and up to the 1960s, only a few seismographs operated simultaneously in the northeastern United States and eastern Canada. These stations were part of regional networks operated by the Jesuit Seismological Association (JSA), the Canadian government, and some American colleges and universities. In these early decades, numerous factors such as the type of instrumental response, lack of accurate time control, nonuniform geographic configuration of stations, use of the graphical method, and limited knowledge of crustal velocities were potential sources of errors and uncertainties in the epicentral coordinates. In the sixties, a major improvement in the coverage came about with the operation of the World Wide Standard Station Network (WWSSN), the Long Range Seismic Monitoring Program (LRSM), and the expansion of the Canadian Network for the Upper Mantle Project. The operational characteristics and station distributions of these networks were primarily oriented towards recording large regional and teleseismic events. The uncertainty to be associated with many local epicenters during the early sixties can occasionally reach a few tens of kilometers for small events.

Table 2.5-7, Table 2.5-8 and Table 2.5-9 list all pertinent information on eastern stations of the JSA, WWSSN, and Canadian networks. Information on LRSM stations, which in general operated for relatively short periods of time, can be found in Poppe, Naab and Derr (1979).

With the late 1960s, an increased interest in understanding the local seismicity has resulted in the implementation of dense seismograph networks in northeastern United States. Presently, seismic data for that region are gathered by the Northeastern United States Seismic Network (NEUSSN) and reported in its bulletin. This agency, incorporated in 1975 and funded by the Nuclear Regulatory Commission (NRC), the United States Geological Survey (USGS), the National Science Foundation (NSF), the New York State Energy and Resource Development Authority, and the New York State Science Services, reports earthquake hypocentral locations and magnitudes determined through the cooperation of the following institutions: Weston Observatory of Boston College (WES), Massachusetts Institute of Technology (MIT), University of Connecticut (UCT), Lamont-Doherty Geological Observatory (LDO), Pennsylvania State University (PSU), Delaware Geological Survey (DGS), and the Maine Geological Survey (MGS). Figure 2.5-40 shows the most recent configuration of NEUSSN, and Table 2.5-10 lists the participating stations.

Seismicity data for adjacent eastern Canada are reported in the annual <u>Canadian Earthquakes</u> Seismological Series of the Earth Physics Branch (EPB) of Canada.

Clearly, this large-density network can provide quite reliable epicentral locations with a small uncertainty of a few kilometers. It is also capable, in many instances, of yielding focal depth information, and for larger events, fault plane solutions.

(c) <u>Significance of Cumulative Seismicity Data</u>

The new information of focal depth is most important in the understanding of regional tectonics. Epicentral maps are two-dimensional while the causative structures of earthquakes are three-dimensional. In many cases, intraplate earthquakes are related to basement structures which have no surficial expressions. Depth estimates of the larger events for which hypocenters have been calculated indicate that the activity tends to be located between 5 and 20 km (i.e., neither at the surface nor at the bottom of the crust, but rather near the middle, or in the upper half of the crustal basement). This trend suggests that many of the basement structures can be detected and defined only by geophysical means.

It has been pointed out that the spatial distribution of epicenters, obtained from a denser configuration of instruments operated only in recent years, is a reliable indicator of the major patterns of the historical seismicity (Sbar and Sykes, 1977). The historical record, complete for more than two centuries at or above a significant threshold (e.g., Intensity VII), yields important information in assessing seismic potential. Although the understanding of causative mechanisms, based on fault plane solutions and focal depths, depends on the more accurate instrumental data recently acquired, the definition of the likely upperbounds characteristic of major active geologic features as well as the locations of large earthquakes likely related to structures are obtained on the basis of the long-term historical seismicity. Thus, the cumulative historical seismicity data, interpreted in the light of a careful review, yields valuable information on the spatial and temporal distribution of the larger and more significant events and on the location of zones of concentrated seismic activity.

A second important point to be made is the spatial coincidence of all larger historical events with the few zones of activity revealed by recent networks operated at a much lower threshhold of detection. This association suggests that the tectonic forces, which are relatively homogeneous over large regions of the continent, give rise to higher stress concentrations and releases in very specific areas where the seismic activity has been evident for centuries. In this context, the occurrence of large significant events becomes spatially predictable. If large earthquakes are still random in time, their occurrence in space is not; it is confined to continuously active areas because these are areas of certain geologic anomalies which have the ability to localize stress.

b. Recent Seismic Studies

Recent seismological research, carried out under the aspect of the New England Seismotectonic Study (Barosh et al., 1979), have findings pertinent to regional seismicity. Among the more important findings and repeated theses of this work are:

1. A very uneven distribution of earthquakes exists throughout New England with concentrated activity in only a few areas.
2. Population and seismograph distributions do not appear to have greatly biased the earthquake distribution results.

3. Seismicity correlates poorly with Paleozoic structures. A better correlation appears to exist between seismicity and Mesozoic features, particularly those related to Cretaceous continental margins.
4. High angle extensional faults are known or predictable in most areas of active seismicity.
5. Active areas are commonly located in lowlands, with elevations below 300 meters (msl). Exceptions are the White Mountains in New England and the Adirondacks in New York.
6. Seismically active lowland areas are presently subsiding. Passamaquoddy Bay and part of the southern Maine coast are good examples of subsidence. The Adirondacks and central New Brunswick, also active, are areas of crustal uplift with associated vertical crustal movements.
7. The epicentral patterns parallel the general trend of geologic structures, thus implying that older faults are currently reactivated.

These summary conclusions have no direct implication on the seismic safety of the Seabrook site, since they do not question the adequacy of the 1755 Cape Ann earthquake as the Safe Shutdown Earthquake. Although Barosh specifically rejects the correlation of seismicity with mafic plutons, and specifically considers Cape Ann as an example of an area where earthquakes occur in embayments because of Cretaceous continental margin sag, his hypothetical tectonic model does not imply at this time any need to modify the present SSE.

c. Seismotectonics and COCORP Studies

Recent availability of data from expanded local seismograph networks, both in the eastern U.S. and Canada, has made possible the spacial and temporal study of earthquake distribution, earthquake mechanisms, inferred stress regime, correlation of seismicity with known geologic structures and preliminary definitions of seismic zones. Sykes (1978), Yang and Aggarwal (1981), Barosh (1982) and Pulli (1983) have reviewed older and recent information, and on that basis formulated their own conclusions on the causes of current earthquakes. Although there is general agreement among the conclusions of these various authors, they nevertheless emphasize different aspects of these interpretations.

The concept of reactivation of faults in the generation of earthquakes is common to Sykes (1978), Yang and Aggarwal (1981) and Barosh (1983). Opinions of these authors differ however, regarding which structures are reactivated. For Sykes, the pre-existing zones of weakness are identified in the "Appalachians as pre-existing faults that trend nearly parallel to present day continental margins as well as along features transverse to the margins." In the case of large historic shocks in Massachusetts coincident with a northwest trending lineament, Sykes still wonders if they could be associated with northeast striking Triassic structures. Regarding association of earthquakes with structures, Yang and Aggarwal are more restrictive and do not support a seismic trend transverse to the Appalachians in New England. They also propose that earthquakes on the eastern margin of the Appalachians occur along existing faults in response to stresses generated by thermally induced horizontal gravity variations in the offshore oceanic lithosphere. They see two distinct seismogenic provinces in the northeast: the Adirondack-Western Quebec Province and the Appalachian Province. Uniformity of horizontal stress orientation within these provinces is among their major conclusions. This position is in agreement with Zoback and Zoback (1980, 1981).

The consistency of stress orientations deduced from fault plane solutions is not fully accepted by Pulli and Toksoz (1981) or Pulli (1983) for the Appalachian Province. They recognize trends among fault plane solutions, but point out that current data were obtained from relatively small to moderate earthquakes with shallow foci. For such shallow events compressive stress distribution is more likely to be influenced by a local feature. Graham and Chiburis (1980) had reached a similar conclusion.

The general agreement among all researchers that seismicity patterns, based on recent instrumental data, is remarkably similar to that in the historical record supports the conclusion of this subsection. This observed pattern is a fundamental element in the selection process of the SSE. Current divergence of opinion on the temporal pattern of earthquakes rates is indicative of the relatively short observation time and the need for better data.

Data generated by the COCORP program (Brown et al., 1983) have made significant contributions to seismic hazard research, defining seismic hazard and associated uncertainty. Again, however, these studies have not produced results which would influence the selection of the SSE for Seabrook. Two recent seismic events at Gaza, New Hampshire, January 18, 1989 (body wave magnitude (m_b) = 4.7) and in New Brunswick, Canada, January 9, 1982 (m_b up to 5.7) have occurred in the site region. A detailed report on these events was docketed in August, 1983 (Weston Geophysical, 1983).

In summary, neither the seismological and geological data or recent research have produced results which would change the selcection of the SSE for Seabrook.

2.5.2.2 Geologic Structures and Tectonic Activity

a. Introduction

The specific regional geologic structures that are significant in determining regional earthquake potential relative to the site are localized in the area of Cape Ann, Massachusetts, and near the Ossipee Mountains, New Hampshire. These structures are characterized by the combination of post-metamorphic fault complexes of crustal dimension in direct spatial association with vertically oriented, cylindrical mafic intrusive plutons of known or geophysically inferred Late Mesozoic age (Boston Edison Co., Pilgrim II PSAR, 1976). The Cape Ann structure lies offshore in the Northeastern Massachusetts Thrust Fault Complex, 10 to 30 miles southeast of the site. The Ossipee Mountains structure, a faulted volcanic caldera in central New Hampshire, is approximately 70 miles north-northwest of the site.

The orientation of the contemporary regional stress field in New England is not well known. The tectonic activity of the region is defined by the sporadic occurrence of small to moderate earthquakes, reflecting localized accumulation and sudden relief of stress which has become concentrated at depth on geologic structures which have different physical properties than the crustal domain which enclose them. Earthquakes in the region are considered to result from fault movements at depths of from about 2 to 15 kilometers in the crust; no dislocations of the ground surface resulting from tectonic faulting have been observed historically.

The tectonic provinces which occur within 200 miles of the site (Figure 2.5-5 and Figure 2.5-36) include the Coastal Anticlinorium (Site Province); Northeastern Massachusetts Thrust Fault Complex; Merrimack Synclinorium; Western New England Foldbelt; Southeastern New England Platform; Long Island Platform; New York Recess; Valley and Ridge; Appalachian Plateau; Adirondack Uplift; and Western Quebec Seismic Zone.

b. Coastal Anticlinorium (Site Province)

The Coastal Anticlinorium is a basement terrane of Cambro-Ordovician metasedimentary and metavolcanic rocks of eugeosynclinal and islandarc derivation intruded by numerous granitic plutons of Middle Paleozoic orogenic origin. The basement rocks locally enclose fragments of Late Precambrian (Avalonian) lithologies and are overlain toward the northeast by supracrustal patches of SiluroDevonian volcaniclastic formations and Late Paleozoic terrestrial clastic rocks including, in New Brunswick, coal seams. Toward the southwest end of the province in southwestern Maine and southeastern New Hampshire, the basement rocks are transected by several alkaline central-complex plutons and by numerous tabular mafic dikes. Structurally, the province is characterized throughout by northeast-trending orogenic folding and faulting; by long, northeast-striking, post-orogenic, strike-slip faults; by northweststriking, high-angle faults of Late Paleozoic to Mesozoic ages in the northeastern-most part of the site region; and by discordant Late Paleozoic to Mesozoic alkaline plutons and mafic dike swarms in the southwestern part.

The location of the eastern boundary of the province, beneath the waters of the Gulf of Maine, is not known. Variations in aeromagnetic and gravity patterns (Kane et al., 1972a; 1972b) in the Gulf of Maine suggest that the southeastern province boundary may lie about 80 miles to the east of the site.

The location of the northwestern boundary, along the trend of the Nonesuch River-Flint Hill-Wekepeke fault zone, is newly postulated on the basis of current research reported by Aleinikoff and Zartman (1978) and Lyons, Boudette and Aleinikoff (1979).

The tectonic history of the province has involved its consolidation as a crustal mass by northwesterly directed orogenic compression in Early Paleozoic time; its subsequent relatively moderate fold and fault deformation and intrusion by granitic masses during an episode of northwesterly directed orogenic compression in Middle Paleozoic (Acadian) time; its translational faulting and south-westerly-directed, right-lateral displacement during a glancing continental collision in Late Paleozoic (Variscan-Alleghanian) time; its block-faulting in the northeast, and central complex and mafic dike intrusive activity in the southwest, under conditions of extensional tectonics and ocean opening in Latest Paleozoic and Mesozoic times. For the past 100 million years, the province has been subjected only to diffuse tectonic arching and, within the last 2 million years, to crustal depressions and rebounding as continental ice sheets advanced and retreated across the region.

There are no definitive data by which the present crustal stress regime is deduced. No capable faults have been identified in the province.

The historical seismicity of the province is characterized by broad areas of little to no historical earthquake activity interrupted locally by clusters of small to moderate events located in eastern-most Maine, south-central Maine, south-coastal Maine, and near Portsmouth in southeastern New Hampshire (see Figure 2.5-36 and Figure 2.5-37). In eastern-most Maine and southwestern New Brunswick, in the area of Passamaquoddy Bay, a cluster of historically reported earthquakes, including one Intensity VI(MM) event, lies in a distinct north-northwesterly orientation in close spatial association with the Late Paleozoic-Mesozoic Oak Bay fault. In south-central Maine, in the Bangor-Orrington area, several Intensity V(MM) and some smaller events are clustered in the vicinity of the post-orogenic, right-lateral Norumbega fault system. In south coastal Maine, more than one dozen earthquakes of Intensity V(MM) and smaller have been reported along an 85-mile northeast-trending zone which coincides with the Norumbega fault system and with the province boundary from near Saco on the southwest, to near China Lake on the northeast. In southeastern New Hampshire, a distinctive cluster of small historical events, with an Intensity V(MM) upperbound, lies in the Portsmouth-Great Bay area where no post-orogenic faulting has been detected.

The largest event in the province is an Intensity VI(MM), Magnitude 4.8 event of April 26, 1957, offshore to the east of Portland, Maine. Partial aeromagnetic coverage (Boston Edison Company, Pilgrim II PSAR, 1976) in the southern part of this area suggests, but cannot demonstrate, that a mafic pluton of White Mountain series affinities may occur in very close spatial association with the epicentral location of this event.

c. <u>Northeastern Massachusetts Thrust Fault Complex</u>

The Northeastern Massachusetts Thrust Fault Complex is made up of crystalline igneous and metamorphic rocks of Late Precambrian (Avalonian) and Early Paleozoic age which have been extensively fractured by thrust faulting and probably earlier, deep-seated, post-orogenic transcurrent (strike-slip) faulting. In northeastern Massachusetts, fault blocks of deformed but unmetamorphosed SiluroDevonian volcaniclastic rocks having European fossil assemblages are contained within the older crystalline terrane; a cylindrical mafic pluton (the Cape Ann pluton) and a possible mafic ring-dike body have been interpreted from magnetic surveys to lie within the fault complex, adjacent to its major crustal fault, about one mile offshore to the north of Cape Ann, Massachusetts.

The tectonic history of the province has involved post-Middle Paleozoic strike-slip faulting of probably great lateral displacement plus Late Paleozoic northwest-over southeast thrust faulting having an aspect of right-lateral displacement. This zone of complex brittle-fracture deformation was then intruded, offshore to the north of Cape Ann, by at least one cylindrical mafic pluton whose geophysical signature is similar to plutons exposed on the land area in New Hampshire and Maine whose ages have been dated as Cretaceous (110 to 120 million years old). As for all of the inland provinces in the site region, the Thrust Complex is not known to have been subjected to other than passive tectonic forces subsequent to Mesozoic time. No capable faults have been identified in the province.

Historical seismicity within the fault complex is the greatest in New England, with one Intensity VIII(MM) event in 1755, one Intensity VII(MM) event in 1727, three Intensity VI(MM) events, and numerous Intensity V(MM) and smaller events (see Figure 2.5-36 and Figure 2.5-37). The two largest earthquakes are considered to have occurred in the area offshore of Cape Ann. The larger offshore events have been correlated with a localized tectonic structure consisting of the geophysically inferred cylindrical mafic pluton tangent to the Bloody Bluff fault, the major fault system of the Complex (Boston Edison Company, Pilgrim II PSAR, 1976). All of the lower level seismicity is spatially correlated with the zone of intense post-metamorphic faulting. As the zone of faulting narrows and becomes less complex toward the southwest end of the province, historical seismicity also markedly diminishes both in frequency and intensity.

d. <u>Merrimack Synclinorium</u>

The Merrimack Synclinorium is a distinctive basement terrane of Siluro-Devonian eugeosynclinal sedimentary rocks, enclosing numerous granitic masses of Middle Paleozoic orogenic origin, and, in southern New Hampshire a plutonic gneiss block (the Massabesic Gneiss) of Late Precambrian age. The basement rocks are transected in eastern New Hampshire and southwestern Maine by more than one dozen alkaline central complex plutons of the White Mountain Plutonic Series, and by numerous, largely northeast-striking mafic dikes. Structurally, the province is characterized by Middle Paleozoic (Acadian) orogenic folding (frequently transverse to the axis of the province), faulting, and high-grade metamorphic facies; by long, northeast-striking, post-metamorphic, strikeslip faults; by the northwesterly transecting zone of isolated, discordant plutons of the White Mountain Plutonic Series; and by mafic dikes of Early to Middle Mesozoic age.

The tectonic history of the province was initiated with its compressional consolidation as part of the North American crust during the Acadian orogeny in Middle Paleozoic time, followed by its uplift and rapid erosion in Late Paleozoic time. The southeastern boundary of the province was probably widely subjected to glancing, compressional strike-slip faulting in Late Paleozoic time during the Variscan continental collision. With the initiation of extensional crustal stress in Early Mesozoic time, numerous mafic dikes were emplaced in the southwestern part of the province, and several volcanic-plutonic complexes were emplaced, including the White Mountain batholith, the largest of all the White Mountain Series plutons. Further volcanic-plutonic activity occurred in Jurassic time. Dynamic tectonism ceased in Middle Cretaceous time, 120-110 million years ago, with the emplacement of several mafic plutons, including the Ossipee Complex, and a scattering of mafic dikes. No capable faults have been identified in the province.

Although a substantial portion of the seismicity in the province is spatially associated with one of three anomalous structural features described below (also see Figure 2.5-36 and Figure 2.5-37), a definitive identification of an earthquake-tectonic relationship can be made in only one case, the Ossipee pluton, where the border fault of an apparent collapsed Caldera is tangent to the north rim of the pluton.

1. In southwestern Maine, the most prominent and repeated seismicity, including two Intensity VI(MM) events of December 23, 1857, and July 15, 1905, are spatially close to the northeast-trending Lewiston-Pittsfield fault zone. Similarly, diffuse seismicity may be correlative with a series of northeast-trending faults in a broad zone to the northwest of the Lewiston-Pittsfield zone, and with the Norumbega fault system along the southeast boundary of the province with the Coastal Anticlinorium. All of these fault systems are post-metamorphic, and are interpreted to be of Carboniferous (Variscan) age.

2. In central New Hampshire, a cluster of earthquakes, including two Intensity VII(MM) events, is spatially correlated with a 850-square mile physiographic-geologic-aeromagnetic anomaly which encloses at least five Mesozoic central complex intrusives, and which may reflect a collapsed volcanic caldera. The published epicenters of the two Intensity VII(MM) events occurred in December 1940, and are spatially correlated with a tectonic structure in which an east northeast-trending border fault of the apparent collapse structure passes tangent to the north rim of a large cylindrical mafic pluton of Middle Cretaceous age, the Ossipee Complex (Boston Edison Company, Pilgrim II PSAR, 1976).

3. In south-central New Hampshire, a diffuse grouping of earthquakes of Intensity V(MM) and smaller occurs in a region in which a few presumed Triassic silicified zones have been mapped but where extensive post-metamorphic mechanical deformation has not yet been identified. The grouping of epicenters here is very closely contained within a discrete area of sillimanite + orthoclase low P/T granulite facies metamorphism (Morgan, 1972), where gravity patterns (Kane et al., 1972b; Nielson et al., 1976) and aeromagnetic patterns (Boston Edison Company, Pilgrim II PSAR, 1976) display both prominent northwest and northeast-trending anomalies. The post-Acadian Concord granite pluton (possible age 330 million years: Lyons and Livingston, 1977) is centrally located in the epicentral grouping; the Late Paleozoic Milford granite pluton (275 million years: Aleinikoff and Zartman, 1978) is located at the south end of the epicentral grouping. This combination of geophysical and geological features is not known to occur elsewhere in the region.

e. Western New England Foldbelt

The Western New England Foldbelt consists of Lower Paleozoic rocks with included Precambrian thrust slices which were consolidated to a crustal block about 450 million years ago during the Taconic orogeny; locally metamorphosed and thrust faulted during the Acadian orogeny; and broken by simple, widely spaced normal faults and locally intruded by a few ring complex plutons along the eastern margin during Mesozoic time. The province is characterized by four parallel north-trending subprovinces: the Middlebury Synclinorium; the Green Mountain Anticlinorium; the Connecticut Valley Synclinorium; and the Bronson Hill Anticlinorium. Early and Middle Paleozoic compressional stress regimes were directed westerly. Based on post-orogenic mafic dike orientations, Early Mesozoic stress was extensional, directed to the southeast; Middle Mesozoic stress was extensional, directed to the south. No capable faults have been identified in the province.

The historical seismicity of the province is of very low frequency and is, with two exceptions, limited to Intensity V(MM) or smaller earthquakes (see Figure 2.5-36). The network of faults in the aseismic western part of the province is predominantly comprised of gravity-slide and thrust faults of Taconic age, welded by Taconic and Acadian metamorphic processes. A substantial portion of the historical earthquakes within the province has occurred along the Bronson Hill Anticlinorium in the eastern part, in spatial association with simple widely spaced, brittle-fracture, normal fault structures of Mesozoic age.

The first exception to the low-intensity characteristic of the province is the Intensity VI(MM), Magnitude m_b = 4.9 event of June 15, 1973, near Worburn, Quebec, about 210 miles north-northeast of the site (Wetmiller, 1975). This event is spatially correlated with an anomalous and localized tectonic structure, consisting of a large cylindrical mafic plug of Middle Cretaceous age, the Megantic complex (Boston Edison Company, Pilgrim II PSAR, 1976), emplaced within a swarm of closely spaced, northwest-dipping normal faults of apparent post-Devonian age (St. Julien and Hubert, 1975, Page 343, Section E-E'). The faults of St. Julien and Hubert (1975) are not defined in plan view in their paper and are not, accordingly, shown on Figure 2.5-2 and Figure 2.5-4.

The second exception is a peculiar intensity VI event which occurred on January 30, 1952, in Burlington, Vermont. Cracks at the surface of frozen ground near the Winooski River as well as cracks in pavement and basement walls were reported. The event was given its Intensity VI most likely because of these reports. This event remains anomalous in nature because of its extremely small felt area, 50 square miles, certainly not characteristic of a true Intensity VI event. Other factors such as an extremely shallow focal depth of a smaller magnitude event, and possibly frost action on a saturated overburden could be envisaged as principal causes of the observed cracks and explosive noises. The true tectonic origin of this event is at least questionable, especially since similar events in New England have been documented as due to ground cracks induced by freezing conditions.

f. Southeastern New England Platform

The province consists of a little-deformed Late Precambrian (Avalonian) granitic basement complex, containing Early and Late Paleozoic intrusive masses, Early to Late Paleozoic supracrustal basins and, to the southeast, north-trending mafic dikes of presumed Triassic age. The supracrustal basins have been open-folded and broken by shallow thrust and normal faults. There are no known regional fault zones within the province. No capable faults have been identified in the province.

Historical seismicity of the province is diffuse, and generally of low frequency and low intensity (see Figure 2.5-36 and Figure 2.5-37). A clustering of epicenters is located in southeastern Rhode Island toward the southern end of the Carboniferous Narragansett Basin where the basin rocks have been deformed locally by Late Paleozoic granitic intrusion and polyphase compressional deformation with up to staurolite-grade metamorphism. A number of small events also has occurred in the relatively more faulted portions of the Boston Basin. The faulting in these areas is not considered to have deep crustal dimensions. The largest earthquake in the province was an Intensity VI-VII(MM) event on May 16, 1791, near East Haddam, Connecticut, where a pronounced northwesterly trending physiographic-geologic-magnetic-gravity fabric is transected by a northeast-striking Mesozoic mafic dike swarm and the boundary of the province.

g. Long Island Platform

The province is interpreted to consist of down-faulted basins in Early Paleozoic or Precambrian basement, filled with Mesozoic sediments and overlain by loosely consolidated Coastal Plain sediments of Cretaceous and Tertiary ages.

No seismic activity has been detected in the province in the site region (see Figure 2.5-36). No capable faults have been identified in the province.

h. New-York Recess

The province consists of Cambro-Ordovician geosynclinal deposits and included Precambrian thrust slices which were consolidated to a crustal block during the Taconic orogeny; locally deformed and metamorphosed by the Acadian orogeny; compressionally faulted, intruded, and thermally altered by forces of the Alleghanian orogeny; broken by normal faulting and intruded by mafic dikes during Triassic continental rifting; and finally, subjected to three episodes of large-scale, left-lateral folding and strike-slip faulting in Late Jurassic time. Geophysically, the southeastern two-thirds of the province in the site region is distinctively characterized by a broad, high-amplitude Bouguer gravity high which locally exhibits zones of steep gradients. No capable faults have been identified in the province.

Historical seismicity in the province in the site region includes three events listed as Intensity VII(MM) in New Jersey and southernmost New York; several Intensity VI(MM) events in New Jersey, southernmost New York, and southwestern Connecticut; and numerous Intensity V(MM) and smaller events (see Figure 2.5-36). In areas for which detailed geologic mapping has been published, as along and to the northwest of the Ramapo fault system, it appears that the higher frequency of seismic activity can be correlated with zones of relatively greater frequency of brittle-fracture faulting.

In his investigations of fault plane solutions of local events, Aggarwal (1977) has noticed that for each earthquake one of the nodal planes trends N-NE or NE. Noting the predominance of NEtrending faults in the Ramapo fault system, he has suggested that this consistency would indicate that earthquakes occur along preexisting faults.

i. Valley and Ridge

The province consists of Cambrian to Pennsylvanian sedimentary rocks which were deformed by thin-skinned folding and thrust faulting during the Alleghanian orogeny, and apparently further slightly deformed by Mesozoic compressional and extensional tectonic forces. Grenvillian basement lies at great depth and is not believed to have been remobilized during deformation of the sedimentary rocks, although it may have been broadly warped (Rodgers, 1970). No capable faults have been identified in the province.

Historical seismicity in the province is of generally low frequency and intensity in the site region (see Figure 2.5-36). There is a scattering of small events beyond the site region to the southwest along the Blue Mountain Structural Front, and to the west in the closely folded and structurally anomalous north-northeast trending Lackawanna syncline in northeastern Pennsylvania.

j. Appalachian Plateau

The province consists of Cambrian to Pennsylvanian platform sedimentary rocks which were mildly deformed into east northeast-trending open folds during the Alleghanian orogeny, and then locally broken by small, discontinuous normal faults of probable Mesozoic age and intruded by mafic dikes of Upper Jurassic age. A south-sloping Grenvillian basement surface underlies the sedimentary rocks at depth, broken locally in the northeastern part of the province by pre-Devonian normal faults. No capable faults have been identified in the province.

Historical seismicity in the province is of very low frequency and intensity (see Figure 2.5-36). No earthquakes have been reported in the province in the site region.

k. Adirondack Uplift

The province consists of Grenvillian-age crystalline rocks overlapped in southern and eastern areas by Cambrian and Ordovician sedimentary rocks. The crystalline rocks are broken by numerous north northeast-trending normal faults which, in some instances, extend into the sedimentary rocks to the south and east. The age of block faulting is predominantly pre-Middle Devonian, but faulting may have been reactivated in Mesozoic and possibly Late Tertiary times along the Champlain-Lake George rift system in the eastern part of the province. No capable faults have been identified in the province.

Historical seismicity has generally been associated with the peripheral regions of the province, in areas where overlapping sedimentary rocks are broken by normal faults trending out from the exposed basement massif (see Figure 2.5-36). A cluster of small, shallow earthquakes has recently been reported in the central part of the Uplift in the area of Blue Mountain Lake.

The largest historical earthquake in the province is an Intensity VII(MM) event which occurred on April 20, 1931, near Lake George, New York, about 148 miles northwest of the site. In detail, the epicentral area is characterized geologically by a network of closely spaced northeast-and northwest-trending normal faults whose displacement is commonly down toward the long, central Lake George rift structure.

l. Western Quebec Seismic Zone

The Western Quebec Seismic Zone is a typical example of a seismotectonic province primarily delineated on the basis of seismological data (see Figure 2.5-36). Defined by Canadian seismologists (Basham, Weichert, Berry, 1979), the elliptically shaped region extends from Lake Champlain in the southeast to near Timiskaming, Quebec, in the northwest. The northeastern and southwestern boundaries are drawn to enclose the historical distribution of seismic activity. The southwestern boundary follows the Ottawa River for a large section, while the northeastern boundary is more arbitrarily selected. Considered by some as part of the "Ottawa-Boston Trend" (Sbar and Sykes, 1973), it was thought to be continuous through New England. However, the Western Quebec Seismic Zone terminates in New York State against the seismically quiescent Western New England Foldbelt Province distinguished by a distinctive gravity high in central and northern Vermont. The Western Quebec Seismic Zone is considered as being relatively the most active region in eastern Canada (Basham, 1977).

In the site region, the province consists geologically of a blockfaulted basin of Grenvillian-age crystalline rocks and overlying Cambrian and Ordovician platform sedimentary rocks. The major normal faults trend east-southeasterly through much of the basin and turn to parallel the north-northeast-trending margin of the province on the east. Middle Cretaceous mafic plugs occur in the eastern part of the province, distributed along east-southeast trends parallel with the major block faulting. No capable faults have been identified in the province.

Characterized by the frequent occurrences of small events in the Magnitude 2 to 4 ranges, the region is also capable of larger but infrequent events in the Magnitude 5.5 to 6.0 range. This capability is attested by the Timiskaming, November 1, 1935, event (Intensity VII(MM)), the Cornwall-Massena, September 5, 1944, event (Intensity VII(MM)), and the September 16, 1732, event, near Montreal, originally assigned an Intensity IX(mm) by Smith (1962) but recently revised to an Intensity VIII(MM) on the basis of additional historical data (Leblanc, 1980). Even though a large portion of the zone shows some correlation of earthquakes with geological and topographical features (Forsyth, 1977), it is not yet clear to what extent the larger events and some clustering of smaller events can be related to structures without carrying out further geophysical investigations.

2.5.2.3 Correlation of Earthquake Activity with Geologic Structures or Tectonic Provinces

a. Site-Significant Eartquake Activity

Epicenters of highest intensity of historically reported, sitesignificant earthquakes are correlated with anomalous tectonic structures in the area of Cape Ann, northeastern Massachusetts and in the Ossipee Mountains, central New Hampshire (see Figure 2.5-36). In each case, highest historical seismicity is correlated with cylindrical mafic plutons which are in direct spatial association with closely spaced, post-metamorphic faults having deep crustal dimensions (Boston Edison Company, Pilgrim II PSAR, 1976).

The larger events of 1755 (VIII(MM)) and 1727 (VII(Mm)) plus continuing smaller events off Cape Ann are correlated with a geophysically inferred mafic pluton which is enclosed in a well defined complex of east-northeast-trending, post-metamorphic (brittle-fracture); strike-slip and thrust faults. This fault complex is bounded on the north by the Clinton-Newbury fault, on the south by the Northern Border fault of the Boston Basin, and on the west where faults of the complex die out or coalesce. Aeromagnetic patterns suggest that the complex also dies out or coalesces about 35 miles offshore to the east, northeast of Cape Ann. Seismicity of the Northeastern Massachusetts Thrust Fault Complex is anomalously high for the region, is greatest in the vicinity of the mafic pluton just offshore of northeastern Cape Ann, and dies out to the west in direct relation to the progressive westerly decrease in fault frequency (see Figure 2.5-37).

The larger events of 1940 (Intensity VII(MM)) in the Ossipee Mountains area are correlated with the Ossipee mafic pluton, which is enclosed in a roughly elliptical structure with an 850-square mile area, interpreted from both geologic and aeromagnetic evidence to reflect a block fault complex of a collapsed volcanic caldera.

Seismicity of the block fault complex is greatest in intensity where the Ossipee pluton lies tangent to a bounding fault of the caldera structure, and repeated smaller events have been recorded elsewhere within or on the border of the caldera structure.

b. <u>Regional Correlation Considerations</u>

Historical seismicity in the region has tended to concentrate in areas or regions also characterized by the presence of high-angle fault systems (see Figure 2.5-36). These fault systems have crustal dimensions and relatively "young" post-orogenic mechanical (brittle-fracture) displacements, and may be of either transcurrent (strike-slip) compressional origin or of block-fault extensional origin. The spatial stability through historical times of the seismically more active areas is indicative of localized stress concentrations and subsequent strain releases associated with discrete structural inhomogeneities. There is no apparent association in the region of anomalous earthquake activity with "old" premetamorphic or synmetamorphic fault structures or with syntectonic granitic batholiths.

In several instances, an apparent higher frequency of seismic activity is spatially associated with areas having a greater frequency of mapped brittle-fracture deformation than in their surrounding terranes. Some large earthquakes are spatially associated with distinctive, individual tectonic structures defined by the combination of discordant, post-metamorphic mafic intrusives lying within crustal fault systems or situated immediately adjacent to individual major crustal faults.

Published geologic reports are not of uniform quality and detail from place to place throughout the region. Where detailed information does exist, however, there appears to be a direct relationship between the degree of post-metamorphic mechanical deformation and the level of seismic activity. Conversely, broad areas in the region which are characterized by their infrequent, widely spaced and low-intensity historical earthquake activity are also characterized tectonically by the apparent absence of deep-crustal, post-metamorphic mechanical deformation.

Seismic activity in the region can be related to the interaction of the present crustal stress regime with tectonic structures that may or may not be evident at ground surface. In the latter case, inhomogeneities of lithology and/or geologic structure can only be inferred through geophysical investigations which detect variations in the physical properties in the subsurface crustal and mantle rocks. Based on geological and geophysical data presently available, a number of correlations of earthquakes with geologic structures can be made, as enumerated below in Subsection 2.5.2.3c.

c. Correlation with Structures

The largest historical earthquakes in New England occurred in 1727 and 1755 off Cape Ann, Massachusetts (Intensities VII(MM) and VIII(MM), respectively), and in 1940 in the Ossipee Mountains, New Hampshire (two Intensity VII(MM) events). As discussed in Subsection 2.5.2.3a above, these earthquakes are correlated with the anomalous structural combinations of cylindrical mafic plutons enclosed in post-metamorphic fault complexes.

The largest historical earthquakes reported in northeastern North America adjacent to New England occurred in 1638, 1663, 1860, 1870 and 1925 near La Malbaie, Quebec (Intensities IX(mm), X(mm), VIII-IX(MM), and IX(MM)-Magnitude 7.0, respectively). The largest earthquakes of this localized seismic source, which is located about 300 miles north of the site, may have produced intensities of up to VI(MM) at the site. Along with hundreds of smaller felt events and instrumental recordings, these large earthquakes occurred on and are correlated with a discrete tectonic structure defined by the intersection of a major northeast-striking crustal fault system with the Charlevoix structure, a 25-mile diameter meteor impact feature dated at about 380 million years age.

The largest historical earthquakes just beyond the 200-mile site region in the Western Quebec Seismic Zone occurred in 1732 near Montreal, Quebec, and in 1944 in the area of Cornwall-Massena near the Ontario-New York border (Intensities VIII(MM), Magnitude 5.6-6.0). The large event near Montreal is located in an eastwest elongate earthquake cluster with two Intensity VII(MM) and smaller events, and is correlated with a spatially coincident structural combination of cylindrical mafic plutons enclosed in a block fault complex. The Cornwall-Massena earthquake is spatially related to the Gloucester fault and a probable mafic intrusive body as evidenced by gravity and magnetic data.

Within the site region there are several areas of "clustered" epicenters whose historical intensities were less than Intensity VIII(MM). In eastern-most Maine, a northwesterly oriented group of epicenters of Intensity VI(MM) and smaller events is spatially correlated with the Late Paleozoic-Mesozoic Oak Bay fault and a pronounced high-gravity ridge. In south-central Maine, an elongate concentration of more than one dozen earthquakes, including two Intensity VI(MM) events, is spatially correlated with the post-metamorphic Lewiston-Pittsfield fault zone, a trend defined by marked offsets of metamorphic isograds, by a pronounced linear gravity anomaly and by deformed radiometric "chrontour" lines. In east-central Connecticut, a concentration of small earthquakes with one Intensity VI-VII(MM) event, coincide spatially with a zone in which closely spaced, northeast-striking normal faults and a mafic dike swarm are transected at a high-angle by a pronounced northwest-oriented physiographic-geologic-magnetic-gravity structural fabric.

Two isolated earthquakes of moderate intensity have occurred in the site region, in 1973 near Woburn, Quebec, and in 1957 offshore to the east of Portland, Maine (Intensity VI-Magnitude 4.9, and Intensity VI-Magnitude 4.8, respectively). The Woburn, Quebec event is spatially correlated with a localized geologic structure consisting of a large cylindrical mafic pluton of Middle Cretaceous age, the Megantic Complex, enclosed within a swarm of closely spaced, northwest-dipping normal faults of post-Devonian age. The offshore-Portland event occurred in an area where partial aeromagnetic coverage suggests, but cannot demonstrate, the possible presence of a cylindrical mafic pluton.

Two recent seismic events at Gaza, New Hampshire, January 18, 1989 (body wave magnitude (m_b = 4.7) and in New Brunswick, Canada, January 9, 1982 (body wave magnitude (m_b) up to 5.7) have occurred in the site region. A detailed report on these events was docketed in August, 1983 (Weston Geophysical, 1983).

The Gaza event falls along a pre-existing north-northeast alignment of instrumental and historic seismicity (Figure 2.5-35). This alignment includes the 1940 Ossipee events (m_b) = 5.5) which are spacially associated with the Ossipee intrusive complex (BECO Pilgrim Unit II Docket, (1976). The Gaza event is about 35 km southwest of the Ossipee complex. The immediate epicentral area is underlain by the Devonian Littleton Formation which contains no regional structural elements. Local detailed mapping, remote sensing and geophysical analysis identified a trend of Jurassic faulting coincident with remote sensing lineaments and the epicentral pattern of the event. The trend is intersected by northwest or east-northeast trending magnetic and remote sensing lineaments. This conjunction of structural elements, geophysical anomalies and Mesozoic igneous activity apparently combine to produce the present-day seismic release. Within this tectonic framework, this event does not effect the earthquake potential for the Seabrook site.

Seismic events occurred in two series in New Brunswick, Canada during January and June of 1982 (Figure 2.5-35). Major faults with histories of multiple movements are known to exist bounding and cutting the North Pole pluton, a major Devonian intrusive in the Miramichi massif. Significant gravity and geophysical anomalies are found in spacial coincidence with these geological structures. Complex geological and structural associations make possible a hypothesis to account for significant stress accumulation along lithologic and structual discontinuities. Analysis of aftershocks of the main shock (m_b) = 5.7) clearly show activity distributed along two intersecting planes (of rupture?) extending to a depth of 7 km. A review of past seismicity in the 1982 epicentral area suggests the structure responsible for the 1982 sequences has been seismically active before (Weston Geophysical, 1983). Detailed mapping of trenches in the epicentral area revealed a fault zone with evidence of multiple and possibly post-glacial movement.

The 1982 New Brunswick activity is thus considered to be related to a local tectonic source. This activity, at a distance of 565 km from the Seabrook site, has no significance with respect to the site SSE or OBE. The Seabrook design could, in fact, accommodate an event of this size even within the site tectonic province.

2.5.2.4 Maximum Earthquake Potential

The selection of the maximum earthquake potential for the Seabrook site is straightforward due to the relative proximity of the Cape Ann historical events of 1727 and 1755. Even though these events have occurred in a different tectonic province and are considered structure-related, because of the proximity to the site of the Northeastern Massachusetts Thrust Fault Complex boundary, it is assumed for licensing purposes that the Intensity VIII(MM) 1755 earthquake could occur "at the site."

It should be observed that such an Intensity VIII(MM) at the site is more than adequately conservative, considering the results of Table 2.5-11, which tabulates the calculated and observed site intensities for all historical events of the site region, as well as for any other large events outside the site region. In this table, the date, location, intensity, magnitude, and epicentral distance to the site are listed for each event which may have resulted at the site in an Intensity III(MM) or greater. The Gupta and Nuttli (1976) attenuation relationship has been used to calculate theoretical site intensities, as this relationship of intensity fall-off with distance is currently considered the most applicable to eastern United States. Whenever possible, historically observed site intensities have been read from published isoseismal maps. A comparison of the calculated values with the observed values shows good agreement. In general, the calculated values tend to be slightly higher than reported ones, suggesting that the Gupta and Nuttli's relationship is somewhat conservative. Figure 2.5-41 shows the attenuation curve used in this study, as well as four others also derived for eastern regions. Isoseismal maps and felt-report plots from which observed site intensities were abstracted are presented in Appendix 2E, Part 1.

It should be noted from Table 2.5-11 that the largest calculated site intensities are associated with the 1755 and 1727 Cape Ann events ("7.1" and "7.0" intensity units, respectively) and are one level lower than the intensity associated with the selected maximum earthquake potential. Since attenuation relationships principally reflect intensity values reported for average soil foundation conditions, the intensity value on bedrock would be at least one intensity less than soil so that the maximum intensity on bedrock would be VI(MM).

The next largest site intensities of "5.8," calculated from the Gupta and Nuttli (1976) relationship, are associated with two La Malbaie, Quebec, earthquakes during the early years of 1534 and 1663. The epicentral distance is approximately 325 miles; the assigned epicentral intensity is relatively high, IX-X and X(MM), and possibly conservative. The 1925, March 1, event from the same area, with an estimated Magnitude 6.6 to 7.0 and a reported epicentral Intensity IX, is probably a better characterization of the La Malbaie earthquake potential. The calculated site Intensity "4.8" appears to be high in comparison with the observed Intensity IV in the immediate site area (see Figure 2E-2O of Appendix 2E). The Ossipee, New Hampshire, events of December 20 and 24, 1940, with a Magnitude 5.4 to 5.8, have a calculated site intensity of "5.1," while the observed reports near the site are well characterized by an Intensity IV(MM) (see Figure 2E-27 of Appendix 2E).

Other events with shorter epicentral distances have a calculated site Intensity V(MM), but well below the selected maximum earthquake potential of an Intensity VIII(MM) assumed near the site.

A more detailed review of some events which could have affected the site area with an Intensity IV or greater and a presentation of substantiating references are given in Appendix 2E, Part II.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

The plant foundations of the Seabrook site rest on sound crystalline bedrock consisting of quartz diorite and included quartzite of the Newburyport pluton of Middle Paleozoic or older age. Compressional wave velocities of the foundation materials range from 16,500 to 18,500 feet per second, and shear wave velocities from 8,000 to 10,000 feet per second, indicating a competent bedrock. Table 2.5-12 and Table 2.5-13 provide a summary of seismic compressional and shear wave velocities, bulk densities, soil properties, shear modulus and other parametric values. Variation of shear modulus with strain level and mean effective stress for structural fill is shown on Figure 2.5-67. Figure 2.5-18 shows groundwater elevation contours at the site.

All structures identified in Table 3.2-1 except electrical manholes, duct banks, and service water piping are placed on competent bedrock or fill concrete extended to competent bedrock.

The procedures used to determine the SSE ground motion in the structural fill is addressed in Subsections 3.7(B).1.4, 3.7(B).2.4, 3.7(B).3.12 and 2.5.4.7.

There are no unusual conditions at the site which would effect seismic wave transmission.

2.5.2.6 Safe Shutdown Earthquake

The horizontal peak acceleration associated with the maximum earthquake potential Intensity VIII(MM) according to the intensity-acceleration relationship established by Trifunac and Brady (1975), shown on Figure 2.5-42, is 0.25g (mean plus one sigma). Assuming that the vertical peak acceleration is two-thirds of the horizontal acceleration (Newmark and Hall, 1977), 0.167g is selected accordingly.

Figure 2.5-43 and Figure 2.5-44 present the horizontal and vertical design spectra for various damping ratios, taking into account the selected design accelerations and Nuclear Regulatory Commission Regulatory Guide 1.60. Such design spectra are considered conservative and capable of accommodating both the earthquake potential of a nearby moderate earthquake as well as of a larger distant one.

Figures 3.7(B)-1 to 3.7(B)-20 describe the Safe Shutdown Earthquake in both the time and frequency domain.

The duration of the earthquake is estimated at 10 to 15 seconds. The 10 to 15 seconds duration of motion is that which is estimated to exceed $.05_g$ (Bolt, 1973, 1981). The account given by Winthrop (1755) is perceived motion. The ability to perceive ground motion is a function of its frequency and duration (Nicholls et al., 1971; Siskind et al., 1980); in general, the higher frequencies and longer durations are associated with lower thresholds of perceptible motion. At frequencies of 1 to 4 Hertz, it is estimated that the threshold of perceptibility would be between .001g and .008g, 6 to 50 times lower than the .05g chosen as the strong motion duration. Because of the early hour of the morning (approximately 4:15 A.M.) and the lack of man-made background noise in 1755, Professor Winthrop undoubtedly perceived the motion at the low end of the threshold of perceptibility and, as he has described, felt several phases of motion and not just that associated with the strong motion (Shear or Lg). This is consistent with the duration of perceptible motion from other earthquakes as stated by Bolt (1973, p. 1307); "the long period vibrations taken with the aftershocks add to the human propensity to exaggerate the duration of shaking (humans can feel a .001g). Some people in the Alaska earthquake reported feeling motions for 150 seconds (Kachadoorian and Plafker, 1967)."

The seismic motion in the offsite borrow supporting seismic Category I electrical manholes, duct banks, and service water pipes will be damped out within 2 seconds after the end of the SSE bedrock seismic motion. This duration was determined by assuming that motion in the offsite borrow was essentially stopped when the amplitude of motion was less than 10 percent of the amplitude at the end of the earthquake. The decay of amplitude after the end of the earthquake is defined (by Richart, et al., 1970):

$$\ln \frac{Z_1}{Z_2} = n \frac{2\pi D}{1 - D^2}$$

where Z_1 = Amplitude at end of earthquake

Z_2 = Amplitude after n cycles of damped free vibration

n = Number of cycles after end of earthquake

D = Percentage of critical damping

Using D = 5 percent, seven cycles are required to reach the 10 percent amplitude level. The natural period of vibration for the greatest thickness of offsite borrow, based on the SHAKE analysis described in Subsection 2.5.4.10b, is less than T = 0.25 sec. This results in a duration of seismic motion in the offsite borrow of less than 2 seconds after the end of bedrock motion. For thinner layers of offsite borrow, the duration of motion will be less than calculated above.

2.5.2.7 Operating Basis Earthquake

An Operating Basis Earthquake equivalent to one-half the Safe Shutdown Earthquake has been used. The corresponding horizontal acceleration is 0.125g, which is equal to the acceleration associated with an Intensity VII(MM) (Trifunac and Brady, 1975). Such an intensity is of the same level as the highest intensity experienced historically in the site vicinity on the poorer soils foundation materials.

The probability of exceeding the OBE during the operating life of the plant is estimated from the fractile seismic hazard curves (Figure 20) determined in the report on "Seismic Hazard at Seabrook Nuclear Station," (Dames and Moore, 1983, Appendix F to Seabrook PRA Study). This hazard study describes all input seismicity and ground motion parameters. Various sensitivities are examined in this referenced study. The New Brunswick and New Hampshire earthquake of January 1982 were included in the seismic hazard analysis, therefore, hazard curves accommodate the occurrence of these recent earthquakes. The median annual frequency of exceeding the OBE peak ground acceleration (0.125g) is 5.25×10^{-4}; this annual frequency is equivalent to a probability of 0.0259 to exceed the OBE acceleration during an assumed 50-year operating life span of the Seabrook plant. Uncertainty on this medium estimate is illustrated by levels of the 84th and 16th percentile seismic hazard curves. The 84th percentile annual frequency of exceeding the OBE is 1.45×10^{-3} and the 16th percentile estimate is 2.00×10^{-4}. These plus and minus one standard error bounds on the median seismic hazard estimate correspond to probabilities of exceeding the OBE during the plant operating life of 0.07 and 0.01, respectively.

2.5.3 Surface Faulting

There is no evidence of surface faulting at the site. During the construction phase of the project, all plant excavations were subjected to detailed geologic mapping on a continuous basis. The detailed mapping revealed that the bedrock at the site was transected by numerous, short faults, as discussed in detail in Subsections 2.5.1.2b.3 and 2.5.1.2b.6. The youngest of these site faults are interpreted from their structural relationships with radiometrically dated mafic dikes to be at least 200 million years old. In the course of the detailed geologic mapping at the site, the bedrock surface was examined generally for evidence of post-glacial offsets, and the bedrock/overburden interface immediately above the more prominent bedrock faults was examined in detail for evidence of post-glacial movement. No surface offsets were found on the bedrock, and no evidence of deformation inglaciomarine clay-silt or outwash sands or crudely stratified tills was detected.

2.5.3.1 Geologic Conditions of the Site

The regional and site geologic conditions are discussed in Subsections 2.5.1.1, 2.5.1.2 and 2.5.4.1.

2.5.3.2 Evidence of Fault Offset

Based on fault-specific geologic and geophysical investigations in the site area (Subsection 2.5.1.2a.5) and on detailed geologic mapping at the site during project construction (Subsection 2.5.1.2b.6), there is no evidence of recent fault offset at or near the site. Post-glacial bedrock offsets of small displacement have been reported at numerous loctions in the site region (Boston Edison Company, Pilgrim II PSAR, 1976), ascribed to frost heaving, post-glacial isostatic rebound or ice shove. These offsets are commonly found in schistose rocks, striking parallel to schistosity. No offsets of this nature have been found at the site.

2.5.3.3 Earthquakes Associated with Capable Faults

No capable faults have been identified in the site region.

2.5.3.4 Investigation of Capable Faults

No capable faults have been identified in the site region.

2.5.3.5 Correlation of Epicenters with Capable Faults

No capable faults have been identified in the site region.

2.5.3.6 Description of Capable Faults

No capable faults have been identified in the site region.

2.5.3.7 Zone Requiring Detailed Faulting Investigation

No capable faults have been identified in the site region; no detailed fault investigations as described in Appendix A to 10 CFR Part 100 are required.

2.5.3.8 Results of Faulting Investigation

The site is not located within a zone requiring detailed faulting investigation (Subsection 2.5.3.7).

2.5.4 Stability of Subsurface Materials and Foundations

2.5.4.1 Geologic Features

As discussed in Subsection 2.5.1.2b.6, there are no hazardous geologic features, nor is there any evidence of hazardous geologic features significant to the site. Specifically, there are no cavernous lithologies, incompetent shear zones, or important zones of deep bedrock weathering at the site. The site is founded on hard, competent crystalline igneous and metamorphic rocks of Mesozoic and older geologic ages which have not been subjected to dynamic tectonic stresses for more than 100 million years. The only withdrawal of subsurface fluids occurring at or adjacent to the site is of minor consequence, pumping groundwater for construction purposes and domestic water supply, and does not cause ground settlement at the site. Based on investigation results discussed in Appendix 2H and observations in deep bedrock excavations during construction of the project, the bedrock at the site is not subject to anomalous unrelieved residual stress.

The previous loading history of the bedrock foundation materials at the site involved several episodes of compressional orogenic stress during Paleozoic time, and one or more periods of extensional crustal stress during Mesozoic time, all prior to 100 million years ago. The bedrock at the site does not display evidence of tectonic loading more recently than Mesozoic time. During episodes of Pleistocene continental glaciation, occurring intermittently during the past 2 million years up until about 25,000 to 15,000 years ago, the site was successively loaded by glacial ice which may have amounted to a mile or more in thickness. There is no evidence of bedrock deformation at the site caused either by the ice loading or by the subsequent post-glacial crustal rebound.

Although Paleozoic and Early Mesozoic deformational events have locally created faults, folds, joints, and slickenside surfaces at the site, there is no evidence that these localized features adversely affect the structural integrity of the crystalline bedrock on which the site is founded.

2.5.4.2 Properties of Subsurface Materials

All seismic Category I structures are founded on sound bedrock or on engineered backfill extending to sound bedrock. Engineered backfill was also placed around all seismic Category I structures.

Engineering properties of the bedrock at the site measured both in the field and in the laboratory are presented in Subsection 2.5.4.2a below.

The engineering backfill which is described in Subsection 2.5.4.5 consists of

- Fill concrete
- Backfill concrete
- Offsite borrow
- Tunnel cuttings or
- Sand-cement
- Controlled Low Strength Material (CLSM)

As shown in Table 2.5-20, fill concrete was used as the engineered backfill beneath the foundations of all seismic Category I structures except for (1) safety-related electrical duct banks, (2) five electrical manholes, and (3) the service water pipes. The latter three items were founded on offsite borrow or tunnel cuttings. Properties of the engineered backfill materials are discussed in Subsection 2.5.4.5.

Properties of the overburden soils at the site are not required for evaluation of safety-related structures since all Category I structures are founded on sound bedrock or engineered backfill extending to sound bedrock, and are surrounded by engineered backfill. The available data on the overburden soils is summarized briefly and referenced in Subsection 2.5.4.2b.

a. <u>Bedrock Properties</u>

The bedrock at the plant site consists primarily of quartz diorite (Newburyport formation) with occasional diabase dikes or inclusions of quartzite (Kittery formation) and very minor inclusions of quartzitic schist. The bedrock conditions in the vicinity of the seismic Category I structures were investigated by 42 borings extending to depths up to 169 ft below ground surface, or up to about 90 ft below the bottom of the reactor excavations. Representative samples of the diorite and quartzitic schist were selected for laboratory testing from various depths in three of the borings: E1-1 at the center of the Unit 1 reactor, E2-1 at the center of the former Unit 2 reactor, and B-7, near the Waste Processing Building. Crosshole and uphole geophysical measurements were made in seven borings near Unit 1. These tests are sufficient to define the bedrock properties in the plant area for static evaluation of foundation-bearing capacity, excavation heave, and foundation settlement. Rock properties were not required for seismic analyses since bedrock was treated as a rigid boundary, as described in Subsection 3.7(B).2.3.

Additional data on the properties of rock in the vicinity of the site were obtained during the field and laboratory studies for design of the circulating water tunnels. The results of laboratory tests on specimens from the tunnel borings were considered when evaluating rock properties for design of seismic Category I foundations. The following index and engineering properties were measured for the rock at the plant site and along the circulating water tunnel alignments and are summarized in Table 2.5-12. These properties were obtained from seismic surveys, rock coring (including oriented cores), borehole testing, and laboratory testing:

- Classification and Description of Rock
- Rock Quality Designation (RQD)
- Permeability of Rock Mass
- Dip and Orientation of Joints, Slickensides and Foliation (Core Orientation)
- Compression (P) and Shear (S) Wave Velocities
- Density of Rock
- Unconfined-Compressive Strength
- Static Modulus
- Poisson's Ratio
- Dynamic Shear Modulus
- In Situ Rock Stress
- Rock Hardness

The various field exploration programs are described in Subsection 2.5.4.3.

1. <u>Classification and Description of Rock</u>

   Rock cores from all borings were classified and described in the field office based on visual examination by a qualified geologist. Rock classification and descriptions are presented on the boring logs. See Subsection 2.5.4.3 and Appendix 2D for the locations of boring logs for each boring program.

   Bedrock exposed in excavations at the site and in the tunnels was also classified by visual examination. Photographs of typical excavations are presented in Subsection 2.5.4.5, and results of excavation logs are summarized in Subsection 2.5.1.2.

2. Rock Quality Designation (RQD)

Rock Quality Designation (RQD) is defined as the ratio of the length of sound pieces of core, 4 inches or longer, recovered in the core barrel to the distance that the core barrel was advanced, expressed as a percent. RQD was measured in ten of the borings made at the locations of site foundation excavations and in all of the borings along the course of the intake and discharge tunnels. An NX size (2 in. diameter) double tube core barrel was used for all rock cores for which RQD was evaluated. The RQD data are shown both numerically and graphically on the boring logs which are presented in Appendix 2F (borings E2-11 through E2-18) and in Reference 121 (Intake and Discharge Tunnel Borings).

The ten site foundation borings for which RQD was measured are summarized below:

Boring No.	Structure	Inclination (degrees from vertical)	Elevation Range (MSL) (in rock)	Avg RQD percent	Avg RQD (%) (below El –40*)
E2-11	Reactor 1	40	+11 to –102	86	89
E2-12	Reactor 1	41	+21 to –104	76	83
E2-13	Reactor 1	41	+30 to –98	76	73
E2-14	Reactor 1	41.5	+27 to –93	67	82
E2-15	Reactor 2	41.5	+4 to –110	66	73
E2-16	Reactor 2	41	+7 to –108	61	66
E2-17	Reactor 2	41	-2 to –111	54	61
E2-18	Reactor 2	39	+3 to –112	49	53
AIT-1	Pumphouse Intake Shaft	Vertical	-6 to –304	75	76
ADT-1	Pumphouse Discharge Shaft	Vertical	+3 to –288	81	83

*El-40 is the approximate excavation grade for Reactors 1 and 2 and the intake and discharge structures.

Borings E2-11 through E2-14 and E2-15 through E2-18 are inclined borings which were made around the perimeter of Reactors 1 and 2, respectively. These borings were performed to provide data for design of the side slopes for the reactor excavations. Borings AIT-1 and ADT-1 were made at the sites of the vertical intake and discharge shafts entering the east side of the pumphouse. The boring locations are shown in Figure 2.5-17.

RQDs obtained from the reactor site angle borings correlated well with conditions encountered in site excavations. The borings indicated generally very poor to fair quality rock within 10 to 20 ft of the bedrock surface. Poor quality rock encountered in shallow excavations commonly required removal so that final excavation surfaces consisted of sound rock, as described in Subsection 2.5.4.14. RQD values below the top 20 ft of bedrock varied across the site depending upon rock type, joint spacing and orientation relative to boring orientation, incipient jointing, and the incidence of dikes and faults. In general, the lowest RQD values were obtained in rock where joints were closely spaced and joint surfaces were highly polished, coated with chlorite or severely weathered. These conditions were observed more commonly in the borings penetrating diorite and quartzite beneath Reactor 2. A reasonable indication of areas where poor quality rock would have to be removed in the deep excavations was provided by RQD values. A few areas of poor quality rock not readily identified by the borings were encountered in the deep excavations and required excavation beyond the design lines. Treatment of areas requiring over excavation is described in Subsection 2.5.4.14.

In the diorite rock at the tunnel shaft locations, Boring AIT-1 indicated somewhat lower RQD above El. -130 than ADT-1. However, during excavation, conditions at the two shafts were quite similar, indicating that the poorer RQD values in AIT-1 can be attributed to the predominance of low angle joints in this boring. These shallow-dipping joints were of minor significance during excavation.

Several other borings of significant depth were made at the site but no RQDs were measured in these borings. These included borings of the B, D, and E series made from 1968 through 1974, logs for which are included in Appendix 2D. Rock quality as represented by fracture spacing on these logs is consistent with the rock quality as seen in excavations and the borings with measured RQD on the site.

RQD values were used primarily to provide general indications of rock quality across the site. RQD values, however, were also used to estimate Young's modulus of the in situ rock mass by using empirical relationships correlating RQD with Young's modulus as determined from laboratory tests on intact samples or from seismic velocities as determined in the field. This use of RQD is discussed in Subsection 2.5.4.5b.

3. Permeability of Rock Mass

Water pressure tests were performed in the bedrock in many of the boreholes drilled for the circulating water tunnels. Each test was performed on a 20-ft length of borehole, and an equivalent permeability of the rock mass was calculated based on the assumption that the water uptake was uniformly distributed throughout the zone tested. Measured permeabilities for the 20-ft zones ranged from 0 to $7x10^{-3}$ cm/sec, as shown on Table 2.5-12.

Permeabilities were computed using the procedure described in pages 544-546 of the Earth Manual (Reference 124). The test procedures and test pressures, water takes, and computed permeabilities are presented in References 120 and 121.

A pumping test was performed in Borehole F5 which is about 8,000 ft east of the plant site on Hampton Beach. The pumping test procedures are summarized in Subsection 2.5.4.3 and described in detail in Reference 120. A depth of approximately 200 ft of bedrock was tested. The equivalent coefficient of permeability for the 200-ft depth was computed to be $1x10^{-3}$ cm/sec.

4. Dip and Orientation of Joints, Slickensides and Foliation

The dip of joints, slickensides and foliation observed in the rock cores were measured and are reported on the boring logs.

Orientation of joints, slickensides and foliation was measured in portions of about 53 borings using the Christensen Core Orientation System described in Subsection 2.5.4.3.2. The oriented core data is contained in Reference 120 and Appendix 2F.

The dip and orientation of joints, slickensides and foliation in exposed bedrock outcrops at the site and in the excavations are described in Subsection 2.5.1.2 and shown in Figure 2.5-16, Figure 2.5-21 and Figure 2.5-22.

5. Compression and Shear Wave Velocities

In situ compression (P) wave velocities for the bedrock types at and in the vicinity of the site were measured by seismic surveys and uphole and crosshole geophysical tests. Compression wave velocities were also measured by laboratory sonic tests. The data indicate that both major rock types (diorite and quartzite) have essentially the same P wave velocity. Both the in situ and laboratory data indicate uniform P wave velocities across the site area, indicating a general uniformity of bedrock properties for engineering design. The range of wave velocities for each type of measurement is shown in Table 2.5-12. The average P wave velocity for the saturated laboratory specimens was 16 percent higher than for the dry specimens. The sonic test results shown in Appendix 2G indicate that an increase in confining pressure from 0 to 3000 psi causes less than a 3 percent increase in P wave velocity. An increase in axial load from 0 to 1000 psi causes less than 4 percent increase in P wave velocity.

In situ shear (S) wave velocities of the diorite were measured in the crosshole and uphole tests. The range of test results is summarized in Table 2.5-12.

The seismic surveys and uphole and crosshole tests are described in Subsection 2.5.4.3. The laboratory sonic wave velocity tests were performed on both dry and saturated intact core specimens using procedures generally in accordance with ASTM D2845. Detailed results of the sonic tests are contained in Appendix 2G and in Reference 120.

6. Rock Density

Values of density were determined for selected rock specimens used in the laboratory testing programs. The procedures used to determine density were generally in accordance with ASTM D2845. Results of these measurements are summarized in Table 2.5-12. Detailed test results are contained in Appendix 2G and Reference 120. An average rock density of 2.80 g/cm^3 was measured for the diorite in borings B-8 and B-9 for use in evaluating the geophysical survey data, as described in Subsection 2.5.4.4.

7. Unconfined-Compressive Strength

Unconfined-compressive strength was measured on 31 air-dry rock core specimens using procedures described in ASTM D2938. The test results are summarized in Table 2.5-12.

The data indicate a wide variation in strengths for each type of rock. The diorite has an average compressive strength of 18,300 psi and the quartzite an average compressive strength of 12,100 psi. The one test on quartzitic schist in the plant area indicates compressive strength similar to the diorite.

Detailed test data are presented in Appendix 2G and Reference 120.

8. Young's Modulus

Values of initial tangent Young's modulus (E_i) for static loading were determined from 25 of the unconfined-compression tests on rock core specimens. The procedures used were generally in accordance with ASTM D3148. On 12 of the specimens, the secant modulus at 50 percent of the ultimate compressive strength (E_{s50}) was calculated, while on the other 13 specimens, the tangent modulus at 50 percent of ultimate compressive strength (E_{t50}) was computed. The range of values for each of these moduli is shown in Table 2.5-12.

The data indicate similar modulus values for both types of rock (diorite and quartzite) at the plant site. The one test on quartzitic schist in the plant area indicated modulus similar to the diorite and the quartzite.

Young's modulus was used in calculating foundation heave and settlement due to excavation and structural loads, respectively. Adjustment of the Young's modulus measured on intact specimens, to account for the average RQD in the field, is discussed in Subsection 2.5.4.5b. Young's modulus was not used in dynamic analyses since the rock was assumed to be a rigid boundary below foundation grades as described in Subsection 3.7(B).2.3.

Detailed test results are contained in Appendix 2G and Reference 120.

9. Poisson's Ratio

Poisson's ratio for static loading was determined for 10 of the unconfined compression tests on rock core specimens. Values were calculated at the start of loading and at a load of 50 percent of ultimate compressive strength. The range of values is shown in Table 2.5-12.

Poisson's ratios for the diorite samples covered the entire range shown, while values for the quartzite (2 tests) were at the low end of the range. Individual test results are tabulated in Appendix 2G.

Poisson's ratios for dynamic loading were determined from the uphole and crosshole geophysical tests. These tests were performed in the diorite only, and summarized in Table 2.5-12.

Values of Poisson's ratio were used in calculating foundation heave and settlement due to excavation and structural loads, respectively. They were not used in dynamic analyses since the rock was assumed to be a rigid boundary below foundation grades as described in Subsection 3.8.5.1.

10. Dynamic Shear Modulus

Values of dynamic shear modulus for the quartz diorite were determined from the uphole and crosshole tests. The range of values is shown in Table 2.5-12.

11. In Situ Rock Stresses

In situ rock stresses were measured in Boring 0C1A, near the center of the Reactor Containment Building. This location was selected because the seismic and geophysical testing indicated higher than average compressional wave velocities in this area. The rock stresses were measured at five points between depths of 33 and 42 ft. The values of horizontal compressive stress are summarized below:

	Range	Average Stress
Largest stress	150 to 2,150 psi	1,240 psi
Smallest stress	50 to 1,570 psi	860 psi

The mean orientation of the largest horizontal stress was N40°E with a variation of up to ±35° for different tests.

Test procedures and detailed results are contained in Appendix 2H.

12. Rock Hardness

Values of hardness were determined for 24 rock core specimens. Four types of hardness and "total" hardness were determined for each specimen. The types of hardness determined were:

(a) Schmidt (L-type) Rebound Hardness, H_R

(b) Shore Scleroscope (C-2 type) Hardness, H_S

(c) Modified Tabor Abrasion Hardness, H_A

(d) Abrasivity, A_R

(e) Total Hardness, $H_T = H_R \sqrt{H_A}$

There are no standard procedures available for these tests. The tests were performed in general accordance with procedures described in Tarkoy (Reference 125).

Test results are contained in Reference 120.

b. Overburden Soil Properties

Since all seismic Category I structures are founded on bedrock, on concrete fill over bedrock, or on controlled backfill over bedrock, and are surrounded by controlled backfill, or CLSM detailed properties of the overburden soils at the site are not required for evaluation of safety-related structures.

The overburden materials at the plant site consist of a surface layer of loamy brown silty fine sand which is generally not more than about 2-ft thick, underlain by a layer of dense silty gravels and sands (glacial till) which extend to bedrock. The total thickness of overburden is in the range of 0 to 20 feet. The standard penetration resistances measured in the dense silty sands and gravels are generally in the range of 40 to 100 blows per foot. Logs of the overburden soils in the plant site borings are contained in Reference 120 and Appendices 2F, 2I and 2J.

The overburden along the circulating water tunnel alignments generally consists of three strata: an upper stratum of sand, a middle stratum of clay, and a lower stratum of glacial till which rests on the bedrock. In the marsh area between the site and Hampton Harbor, a stratum of peat up to 30-ft thick was encountered. The total thickness of overburden ranges from 0 to over 120 ft.

The thickness of each stratum and standard penetration test blowcounts in the overburden are shown on the boring logs in References 120 and 121.

Laboratory tests were performed on selected samples from the plant site and tunnel borings to determine the general properties of the overburden. Table 2.5-14 contains an index of the tests performed, the test methods used, and the reference in which the test data may be located.

2.5.4.3 Exploration

An index of the exploration programs is contained in Table 2.5-15 indicating the location of the boring logs and/or detailed report prepared for each program.

Subsurface explorations were performed to determine soil and rock conditions at the site and along the circulating water tunnel alignments, and to investigate geologic features in the vicinity of the site. The explorations were performed in several different programs during the period 1968 through 1979, and consisted of over 340 borings in soil and rock, ten shallow trenches, 200 seismic refraction and seismic reflection lines, uphole and cross-hole velocity measurements, water pressure permeability tests, one pumping test, and in situ stress measurements in a boring near the center of the Containment for Unit 1. Soil and/or rock samples were obtained from each of the borings, and rock core was oriented in many of the borings to determine the strike and dip of joints, fractures, foliation, and contacts between different rock types.

The subsurface explorations were performed under the direct field supervision of a qualified geologist or geotechnical engineer. The rock core from all borings was logged in the field office by a qualified geologist.

Following, in chronological order, is a brief description of each of the subsurface investigation programs.

a. Initial Borings (A, B, C and P Series)

Between October 1968 and July 1969, 157 borings (A, B, C and P Series) were drilled on the plant site and in the vicinity of the site to obtain data on the bedrock and overburden soils.

Forty-eight borings (B-Series) were drilled in the site area to depths of 20 to 176 ft. The thickness of overburden was 0 to 94.5 ft. Split-spoon samples and Standard Penetration Test N-values were obtained in the overburden in all but five of the borings. Rock cores were obtained in each boring from the bedrock surface to the bottom of the boring.

Fourteen observation wells were installed in 12 of the borings using one of two methods. Where bedrock occurred near the surface (less than 8 ft), the 4-in. steel drill casing was seated on the rock and left in place, forming on open-hole observation well in the bedrock. Where the bedrock was deeper than about 8 ft, plastic observation well with gravel pack was installed. In two of the borings, plastic observation wells were installed at two different depths.

Groundwater conditions at the site are discussed in Subsection 2.4.13. Driller's logs for the B-Series borings are contained in Reference 122. Detailed rock descriptions for selected borings are contained in Appendix 2D.

The locations of the B-Series borings are given in Figure 2.5-17. Geologic profiles of the site area are given in Figure 2.5-11, Figure 2.5-16 and Figure 2.5-20.

Twenty auger borings (A-Series) were drilled in the Hampton Beach State Park (barrier beach) to investigate soil conditions. The depths of the borings ranged from 49 to 56 ft. Borings were terminated in soil or at refusal, with no rock coring. Bulk samples from the auger were taken at approximately 5-ft intervals. Boring logs for the A-Series borings are contained in Reference 122.

Seventy-eight borings (C-Series) were drilled in the tidal marsh east of the site to obtain split-spoon soil samples and rock cores. The borings ranged in depth from 7 to 52 ft. The thickness of overburden soils ranged from 2 to over 52 ft. Logs of the C-Series borings are shown in Reference 122. Locations of the borings are shown on Figure 2.5-9 and Figure 2.5-10.

Eleven P-Series borings were drilled offshore from the barrier beach to obtain split-spoon samples of the overburden soils. Borings were terminated in soil or at refusal. The depth of the borings ranged from 14 to 17.5 ft. Logs of the borings are contained in Reference 122. Locations of the borings are shown on Figure 2.5-9.

b. <u>Additional Site Borings (D and E Series)</u>

In November and December of 1972, 24 additional borings (D and E-Series) were made to define further the soil and bedrock conditions in the site area. The depths of the borings ranged from 24 to 171 ft. Overburden soils ranged from 1 to 25 ft. Information obtained included split-spoon soil sample descriptions, standard penetrations test blowcounts in soils, rock core descriptions, relative frequency of core breaks and partings, and joint angles.

The locations of the D- and E-Series borings are shown in Figure 2.5-17. Rock core logs and soil sample descriptions are given in Appendix 2D and 2J, respectively.

c. <u>Seismic and Fathometer Surveys</u>

Seismic refraction and reflections surveys were performed during the fall of 1968 and during March and April 1973 in five zones at and in the vicinity of the site:

1. Plant site
2. Tidal marsh east of site
3. Hampton Harbor
4. State Park - State Beach (barrier beach)
5. Offshore from the barrier beach

The objective of these surveys was to determine the depth of bedrock and to locate major seismic boundaries in the overburden. Fathometer surveys were performed in Hampton Harbor and offshore of the barrier beach in March and April 1973 to determine bottom contours.

The results of the seismic and fathometer surveys are discussed in Subsection 2.5.4.4. Plans of the survey lines and profiles and contour maps of the survey data are contained in Appendix 2K.

The depth of major strata changes and the type of soil and rock in each major stratum were confirmed by the C and P series borings. The locations of these borings relative to the seismic lines are shown on the plans in Appendix 2K. Depths to the bottom of the boring and to bedrock or refusal are shown on the seismic profiles.

d. In Situ Wave Velocity Measurements

Compression (P) and shear (s) wave velocity measurements were made in the boreholes at the proposed reactor locations using up-hole and cross-hole techniques. Up-hole data were obtained in Boring B-38 and cross-hole measurements were made in Borings B-22, 35, 37, 38, 39, 40 and 41. The locations of these borings are shown in Figure 2.5-17. The results of the in situ velocity measurements are summarized in Table 2.5-12.

Details of the test procedure are given in Subsection 2.5.4.4.

e. Subsurface Investigations for Circulating Water Tunnels (AAIT, AIT, ADT and F Series)

1. Boring Program

During the period April 1973 to May 1974, geotechnical investigations were performed to provide engineering and geologic data pertinent to the design and construction of the circulating water tunnels and the vertical shafts at the plant and ocean ends of the tunnels. These geotechnical investigations also provide additional geologic data needed for interpreting the regional and site geology.

One hundred-twelve borings (AAIT, AIT, ADT, and F Series) were made along the discharge tunnel alignment and three alternate intake tunnel alignments. The boring program included angle borings, core orientation, and in situ permeability measurements.

Borings were made to obtain the following information:

(a) Geologic classification and description of soil and rock types

(b) Degree of fracturing, core recovery, and RQD of bedrock

(c) Degree of weathering of bedrock

(d) Orientation of foliation

(e) Orientation and frequency of joints and other fractures

(f) Orientation of contacts between different rock types, including orientation of dikes

(g) Water intake and the equivalent permeability of the bedrock

(h) Strength, hardness, and abrasivity of the bedrock for evaluation of tunnel excavation procedures

(i) Engineering classification and description of overburden.

The layout of the borings for the circulating water tunnels is shown on Figure 2.5-9 and Figure 2.5-10. Boring logs are contained in Reference 120. Profiles of soil and bedrock surface, rock type, core recovery, RQD, rock weathering, permeability and high angle joint frequency are also shown in Reference 120. Subsurface conditions for the circulating water tunnels are summarized in a paper by Desai et al., (Reference 119).

Split-spoon samples and standard penetration tests were taken at 5-ft intervals in soil until bedrock or refusal was encountered. Then coring was performed using NX (2 in. ID) conventional core barrels and NQ (1 in. ID) wireline core barrels. Coring was generally done to a depth of 50 ft below the tunnel invert elevation in core runs up to 10 ft long.

Prior to the selection of deep rock tunnels for the Circulating Water System, conduits were proposed to pass through the overburden sands on the marsh and barrier beach. To permit laboratory investigation of the cyclic mobility potential of these overburden sands, undisturbed sand samples were obtained in Borings F1A and F2. The cylic mobility potential of these overburden sands is discussed in Reference 123.

2. Core Orientation

Rock core was oriented in 46 of the 112 borings, using the Cristensen Core Orientation System, to determine the strikes and dips of joints, fractures, foliation, and contacts between rock types. The equipment consists of a special core barrel with a scribe attached to the bottom of the barrel and a compass attached to the top to indicate the orientation of the scribe. A camera mounted just above the compass photographs the compass at preset, constant time intervals. During drilling operations, the vibrations cause the photographs to be blurred. Therefore, the drilling is stopped at selected depth intervals, generally 1 ft, to allow the compass to stabilize and to obtain a clear photograph. The drilling depth and elapsed time at each interruption in the drilling are recorded. These data and the preset photograph interval are then used to compute the depth at which each clear photograph was taken.

The orientation of the reference line at each depth is determined from the corresponding compass photograph. Knowing the orientation of the scribe line, pieces of core can then be properly positioned in an orientation device (goniometer) which permits direct measurement of the strike and dip of various features.

All of the oriented core data is presented in Reference 120, including stereonets showing pole projections for all planar features and line projections for linear features.

3. Permeability Tests

Water pressure tests were performed in the rock portion of most of the borings along the tunnel lines. The pressures, water takes, and computed permeabilities are listed on the boring logs.

Some borings were pressure tested in approximately 20-ft intervals during advancement of the boring using a single BX or NX packer, but most borings were pressure tested after the boring was completely drilled. In the later case a double-packer assembly was used to test a 20-ft zone of borehole. The maximum pressure used during the test was 1 psi per foot of depth.

The tests were performed by first testing at one-half the maximum pressure, then maximum, then one-half maximum. The flows were measured using water flow meters. Permeability profiles for the intake and discharge tunnels are shown in Reference 120.

A pumping test was performed in borehole F5 in September 1973 to obtain data regarding permeability of the bedrock and to provide data for estimating water inflow during construction. A submersible pump was located in the cased borehole at a depth of 116 ft, about 6 ft above the bedrock surface. Frequent determinations were made of the pumping rate and the cumulative volume pumped over a continuous pumping interval of 54 hours. Twelve water samples for chloride determinations were obtained at various times during the test to monitor changes in salinity of the discharge. The results of the pumping test are contained in Reference 120.

4. In Situ Temperature Measurements

In situ temperature measurements were made in Boring F2 to determine the temperature profile prior to construction and operation of the circulating water tunnels. To allow temperatures to stabilize in the boring, the measurements were made on the day after drilling of the boring was completed. The measurements were taken at various depths using a thermistor mounted in a probe. Measurements were taken both as the probe was inserted and then as it was withdrawn from the hole.

A simplified profile of Boring F2 and a plot of temperature versus depth is shown on Figure 2.5-45.

f. In Situ Rock Stress Measurements

In situ rock stress measurements were made in June and July of 1973 at depths of 33 to 43 ft in vertical Boring OC1A, using the overcoring technique. This boring is about 14 ft from the center of Reactor 1. Test results are summarized in Table 2.5-12. Detailed results and test procedures are contained in Appendix 2H.

These rock stress measurements were performed in the diorite, at a location where the seismic surveys indicated higher than average P wave and S wave velocities, to investigate the possibility of high residual in situ stresses. The measured in situ horizontal stresses are similar in magnitude and direction to those published for other areas of New England.

g. Scotland Road Fault Investigation

1. General

Field investigations were performed during the period of November 1973 to March 1974 to locate and examine the Scotland Road Fault and its overlying Pleistocene deposits. Investigations included ground and aerial reconnaissance, a stadia survey of the area, a seismic refraction survey, nine borings, and four backhoe trenches. A base map showing the location of the field investigations is presented in Appendix 2C and on Figure 2.5-8.

2. Seismic Refraction

A seismic refraction survey was conducted across the study area during the period of November 5-9, 1973 to determine the thickness of unconsolidated overburden and weathered rock materials as well as seismic velocities of local geologic materials. The survey consisted of a 1,000-ft line trending NW and five cross lines. The results of this refraction survey are shown on a velocity profile of subsurface materials in Appendix 2C.

3. Borings

During the period of December 4, 1973 to February 13, 1974, seven vertical and two angle borings (SRF Series) were drilled along the center-line of the seismic refraction surveys to locate, define, and sample the Scotland Road Fault zone. The boring logs and a geologic profile of the fault zone based on the borings is shown in Appendix 2C.

4. Trenches

During the period of November 20, 1973 to March 4, 1974, four backhoe trenches were excavated in the study area to expose and examine the glacial marine clay which overlies the Scotland Road Fault zone. Trench locations and profiles are presented in Appendix 2C.

h. Portsmouth Fault Investigation

Field investigations were conducted along the general path of the inferred Portsmouth Fault (see Subsection 2.5.1.1) in an attempt to determine the presence, location, orientation, and physical characteristics of the feature, and to examine the nature and structure of unconsolidated deposits which overlie bedrock in the area. Investigation included ground reconnaissance, examination of aerial photographs, magnetic surveys, four core borings (PF series) and two backhoe trenches. The Portsmouth Fault investigations did not locate the inferred fault, nor did it reveal the existence of any major structural feature trending toward Hampton from Portsmouth.

The borings and trench locations and geologic profile are shown in Appendix 2C.

i. Inclined Borings at Reactor Sites (E2-Series)

In May and June of 1974, eight inclined borings (E2 Series) were made around the perimeter of two proposed reactor excavations to determine the frequency and orientation of fractures in the rock. The borings ranged in length from 165 to 169 ft and in inclination from 39° to 41.5°, measured from the vertical. The borings were advanced through the soil by washing since the inclination of the holes made it impractical to take conventional split-spoon samples or to measure standard penetration resistances. The rock core was oriented from near the rock surface to the bottom of the hole in five of the eight borings using the procedures described in Subsection 2.5.4.3e.2. In the other three borings, rock core was oriented for only part of the total length.

A plan of the boring locations, boring logs, and generalized dip and strike of the joints are shown in Appendix 2F.

j. Additional Plant Site Borings and Test Pit (G-Series and TP-100)

In September and October of 1974, 12 borings (G-Series) and one test pit (TP-100) were made to determine soil and bedrock conditions for the design and construction of the water and oil storage tanks, settling basin, retaining wall, seawell, and riprap structures. In nine of the borings (G-1 through G-8, G-12) standard penetration tests and split-spoon samples were taken in the overburden soils, and the borings were terminated at refusal. The remaining three borings (G-9 through G-11) were washed through the overburden with no sampling, and then rock was cored for 15 ft using an NX core barrel.

One test pit was excavated near the center of the settling basin to obtain a bulk sample of the overburden soils for laboratory testing. See Subsection 2.5.4.2b for references to the laboratory testing performed.

The location of the borings are shown on Figure 2.5-17. Boring logs, test pit log and test pit location are contained in Appendix 2I.

k. Intake Tunnel Extension Borings (AIT-Series continued)

From June through August 1975, 19 borings (AIT-Series, continued) were drilled to investigate geologic conditions along the 3400-ft intake tunnel extension and at intake shaft locations. The locations of these borings are shown on Figure 2.5-9 and Figure 2.5-11. The depth of the borings below ocean bottom ranged from approximately 190 ft to 220 ft, except for one boring which was 84 ft deep.

Standard penetration tests and split-spoon samples were taken in the ocean bottom sediments in each boring.

Rock cores were taken using NX (2in.) or NQ wireline (1in.) core barrels. Core recovery, RQD, and distribution of high angle joints (dip greater than 50°) were measured for each section of rock core. Water pressure tests were performed in all but one boring to determine permeability of the rock. Schmidt Hammer hardness was measured on selected core sections from each boring.

The data from these tests and measurements are shown on the boring logs in Reference 121. Profiles of soil and bedrock surface, rock type, core recovery, RQD, rock weathering, permeability and high angle joint frequency are also shown in Reference 121. Subsurface conditions along the tunnels are summarized in the paper by Desai et al., (Reference 119).

l. Exploratory Trenches at Reactor 2

Four test trenches about 200-ft long and 20-ft wide were excavated in an "X" pattern at the location of the Unit 2 Containment to examine the stratigraphy of the overburden and the soil-rock contact. Previous borings in this area had indicated the presence of several faults in the bedrock, and the trenches were excavated to determine whether any fault-related displacement had occurred in the overburden. No evidence of such displacement was observed. The overburden and bedrock in this area subsequently have been mapped in greater detail during the actual construction excavation as described in Subsection 2.5.1.2b.7.

The location of the trenches and the report on the trench program are contained in Appendix 2L.

m. Geologic Mapping of Construction Excavations

Geologic mapping of the rock and overburden soils encountered in the construction excavation is discussed in Subsection 2.5.1.2b.6.

2.5.4.4 Geophysical Surveys

Seismic profiles, including compressional wave velocity values and a bedrock contour maps of the offshore areas, based on the seismic and test boring data, are presented in Appendix 2K.

Table 2.5-12 summarizes the in situ compressional "P" and shear "S" wave velocity measurements, along with the corresponding elastic moduli values. "P" and "S" wave velocity measurements were made in the boreholes at reactor locations using uphole and cross-hole techniques. Uphole data were obtained in Boring B-38 by shooting in the drill hole and recording at the surface. Cross-hole data were obtained by lowering four three-component seismometers into four different drill holes to a common depth and shooting a small explosive charge in a fifth drill hole at the same depth. This procedure was repeated using different shot hole and recording hole combinations. Borings used in the cross-hole measurements included Borings B-22, B-35, B-37, B-38, B-39, B-40 and B-41. Density values for the bedrock used in the computation of the elastic moduli were obtained from core samples in Borings B-8 and B-9.

2.5.4.5 Excavations and Backfill

a. Extent of Excavation and Backfills

The extent of excavations for seismic Category I structures is shown on Figure 2.5-46. Photographs of typical excavations are shown on Figure 2.5-50, Figure 2.5-51, Figure 2.5-52.

All seismic Category I structures are founded on sound bedrock or on engineered backfill extending to sound bedrock. Engineered backfill was also placed around all Seismic Category I structures. The engineered backfill consists of either fill concrete, backfill concrete, offsite borrow, tunnel cuttings, CLSM, or sand-cement, as described in Subsection 2.5.4.5c below. As shown in Table 2.5-20, fill concrete was used as the engineered backfill beneath the foundations of all seismic Category I structures except for safety-related electrical duct banks, five electrical manholes and service water pipes which are founded on offsite borrow, CLSM, or tunnel cuttings.

The extent of engineered backfill beneath and around the seismic Category I structures and safety-related electrical duct banks, electrical manholes and service water pipes are shown on Figure 2.5-47, Figure 2.5-49, Figure 2.5-50, Figure 2.5-51 and Figure 2.5-52. The locations of these sections are shown on Figure 2.5-46 and Figure 2.5-47.

Random fill was used for general site grading in areas not requiring engineered backfill.

b. Dewatering and Excavation Methods

The bedrock in the main plant area was exposed by removing the overburden using conventional techniques. The design elevations for the various building foundations were reached by controlled blasting of bedrock, observing all applicable codes and standards. Vibrations were monitored during blasting to verify adherence to specified vibration criteria to preclude any damage to bedrock at or below the foundation level or to adjacent structures or in-place concrete. The NRC representatives periodically inspected the excavation progress.

After completion of excavation, all bedrock surfaces were thoroughly cleaned, using a variety of methods, to remove all loose fragments, dirt and debris. Following cleaning, all bedrock surfaces supporting seismic Category I structures were inspected and mapped in detail by a geologist familiar with the rock foundation requirements and the geologic and engineering properties of the rock mass. The inspection included an examination and evaluation of the physical characteristics of the rock mass such as pattern and distribution of jointing and the amount and degree of weathering. Bedrock excavation included removal of very small amounts of severely weathered rock which were present at final grade in a few isolated areas. The pertinent information is included in Subsection 2.5.1.2b.6.

Dewatering methods used during placement of engineered backfill are described in Subsection 2.4.13.5.

During the comparatively short period of exposure prior to placing of engineered backfill, no special protection of the base rock was required.

This rock is a sound diorite and quartzite with diabase dikes, as described in Subsection 2.5.1.2b.6, and has very low porosity, tight joints and very low permeability.

Measurements of heave or rebound of the rock in the excavations were not taken. However, no instances of rock behavior or excavation and foundation movements attributable to heave were observed. This is consistent with expected behavior, since the predicted maximum rock heave in seismic Category I excavations was 0.25 in. in the center of the bottom of the reactor excavations. Heave of this magnitude would have no significant effect on the properties of the rock below the excavations or on the performance of the structures placed in the excavations.

The heave was estimated using a Boussinesq elastic pressure distribution for the unloading of 80 ft of soil and rock overburden for a 200-ft diameter area. The equation used to compute the heave was:

$$\delta = \frac{2\,qr\left(1-v^2\right)}{E}$$

where δ = heave, in.

q = magnitude of unloading, psi

r = radius of unloaded area, in.

v = Poisson's ratio

E^{field} = In situ Young's modulus

The elastic modulus used for the heave analysis was $E_{field} = 1.0 \times 10^6$ psi. This value of E_{field} was determined by correcting the average laboratory modulus for intact rock at the reactor locations, $E_{lab} = 10 \times 10^6$ psi (Appendix 2G, Table 2G-1), to account for the average RQD of the rock below the excavation using empirical data presented in Hendron (1968). For the average RQD = 60 percent at the Unit 2 reactor site, which is lower than the average at the Unit 1 reactor site, the correction factor is $E_{field}/E_{lab} = 0.1$.

c. Engineered Backfill

All backfill used under and around all Category I structures is engineered backfill consisting of either fill concrete, backfill concrete, offsite borrow, tunnel cuttings, or sand-cement. Approximately 500,000 cu yds of engineered backfill were required in safety-related areas. In addition, approximately 500,000 cu yds of nonsafety-related engineered backfill and random fill were required.

The six types of engineered backfill are described below:

1. Fill Concrete

Fill concrete was used under all Category I structures except electrical ductbanks, five electrical manholes, and service water piping, from the top of sound bedrock to the bottom of the structure. The fill concrete has a minimum 28-day compressive strength of 3000 psi, and was produced and tested in accordance with the same procedures used for Category I Structural Concrete described in Subsection 3.8.3.6a.2. The extent of fill concrete is shown on typical sections in Figure 2.5-47, Figure 2.5-48, Figure 2.5-49, and Figure 2.5-50.

Results for unconfined compression tests on fill concrete are shown in Figure 2.5-59. These results cover a six-month period from May 24 to Nov. 11, 1978 and represent all test data for fill concrete placed under the containment mat for the former Unit 2. These results are representative of fill concrete placed beneath all seismic Category I structures.

2. Backfill Concrete

Backfill concrete was used to backfill between the structure wall and the rock excavation wall for all Category I structures that were founded below the bedrock surface. Backfill concrete has a minimum 28-day compressive strength of 2000 psi and was produced and tested in accordance with the same procedures used for Category I Structural Concrete described in Subsection 3.8.3.6a.2. The extent of backfill concrete is shown on typical sections in Figure 2.5-48, Figure 2.5-49 and Figure 2.5-50.

Representative results for unconfined compression tests on backfill concrete are shown in Figure 2.5-60. These results are for backfill concrete placed in various safety-related areas of the site during the period May 30, 1978 to October 30, 1980.

3. Offsite Borrow

Offsite borrow was placed under, adjacent to and above safety-related duct banks, manholes, service water pipes, and adjacent to seismic Category I structures above the bedrock surface.

The maximum thickness of offsite borrow beneath safety-related electrical duct banks was 18 ft. Typical profiles of offsite borrow beneath safety-related electrical ductbanks are shown in Sections KK, LL, and MM on Figure 2.5-50.

The maximum thickness of offsite borrow placed beneath safety-related electrical manholes was 18 ft. Typical profiles of offsite borrow beneath safety-related electrical manholes are shown in Figure 2.5-50, Sheet 1, Section MM and Figure 2.5-50, Sheet 2, Sections NN and PP.

The thickness of offsite borrow beneath safety-related service water pipes ranged from approximately 2 to 15 ft. Typical profiles in which the depth of offsite borrow beneath safety-related service water pipes ranges from 2 to 5 ft are shown in Figure 2.5-50, Sections JJ, KK, LL, and MM. A typical profile in which the depth of offsite borrow beneath safety-related service water pipes ranges from approximately 10 to 15 ft is shown in Figure 2.5-49, Section II.

The maximum thickness of offsite borrow placed adjacent to safety-related structures is approximately 63 ft along the west wall of the Discharge Transition Structure and the east wall of the Service Water Pumphouse.

The offsite borrow was obtained principally from the following locations: Pyburn pit, Kensington, N.H. about 3 miles from the site; Brentwood pit, Brentwood, N.H. about 17 miles from the site, Beard pit, Dover, N.H. about 25 miles from the site, and Lee pit, Lee, N.H. about 23 miles from the site. The deposits in these pits consist of stratified sand with some gravel and cobbles and/or outwash consisting of coarse sand with some fine gravel. Laboratory engineering properties of the material were investigated and are summarized in Table 2.5-16. A test fill was completed and is described in Subsection 2.5.4.5d.

Principal specification requirements for the offsite borrow were:

(a) Offsite borrow met the following gradation requirements:

Sieve Size	Percent Passing
1½"	100
¾"	100-95
#4	95-50
#10	86-30
#20	70-15
#40	50-7
#60	32-3
#200 (washed)	10-0.2

The allowable variation for each limit (fine and coarse side of the gradation band) for sieve size other than 1½" and #200 did not exceed an aggregate total of ten percent, with individual maximum variation of five percent for the fine side and ten percent for the coarse side as long as the coefficient of uniformity C_u of the offsite borrow was maintained greater than or equal to three (3).

Stratified deposits were blended at the borrow pits, omitting any fine grained soils, and frequent gradation checks assured conformance to required standards.

(b) The offsite borrow was placed in uniform layers of eight inches or less and compacted at or near optimum moisture content to obtain not less than 95 percent of the maximum density shown on the dry weight curve as determined by the Modified Proctor Compaction Test ASTM D1557-70. During the compaction process, moisture control was maintained at or near the optimum levels by continuous wetting during the compaction effort as required.

(c) Compaction verifications were accomplished by performing in-place density tests in accordance with ASTM D1556 or ASTM D2922 and D3017.

(d) Testing criteria:

(1) At least one gradation test per day or every 2,000 cubic yards of offsite borrow delivered.

(2) One compaction check per lift for every 2000 linear feet maximum where hand-operated compaction equipment was utilized.

(3) One compaction check per lift for every 20,000 SF of plan view area (largest plan view dimension not to exceed 200 ft) where heavy self-propelled compaction equipment was utilized.

(4) One-point modified Proctor compaction test for each in-place density test, used to estimate maximum dry density for each density test by interpolation from a family of compaction curves. See Figure 2.5-65 for a summary plot of compaction curves.

The frequency of these tests was increased when considered necessary by the inspectors or engineers in charge.

Figure 2.5-51 shows actual gradation band of offsite borrow as delivered, developed from sieve analyses from representative samples of material used, superimposed over the specified gradation band.

Every layer of offsite borrow was compacted to a dry density of at least 95 percent of the maximum dry density, as determined by ASTM D1557. Any layer which did not attain the required 95 percent compaction was recompacted and retested until the specified minimum degree of compaction was met.

During post construction activities approximately 48 cubic yards of existing engineered backfill were removed then replaced and compacted but not tested. The untested backfill is located on the north side of the Cooling Tower, between column lines K and N and within 10 feet of the structure. This material was placed and compacted in 8" lifts in the top 3½ feet of an area of approximately 370 square feet.

Representative field density test results for offsite borrow placed in the plant areas during typical winter, spring, and summer periods are shown in Figure 2.5-52, Figure 2.5-53 and Figure 2.5-54, respectively. As seen in these three figures, the offsite borrow was compacted to at least 95 percent of ASTM D1557-70 compaction.

Offsite borrow was also used as fill behind the revetments, as shown in Figure 2.5-68 and Figure 2.5-69. This offsite borrow was compacted to at least 90 percent of maximum dry density determined by ASTM D1557-70. Typical field density test results of the offsite borrow placed to at least 90 percent compaction are shown in Figure 2.5-56. Near the railroad tracks at the west end of Revetment A, the offsite borrow was placed to at least 95 percent compaction, as shown on Figure 2.5-55.

4. Sand-Cement

Sand-cement was used in only one safety-related area to backfill adjacent to and above the service water piping placed in a trench excavated in rock, from N9774, E6250 to N9774, E6430. Figure 2.5-50, Section JJ, shows the service water pipe installed in this trench, as well as the various engineered backfill materials placed beneath and around the pipe. Note that engineered backfill from the top of bedrock to the invert of the pipes was offsite borrow. Sand-cement was used to backfill from the invert to a level about 6 ft above the top of the pipes.

The processed sand used for sand-cement was obtained from Ossipee and Dover, N.H. The cement was Type II.

The mix, based on trial batch design, consisted of the following:

Batch	Quantity
Sand, lbs.	2735
Cement, lbs.	169 (6.2 percent of total sand)
Water, gallons	56.5

Laboratory engineering properties of the acceptable mix described above are summarized in Table 2.5-17.

The specified 28-day compressive strength was 100 psi. The ingredients (sand, cement, and water) were subject to specifications, quality, and testing requirements similar to those for materials used for Category I concrete, as described in Subsection 3.8.4.6.

Table 2.5-18 shows all the results of the field tests including pertinent laboratory test results.

Figure 2.5-58 shows plots of strength gain recorded from the tests performed on cylinders at specified periods, i.e., 7, 28, and 90 days of curing.

Quality control procedures similar to those for producing and placement of Category I concrete were implemented, including testing requirements to verify the strength of the sand-cement.

5. Tunnel Cuttings

Tunnel cuttings were placed in two safety-related areas of the site in the vicinity of the Plant Administration Building. The coordinates of these two areas are approximately N10160 to 10220, E5290 to 5360; and N10140 to 10210, E5420 to 5550. The greatest thickness of tunnel cuttings in safety-related areas of the site is about 15 ft beneath manhole W19/20 in the second area noted above. A profile of the tunnel cuttings beneath and adjacent to manhole W19/20 is shown in Figure 2.5-50, Section 0-0.

Tunnel cuttings were not placed against nor within a 10 ft horizontal distance of the walls of any seismic Category I building.

Tunnel cuttings were produced by the tunnel boring machines during excavation of the circulating water tunnels. The tunnel cuttings were predominantly quartzite and quartz diorite with occasional small amounts of quartzitic schist and diabase dikes. All rock types were hard and sound, with similar properties as indicated in Subsection 2.5.4.2. The gradation band for the tunnel cuttings is shown in Figure 2.5-61. Engineering properties determined from laboratory and field tests on quartzite tunnel cuttings are summarized in Table 2.5-19.

The test fills of the quartzite tunnel cuttings were con-structed, and plate load tests were conducted on each fill as described in Subsection 2.5.4.5d.

The specification requirements were as follows:

(a) The gradation of the material was:

Sieve Size	U.S. Standard Size
3"	100
1½"	100-50
¾"	100-25
"	100-12
#4	75-8
#10	50-5
#40	25-2
#200 (washed)	12-0

The coefficient of uniformity C_u of the tunnel cuttings was greater than or equal to five.

Maximum size of stones was three inches. Elongated stones larger than 3 inches and up to a maximum of 6 inches but passing through 3-inch sieve were accepted. Elongated stones larger than six inches, as visually noticeable, were removed during placement as structural fill.

(b) In safety-related areas, the tunnel cuttings were placed in uniform layers of eight inches or less and compacted to achieve not less than 95 percent of the maximum density as determined by the Revised Modified Proctor Compaction Test ASTM D1557-70. Materials passing the 1½ inch sieve, rather than passing the ¾ inch sieve, were used as the maximum size. During the compaction process, material moisture conditions were maintained to ±1 percent of optimum, where practicable.

(c) Compaction verifications were accomplished by performing in-place density tests in accordance with ASTM D2922 and D3017, with appropriate corrections to account for materials retained on the 1½ inch sieve and a moisture bias correction, if any.

(d) Testing criteria:

(1) At least one gradation test per day or every 2,000 cu. yd. of tunnel cuttings.

(2) One compaction check per lift for every 200 linear feet maximum where hand-operated compaction equipment was utilized.

(3) One compaction check per lift for every 20,000 sq. ft of plan view area (largest plan view dimension not to exceed 200 ft), where heavy self-propelled compaction equipment was utilized.

(4) One-point modified Proctor compaction test for each in-place density test, used to estimate maximum dry density for each density test by interpolation from a family of compaction curves. See Figure 2.5-64 for a summary plot of compaction curves. Figure 2.5-57 shows the field density test results for all safety-related tunnel cuttings placed in the areas noted above.

6. Controlled Low Strength Material (CLSM)

Controlled Low Strength Material (CLSM) is defined and detailed by the American Concrete Institute in their Committee Report on CLSM ACI 229R-99 (Reapproved 2005) to be a cementitious, flowable, self-leveling, self-compacting backfill material that hardens to 100% density. CLSM is a relatively new engineered backfill material mixture that has been tested and demonstrated to have many benefits compared to the traditional compacted granular backfill. The CLSM specified for use at Seabrook Station is a mixture of sand, Portland cement, fly ash, and water, with an air entrainment admixture that has been tested to demonstrate the required physical geotechnical properties and attributes.

Table 2.5-13 provides a comparison summary of the basic material properties and Table 2.5-22 presents a summary of properties of the CLSM developed from laboratory testing.

One of the key attributes of CLSM backfill, beyond its ease of installation and safety features, is the specified minimum and the controlled maximum 28-day compressive strength. Having a minimum 28-day compressive strength of 50 psi assures that the CLSM will have the required bearing capacity similar or better than a compacted granular backfill and sufficient strength for all loading requirements. Having a controlled maximum 28-day compressive strength of 150 psi assures that the CLSM will be "excavatable" (i.e. removable by hand tools) for future maintenance/inspections if required.

The CLSM, design mix specified herein, has been tested, evaluated, and demonstrated to have the required physical properties and attributes for use as engineered backfill for safety-related and non-safety related applications without limitation.

d. Test Fills

Four test fills were constructed of the engineered backfill materials, one using the offsite borrow and three using the tunnel cuttings. Plate load tests were performed on each test fill. The test fills were constructed to:

1. Measure the in situ modulus of the offsite borrow.
2. Measure the in situ modulus of the tunnel cuttings and demonstrate that the tunnel cuttings would have modulus values equal or superior to the offsite borrow.
3. Develop procedures for placement, compaction and testing of the tunnel cuttings.

   Engineering properties for the offsite borrow and tunnel cuttings, determined from the test fills and from laboratory tests, are summarized in Table 2.5-16 and Table 2.5-19. Detailed procedures and results of the test fill study are contained in Appendix 2N.

e. Random Fill

Random fill was used for nonsafety-related general site backfill and grading in areas not requiring engineered backfill. Random fill consisted of offsite borrow, tunnel cuttings, and soil from onsite excavations with less stringent placement requirements than the engineered backfill. Random fill was placed in 8-to-12-in. lifts and compacted to at least 90 percent of the maximum dry density determined by ASTM D1557-70.

In the plant area the maximum vertical thickness of random fill is about 40 ft at a location between the Circulating Water/Service Water Pumphouse and intake/discharge transition structure, as shown on Section A-A in Figure 2.5-48. At this location engineered backfill with a minimum horizontal extent of 10 ft was placed against the walls of the structures. Random fill was placed between the areas of engineered backfill.

Beyond the plant site, areas where random fill was used to raise the general site grade may be determined by comparing Figure 2.4-1 (plan of the final plant grade) and Figure 2.5-15 (plan of original site contours).

For small local excavations and backfilling in areas not requiring engineered backfill, the testing of the compacted material may be waived upon review of its size and location.

2.5.4.6 Groundwater Conditions

As stated in Subsection 2.4.13.2, the groundwater table in the site area is mostly in till or bedrock at depths no greater than 17 feet and usually less than 10 ft below the original ground surface. Groundwater contours at the site prior to construction are shown in Figure 2.4-28 and Figure 2.4-29. By assuming groundwater at elevation +20.0 feet MSL (the finished plant grade), the most severe case for hydrostatic loading is considered for design of structures.

In addition, all seismic Category I structures are founded on rock or on engineered backfill over rock, and no differential settlements due to changing groundwater conditions are expected.

The plant did not originally employ a permanent dewatering system. Therefore, all subsurface portions of safety-related structures, systems, and components were designed for hydrostatic pressure and uplift due to the assumed groundwater level at elevation +20.0 feet MSL.

Information on dewatering during construction can be found in Subsection 2.4.13.5.

During construction, groundwater inflow was minimal. Total inflow into various building excavations on site was estimated to range from 0 to 15 gallons per minute during survey in 1977 and 1978. These inflows are within the range of what to expect from the type of bedrock in the vicinity of the site.

A summary of permeability for the glacial and bedrock materials in the Seabrook area is listed in Table 2.4-23.

For information on groundwater fluctuations, direction of groundwater flow, gradients and velocities refer to Subsection 2.4.13.2.

Refer to Subsection 2.4.13.4 for a discussion of the groundwater monitoring program.

Since all safety-related structures are founded on sound bedrock or on engineered backfill extending to sound bedrock, there is no potential for subsidence.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

All seismic Category I structures are founded on sound bedrock or engineered backfill extending to sound bedrock, as described in Subsection 2.5.4.5. The seismic design of seismic Category I structures is discussed in Subsections 3.7(B).2 and 3.7(B).3.

For all seismic analyses the rock was treated as a fixed boundary, as described in Subsection 3.7(B).2.3. Therefore, no dynamic rock properties were required for the seismic analyses.

Four seismic Category I electrical manholes are founded on offsite borrow, with a maximum thickness of 18 ft below the base of manhole W33/34. Seismic amplification in the maximum thickness offsite borrow was analyzed using the lumped-mass and spring approach described in Subsection 3.7(B).2.4. During design, estimated values of shear wave velocity $c_s = 650$ ft/sec and shear modulus, G = 13,890 psi were used. Based on data in Richart et al., (1970), $c_s = 650$ ft/sec is a conservative (low) value for the offsite borrow. These values were assumed to be constant for the 18-ft thick layer of offsite borrow. The amplified accelerations were used for the structural analyses of walls of manholes on offsite borrow.

Subsequent to design, the shear modulus for offsite borrow beneath the manholes was backfigured from the results of the plate load test described in Subsection 2.5.4.5d. The Young's modulus for the cyclic portion of the plate loading, E = 24,800 psi, and the Poisson's ratio from drained triaxial tests, v = 0.3, were used to calculate a value of G = 9,550 psi. The average degree of compaction for the offsite borrow test fill was the same as for the offsite borrow plated during construction. Assuming that the 18-in. diameter plate influenced a 24- to 36- in. thick layer of soil beneath the plate, the average shear strain, γ, in the soil during the unload-reload cycle was $\gamma = 3.6\text{x}10^{-3}$ in./in. to $5.4\text{x}10^{-3}$ in./in. The value of shear modulus at low strain (10^{-6} in./in.), G_{max}, was then determined using the relationship between shear modulus and shear strain for sand presented in Seed and Idriss (1970). The average octahedral stress, $_m$ = 4,000 psf, in the zone beneath the plate was calculated using the elastic solutions for a rigid plate with an average load of 6 tsf. Values of G_{max} for other effective stress levels were than computed using the relation:

$$G_{2max} = G_{1max}\sqrt{\frac{\bar{\sigma}_{m2}}{\sigma_{m1}}}$$

where G_{1max} and $\bar{\sigma}_{m1}$ were values from the plate load test.

A plot of G_{max} vs $\bar{\sigma}_m$ for the offsite borrow, based on the plate load test data, is shown in Figure 2.5-71. As noted in Subsection 3.7(B).2.4, the seismic design of the manholes was checked using the shear modulus values backfigured from the plate load test, and was found to be satisfactory.

The seismic Category I electrical ductbanks which are founded on offsite borrow were analyzed using the procedures described in Subsection 3.7(B).2.4. The dynamic properties backfigured from the plate load test as described above were used for these analyses.

Electrical manhole W19/20 is founded on 15 ft of tunnel cuttings with a few layers of offsite borrow. Analysis of the amplification for this manhole was performed as described in Subsection 3.7(B).2.4, with an average shear modulus determined from the plate-load test on tunnel cuttings using the procedure described above.

The seismic Category I service water pipes are supported on offsite borrow generally 2 ft thick, but in certain areas up to a maximum of 15 ft thick, as described in Subsection 2.5.4.5c.3. All seismic Category I pipe is surrounded by offsite borrow except for a 180-ft length of trench near the Service Water Pumphouse where the pipe is surrounded by sand-cement as described in Subsection 2.5.4.5c.4. The thickness of cover over the pipes is from 12 to 24 ft except for the pipes between the Service Water Pumphouse and intake/discharge structures where the cover is up to 60 ft. The seismic stresses in these pipes were analyzed using the procedures of Iqbal and Goodling, as described in Subsection 3.7(B).3.12.

CLSM, as described in Subsection 2.5.4.5c.6, recently added to the list of engineered backfill materials that may be used. CLSM is a relatively new engineered backfill material mixture intended for restoration of excavations that has been tested and demonstrated to have many benefits compared to the traditional compacted granular backfill. Controlled laboratory testing and an Engineering Evaluation, by Altran Solutions of Boston, MA, of a specific CLSM design mix provides the basis for acceptance of the CLSM as an engineered backfill material for both safety and non-safety related applications. That evaluation, using the material properties of the CLSM engineered backfill (Table 2.5-22) and the procedures of Iqbal and Goodling, as described in Subsection 3.7(B).3.12 in a parametric study (i.e. varying pipe size and depth), demonstrate acceptable stress levels at bounding conditions for all design load combinations and concluded that the CLSM mix specified herein was an acceptable engineered backfill.

For the pipe surrounded by offsite borrow, the following design parameters were used in the analyses:

Unit Weight (buoyant)	$\gamma_b = 60$ pcf
Void ratio	$e = 0.4$
Poisson's ratio	$v = 0.4$
Coefficient of Lateral Pressure	$K_o = 0.5$
Coefficient of Subgrade Reaction	$k_o = 300$ lb./in.^{2}/in.
Coefficient of Friction (steel pipe to soil)	$\mu = 0.3$
Maximum soil particle velocity	
for OBE	$V_m = 6$ in./sec
for SSE	$V_m = 12$ in./sec
Shear wave velocity	$C_s = 770$ ft/sec
Seismic soil strain	
for OBE	$\epsilon_m = 0.000325$ in./in.
for SSE	$\epsilon_m = 0.000650$ in./in.

The values of unit weight, Poisson's ratio, void ratio, and coefficient of lateral earth pressure are conservative values, based on the results of field density measurements shown in Figure 2.5-52, Figure 2.5-53, Figure 2.5-54 and Figure 2.5-55 and triaxial tests shown in Table 2.5-16. The coefficient of subgrade reaction is a lower-bound value from Figure 5 in Appendix 2N, which is based on the results of triaxial compression tests. Since the lower-bound value of k_o is not necessarily conservative, a check analysis is being performed with higher values of k_o. The coefficient of friction between soil and pipe is conservative, based on the measured friction angles from triaxial tests and the reduction factors recommended in Iqbal and Goodling. The maximum soil particle velocity, shear wave velocity, and seismic soil strain were determined using the procedures described in Iqbal and Goodling.

The section of service water pipe surrounded by sand-cement was analyzed using an average shear modulus, G = 10,100 psi backfigured from the initial Young's modulus measured in the consolidated drained triaxial compression tests described in Appendix 2M. The average in situ octahedral effective stress in the 8-ft thick sand-cement layer (see Section J-J on Figure 2.5-50) is σ_m = 7.1 psi; therefore the modulus values are based on the three triaxial tests with σ_c = 7.1 psi. Poisson's ratio, v = 0.2, measured during the tests was used to convert Young's modulus to shear modulus. The stress-strain curves indicate essentially constant initial modulus for loading up to about 20 percent of the compressive strength. This is consistent with data in Dupas and Pecker (1979) which indicates that the shear modulus of sand-cement is significantly less affected by strain level than is an uncemented sand. Since the seismic stresses in the sand-cement are less than about 4 percent of the unconfined strength (see Subsection 2.5.4.8), the shear modulus for sand-cement was not varied with strain level.

An evaluation of the CLSM backfill was performed by Altran Solutions of Boston, MA for the purpose of comparing the compacted engineered backfill with the CLSM backfill with the objective of demonstrating that the CLSM backfill was not detrimental to buried piping systems from a pipe stress perspective. The buried Service Water pipes, having extensive sections of buried piping, were subjected to extensive and rigorous analysis. Altran Solutions used the Service Water calculation methodology and format as the basis of a parametric study to demonstrate that the use of CLSM as an engineered backfill on an as-desired basis is not detrimental to the buried pipes. A representative calculation was performed for each safety related buried piping system and pipe size using CLSM material properties in such a way to represent a bounding evaluation. The calculations performed do not represent a complete re-evaluation of the entire buried piping system, but does demonstrate the use of CLSM backfill is not detrimental to pipe stress.

The methodology for the CLSM evaluation first focused on identifying the buried safety related piping which identified a total of nineteen lines consisting of 5 unique pipe sizes of which the worst-case service configurations were used for the bounding analysis. Next, using the same analytical methodology of Iqbal and Goodling, as used for the original buried piping analyses and as described in Subsection 3.7(B).3.12 Altran Solutions conducted a parametric study, varying input parameters, to demonstrate acceptable stress levels for the various bounding configurations and conditions for all design load combinations. The analysis focused on the stresses at the buried pipe elbows using the Flexible Approach to determine deflections, moments, and shear loads, as in the original analysis.

The material properties used for evaluations were developed from laboratory testing of the CLSM. The evaluation used Young's Modulus values ranging from 8,250 psi to 81,900 psi enveloping the value of 24,800 psi used in the original service water buried piping analysis. The Coefficient of friction for steel is typically noted as 0.3, however the original service water buried piping analysis justified the use of a value of 0.5 as being more representative since the buried pipes are coated or wrapped providing increased frictional resistance compared to that of smooth uncoated steel, this value was also used for the CLSM evaluations. Maximum particle velocity is a function of ground acceleration during a seismic event and remains unchanged at 6 in/sec for OBE and SSE. Shear wave velocity of the CLSM was determined by testing to be 2310 ft/sec. The original buried pipe stress calculation used a shear wave velocity value multiplied by 2.0 or the compression wave velocity multiplied by 1.0. Since a lower shear wave velocity results in higher pipe stresses, the lower value of 2310 ft/sec from the lab test report was conservatively selected to bound the results. A review of the piping materials, design pressures, and temperature determined that using the pipe materials, design and operating pressures and temperatures would provide bounding conditions for the other buried systems. Based on the parametric calculations performed by Altran Solutions, comparing existing backfill material properties and resulting pipe stresses to the CLSM properties and resulting pipe stresses, it was concluded that the use of CLSM as an engineered backfill material will not be detrimental to the ability of buried piping to accommodate all specified design loadings.

2.5.4.8 Liquefaction Potential (Including Cyclic Mobility)

a. Definitions

There are two different phenomena associated with the behavior of saturated sands under cyclic loading, namely, liquefaction and cyclic mobility. Where the stability of soils under seismic loading is in question, these two phenomena should be considered separately. Their definitions are as follows:

1. Liquefaction is a phenomenon wherein a saturated sand loses a large percentage of its shear strength (due to increased pore pressure and reduced effective stress induced by monotonic or cyclic loading) and flows in a manner resembling a liquid until the shear stresses in the mass are less than the reduced shear strength.

2. Cyclic Mobility is the progressive softening (reduction in modulus) of a saturated sand when subjected to undrained cyclic loading. Cyclic mobility does not necessarily lead to a reduction in subsequent undrained shear strength, but may lead to excessive deformations.

b. Analysis

The foundation materials adjacent to or under all seismic Category I structures are of the following types:

1. Bedrock
2. Fill concrete extending to bedrock
3. Backfill concrete
4. Offsite Borrow
5. Tunnel Cuttings
6. Sand-cement
7. Controlled Low Strength Material (CLSM).

The bedrock at the plant site is primarily sound quartz diorite with occasional mica schist inclusions and diabase dikes. Properties of the bedrock are presented in Subsection 2.5.4.2a. The fill concrete has a minimum compressive strength of 3,000 psi and the backfill concrete has a minimum compressive strength of 2,000 psi, as discussed in Subsection 2.5.4.5. There is no potential for liquefaction or cyclic mobility of the bedrock, fill concrete or backfill concrete.

The properties of the offsite borrow and tunnel cuttings are presented in Subsection 2.5.4.5. The field control of engineered backfill placement, including results of measurements of in-place density and percent compaction, is discussed in Subsection 2.5.4.5.

The offsite borrow, which is classified as SW (Unified Soil Classification System) and compacted to at least 95 percent of maximum dry density determined by ASTM D1557, is not susceptible to liquefaction as defined above. This conclusion is based on the results of the triaxial tests presented in Appendix 2M, which indicate that both drained and undrained tests on specimens at 95 percent compaction are dilative during shear. Thus a pore pressure increase cannot be sustained during shear, and liquefaction is not possible.

The resistance of the offsite borrow to cyclic deformations (cyclic mobility) was not measured during the laboratory or field testing programs, since the extensive body of published literature of cyclic testing indicates that specimens of widely graded sand or sand and gravel compacted to at least 95 percent of maximum dry density develop less than ±2.5 percent strain when subjected to 5 to 10 cycles of stress equivalent to an 0.25g earthquake. A testing program on soils with gradations similar to the offsite borrow and with compaction at a relative density of 80 percent, which was equivalent to 95 percent compaction, is reported in the Soils Report of the Pilgrim Unit 2 PSAR (1977). On the basis of these data, it was concluded that the offsite borrow was adequately resistant to cyclic deformation under the SSE loading.

The tunnel cuttings described in Subsection 2.5.4.5c.5 are composed of a widely graded angular crushed quartzite compacted to 95 percent of maximum dry density measured by ASTM D1557. Although not tested in the laboratory, the tunnel cuttings are considered to be more dilative and hence more resistant to liquefaction than the offsite borrow, due to the significantly coarser particle size and higher in-place density of the tunnel cuttings. The dilative behavior of dense Oroville gravel, with gradation similar to the tunnel cuttings but with rounded to subrounded particles, has been demonstrated by large scale triaxial tests reported in Banerjee, Seed and Chan (1979). Cyclic testing on the Oroville gravel reported in Wong, Seed and Chan (1974) indicated that the cyclic resistance of compacted materials increases substantially with increasing particle size. Therefore, the tunnel cuttings are considered to be more resistant to cyclic deformation (cyclic mobility) than the offsite borrow. In addition, the moduli measured in the repeated loading portions of the plate load tests described in Subsection 2.5.4.5d were significantly higher for the tunnel cuttings than for the offsite borrow, from which one can infer higher resistance to cyclic loading. Thus, it was concluded that the tunnel cuttings are more resistant to both liquefaction and cyclic deformations than the offsite borrow.

The sand-cement, described in Subsection 2.5.4.5c.4, was used as backfill adjacent to and above one 180-ft long section of service water pipe. Since the strength of sand-cement is derived primarily from cementation at grain contacts, and not from interparticle friction, loss of strength due to buildup of pore pressure, i.e., liquefaction, is not possible. The resistance of sand-cement to cyclic deformations was not measured in the laboratory since the in situ cyclic shear stress induced by the SSE in the sand-cement is less than 4 percent of the specified minimum compressive strength of 100 psi at 28 days, and less than 2 percent of the minimum compressive strength measured on field test cylinders at 90 days, as shown in Figure 2.5-58. Cyclic stresses of this magnitude would not be sufficient to break the cementation bonds, hence no significant cyclic deformations could occur. There are no known subsurface conditions at the site which would lead to future loss of strength in the sand-cement. It is likely that the strength will increase with time, since the strength increased with time in the laboratory. Thus, it was concluded that the sand-cement is adequately resistant to both liquefaction and cyclic deformations.

The Controlled Low Strength Material (CLSM), described in Section 2.5.4.5c.6, is a flowable, cementitious, self-compacting, fine aggregate material designed to be an excavatable engineered backfill with physical properties and attributes making it preferable to traditional compacted granular backfills. The CLSM hardens to 100 percent compaction and its shear strength is derived from the cementation of the aggregate at grain contacts, similar to a sand-cement mixture, and not due to inter-particle frictional forces that can be lost or reduced with increased pore pressure as with compacted backfills and therefore, liquefaction is not possible with CLSM backfill. The resistance of CLSM to cyclic deformations was not tested, as with the sand-cement backfill, since the cyclic shear stress induced by the SSE of less than 4 psi is significantly below the certified minimum 28-day unconfined compressive strength of 50 psi. Cyclic stresses of this magnitude are not sufficient to break the cementation bonds and therefore no significant cyclic deformations could occur. There are no known subsurface conditions which would lead to future loss of strength in the CLSM backfill. Long term strength tests indicate that the CLSM has a modest increase in strength over time but the rate of increase drops off to reach a maximum strength as designed to maintain the excavatability property which increases the margin against liquefaction and seismic cyclic deformation. It is concluded and demonstrated by a rigorous parametric evaluation that the CLSM design mix specified herein will perform its design function as intended without any detrimental impact to the piping, and will be adequately resistant to both liquefaction and cyclic deformations.

2.5.4.9 Earthquake Design Basis

The evaluation of the maximum earthquake potential is presented in Subsection 2.5.2.4. Based on this analysis, the maximum earthquake potential consists of an Intensity VIII (MM) earthquake. For this earthquake, the following peak accelerations have been derived in Subsections 2.5.2.6 and 2.5.2.7.

Design Earthquake	Horizontal Peak Acceleration, g's	Vertical Peak Acceleration, g's
OBE	0.125	0.083
SSE	0.25	0.167

The horizontal and vertical design spectra for the SSE are shown on Figure 2.5-43 and Figure 2.5-44.

The seismic response of seismic Category I service water pipelines, electrical ductbanks and electrical manholes founded on offsite borrow, Controlled Low Strength Material (CLSM), or tunnel cuttings was analyzed using a peak horizontal acceleration of 0.25 g.

2.5.4.10 Static Stability

All seismic Category I structures are founded on sound bedrock or on engineered backfill extending to sound bedrock. As shown in Table 2.5-20, fill concrete was used as the engineered backfill beneath all seismic Category I structures except for safety-related electrical duct banks, five electrical manholes, and the service water pipes, which were founded on offsite borrow or tunnel cuttings.

a. Bearing Capacity and Static Settlement

Navdocks DM-7 (1963) was used to estimate the allowable bearing pressure for structures founded on bedrock. In Table 11-1 of Navdocks DM-7 (1963), an allowable bearing pressures for hard crystalline rocks of 80 tsf is recommended. (Note: The allowable bearing pressure is the pressure that can be applied in the field. The ultimate bearing capacity is 6 to 10 times higher than the allowable value).

An alternate technique for estimating allowable bearing pressure on rock is to multiply the unconfined compressive strength by 0.2 to 0.3 to adjust for the presence of rock defects, as suggested by Bowles (1977, p. 143). For the rock at this site, the lowest measured unconfined compressive strength in the zone of interest was 5970 psi (Table 2G-1). Using the factor 0.2, Bowles' approach gives a value of 86 tsf for the allowable bearing pressure. This value is similar to that recommended in Navdocks DM-7.

Some structures are founded on fill concrete, which has a 90-day unconfined compressive strength of 5400 psi (Figure 2.5-59).

Following Bowles' (1977, p. 143) suggestion to use no more than the unconfined compressive strength of the concrete as a working compressive strength of the rock, and using the factor of 0.2, the allowable bearing pressure on the rock is calculated to be 78 tsf. Thus, an allowable bearing pressure of 80 tsf is suitable for foundations on rock, with or without fill concrete between the two. To be conservative, an allowable bearing pressure of 60 tsf was used.

The actual bearing pressure beneath the major seismic Category I buildings is shown on Table 2.5-20. The foundation type and dimensions are presented in Table 3.8-15. The most highly loaded foundation is the ring wall around the Containment Enclosure Buildings which has a maximum bearing pressure of 36 tsf.

The maximum estimated settlement for any seismic Category I structure is 0.5 in. for the combined loading of the Containment structure and containment enclosure structure. Of this settlement, approximately 0.25 in. represents recompression of the heave resulting from the excavation, as described in Subsection 2.5.4.5b. The differential movement between these two structures is estimated to be less than 0.1 in. The maximum settlement was estimated using the relationship:

$$\delta = \frac{2\,qr\,(1 - v^2)}{E}$$

where δ = total settlement at center of foundation, in.

E = average modulus of elasticity, psi

q = foundation bearing pressure, psi

r = radius of loaded areas, in.

v = Poisson's ratio

An average modulus $E = 1 \times 10^6$ psi, corrected for RQD as described in Subsection 2.5.4.5b, was used for the analysis. The weighted average loading for the combined Containment structure and containment enclosure structure was $q = 17.2$ tsf over a radius of 86.5 ft. The value of Poisson's ratio, $v = 0.2$, used for this analysis was conservative, based on the compression test data in Table 2.5-12.

The estimated settlement will occur as elastic compression during construction as the load is added. No significant post construction settlements or differential settlements for foundations on rock or fill concrete are anticipated.

For the manholes supported on offsite borrow or tunnel cuttings, an allowable bearing pressure of 2.5 tsf was established. The minimum base size for the manholes is about 18 ft by 18.5 ft, with a minimum embedment of 9.5 ft. The ultimate bearing capacity was calculated using the Terzaghi bearing capacity formula

$$q_u = 1/2B\gamma N_\gamma + \gamma D_f N_q$$

Using a submerged unit weight, $\gamma_b = 72$ pcf and bearing capacity factors $N_\gamma = 50$ and $N_q = 40$ for a minimum friction angle $\varphi = 36°$ as determined from triaxial tests, the ultimate bearing capacity for the offsite borrow is $q_u = 30$ tsf. For a submerged unit weight $\gamma_b = 97$ pcf and bearing capacity factors $N_\gamma = 70$ and $N_q = 70$ for an assumed $\varphi = 40°$, the ultimate bearing capacity of the tunnel cuttings is $q_u = 63$ tsf. Thus, the allowable bearing pressure provides factors of safety of 12 and 25 against ultimate bearing capacity failure for the offsite borrow and tunnel cuttings, respectively.

The maximum bearing pressure beneath the base of the manholes is 1.4 tsf, assuming the water table is below the bottom of the manhole. Using the elastic settlement formula described above, with E = 10,500 psi from the plate load test data and $v = 0.3$ from the triaxial test data, the maximum settlement for manholes on the offsite borrow is $\delta = 0.5$ in. For the tunnel cuttings, with E = 24,000 psi from the plate load test data and estimated $v = 0.3$, the maximum settlement of the one manhole on tunnel cuttings is $\delta = 0.2$ in.

The estimated settlement will occur during the construction of the manholes and as backfill is placed around the manholes. No significant post construction or differential settlements of the manholes founded on offsite borrow or tunnel cuttings is expected, unless a seismic event occurs, which is covered in Subsection 2.5.4.10b.

Bearing capacity of CLSM and any backfill is directly related to such factors as the physical properties of the material and more importantly depth, ground water level, dimensions of the footing or bearing plate, and any potential live load surcharge in addition to the expected deadweight. The bearing capacity of the CLSM determination is presented below in a manner to allow relative comparative evaluation using the configuration of the safety related electrical manhole evaluated above.

The bearing capacity of CLSM is determined following the same methodology used for the compacted offsite borrow using Terzaghi's Bearing Capacity Equation:

$$q_{ult} = \frac{Q''_{ult}}{B} = \frac{1}{2}\gamma_1 B N_\gamma + \left(q_2 + D_f \gamma_2\right) N_q + c N_c$$

Where the bearing capacity factors; N…, Nq, and Nc are solely based on the friction angle (Φ or phi) of the bearing material and are taken from standardized tables or figures available in geotechnical engineering publications. B = width of footing, N = depth of footing, and … is conservatively taken as the buoyant unit weight as the Seabrook design ground water elevation is at grade.

The friction angle (Φ or phi) of the CLSM (Table 2.5-22) was determined from laboratory triaxial testing to be 25.7 degrees with a cohesion intercept value of 14.6 psi or 2100 psf. The bearing capacity factors for the CLSM using the factor chart are found to be; N… = 10, Nq = 13, and Nc = 26. When the friction angle is less than 28 degrees, such as with CLSM, a reduction factor of 2/3 is applied to the cohesion intercept.

Solving for the ultimate bearing capacity of CLSM yields:

q_{ult} = (.5)(41pcf)(14ft)(10) + [100psf + (9.5ft)(41pcf)](13) +(2/3)(2100psf)(26) = 45,600 psf or 22.8 tsf

With an Allowable Bearing Capacity of CLSM set at 2.5 tsf to match that of the Compacted Offsite Borrow Engineered Backfill yields a Factor of Safety in excess of 9.0 against CLSM backfill collapse.

CLSM placed at excavations around or under piping and manholes self-levels and self-compacts to 100 percent density as it hardens into a solid mass and as such does not exhibit post installation settlement as compacted granular backfill material.

b. Settlement Due to Seismic Loading

The settlement resulting from the SSE loading was also estimated for the seismic Category I structures founded on offsite borrow or tunnel cuttings, using the relationship between horizontal cyclic shear strain in the soil during the earthquake and accumulated vertical strain described in Silver and Seed (1971) and Seed and Silver (1972). The peak horizontal cyclic shear strains were determined for the thickest layers of offsite borrow and tunnel cuttings below seismic Category I structures (see Subsection 2.5.4.5c) using the one-dimensional computer program SHAKE (Schnabel et al., 1972) with 3 to 7 soil layers below the structure. The SHAKE analyses were performed using the lower values of shear moduli from the plate load tests (see Subsection 2.5.4.7) and the shear modulus reduction curve and damping values from Seed and Idriss (1970). Using the Seed and Silver (1971) data for relative density, D_r = 80 percent and 10 cycles of loading, the maximum seismic settlement for seismic Category I structures is 0.1 in. for the service water pipes located in the area with 15 ft (maximum thickness) of offsite borrow beneath the pipes. Seismic settlements of this magnitude will not affect the performance of the seismic Category I manholes, ductbanks or service water pipes during or after the SSE event.

CLSM placed at excavations around or under piping and manholes self-levels and self-compacts to 100 percent density as it hardens into a solid mass. The shear strength of CLSM is primarily due to the inter-particle cementation and not intra-particle interlocking, even though the fine aggregate of the CLSM does become interlocked. The stresses induced by seismic loading are well below the shear strength of the CLSM and as such the CLSM mass will not be subjected to seismic differential displacements and consolidation (i.e. settlement) of granular particles as with a compacted granular material.

c. Static and Dynamic Lateral Pressures

Lateral earth pressures for Category I structures surrounded by offsite borrow were computed for both static and seismic conditions using the pressure diagrams shown in Figure 2.5-66 and Figure 2.5-67. The static coefficients of at-rest earth pressure, K_o, and active earth pressure, K_a, are conservative values, based on the minimum friction angle of 36° measured in triaxial tests. Static water pressures were computed using the maximum groundwater elevation at the ground surface, El +20. For the rigid wall, a static lateral compaction pressure was included for the full height of the wall.

The dynamic lateral pressure coefficient, K_h, for nonrigid walls was calculated using the procedures described in Seed and Whitman (1970). As discussed in Seed and Whitman, the effect of vertical acceleration on the dynamic lateral pressure for nonrigid walls is negligible for the case where vertical acceleration is one half the horizontal acceleration.

The magnitude and distribution of dynamic lateral pressure of rigid walls was based on discussions with Dr. H.B. Seed (Dalal, 1975). Based on his experience Dr. Seed recommended that an approximately uniform pressure distribution would be appropriate for rigid box-type structures founded on rock and surrounded by soil. The lateral pressure would be one half the maximum pressure from the Seed and Whitman (1970) method, increased by an empirical factor of 3 to account for the difference in stiffness between the structure on rock and the surrounding soil. Thus, the coefficient of dynamic earth pressure, K_D, is

$$\begin{aligned} K_D &= \tfrac{1}{2} \times 3 \times \Delta K_{AE} \\ &= \tfrac{1}{2} \times 3 \times 3/4 \times a_{max} \\ &= 1.125\ a_{max} \end{aligned}$$

Where $\Delta K_{AE} = 3/4\ a_{max}$ is the dynamic component of lateral earth pressure from Seed and Whitman (1970). The uniform pressure distribution is conservative, since the actual dynamic component of lateral pressure must go to zero at the base of the wall, where there is no relative dynamic motion between the structure and the surrounding soil. Near the very top of the wall, the horizontal earthquake pressure is limited to be equal to the passive resistance of the soil. The rigid walls of all seismic Category I buildings, except five manholes, were founded on sound bedrock or fill concrete extending to sound bedrock. For these walls, the bedrock accelerations were used to compute dynamic lateral pressures. For the five manholes founded on offsite borrow or tunnel cuttings, the amplified soil accelerations at the base of the manholes described in Subsection 2.5.4.7 were used for design.

The maximum lateral pressures for any seismic Category I structure occur at the east wall of the Service Water Pumphouse and the west wall of the discharge transition structure, where the thickness of the offsite borrow is 63 ft. These are rigid walls with the following lateral pressures at the base:

Static At-rest Soil Pressure	1,970 psf
Hydrostatic Pressure	3,930 psf
Permanent Surcharge	0
Live Load Surcharge	250 psf
Dynamic Soil Pressure (SSE)	2,210 psf

Using the above values, the total horizontal earth load, excluding the live load and the hydrostatic water pressure, is calculated to be 190 k/ft. The at-rest earth load alone is 62 k/ft. Hence, the total earth load during an earthquake is 3.1 times the at-rest ($K_o = 0.5$) earth pressure. The hydrostatic water and the live load surcharge effects are added to the earthquake load for design of the walls. The effects of compaction need not be included during an earthquake since the shaking dissipates compaction pre-stress effects.

Tunnel cuttings were not placed against nor within 10 ft horizontal distance of any seismic Category I building wall. Therefore, analyses of lateral loads due to tunnel cuttings were not required.

For cases where CLSM is used against a foundation or wall of a Category I structure consideration of the effect of static and dynamic lateral pressures is presented by comparison of the static and dynamic lateral earth pressures for the offsite borrow compacted backfill, above.

Foundations walls are rigid structures that do not yield or tilt in or out as would a retaining wall, and as such "Active Earth Pressure (Ka)" from a sliding wedge of soil cannot develop. For this evaluation the pressure induced to the building foundation wall by backfill material is due to the static lateral pressure exerted by the deadweight of backfill material "at-rest". The amount of "at-rest lateral pressure" exerted onto a building foundation by a backfilled material is determined much like the hydrostatic pressure caused by a liquid against a retaining wall. Unlike liquids, backfill materials have inherent shear strength and are self-supportive to an extent, which there will be some lateral pressure induced against its retaining or confining structure(s). The amount of static lateral pressure induced by the backfill material is dependent on the material properties of the backfill and is derived once the "At-Rest Earth Pressure Coefficient" for that backfill material is determined, as follows:

K_0 = At-Rest Earth Pressure Coefficient

K_{0g} = [1 - sin(PHI)] for granular soils, PHI = effective angle of internal friction

K_{0c} = [Nu/(1 - Nu)] for cohesive soils, Nu = Poisson's ratio

For the compacted backfill, K_{0g} = (1-sin36) = 0.412

For CLSM, K_{0c} = (0.34)/(1-0.34) = 0.52 (worst case)

P_h = Lateral Pressure caused by backfill

P_h = (P_v)(K_0)

P_v = Vertical Stress at depth z = (gamma-sat)(z), gamma-sat is saturated density, z = depth

Thus, P_h = (gamma-sat)(z) (K_0) for the fully saturated depth of backfill

Applying the At-Rest Earth Pressure Coefficients yields;

P_h = (60)(63) (0.412) = 1557 psf due to compacted offsite borrow

P_h = (41)(63)(0.52) = 1343 ps due to CLSM backfill

Given that the other contributing factors to lateral pressure remain unchanged (i.e. surcharge and hydrostatic pressure), by comparison the CLSM induces less lateral static pressure than the compacted backfill at the worst-case depth. Also, by comparison and having all other contributing factors remain unchanged, the lower density of the CLSM (both dry and saturated) compared to compacted backfill, any dynamic pressures induced on the retaining structure or foundation wall during a seismic event will be proportionally less than the lateral dynamic pressures induced by a compacted backfill previously evaluated to be acceptable given the reduced mass.

Therefore, by comparison, static or dynamic pressures induced by a CLSM engineered backfill are bounded by the previous analysis for compacted granular backfill, and as such does not create any negative impact when placed against the walls or foundations of the Category I structures.

2.5.4.11 Design Criteria

Since all seismic Category I structures are founded on rock or on engineered backfill of shallow depth over rock, there is no need for stability studies of subsurface materials.

2.5.4.12 Techniques to Improve Subsurface Conditions

The bedrock at the site consists of Newburyport quartz diorite, Kittery metasediments, and a number of diabase dikes. (Reference Subsection 2.5.1). All of this rock is hard and strong, with compressive strengths between 10,000 and 35,000 psi. The site engineering geologist made documented inspections of all bearing surfaces. Based on these inspections, all severely weathered and very closely jointed rock was removed from all areas, including those where it might affect structure stability, foundation bearing capacity or wall loadings. This resulted in overexcavation in some areas; see Subsections 2.5.4.14 and 2.5.1.2b.6.

Only sound, fresh rock was utilized for the support of foundations of safety-related structures. Thus, no improvements of subsurface conditions were required.

2.5.4.13 Subsurface Instrumentation

No subsurface instrumentation was required for foundations of seismic Category I structures.

2.5.4.14 Construction Notes

No construction problems which would adversely affect the safety of the plant were encountered. Some procedures and design details, however, were adopted as a result of conditions which developed during construction and subsequent maintenance activities. These are discussed below.

a. Dewatering

The amount of water which had to be pumped was relatively small, approximately 15 gpm maximum per building excavation. This water in the excavation resulted from runoff and groundwater seepage along fractures in bedrock of the excavation. Direct pumping from open sumps strategically placed handled all dewatering of the excavation, and no other techniques, such as well points or slurry walls, were required.

b. Excavation Techniques

The overburden was removed by conventional means, and the bedrock surface exposed. Rock excavation was done using controlled blasting techniques such as pre-splitting. Blast monitoring was continuously maintained to assure that no damage was induced to adjacent structures; blast monitoring was also useful in minimizing damage to adjacent rock.

Evaluation of the stability of all excavated slopes during construction was accomplished by the following procedures:

1. Close supervision of the results of controlled blasting, blast monitoring, and the blasting scheme
2. Thorough scaling and cleaning of the walls and continuing surveillance of their stability during construction
3. Removing of weathered, fractured and jointed zones of rock which have questionable stability.

No material was left above the level of the foundation mat which could induce a seismic or static load on a wall, unless the wall had been designed for that load. No material remained below the level of the foundation mat which could affect the stability or bearing capacity of the foundation.

In addition, to provide increased safety for the workmen in the excavations, the following procedures were employed:

1. Placing, as required, of 8- to 16- foot long rock bolts which were fully encapsulated with resin
2. Placing, in conjunction with the rock bolts, of chain link fence and steel straps to prevent smaller pieces of rock from falling into the excavations.

These measures were for construction safety purposes only and were completely supplanted by fill concrete and backfill concrete placed against these surfaces.

c. <u>Overbreak and Overexcavation</u>

Overbreak and overexcavation resulted from

1. Occasional overblasting
2. Adversely oriented or locally very closely spaced rock jointing, which imparted a blocky fabric characteristic to the rock.

Zones of overexcavation or overbreak were filled with fill concrete to the underside of the structures, assuring that all loads were carried to sound bedrock. Backfill concrete was used to fill zones of overbreak on the sides of excavations in bedrock.

d. <u>Fill and Backfill Concrete</u>

Fill concrete and backfill concrete are used under and around structures to provide for the transfer of loads to the bedrock, either because bedrock is at a lower than anticipated elevation or because excavation was continued to a lower elevation in order to obtain stable bedrock.

Except for electrical manholes, electrical ductbanks and service water pipes indicated in Table 2.5-20, all safety-related structures are founded on sound bedrock or on fill concrete extending to sound bedrock. Fill concrete was subjected to QA inspection during placement. A minimum compressive strength of 3,000 psi at 28 days was attained for the fill concrete; for backfill concrete this Figure is 2,000 psi.

e. Use of Tunnel Cuttings

Tunnel cuttings were used as engineered backfill beneath electrical manhole W19/20. The tunnel cuttings were evaluated, tested, and accepted for use as engineered backfill as described in Subsections 2.5.4.5c and 2.5.4.5d.

f. Use of Sand-Cement

Sand-cement was used as engineered backfill in only one safety-related area, as backfill adjacent to and above the service water pipes in a trench excavated in rock, from N9774, E6250 to N9774, E6430. In this location, the engineered backfill from the top of sound bedrock to the pipe invert was offsite borrow. The sand-cement extended from the invert to a level about 6 ft above the top of the pipes. Properties of the sand-cement are described in Subsection 2.5.4.5c.

g. Use of Controlled Low Strength Material (CLSM)

CLSM is used in lieu of traditional compacted granular backfill material at multiple locations around the site for restoration of excavations in support of maintenance or inspection activities. CLSM is a relatively new engineered backfill material mixture which is significantly safer to install than traditional granular backfills, more efficient and cost effective, while providing the required support and protective properties as described in Subsection 2.5.4.5c.6 for the buried utilities.

2.5.5 Stability of Slopes

There are no offsite natural or man-made slopes (cut or fill), the failure of which could adversely affect the safety of the plant.

The only onsite slope requiring analysis is the stone revetment for the protection of the site during peak PMH surge activity as described in Subsection 2.4.5.5, Protective Structures. The stone revetment is not a Category I structure.

2.5.5.1 Slope Characteristics (Revetment

The stone revetment, located as shown in Figure 2.5-47, has a maximum height of 19 ft, of which 6 ft are buried below the ground surface. Thus, the maximum net revetment height is 13 ft. Subsequent references to revetment height are in terms of net height above finish grade at the toe. Although no special exploration programs were performed for the revetment, the general site borings described in Subsection 2.5.4.3 indicate the natural soils at the revetment locations are 1 to 2 ft of topsoil underlain by dense glacial till extending to bedrock. The glacial till is described in Subsection 2.5.4.2b. The depth to bedrock varies from 0 to approximately 40 ft beneath the revetment, as determined from the contours in Figure 2.5-15 and Figure 2.5-17. The topsoil was stripped and the revetments are founded on bedrock or on glacial till. The fill behind the revetment consists of offsite borrow compacted to at least 90 percent of maximum dry density determined by ASTM D1557, except in the area near the railroad tracks, identified on Figure 2.4-55, where the offsite borrow was compacted to at least 95 percent of maximum dry density. The fill materials are described in Subsection 2.5.4.5c.

2.5.5.2 Design Criteria and Analysis of Revetment

Wave design of the revetment is described in Subsection 2.4.5.5. Seismic analysis of the revetment was performed to determine the deformations during an SSE event. The analysis was based on a time history of acceleration and seismic shear stress from the 2-dimensional finite element program FLUSH (Lysmer et al., 1975) followed by a computer integration of rigid-body displacements using a Newmark type analysis for a wedge failure surface. Two cross sections of revetment were considered in the analysis:

a. Section Q-Q - Revetment A, thickest underlying soil (about 40 ft) with revetement height of 10 ft

b. Section R-R - Revetment A, highest section (13 ft)

The locations of these sections are shown on Figure 2.5-47. The soil profiles and finite element grids for these sections are shown in Figure 2.5-68 and Figure 2.5-61.

The horizontal earthquake motion input at the bedrock surface was modeled using the Housner artificial record, scaled to match the design SSE spectrum for 5 percent damping shown in Figure 2.5-43 for the range of frequencies of 2 to 8 Hz, which brackets the natural frequencies of all three soil/revetment sections. An upper cutoff frequency of 15 Hz was used in the analysis. The duration of the design earthquake motion was 20 seconds.

Soil properties used for the analysis were conservative values based primarily on published data in the literature. The properties are summarized in Table 2.5-21. The shear modulus at low strain (less than 10^{-6} in./in.) for each element was determined using the relation:

$$G_{max} = 1000\ K_2\ (\bar{\sigma}_m)^{1/2}$$

where

G_{max} = shear modulus at 10^{-6} in./in. shear strain (psf)

K_2 = shear modulus parameter, constant for a given soil type, density and strain level

m = octahedral effective stress (psf)

The K_2 value for the revetment stone was based on the average value for the rockfill shell in Oroville dam, (California Department of Water Resources, 1979) determined from cyclic triaxial data and from actual performance of the embankment during the 1975 Oroville earthquake. The K_2 value for the glacial till was based on values for the deep alluvium at San Fernando Dams, reported in Seed et al., (1973) and on in situ measurements of shear wave velocity for a similar till in Boston (GEI, 1976). The K_2 value for the offsite borrow was selected to represent the material with 90 percent compaction using data from Seed and Idriss (1970). Values of unit weight and Poisson's ratio for the offsite borrow were based on Table 2.5-16. For the rockfill and glacial till, values of unit weight and Poisson's ratio were estimated based on typical values in the literature. A damping ratio of 0.5 percent at low strain was used for each of the soil types. The variation in shear modulus and damping with strain level were based on the curves presented in Seed and Idriss (1970). For these analyses, the water level in the revetment and fill behind the revetment was assumed to be at El +14.5 MSL.

For evaluation of displacements, five trial wedges were selected through each revetment, as indicated on Figure 2.5-70. For each wedge, the horizontal yield acceleration required to reduce to factor of safety of the wedge to 1.0 was computed using a pseudo-static wedge analysis (U.S. Army Corps of Engineers, 1970). The friction angles of the various materials which were used to compute the yield accelerations are shown in Table 2.5-21. The friction angle for the offsite borrow was based on triaxial test data presented in Table 2.5-16. Values of friction angle for the revetment stone and glacial till were estimated based on data in Marsal (1972) and GEI (1981), respectively. The friction angle between the filter fabric (Polyfilter X) and the adjacent soil was estimated based on the data presented in Haliburton et al., (1978).

The time history of average earthquake acceleration for a given wedge was then compared to the yield acceleration for that wedge. Whenever the wedge acceleration exceeded the yield acceleration, horizontal displacement was assumed to occur. The total horizontal displacement was determined by accumulating displacements through the duration of the earthquake. Settlement was computed by assuming that the computed horizontal displacement represented the horizontal component of downslope crest movement along the back side of the wedge, as shown in Figure 2.5-70. The assumed displacements at the base of the wedge are also shown on Figure 2.5-70.

The analyses indicate that the largest overall crest settlement for Revetment A resulting from the SSE event will be about 0.5 ft for Wedge 1 at Section R-R. The analyses also indicate that the cap-stone at Revetment A (Wedge 3) may slump an additional 0.5 to 1.5 ft, resulting in a total settlement of 1.0 to 2.0 ft for the capstone. Based on the hydrologic and wave runup analyses described in Subsection 2.4.5.5, the settlements at Revetment A, B or C resulting from the SSE event would not significantly affect the performance of the revetment.

The static stability of the highest section of the revetment, was also analyzed using the wedge analysis described by the U.S. Army Corps of Engineers (1970). The wedges analyzed were those shown on Figure 2.5-71, plus a combined wedge consisting of the upper portion of Wedge 3 and the lower portion of Wedge 4. The properties used in the analysis were as given in Table 2.5-21. The minimum static factor of safety, $F_s = 1.51$, calculated for Wedge 4 is satisfactorily for permanent slopes, based on the criteria given in U.S. Army Corps of Engineers (1970). This minimum factor of safety is considered to be a very conservative value due to the very conservative friction angle ($\varphi = 36°$) used for the revetment stone. Using a best estimate of friction angle at low confining pressure, $\varphi = 46°$ based on data in Marsal (1972), the minimum static factor of safety is $F_s = 2.15$.

2.5.5.3 Logs of Borings

The general site exploration programs and boring logs are described and referenced in Subsection 2.5.4.3.

2.5.5.4 Compacted Fill

Compacted fill behind the revetment is described in Subsection 2.5.5.1 and properties of the fill materials are presented in Subsection 2.5.4.5c.

2.5.6 Embankments and Dams

There are no offsite embankments or dams the failure of which could adversely effect the safety-related facilities at the Seabrook site.

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SEABROOK STATION
UPDATED FINAL SAFETY ANALYSIS REPORT

CHAPTER 2
SITE CHARACTERISTICS

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TABLE 2.1-1 **POPULATIONS OF MUNICIPALITIES WHOLLY OR PARTIALLY WITHIN 10 MILES OF THE SITE**

	1970(1)	1980(2)	1983(4)
New Hampshire			
Brentwood	1,468	2,170	2,668
East Kingston	838	1,190	1,376
Exeter	8,892	10,720	11,230
Greenland	1,784	2,210	2,564
Hampton	8,011	10,820	12,278
Hampton Falls	1,254	1,500	1,602
Kensington	1,044	1,350	1,518
Kingston	2,882	4,640	5,018
Newfields	843	1,000	1,060
Newton	1,920	4,060	4,678
North Hampton	3,259	4,910	5,888
Portsmouth	25,717	28,430	28,580
Rye	4,083	5,230	6,034
Seabrook	3,053	6,000	6,672
South Hampton	558	800	920
Stratham	1,512	2,500	3,040
Massachusetts			
Amesbury	11,388	16,560(3)	17,000
Haverhill	46,120	46,340	47,300
Merrimac	4,245	4,710	4,800
Newbury	3,804	4,920	5,010

	1970[(1)]	1980[(2)]	1983[(4)]
Newburyport	15,807	16,740	17,000
Salisbury	4,179	5,150	5,250
West Newbury	2,254	2,690	2,750

(1) U. S. Census of Population, 1970

(2) Interim Revisions, New Hampshire Population Projections for Towns and Cities to the Year 2000. August 1977. NH Office of Comprehensive Planning. Projected 1980 populations for East Kingston, Exeter, Seabrook, and Stratham are less than 1978 population estimates for the same communities, Rockingham and Stratford County Population Data: 1978 Estimates - Rockingham Stratford Census Project. This is also noted for these same communities and Portsmouth in the 1978 Population Estimates of N. H. Cities and Towns, prepared by the NH Office of Comprehensive Planning, August 1979.

(3) Population Projections 1980-1985, Massachusetts Department of Public Health, Office of State Health Planning, August 1978.

(4) Estimates based on same sources indicated in footnotes (2) and (3) and interpolated for 1983.

TABLE 2.1-2 PROJECTED POPULATION BY SECTOR – 0 TO 10 MILES

Sector	Year	0-1 Mile	1-2 Miles	2-3 Miles	3-4 Miles	4-5 Miles	5-10 Miles	Cumulative Totals By Sector
N	1980	20	80	470	700	700	4,440	6,110
	1983	20	80	530	800	470	5,190	7,090
	1990	20	100	700	1,050	630	6,920	9,420
	2000	20	100	760	1,150	760	9,420	12,210
	2010	20	100	840	1,270	970	13,010	16,210
	2020	20	100	920	1,400	1,220	18,210	21,870
	2025	20	100	970	1,470	1,380	22,010	25,950
NNE	1980	0	0	1,700	1,980	370	8,180	12,230
	1983	0	0	1,930	2,250	430	8,820	13,430
	1990	0	0	2,540	2,960	580	10,300	16,380
	2000	0	0	2,780	3,250	720	12,260	19,010
	2010	0	0	3,060	3,570	910	14,830	22,370
	2020	0	0	3,370	3,930	1,150	18,200	26,650
	2025	0	0	3,540	4,120	1,310	20,410	29,380
NE	1980	0	70	790	1,350	820	980	4,010
	1983	0	70	900	1,540	940	1,140	4,590
	1990	0	100	1,180	2,020	1,240	1,520	6,060
	2000	0	110	1,290	2,220	1,400	2,000	7,020
	2010	0	120	1,420	2,440	1,590	2,640	8,210
	2020	0	130	1,570	2,680	1,800	3,480	9,660
	2025	0	140	1,640	2,820	1,940	4,040	10,580

Sector	Year	0-1 Mile	1-2 Miles	2-3 Miles	3-4 Miles	4-5 Miles	5-10 Miles	Cumulative Totals By Sector
ENE	1980	0	440	820	110	0	0	1,370
	1983	0	500	930	120	0	0	1,550
	1990	0	670	1,230	160	0	0	2,060
	2000	0	730	1,350	180	0	0	2,260
	2010	0	800	1,480	200	0	0	2,480
	2020	0	880	1,630	220	0	0	2,730
	2025	0	920	1,710	230	0	0	2,860
E	1980	0	480	0	0	0	0	480
	1983	0	540	0	0	0	0	540
	1990	0	710	0	0	0	0	710
	2000	0	780	0	0	0	0	780
	2010	0	860	0	0	0	0	860
	2020	0	940	0	0	0	0	940
	2025	0	990	0	0	0	0	990
ESE	1980	0	930	0	0	0	0	930
	1983	0	1,040	0	0	0	0	1,040
	1990	0	1,290	0	0	0	0	1,290
	2000	0	1,530	0	0	0	0	1,530
	2010	0	1,830	0	0	0	0	1,830
	2020	0	2,190	0	0	0	0	2,190

Sector	Year	0-1 Mile	1-2 Miles	2-3 Miles	3-4 Miles	4-5 Miles	5-10 Miles	Cumulative Totals By Sector
	2025	0	2,410	0	0	0	0	2,410
SE	1980	0	50	530	0	0	0	580
	1983	0	60	570	0	0	0	630
	1990	0	70	640	0	0	0	710
	2000	0	90	730	0	0	0	820
	2010	0	110	840	0	0	0	950
	2020	0	130	970	0	0	0	1,100
	2025	0	150	1,040	0	0	0	1,190
SSE	1980	10	90	260	330	520	4,340	5,550
	1983	10	100	280	330	520	4,420	5,560
	1990	20	120	320	340	550	4,600	5,950
	2000	20	150	360	360	570	4,790	6,250
	2010	20	180	420	370	590	4,980	6,560
	2020	30	220	490	390	600	5,180	6,910
	2025	30	250	530	400	630	5,280	7,120
S	1980	120	250	540	570	990	7,620	10,090
	1983	140	270	600	580	1,010	7,760	10,360
	1990	170	330	710	600	1,050	8,080	10,940
	2000	210	410	750	620	1,100	8,400	11,490
	2010	250	500	1,030	650	1,140	8,730	12,300

Sector	Year	0-1 Mile	1-2 Miles	2-3 Miles	3-4 Miles	4-5 Miles	5-10 Miles	Cumulative Totals By Sector
	2020	310	600	1,240	680	1,190	9,080	13,100
	2025	340	670	1,380	690	1,210	9,270	13,560
SSW	1980	250	280	420	510	400	9,000	10,860
	1983	280	310	440	510	400	9,160	11,100
	1990	340	380	470	540	420	9,540	11,690
	2000	410	460	510	540	440	9,920	12,280
	2010	500	560	560	580	450	10,320	12,970
	2020	610	680	610	600	470	10,730	13,700
	2025	680	750	650	610	480	10,950	14,120
SW	1980	60	670	390	230	3,290	11,720	16,360
	1983	60	750	390	230	3,350	11,930	6,710
	1990	80	910	410	240	3,480	12,420	17,540
	2000	100	1,110	430	250	3,620	12,920	18,430
	2010	120	1,350	440	260	3,770	13,430	19,370
	2020	140	1,650	460	270	3,920	13,970	20,410
	2025	160	1,830	470	280	4,000	14,530	21,270
WSW	1980	0	670	650	300	3,370	7,570	12,560
	1983	0	750	710	310	3,420	7,800	12,990
	1990	0	910	850	340	3,560	8,340	14,000
	2000	0	1,110	1,020	370	3,710	9,030	15,240

Sector	Year	0-1 Mile	1-2 Miles	2-3 Miles	3-4 Miles	4-5 Miles	5-10 Miles	Cumulative Totals By Sector
	2010	0	1,360	1,220	400	3,860	9,930	16,760
	2020	0	1,660	1,470	440	4,010	11,170	18,740
	2025	0	1,850	1,620	460	4,090	12,060	20,060
W	1980	110	680	270	320	650	2,230	4,260
	1983	120	750	290	360	740	2,560	4,820
	1990	140	920	350	450	950	3,320	6,130
	2000	170	1,120	410	530	1,330	4,510	8,070
	2010	210	1,360	480	640	1,920	6,250	10,860
	2020	260	1,660	670	760	2,890	8,820	14,910
	2025	290	1,850	620	840	3,580	10,750	17,930
WNW	1980	170	70	250	70	650	2,660	3,880
	1983	180	80	270	80	740	2,960	4,310
	1990	190	90	320	90	930	3,660	5,280
	2000	210	90	320	90	1,080	4,730	6,520
	2010	240	100	330	100	1,250	6,200	8,220
	2020	270	100	340	100	1,450	8,250	10,510
	2025	280	100	340	100	1,560	9,700	12,080
NW	1980	20	220	150	120	120	6,440	7,070
	1983	30	240	160	120	120	6,760	7,430
	1990	30	280	190	140	140	7,540	8,320

Sector	Year	0-1 Mile	1-2 Miles	2-3 Miles	3-4 Miles	4-5 Miles	5-10 Miles	Cumulative Totals By Sector
	2000	30	280	190	150	150	8,550	9,350
	2010	30	290	200	150	150	9,750	10,570
	2020	30	290	200	150	150	11,170	11,990
	2025	30	300	210	160	150	12,030	12,880
NNW	1980	30	270	140	170	290	3,480	4,380
	1983	30	280	160	200	330	4,050	5,050
	1990	30	330	190	260	440	5,360	6,610
	2000	30	340	190	280	480	7,600	8,920
	2010	40	340	220	310	530	11,130	12,570
	2020	40	350	230	340	580	16,680	18,220
	2025	40	360	240	360	610	21,110	22,720

Table 2.1-2

PROJECTED POPULATION BY SECTOR - 0 TO 10 MILES

	Year	0-1 Mile	1-2 Miles	2-3 Miles	3-4 Miles	4-5 Miles	5-10 Miles
Cumula-	1980	790	5,250	7,390	6,760	11,870	68,660
tive	1983	870	5,820	8,160	7,430	12,470	72,550
Ring	1990	1,020	7,210	10,100	9,190	13,970	81,600
Totals	2000	1,200	8,410	11,090	9,990	15,360	94,130
-	2010	1,430	9,850	12,540	10,940	17,130	111,200
-	2020	1,710	11,570	14,070	11,960	19,380	134,940
-	2025	1,870	12,650	14,960	12,540	20,940	152,140

	Year	0-1 Mile	1-2 Miles	2-3 Miles	3-4 Miles	4-5 Miles	5-10 Miles
Cumula-	1980	790	6,040	13,430	20,190	32,060	100,720
tive	1983	870	6,690	14,850	22,280	34,750	107,300
Totals	1990	1,020	8,230	18,330	27,520	41,490	123,090
-	2000	1,200	9,610	20,700	30,690	46,050	140,180
-	2010	1,430	11,280	23,820	34,760	51,890	163,090
-	2020	1,710	13,280	27,350	39,310	58,690	193,630
-	2025	1,870	14,520	29,480	42,020	62,960	215,100

TABLE 2.1-3 TOTAL ESTIMATED SEASONAL AND YEAR ROUND LIVING UNITS FOR TOWNS WITHIN 5 MILES OF SEABROOK STATION (BASED ON 1978-79 ELECTRIC METER USE DATA)

		Estimated Number Seasonal Living Units	Estimated Number Year Round Living Units	Total Seasonal & Year Round Living Units
New Hampshire*				
(1)	Hampton	2526	4084	6610
	Hampton Beach	(2425)	(1721)	(4146)
	Hampton	(101)	(2363)	(2464)
(2)	Hampton Falls	64	439	503
(3)	Kensington	28	429	457
(4)	South Hampton	13	217	230
(5)	Seabrook	429	2444	2936
	Total New Hampshire	3060	7613	10736
Massachusetts				
(6)	Amesbury	373	4368	4741
(7)	Salisbury	857	2048	2905
	Total Massachusetts	1230	6416	7646
	Total New Hampshire & Massachusetts	4290	14029	18382

* North Hampton not included, since electric data not available.

** Note: Estimates of seasonal units based on electric meter use for an annual period. Individual meters with residential rate codes were reviewed for the "Seasonal Months" of July and August and compared to the "Off-Season Months" of November and March. Seasonal meters were defined as those for which the "seasonal" electric use (i.e., KWhr/Mo) was at least three (3) times greater than the "Off-Season" use of electricity. Meters not classified as "seasonal" were classified as year-round and assumed to be associated with the permanent resident population.

TABLE 2.1-4 PROJECTED POPULATION BY SECTOR - 0 TO 50 MILES

Sector	Year	0-10 Miles	10-20 Miles	20-30 Miles	30-40 Miles	40-50 Miles	Cumulative Totals By Sector
N	1980	6,100	2,800	30,400	20,000	6,200	85,500
	1983	7,100	23,500	30,800	20,500	6,300	88,200
	1990	9,400	25,200	31,800	21,500	6,600	94,500
	2000	12,200	27,400	32,700	22,300	6,900	101,500
	2010	16,200	28,400	33,800	23,200	7,200	108,800
	2020	21,900	30,400	35,000	24,100	7,500	118,900
	2025	26,000	35,000	35,600	24,600	7,700	128,900
NNE	1980	12,200	30,600	10,300	16,500	38,900	108,500
	1983	13,400	31,300	10,500	16,800	39,700	111,700
	1990	16,400	32,800	11,000	17,600	41,700	119,500
	2000	19,000	34,800	11,500	18,300	43,300	126,900
	2010	22,400	37,000	11,900	19,100	45,000	135,400
	2020	26,700	39,700	12,400	19,800	46,900	145,500
	2025	29,400	41,300	12,700	20,200	47,800	151,400
NE	1980	4,000	2,100	0	0	0	6,100
	1983	4,600	2,200	0	0	0	6,800
	1990	6,100	2,500	0	0	0	8,600
	2000	7,000	2,800	0	0	0	9,800
	2010	8,200	3,200	0	0	0	11,400
	2020	9,700	3,700	0	0	0	13,400

Sector	Year	0-10 Miles	10-20 Miles	20-30 Miles	30-40 Miles	40-50 Miles	Cumulative Totals By Sector
	2025	10,600	4,100	0	0	0	14,700
ENE	1980	1,400	0	0	0	0	1,400
	1983	1,600	0	0	0	0	1,600
	1990	2,100	0	0	0	0	2,100
	2000	2,300	0	0	0	0	2,300
	2010	2,500	0	0	0	0	2,500
	2020	2,700	0	0	0	0	2,700
	2025	2,900	0	0	0	0	2,900
E	1980	500	0	0	0	0	500
	1983	500	0	0	0	0	500
	1990	700	0	0	0	0	700
	2000	800	0	0	0	0	800
	2010	900	0	0	0	0	900
	2020	900	0	0	0	0	900
	2025	1,000	0	0	0	0	1,000
ESE	1980	900	0	0	0	0	900
	1983	1,000	0	0	0	0	1,000
	1990	1,300	0	0	0	0	1,300
	2000	1,500	0	0	0	0	1,500
	2010	1,800	0	0	0	0	1,800

Sector	Year	0-10 Miles	10-20 Miles	20-30 Miles	30-40 Miles	40-50 Miles	Cumulative Totals By Sector
	2020	2,200	0	0	0	0	2,200
	2025	2,400	0	0	0	0	2,400
SE	1980	600	8,100	800	0	0	9,500
	1983	600	8,200	900	0	0	9,700
	1990	700	8,500	900	0	0	10,100
	2000	800	8,900	900	0	0	10,600
	2010	1,000	9,200	1,000	0	0	11,200
	2020	1,100	9,600	1,000	0	0	11,700
	2025	1,200	10,000	1,000	0	0	12,200
SSE	1980	5,600	13,400	22,000	0	0	41,000
	1983	5,700	13,700	22,400	0	0	41,800
	1990	6,000	14,200	23,300	0	0	43,500
	2000	6,300	14,800	24,300	0	0	45,400
	2010	6,600	15,400	25,200	0	0	47,200
	2020	6,900	16,000	26,300	0	0	49,200
	2025	7,100	16,300	26,800	0	0	50,200
S	1980	10,100	18,600	171,400	97,500	206,700	504,300
	1983	10,400	18,900	174,500	99,300	210,500	513,600
	1990	10,900	19,700	181,700	103,400	219,100	534,800
	2000	11,500	20,500	189,000	107,500	227,900	556,400

Sector	Year	0-10 Miles	10-20 Miles	20-30 Miles	30-40 Miles	40-50 Miles	Cumulative Totals By Sector
	2010	12,300	21,300	196,500	111,800	237,000	578,900
	2020	13,100	22,200	204,400	116,300	246,500	602,500
	2025	13,600	22,600	210,000	118,600	251,400	616,200
SSW	1980	10,900	20,200	161,100	869,000	801,500	1,862,700
	1983	11,100	20,600	164,000	884,600	815,900	1,896,200
	1990	11,700	21,400	170,800	921,100	849,600	1,974,600
	2000	12,300	22,300	177,600	957,100	883,600	2,052,900
	2010	13,000	23,200	184,700	996,300	918,900	2,136,100
	2020	13,700	24,100	192,100	1,036,100	955,700	2,221,700
	2025	14,100	24,600	196,000	1,056,800	974,800	2,266,300
SW	1980	16,400	66,300	185,600	176,000	127,300	571,600
	1983	16,700	67,500	189,000	179,100	129,600	581,900
	1990	17,500	70,300	196,700	186,500	135,000	606,000
	2000	18,400	73,100	204,600	194,000	140,400	630,500
	2010	19,400	76,100	212,800	201,800	146,000	656,100
	2020	20,400	79,100	221,300	209,800	151,800	682,400
	2025	21,300	80,700	225,700	214,000	154,900	696,600
WSW	1980	12,600	26,100	95,000	113,700	34,700	282,100
	1983	13,000	26,500	100,200	120,300	35,800	295,800
	1990	14,000	27,600	112,500	135,700	38,500	328,300

Sector	Year	0-10 Miles	10-20 Miles	20-30 Miles	30-40 Miles	40-50 Miles	Cumulative Totals By Sector
	2000	15,200	28,700	125,800	151,200	41,200	362,100
	2010	16,800	29,900	141,100	168,100	44,300	400,200
	2020	18,700	31,100	158,500	187,200	47,900	443,400
	2025	20,100	31,700	168,400	198,000	50,200	468,400
W	1980	4,300	15,600	47,600	84,900	21,500	173,900
	1983	4,800	17,300	50,400	90,700	23,200	186,400
	1990	6,100	21,100	57,000	104,100	27,000	215,300
	2000	8,100	24,300	63,600	118,800	30,800	245,600
	2010	10,900	28,100	71,300	136,500	35,200	282,000
	2020	14,900	32,500	80,200	58,000	40,300	325,900
	2025	17,900	35,000	85,400	171,000	43,300	352,600
WNW	1980	3,900	7,900	12,900	61,400	33,300	119,400
	1983	4,300	9,000	13,400	63,300	34,500	124,500
	1990	5,300	11,700	14,800	67,700	37,300	136,800
	2000	6,500	16,300	16,200	72,700	40,600	152,300
	2010	8,200	23,800	17,800	78,200	44,300	172,300
	2020	10,500	36,200	19,600	84,400	48,300	199,000
	2025	12,100	46,900	20,700	87,900	50,600	218,200
NW	1980	7,100	8,000	3,600	10,300	16,100	45,100
	1983	7,400	8,800	3,700	10,600	16,700	47,200

Sector	Year	0-10 Miles	10-20 Miles	20-30 Miles	30-40 Miles	40-50 Miles	Cumulative Totals By Sector
	1990	8,300	10,600	4,000	11,300	18,100	52,300
	2000	9,400	13,200	4,200	12,100	19,800	58,700
	2010	10,600	16,700	4,500	12,900	21,700	66,400
	2020	12,000	21,300	4,700	14,000	23,800	75,800
	2025	12,900	27,500	4,900	14,500	26,200	86,000
NNW	1980	4,400	13,900	18,500	14,200	5,400	56,400
	1983	5,100	14,400	18,900	14,400	5,600	58,400
	1990	6,600	15,400	19,800	15,000	6,000	62,800
	2000	8,900	16,900	20,700	15,900	6,400	68,800
	2010	12,600	19,000	21,600	16,800	7,000	77,000
	2020	18,200	22,100	22,700	17,900	7,500	88,400
	2025	22,700	24,400	23,200	18,500	7,900	96,700

	Year	0-10 Miles	10-20 Miles	20-30 Miles	30-40 Miles	40-50 Miles	
Incre-	1980	100,700	253,600	759,200	1,463,500	1,291,600	
mental	1983	107,300	261,900	778,700	1,499,600	1,317,800	
Ring	1990	123,100	281,000	824,300	1,583,900	1,378,900	
Totals	2000	140,200	304,000	871,100	1,669,900	1,440,900	
	2010	163,100	331,300	922,200	1,764,700	1,506,600	
	2020	193,600	368,000	978,200	1,867,600	1,576,200	
	2025	215,100	400,100	1,010,400	1,924,100	1,614,800	

	Year	0-10 Miles	10-20 Miles	20-30 Miles	30-40 Miles	40-50 Miles	
Cumula-	1980	100,700	354,300	1,113,500	2,577,000	3,868,600	
tive	1983	107,300	369,200	1,147,900	2,647,500	3,965,300	
Totals	1990	123,100	404,100	1,228,400	2,812,300	4,191,200	
	2000	140,200	444,200	1,315,300	2,985,200	4,426,100	
	2010	163,100	494,400	1,416,600	3,181,300	4,687,900	
	2020	193,600	561,600	1,539,800	3,407,400	4,983,600	
	2025	215,100	615,200	1,625,600	3,549,700	5,164,500	

TABLE 2.1-5 **OVERNIGHT ACCOMMODATIONS**

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
HAMPTON				
American Hotel, Apt.	(E 1-2)	6 apts.	20	Y
Ashworth Hotel Inc.	(ENE 1-2)	111	358	Y
Barnes Motel	(NE 3-4)	14	34	6 units-Y 8 units-S
Blue Lantern Motel	(NE 3-4)	20	70	S
Century House Motel	(ENE 2-3)	24	48	S
Grayhurst Guest House	(ENE 1-2)	11 rooms 2 apts. 1 cottage	48	S Y
Jen's Ocean Manor	(NE 2-3)	10 apts.	90	S
Rainbow Village Hotel	(E 1-2)	11	30	Y
Rockey's Real Estate	(NE 2-3)	23	207	4-5 rooms-Y
Sea Squire Motor Lodge	(NE 4-5)	12	24	S
Sheraton Motor Inn Lamie's Tavern	(N 2-3)	30	118	Y
Town & Beach Motel	(N 3-4)	23	60	Y
Villager Motel	(N 2-3)	17	34	S
Hillcrest Motor Inn	(ENE 1-2)	31	93	S
Westport Motel	(ENE 1-2)	16	64	S
Pine Haven Motel	(N 2-3)	16	32	Y
Wishing Well	(N 2-3)	16	60	Y
Dalton	(N 2-3)	13	26	Y
Allen's House	(ENE 2-3)	6	12	NA
Nor' Easter Motel	(ENE 2-3)	6-8	16	NA
Wave Motel	(ENE 2-3)	17	34	NA
Boulevard Motel	(ENE 2-3)	25	50	NA

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
Sun 'n Surf	(ENE 2-3)	21	60	12-Y 9-S
Santa Maria Duffey's Island View Duffey's Manor	(ENE 2-3)	60	120	Y
Reef Motel	(ENE 2-3)	14	28	S
Drop Anchor Motel	(NE 2-3)	15	45	S
Motel	(NE 3-4)	3 apts.	6	NA
Yearly Rentals	(NE 3-4)	5	10	Y
Ocean Edge	(NE 3-4)	6	15	S
Spindrift	(NE 3-4)	32	64	NA
Bailey Motel	(NE 3-4)	28 apts.	56	NA
The Carriage	(NE 2-3)	24	48	9-Y (avg. 6 in winter) 15-S
Martins Motor Court	(NNE 2-3)	27	54	NA
Twin Oaks	(NE 2-3)	17	68	S
White Gables Motor Court	(NE 2-3)	10	30	S
Belle & Regis Motel	(ENE 1-2)	7	25	S
Lincoln House	(E 1-2)	17	40	S
Rocky Wold Motel	(ENE 2-3)	14	50	S
Sea Farer Guest House	(ENE 1-2)	10	25	Y
Windjammer Motel	(NE 3-4)	16	48	S
Seascape Motel	(NE 3-4)	19	57	S
Rhea's Motel	(NE 3-4)	6	20	S
Acorn Village Motel	(NE 3-4)	5 rooms 1 house 14 cottages	90	7-Y
Cove Motel	(NE 3-4)	25	75	S
McGurk Rentals	(ENE 1-2)	--	25	S
St. Jean Court	(ENE 1-2)	11	50	S

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
Bea Cottages	(ENE 1-2)	12-15	30	NA
Felka Cottages	(ENE 1-2)	6	12	NA
Marguerite Motel	(ENE 1-2)	42	84	S
Jane's Court	(NE 3-4)	8	16	NA
Ocean Motel	(NE 3-4)	20	80	S
Holiday Shores	(NE 3-4)	16	32	NA
Broadview Guest House	(E 1-2)	4-5	10	NA
Mayflower Inn	(E 1-2)	7	18	Y
Surfside	(E 1-2)	23	46	S
Drop In Cottages	(E 1-2)	9	18	NA
Joy's Rooms and Cottages	(E 1-2)	3	6	NA
Blue Jay Motel	(E 1-2)	16	32	S
Seaside Chalet	(E 1-2)	11	66	S
Brownie's	(E 1-2)	20	40	S
The Cavalier	(ENE 1-2)	10	20	NA
The Austin	(ENE 1-2)	8	16	NA
Seaside Village Motel	(NE 4-5)	22	70	S
Kitty-Lou Cottages & Motel	(ENE 1-2)	17	50	S
Duthie's	(ENE 3-4)	12	24	NA
Sherry Marie	(ENE 1-2)	5	10	NA
Greycliff Rooms	(ENE 1-2)	5	10	S
Eastern Shore Apts	(ENE 1-2)	2	4	S
Deerfield Rooms	(ENE 1-2)	8	15	S
Danahy Apts. & Rms.	(ENE 1-2)	11	22	NA
Ocean Breeze Motel	(E 1-2)	8	24	S
Beach View	(E 1-2)	30	120	Y = 25 units off season 1 prs./unit
The Puritan	(NNE 1-2)	25	50	S

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
Surf Hotel	(ENE 2-3)	36	72	NA
East Wind	(ENE 2-3)	25-30	60	NA
Atlantic (all of them)	(ENE 2-3)	60	180	Y
Seven Gables	(ENE 2-3)	21	42	S
Ocean Spray Court	(ENE 2-3)	35	70	NA
Jonathan's	(ENE 2-3)	16	64	5-Y/11-S
Merrimac	(ENE 2-3)	20	40	S
Sea Den	(ENE 2-3)	43	172	April - October
Ocean Air Apts.and Cottages	(ENE 2-3)	7	14	NA
Riviera Motel	(ENE 2-3)	18	35	S
The Edgewater Cabins	(ENE 2-3)	12	40	S
Sea Breeze Village Cabins	(ENE 2-3)	17	34	NA
Hollywood Motel	(ENE 2-3)	6	20	S
The Alecia Apt. & Kitchenettes	(ENE 2-3)	8	16	NA
Vista Motel	(ENE 2-3)	23	46	S
Colony Motel	(ENE 1-2)	32	64	NA
Main Sail Motel	(ENE 1-2)	9 cottages 16 rooms 8 apts	100	Rooms, apt-Y Cottages-S
Sand Motel	(ENE 1-2)	61	225	23-Y
Sunny Motel	(ENE 1-2)	3	6	S
Chesterfield	(ENE 1-2)	6 apts.	12	NA
Wilbert Motel	(ENE 1-2)	12	24	NA
Sea Horse	(ENE 1-2)	4-5	10	NA
Canadian House	(ENE 1-2)	18	72	S
Longview Apts.	(ENE 1-2)	8-10 at 1&2 bedrooms	40	Y
Sunset Haven	(ENE 1-2)	8 apts.	16	NA
Westport Motel	(ENE 1-2)	16	64	S

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
Mari Anne	(E 1-2)	24 apts.	40	10-15/Y
Motel Drift	(E 1-2)	23	78	Y
Voyager Motor Lodge	(E 1-2)	8	32	Y
Surfside Chalet	(E 1-2)	10	20	NA
Harris Sea Ranch	(E 1-2)	30	70	S
Springfield Motor Court	(E 1-2)	6	12	NA
The Pelham	(E 1-2)	36	144	S
Moulton Hotel	(ENE 1-2)	23	92	S
Green Briar	(ENE 1-2)	21	42	NA
The OceanSide Guest House	(ENE 1-2)	15	60	S
Young's	(ENE 2-3)	61	244	Y
Laughlin's Grand View	(ENE 2-3)	30	60	NA
The Algiers	(ENE 2-3)	22	44	Y
The Vandemere	(ENE 2-3)	20	80	S
The Shirley	(ENE 2-3)	14	40	S
The Kentville	(ENE 2-3)	30	60	NA
Duffey's Apts. & Inn	(ENE 1-2)	20 rooms 6 apts.	61	S
Elmdale	(ENE 1-2)	10	20	NA
Admiral's Choice	(ENE 1-2)	9	18	NA
Sea Mist	(ENE 1-2)	6	12	Y
Bromfield	(ENE 1-2)	5	10	NA
Windsor Motel	(ENE 1-2)	11	37	S
The Pirates' Den (same no. as Admiral's)	(ENE 1-2)	12	24	NA
Star & Key Cottages	(ENE 1-2)	5-10	20	Y
The Helm Rooms	(ENE 1-2)	7	14	Y
Days End	(E 1-2)	10-12	24	NA

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
Springfield	(E 1-2)	30	60	Y
Paine's Guest House	(ENE 1-2)	9	31	S
Silver Spring Kitchenettes	(ENE 1-2)	9	27	S
Vellia	(ENE 1-2)	16	32	Y
The Dorna Room	(ENE 2-3)	8	16	NA
Lismore	(ENE 2-3)	8	16	S
Bon Air	(ENE 2-3)	7	14	NA
June Ville	(ENE 2-3)	7	21	S
Wahl Cottages	(ENE 1-2)	9	18	NA
Adrian's Cottages	(ENE 1-2)	3	6	Y
Clifford House	(ENE 1-2)	3	6	NA
Bartlett Rooms	(ENE 1-2)	8-10	20	NA
Shore Winds	(ENE 1-2)	8	16	NA
Tropical Inn	(ENE 1-2)	5	10	NA
Kenmore Inn	(ENE 1-2)	5	10	NA
Janmere Inn	(ENE 1-2)	50	100	S
Dolphyn	(ENE 1-2)	29	38	S
Bobbie's & Sandpiper Kimball R.E.	(ENE 1-2)	6	12	NA
Styers Motel	(ENE 1-2)	3	6	NA
Motel Tides	(ENE 1-2)	24	48	S
Royal Crest Motor Inn	(ENE 1-2)	40	80	NA
Sunset Chalet	(ENE 1-2)	8	16	NA
Conn. Village Guest House	(ENE 1-2)	10	20	NA
The Milton House	(ENE 1-2)	10	20	NA
Richmond Motel & Apts.	(ENE 1-2)	4	8	Y
Laurentain Motel	(ENE 1-2)	61	184	S

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
The Avon	(ENE 1-2)	25	50	NA
Hollingsworth	(ENE 1-2)	63	120	S
Debonair	(ENE 1-2)	19	38	S
Seagate	(ENE 1-2)	15	60	Y
Beachcroft	(ENE 1-2)	15	98	S
Royall Hampton Inn	(ENE 1-2)	43-48	96	NA
The Lift Inn	(E 1-2)	4	8	S
Hampton House	(E 1-2)	1	2	NA
The Old Salt	(ENE 1-2)	8	14	S
Nourage Guest House	(ENE 1-2)	3 Apt 1 Cottage	16	Y
Beacon Hotel	(E 1-2)	14	33	S
Mermaid Apts.	(ENE 1-2)	8	16	NA
Apts.	(ENE 1-2)	4	8	NA
A Street Apts.	(ENE 1-2)	6	12	NA
Boar's Head Inn	(ENE 2-3)	12	24	S
Colonial Inn	(E 1-2)	NA	NA	NA
Drolet Jean L.	(E 1-2)	60	180	S
HAMPTON TOTAL			8068	
AMESBURY				
Susse Chalet Motor Lodge	(SW 4-5)	60	180	Y

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
Alan's Motel & Diner	(SW 4-5)	28	36	Y
AMESBURY TOTAL			216	
SALISBURY				
Ocean Gate	(SSE 4-5)	52	200	S
Clear Inn House	(SSE 4-5)	22	45	Y
Colonial Arms Hotel	(SSE 4-5)	10	16	Y
Johnson's Hotel	(SSE 4-5)	6	14	Y
Sea Crest Motel	(SSE 3-4)	18	66	S
Knotty Pine Motel	(S 4-5)	21	50	S
Village Inn Motel	(SSW 2-3)	10	20	Y
Beach Road Grove Cabins	(S 4-5)	27	54	Y
Henry Sun & Sand and George's Motel & Cottages	(S 4-5)	Both motels combined--100 units	250	S
El Rancho	(SSE 4-5)	37	101	S
Beach Way Motor Court	(S 4-5)	33	66	S
Sunshine Cabins	(SSW 2-3)	3	6	NA
Olde Farm Motel	(SSW 2-3)	10	20	Y
Hearthstones	(S 3-4)	5	10	NA
Edwards By The Sea	(SSE 4-5)	24	32	S
Sundowner Motel	(SSE 4-5)	11	17	S

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
Surfway	(SSE 4-5)	27	54	NA
The Hungry Traveller	(SSE 4-5)	4	8	NA
The Dunes	(SSE 4-5)	8 units 2 rooms	32	NA
McCarthy's	(SSE 4-5)	10	20	NA
Haggarty's	(SSE 3-4)	14 with 1-2 rooms	56	S
Sea Gate Cottages	(SSE 3-4)	8	40	Y
Sun 'n Sand	(SSE 4-5)	10	20	NA
SALISBURY TOTAL			1197	
SEABROOK				
Hawaiian Garden Motor Inn	(W 1-2)	27	34	Y
Seabrook Cottages & Motel Sleepy Hollow	(WSW 1-2)	17	34	NA
Phoenicia Motel	(SW 1-2)	18	36	NA
Village Motel	(SW 1-2)	11 Cottages	22	NA
Spruce Manor	(WSW 1-2)	17	34	NA
McInnis	(SW 1-2)	10	20	NA
Stoddard Cabins	NA	NA	NA	NA
Best Western	(W 1-2)	53	106	Y

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
SEABROOK TOTAL			286	
NORTH HAMPTON				
Langiells Motor Court	(N 4-5)	4	4	Y
Knox Motel	(N 4-5)	10	26	Y
Gregory's Motel	(N 4-5)	14	32	S
Whispering Pines	(N 4-5)	9	18	S
King Motel	(N 4-5)	16	96	Y
Slumber Manor	(N 4-5)	8	16	
Seaside Inn	(NE 4-5)	NA	NA	NA
NORTH HAMPTON TOTAL			192	
HAMPTON FALLS				
Blue Mist Motel	(NNW 1-2)	6 winter 20 summer	40	Y
New England Village Cabins	(NNW 1-2)	10	20	NA
HAMPTON FALLS TOTAL			60	
KENSINGTON - NONE within 10-mile radius				

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
SOUTH HAMPTON - NONE within 10-mile radius				
RYE				
Historic Cable House	(NE 8-9)	8 – Large 1 - Small	26	S
Salty Breeze Inn	(NE 9-10)	7 - Cabins	40	S
Rye Harbor Motel	(NE 8-9)	16	32	NA
Farragut Hotel	(NE 7-8)	72	144	NA
Sleepy Hollow Motel	(NNE 7-8)	19 units	50	1 or 2 days in August
Rye Beach Motel & Cottages	(NE 8-9)	7	14	S
Dunes Motel	(NE 7-8)	36	72	
Crown Colony Cottages	(NE 9-10)	3 motel units 12 efficiency 3 apartments 1 house 19 units total	84	S
RYE TOTAL			462	
EXETER				
The Exeter Inn	(NW 8-9)	55	110	Y
Hearthside Motor Inn	(NW 7-8)	35	138	Y

Guest House Hotel/Motel	Location (sector)	No. of Rooms	Maximum Capacity	Y=All Year S=Seasonal
EXETER TOTAL			248	
NEWBURYPORT				
Essex Street Inn (Rooming House)	(S 6-7)	20	20	Y
Everett Clarke Cottages	(SSE 5-6)	8 (2-3 B/Rms)	34	S
Hunter Guest Houses	(SSE 7-8)	2 cottages 2-3 Rms. 6 people in each.	12	S
Kenmore Cottages	(SSE 7-8)	6 cottages/1 apt.	24	Y
Walton's Ocean Front	(SSE 7-8)	17	50	S
Benj. Choate House	(SSW 5-6)	5	14	Y
Civic Center	(S 6-7)	3	65	June-Labor Day or by reservation in winter (groups 15)
Windsor House	(S 6-7)	6	15	Y
Beaumanor at Amesbury	(SSW 5-6)	6	11	Y
NEWBURYPORT TOTAL			245	
PORTSMOUTH				
Wren's Nest Motel	(NNE 8-9)	14 rooms 7 Cottages	50	NA

TABLE 2.1-6 **CAMPING FACILITIES WITHIN 10 MILES OF THE SITE**

Name	Location (Sector)	Approximate No. of Sites	Estimated Capacity	Season
Tidewater	Hampton (N 2-3)	100 trailers + 25 tent sites = 125 total	500	May - October
Wakeda	Hampton Falls (NW 4-5)	300	1,200	May 1 - Oct. 15
Adams	Seabrook (S 1-2)	75	300	May 15 - Oct. 1
Shel Al Mobile Estates & Camping	N. Hampton (N 5-6)	190	800	May 15 - Oct. 1
Hampton Beach Trailer	Hampton (NE 3-4)	190	760	May 1 - Oct. 1
Rusnick Campground (daycamp)	Salisbury (SSW 2-3)	NA	NA	NA
Pike's Camping Area	Salisbury (S 4-5)	40 trailer + 40 tent sites = 80 total	400	NA

* No camping facilities identified within 0-5 miles in either Kensington, South Hampton or Amesbury.

Salisbury Beach State Reservation (camping area only)	Salisbury (SSE 5-6)	350 trailer + 135 tent sites = 485 total	1,940	NA
Weemac Campground	Amesbury (WSW 7-8)	100 sites 7 cabins	556	Mid-May - Mid-Oct.
Camp Bauercrest	Amesbury (WSW 7-8)			Summer Camp
Tuxbury Pond Camping Area	S. Hampton (WSW 7-8)	130 sites	520	Mid-May-Oct. 1
Camp Holiday	Amesbury (WSW 7-8)		NA	
Camp Treefoil (Camp Kent)	Amesbury (WSW 5-6)		NA	Summer Camp
Pinebrook Campground	Kingston (WSW 10)		NA	
The Green Gate Camping Area	Exeter (NW 7-8)	80 sites	400	May 26 - Oct. 1
Exeter Elms Campground	S. Exeter (NW 7-8)	68 sites	272	May 30 - Oct. 1
		TOTAL		7648

TABLE 2.1-7 **BEACH PARKING LOT CAPACITIES AND OBSERVED PEAKS 0-10 MILE RADIUS**

Sector	Marked Spaces [(1)] (including leased)	Estimated Unmarked Spaces	Estimated Total Parking Lot Capacity[(2)]	Observed Single Peak Day Count 7/22/79
SSE 8-9	0	71	71	13
SSE 7-8	180	419	599	396
SSE 6-7	125	300	425	389
SSE 5-6	1861	60	1921	1130
SSE 4-5	876 (30)	738	1614	1187
SSE 3-4	297	714	1011	461
SE 2-3	61	141	202	188
ESE 1-2	0	317	317	289
E 1-2	0	2551	2551	1914
ENE 1-2	1882 (318)	620	2502	2283
ENE 2-3	435 (211)	479	914	875
ENE 3-4	0	35	35	13
NE 3-4	562	52	614	547
NE 4-5	135	177	312	190
NE 5-6	115	36	151	151
NE 7-8	80	167	247	247
NE 8-9	0	409	409	248
Total	**6609 (559)**	**7286**	**13,895**	**10,521**

(all sectors)

1. Estimate of leased spaces in brackets and included in estimate of total marked parking lot spaces.

2. Includes marked and unmarked parking lot spaces.

TABLE 2.1-8 BEACH AREA PARKING LOT USAGE SALISBURY/SEABROOK/HAMPTON BEACH (WEEKENDS - 1979)

DATE	HR[(1)]	DAY CODE[(2)]	CAR COUNT	% TOTAL CAPACITY
6/09/79	13	6	3682	31
6/10/79	13	7	4981	42
6/16/79	13	6	6875	58
6/17/79	13	7	6442	54
6/23/79	13	6	3014	25
6/25/79	13	7	4852	41
7/07/79	15	6	5110	43
7/08/79	15	7	8376	70
7/14/79	14	6	6397	54
7/21/79	13	6	7648	64
7/22/79	13	7	9097	76
7/28/79	16	6	3850	32
7/29/79	13	7	7637	64
8/04/79	13	6	5598	47
8/11/79	13	6	2010	17
8/18/79	13	6	2579	22
8/25/79	14	6	2075	17
8/26/79	13	7	5765	48
9/01/79	13	6	4683	39
9/02/79	13	7	5618	47
9/08/79	13	6	1193	10
9/16/79	13	7	1304	11

TOTAL CAPACITY IN LOTS = 11,993 including leased spaces.

(1) 12 = noon, 13 = 1:00 p.m., etc.

(2) Key for day code:

1 = Monday
2 = Tuesday
3 = Wednesday
4 = Thursday
5 = Friday
6 = Saturday
7 = Sunday

TABLE 2.1-9 BEACH AREA PARKING LOT USAGE SALISBURY/SEABROOK/HAMPTON BEACH (WEEKDAYS – 1979)

DATE	HR(2)	DAY CODE(3)	CAR COUNT	% TOTAL CAPACITY
6/08/79	13	5	1547	13
6/11/79	13	1	1283	11
6/14/79	13	4	2069	17
6/15/79	13	5	3471	29
6/18/79	14	1	1660	14
6/19/79	13	2	1896	16
6/20/79	13	3	2909	24
6/21/79	13	4	3130	26
6/22/79	13	5	1976	17
6/25/79	14	1	1351	11
6/27/79	13	3	3045	26
6/28/79	14	4	2058	17
6/29/79	13	5	1535	13
7/02/79	14	1	2871	24
7/03/79	13	2	4135	35
7/04/79(1)	14	3	6613	55
7/05/79	13	4	2174	18
7/06/79	13	5	3216	27
7/09/79	13	1	4703	39
7/10/79	13	2	4888	41
7/11/79	13	3	2821	24
7/13/79	13	5	4703	39
7/17/79	13	2	2611	22
7/19/79	13	4	4787	40
7/20/79	13	5	5099	43
7/23/79	15	1	4268	36
7/24/79	14	2	4415	37
7/26/79	13	4	3192	27
7/27/79	13	5	3852	32
7/30/79	13	1	4425	37
7/31/79	13	2	4622	39
8/01/79	14	3	3471	29
8/02/79	16	4	2564	22
8/03/79(1)	13	5	4731	40
8/06/79	13	1	3872	32
8/07/79	13	2	4514	38
8/08/79	13	3	4494	38
8/09/79	14	4	4058	34
8/15/79	13	3	2371	20
8/23/79	13	4	2043	17
8/31/79	18	5	1321	11

DATE	HR[2]	DAY CODE[3]	CAR COUNT	% TOTAL CAPACITY
9/03/79	13	1	2791	23
9/12/79	13	3	588	5
9/19/79	13	3	328	3

TOTAL CAPACITY IN LOTS = 11,993 including leased spaces.

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(1) Holiday date included with weekend survey dates.

(2) 12 = noon, 13 = 1:00 p.m., etc.

(3) Key for day code:

1 = Monday
2 = Tuesday
3 = Wednesday
4 = Thursday
5 = Friday
6 = Saturday
7 = Sunday

TABLE 2.1-10 BEACH TRAVEL ORIGIN DESTINATION SURVEY RESULTS WEEKDAY – WEEKEND TOTALS

Trip Origin (Radial Distance From Seabrook Station	Summary of Weekday Survey Results		Summary of Weekend Survey Results		Total All Survey Days	
	Number of Surveys	%	Number of Surveys	%	Number of Surveys	%
0-5	100	6.3	54	3.3	154	4.8
5-10	119	7.5	57	3.5	176	5.5
10-20	128	8.1	65	4.0	193	6.0
20-30	264	6.7	234	14.4	498	15.5
30-40	333	21.0	427	26.3	760	23.7
40-50	88	5.6	134	8.3	222	6.9
50+	552	34.8	651	40.1	1203	37.5
Total	1584	100.0	1622	100.0	3206	100.0

TABLE 2.1-11 MONTHLY PEAK ATTENDANCE AT SEABROOK GREYHOUND PARK

January 1977 - September 1979

Year 1977	Daily (Peak)	Year 1978	Daily (Peak)	Year 1979	Daily (Peak)
Jan. (1/22-evening)	3,516	Jan. (1/6-day)	3,837	Jan. (1/27-evening)	3,686
		(1/28-evening)	4,400		
Feb. (2/26-evening)	5,138	Feb. (2/20-matinee)	4,915	Feb. (2/19-matinee)	4,391
March (3/5-evening)	5,508	March (3/11-evening)	5,144	March. (3/24-matinee)	3,447
April (4/30-evening)	4,225	April (4/29-evening)	4,473	April (4/14-matinee)	3,429
May (5/28-evening)	4,038	May (5/6-evening)	4,512	May (5/28-matinee)	4,095
June (6/25-evening)	4,667	June (6/17-evening)	4,187	June (6/12-matinee)	3,801
<u>July (7/23-evening)</u>	6,230	July (7/4-matinee)	5,186	July (7/28-evening)	4,415
		(7/28-evening)	5,333		
Aug. (8/20-evening)	5,477	<u>Aug. (8/26-evening)</u>*	6,960	Aug. (8/11-evening)	4,944
Sept. (9/3-evening)	5,772	Sept. (9/2-evening)	5,032	<u>Sept. (9/1-evening)</u>*	7,027
				(9/17-matinee)	3,800
Oct. (10/15-evening)	3,916	Oct. (10/28-evening)	3,015		
Nov. (11/5-evening)	3,915	Nov. (11/11-matinee)	3,145		
Dec. (12/26-matinee)	4,225	Dec. (12/29-matinee)	3,127		

* Highest daily attendance for year.

TABLE 2.1-12 **AERIAL BOATING SURVEY – OCCUPIED BOATS**

Photo Date[(1)]	Brown's River	Hampton River	Blackwater River	Atlantic Ocean (5-Mile Radius)
7/13	0	6	0	10
7/14*	0	15	3	5
7/19	0	1	2	20
7/20	0	0	0	31
7/21*	0	4	2	72
7/22*	0	-	-	81
7/23	0	-	-	28
7/24	0	0	0	30
7/26	0	2	0	5
7/27	0	4	3	11
7/28*	0	2	0	13
7/29*	0	4	3	180
7/30	0	1	2	8
7/31	0	1	0	5
8/1	0	0	1	1
8/2	0	1	0	13
8/3	0	0	0	7
8/4*	0	1	0	105
8/5*	0	2	1	45
8/6	0	1	0	15
8/7	0	0	0	27
8/8	0	5	0	33
8/9-6 AM	0	0	0	9
8/9-9 AM	0	1	0	16
8/9-11 AM	0	2	0	27
8/9-2 PM	0	4	0	55
8/9-5 PM	0	2	0	37
8/9-7 PM	0	0	0	15
8/11*	0	3	1	28
8/13	0	1	0	8
8/14	0	2	0	15
8/15	0	2	1	18
8/16	0	0	0	10
8/18*	0	6	0	52
8/20	0	4	0	26
8/21	0	3	0	24
8/23	0	2	1	14
8/25*	5	1	1	21
8/26*	1	5	1	352**
8/30	0	1	0	5

Photo Date[(1)]	Brown's River	Hampton River	Blackwater River	Atlantic Ocean (5-Mile Radius)
9/1*	1	7	2	300
9/2-7 AM*	0	3	1	9
9/2-10 AM*	0	1	0	-
9/2-1 PM*	1	5	1	200
9/2-4 PM*	1	5	3	200
9/2-6 PM*	0	4	1	50
9/3*	0	5	0	75
9/8*	0	4	0	150
9/12	0	0	0	10
9/16*	0	2	0	98
9/19	0	0	0	0
9/23*	0	3	0	5

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(1) Photos taken in early afternoon
* Indicates Saturday, Sunday, or holiday.
** 10-mile radius boat count.
NOTE: No occupied boats were observed on Hunt's Island Creek, the Boston and Maine Railroad landing, Farm Dock Landing, or Walton Landing during the survey period.

TABLE 2.1-13 MAJOR EMPLOYERS WITHIN 10 MILES OF THE SEABROOK SITE AND ESTIMATED NUMBER OF EMPLOYEES

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [1]
Hampton			
J.D. Cahill Co.	Scott Road (N 4-5)	Polyethylene coated paperboard	40
Charles Greenman Co.	70 High Street (NNE 4-5)	Leather and rubber soles	14
Hampton Machinery	Exeter Road (N 4-5)	Tannery equipment repairing	30
Hopkin Hunt Co.	Colonial Circle (N 4-5)	Special industrial machinery	3
Palmer & Sicard	Lafayette Road (N 3-4)	Sheet metal for heating and ventilating	62
Pearse Leather	7 Kershaw Avenue (NNE 4-5)	Contract leather finishing	12
Foss Manufacturing	Foss Road (N 3-4)	Nonwoven textiles	12
Whites Welding	6 Kershaw Avenue (NNE 4-5)	Welding	1
Advanced Speaker	432 Lafayette Road (N 4-5)	Speakers	25
Exeter Instruments	70 High Street (NNE 4-5)	Medical instruments	5
Hampton Water Works	52 High Street (NNE 2-3)	Water and sewer	N/A
Wands Inc.	1 Lafayette Road (N 1-2)	Oil heating equipment	20
Wheelabrator-Frye Inc.	Liberty Lane	Pollution control	180

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [(1)]
	(N 3-4)		
Garnet Lumber Co.	5 Dearborn Avenue (NNE 3-4)	Lumber	4
Merrill Lumber Co.	5 Dearborn Avenue (NNE 3-4)	Lumber	6
Mibo, Inc.	12 Evergreen Road (N 3-4)	Buckles and bows	6
Rockingham County Newspapers	Depot Square (NNE 3-4)	Newspaper publishing	12
Stark-MacDonald, Inc.	40 Sweetbriar Lane (N 3-4)	Leather material	2
TDR Electronics	625 Lafayette Road (NNE 3-4)	Time delay relays	7
Hampton Falls			
Golden Eagle Coppersmiths	Lafayette Road (N 1-2)	Weathervanes and lanterns	10
Stillmeadow Glass Works	Lafayette Road (NW 1-2)	Blown glass for labs	9
Kensington			
None			
North Hampton			
Arc-Way Welding	203 Lafayette Road (N 4-5)	Steel fabrication	1
Giant Lift Equipment Co. Inc.	136 Lafayette Road (N 4-5)	Vertical lift equipment	16
LTP Enterprises Inc.	34 Lafayette Road (N 4-5)	Structure fiberglass	10

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [1]
Hampton Pattern Works	91 Post Road (N 5-6)	Wood and metal patterns	6
Seabrook			
Adhesive Machinery Corp. (Ornsteen Chemicals)	Folly Mill Road (WSW 2-3)	Hot melt adhesives and applicating equipment	38
Cargocaire Engineering Corp.	Route 107 (W 2-3)	Industrial dehumidifiers	20
Circle Machine Co.	Stard Road (W 1-2)	Shoe machinery	48
Hale Bros.	Stard Road (W 1-2)	Small chains	8
House of White Birches	Folly Mill Road (SW 1-2)	Publishing books and magazines	32
K. J. Quinn & Co.	Folly Mill Road (SW 1-2)	Industrial coatings and polyurathane elastomers	40
Rockingham Fireworks Manufacturing Co.	Lafayette Road (W 1-2)	Fireworks	4
Spherex Inc.	Walton Road (S 1-2)	Light duty wheels	75
Tower Press Inc.	Folly Mill Road (SW 1-2)	Magazine publishing	50
USM, Bailey Division	Lafayette Road (WSW 1-2)	Plastic, rubber and metal	930
Welpro Inc.	New Zealand Road (W 1-2)	Ladies shoes	350
Withey Press	Lafayette Road (SW 1-2)	Commercial printing	24

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees (1)
Protective Materials Corp.*	Folly Mill Road (WSW 2-3)	Firearms parts	25
D.G. O' Brien Inc.	1 Chase Park (W 1-2)	Electrical connector, atomic reactor parts	100
Amesbury Machine Shop	(W 1-2)		50
*Data from Town of Seabrook Planner			
South Hampton			
None			
Salisbury			
Austin Precision Tool	40 Ferry Street (S 4-5)	Precision parts and gages	4
Barton Corp.	40 Ferry Street (S 4-5)	Custom shipping boxes and crates	25
Manson Boat Works	68 Bridge Road (S 4-5)	Boat building and repairing	25
Tucker Machine Corp.	284 Elm St. Rte. 110 (SSW 4-5)	Screw machine products	10
Vaughn Corp.	386 Elm Street (SW 4-5)	Stonelined water heaters and tanks, solar heaters	65
Vaughn Woodworking Inc.	386 Elm Street (SW 4-5)	Wirebound boxes and crates	9
Weld Machine Corp.	47 Lafayette Road (SSW 2-3)	Machining, prototype hand screw milling	5
Elm Knoll Farm	240 Main Street (SW 3-4)	Lumber	3
Handicapped Artists	8 Sandy Lane (S 4-5)	Prints booklets, etc.	10

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [1]
Amesbury			
Advanced Absorber Products	10 Morrill Street (SW 4-5)	Microwave absorbers and radomes	21
Amesbury Chair	63 Clinton Street (WSW 4-5)	Chairs	5
Amesbury Metal Product	39 Oakland Street (SW 4-5)	Metal stamping, fluorescent lighting fixtures, metal plating	100
Vulcan Plastic Inc.	Noel St. (SW 5-6)	Injection molder and finisher heels	200
Amesbury Tool & Die Corp.	24 Oakland Street (SW 4-5)	Tool and die stampings	11
Bartley Machine and Manufacturing	Water Street (SW 4-5)	Machinery parts	19
Bocra Engineering	R Street (WSW 4-5)	Special tools and dies, jigs and fixtures	24
Cado Fabricating	144 Elm Street (SW 4-5)	Transit cases, consoles (machine work only)	65
Cargocaire Engineering	6 Chestnut Street (SW 4-5)	Dehumidifiers, heat exchangers	150
New Plant Building	Monroe Street (SW 4-5)		150
Dalton Manufacturing	5 Clark Street (WSW 4-5)	Display fixtures and educational materials	6
Durasol Drug & Chemical	1 Oakland Street (SW 4-5)	Erasers, dental adhesives, cleaners	20
Henschel Corp.	14 Cedar Street (WSW 4-5)	Marine signal systems, communication systems	150

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [1]
LeBaron-Bonney Co.	14 Washington (SW 4-5)	Upholstery and top product kits	55
MAT Reinforced Plastic	79 Elm Street (SW 4-5)	Molded fiberglass products	20
Merrimac Valley Foundry	58 Mill St. (SW 5-6)	Iron castings, brass, bronze aluminum	50
North Shore Weeklies	21 Elm Street (SW 4-5)	Newspapers and printing	60
Oakland Industries	11 Oakland Street (SW 4-5)	Sheet metal fabrication	35
R&G Manufacturing (Amesbury Chair)	63 Clinton Street (SW 4-5)	Metal kitchen cabinets	65
Reid Foundry	Mill Street (SW 5-6)	Grey iron castings	25
Sagamore Industrial Finishes	Rocky Hill Road (SW 4-5)	Industrial finishes	11
Scandia Plastic	36 High Street (WSW 5-6)	Extrusion of plastic tubing	32
Alexander Syvinski	38 Collins Avenue (SW 4-5)	Leather tanning and finishing	99
Dreamboat Corp.	10 Merrill Street (WSW 4-5)	Boat building and repairing	9
Whittier Press and North Shore Weeklies	21 Elm Street (SW 4-5)	Commercial printing	4
Brazonics, Inc.	Haverhill Road (SW 6-7)	Primary metals	80
Flexaust Company	Chestnut Street (SW 5-6)	Flexible hose	50

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees (1)
Haverhill Gas Company	Hunt Road (SW 6-7)	Natural gas	139
Maple Wood Products Co., Inc.	60 Merrimac Street (SW 5-6)	Toys and furniture	56
Michele Silverware & Jewelry Co., Inc.	36 Main Street (SW 5-6)	Jewelry	40
Microfab, Inc.	Haverhill Road (SW 6-7)	Printed circuit boards	190
Christesen Machine Co. Inc.	Haverhill Road (SW 6-7)	Machinery and parts	3
Country Kitchens	34 Pond Street (SW 5-6)	Kitchen and bath vanity cabinets	2
Denis Brass Foundry	250 Main Street (SW 5-6)	Brass and aluminum castings	10
R.E. Kimball & Co.	73 Merrimac Street (SW 5-6)	Jellies, jams, and relishes	3
Lowell's Boat Shop	459 Main Street (SW 5-6)	Boats	6
Erikson-Hedlund Stamponic Co.	39 Oakland Street (SW 4-5)	Tools, dies	8
The Old Newbury Crafters, Inc.	36 Main Street (SW 5-6)	Silverware	10
Merrimac			
Metal Finishing, Inc.	2 Littles Court (WSW 8-9)	Metal finishing	23
Engel-Lewis Counter Co, Inc.	Liberty Street (WSW 8-9)	Shoe counters	150

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [(1)]
Will-Mor Engineering Co, Inc.	27 E. Main Street (WSW 8-9)	Tools and machine parts	15
Newbury			
Newburyport Press, Inc.	80 Hanover Street (S 7-8)	Printing	18
Parker River Marine	Route 1A (S 9-10)	Marine equipment	6
Newburyport			
A Rhodes Co., Inc.	46 Water Street (S 6-7)	Shirts	27
Amesbury Specialty Co., Inc.	Parker Street (S 6-7)	?	50
Bay State Carbide Tool Corp.	126 Merrimac Street (SSW 5-6)	Tools	30
Berkshire Manufactured Products, Inc.	116 Parker Street (SSW 6-7)	Precision stampings	75
Circle Finishing, Inc.	Rt. 1 Traffic Circle (S 7-8)	Plating	22
Coca-Cola Bottling Co.	504 Merrimac Street (SSW 5-6)	Bottling	19
Contherm Corp.	Newburyport Turnpike (S7-8)	Heat exchangers	37
Geonautics, Inc.	44 Merrimac Street (SSW 5-6)	Plastic molds	40
Gould, Inc.	374 Merrimac Street (SSW 5-6)	Fuses	500
Kemtron Electron	14 Prince Place	Electronic components	25

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [1]
Products, Inc.	(S 6-7)		
Leary's Beverages, Inc.	504 Merrimac Street (SSW 5-6)	Bottling	80
M & V Electroplating Corp.	5 Greenleaf Street (S 6-7)	Electroplating	64
Newbury Tanning Corp.	12 Federal Street (S 6-7)	Leather finishing	80
Newburyport Daily News	23 Liberty Street (S 6-7)	Newspaper publishing	30
Owens-Illinois, Inc.	Parker Street (SSW 6-7)	Plastic products	200
S. Starensier, Inc.	5 Perkins Way (SSW 6-7)	Fabrics	99
Stride Rite Corp.	Perkins Way (SSW 6-7)	Footwear	100
Towle Mfg. Company	200 Merrimac Street (SSW 5-6)	Silverware	1800
Waverly News Co., Inc.	17 State Street (S 6-7)	Printing	22
Essex Tool & Die, Inc.	Bridge Road (SSW 5-6)	Precision tools and dies	5
International Light Inc.	Dexter Industrial Green (SSW 6-7)	Electro-optical instrumentation	19
Littlefield Press	2 Federal Street (S 6-7)	Commercial printing	9
Piel Craftsmen Co.	307 High Street (SSW 5-6)	Ship models	2
Rivco, Inc.	10 Prince Place (S 6-7)	Rivet setting tools	5

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [(1)]
Stem Chemicals, Inc.	7 Mulliken Way (SSW 7-8)	Chemicals	16
Lewis D. Bartley	7 Spofford Street (SSW 5-6)	Metal stampings	3
Alfa-Laval, Inc.	Route l (SSW 6-7)	Heat exchangers	37
West Newbury			
None			
Exeter			
Alrose Shoe Co., Inc.	1 Rockingham Street (NW 8-9)	Footwear	150
Brockhouse Corporation	Exeter Industrial Park (NW 8-9)	Metal fabrication	200
Chemtan Co., Inc.	Hampton Road (NW 7-8)	Leather chemicals	20
Clemson Automotive Fabrics	Chestnut Street (NW 7-8)	Textile finishing	200
Exeter Footwear, Inc.	93 Court Street (NW 7-8)	Women's footwear	100
Exeter Machine Products	Court Street (NW 7-8)	Screw machine products	22
Exeter News-Letter Co.	255 Water Street (NW 7-8)	Newpaper publisher	58
Blue Ribbon Sports, Inc.	156 Front Street (NW 8-9)	Sport shoes	110
GTE Sylvania, Inc.	Portsmouth Avenue (NNW 7-8)	Electrical equipment	500

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [(1)]
Ideal Tape Co.	Industrial Park, Off Epping Road (NW 8-9)	Tapes and adhesives	12
Prescott RE Mfg. Co., Inc.	10 Railroad (NW 8-9)	Pump equipment	12
Vaporpak, Inc.	Hampton Road (NW 7-8)	Fuel Catalyst System	20
Hampshire Controls	P.O. Box M (NW 7-8)	Electronic controls	N/A
Curtain Shop	43 Water Street (NW 7-8)	Draperies	5
Drew-It Corp.	256 Front Street (NW 8-9)	Can-crushers	5
Miljo Chemical Co., Inc.	94 Epping Road (NW 8-9)	Leather coatings	3
Squamscott Press	17 Court Street (NW 7-8)	Printing	2
Tyco Laboratories Inc.	Tyco Park (NW 7-8)	Electronic	33
Exeter & Hampton Electric Co.	225 Water Street (NW 7-8)	Electric light and power	139
Freedom Shoe Co., Inc.	15 Front Street (NW 7-8)	Sport shoes	N/A
Milliken & Company	Chestnut Street (NW 7-8)	Industrial cotton finishing	200
Wise Shoe Co., Inc.	156 Front Street (NW 8-9)	Shoes	300
Raw Thong Corp.	96 High Street (NW 7-8)	Rawhide laces	8

Name of Firm	Address (Sector)	Type of Manufacturing	Approximate Number of Employees [(1)]
Donnelly Mfg. Co.	Industrial Park, Epping Road (NW 8-9)	Sheet metal fabrication	N/A
Import Leather, Inc.	Industrial Park, Epping Road (NW 8-9)	Leather imports	N/A
Laurel Farms Dairy, Inc.	Pickpocket Road (NW 8-9)	Dairy	13
Regall Coatings, Inc.	94 Epping Road (NW 8-9)	Coatings for plastics	4
Greenland			
GTE Sylvania, Inc.	Route 101 (N 10)	Glass tubing	74
Ocean and Forest Products Co.	755 Portsmouth Ave. (N 9-10)	Sweeping compounds	7

(1) Sources
a. Directory of Massachusetts Manufacturers: 1981-82 Edition, George D. Hall Company, 1981
b. Directory of New England Manufacturers: 1980, New England Council, 1980
c. MacRae's New Hampshire State Industrial Directory: 1982, MacRae's Blue Book, Inc., 1981
d. Exeter, N.H. Industrial & Manufacturer's Guide and Exeter Grown Products, Exeter Area Chamber of Commerce, 1981

TABLE 2.1-14 SCHOOLS WITHIN 10 MILES OF THE SEABROOK SITE

TOWN	SCHOOL	GRADES	DISTANCE & DIRECTION (SECTOR)	(B) NO. OF STUDENTS	(C) NO. OF TEACHERS	(D) NO. OF STAFF	(A) TOTAL FALL 1979 POPULATION (B+C+D=A)
Hampton	Aslans' Pride Nursery School 200 High Street	N	NNE 3m (NNE 2-3)	20 - 3 da/wk 18 - 2 da/wk 38 students	3	0	41
Hampton	Center Elementary Sch. Winnacunnet Road	K-4	NNE 2½m (NNE 2-3)	351	25	12	388
Hampton	Marston Elementary Off of High Street	1-4	NNE 3m (NNE 2-3)	290	22	10	322
Hampton	Hampton Academy Jr. High School 29 Academy Avenue	5-8 s.p.	NNE 3m (NNE 2-3)	537	37	16	590
Hampton	Winnacunnet Coopera-tive High School Landing Road	9-12	NNE 2½m NNE(2-3)	88	36	1,442	1,318
Hampton	Sacred Heart Elementary	1-8	NNE 3m (NNE 2-3)	212	8	27	247
Hampton Falls	Hampton Falls Kinder-garten & Nursery Sch. Rt. 84	N&K	NW 1½m (NW 1-2)	38	3 full-time 1 part-time	0	42

TOWN	SCHOOL	GRADES	DISTANCE & DIRECTION (SECTOR)	(B) NO. OF STUDENTS	(C) NO. OF TEACHERS	(D) NO. OF STAFF	(A) TOTAL FALL 1979 POPULATION (B+C+D=A)
Hampton Falls	Lincoln-Ackerman Elementary Exeter Road	1-8	NW 1½m (NW 1-2)	178	15	12	205
Kensington	Kensington Elementary	1-6	WNW 5m (WNW 4-5)	156	6 full-time 9 part-time	17	188
North Hampton	Busy Beaver Kinder-garten 17 Pine Street	K	NNE 5m (NNE 4-5)	23 (divided into 2 shifts)	2	0	25
North Hampton	Montessori School of Creative Learning 229 Atlantic Avenue	Up to 14 years old	N 5½m (N 5-6)	25 morning 15 afternoon	2	3-5	47
North Hampton	N. Hampton Elementary Atlantic Ave	K-8	NNE 5¼m (NNE 5-6)	459	35	17	511
Seabrook	Seabrook Elementary and Jr. High Walton Road	K-8	S 1¼m (S 1-2)	671	44	24	739
South Hampton	Barnard Elementary Jewell Street	K-8	WSW 6m (WSW 5-6)	89	9	3	101
Amesbury	Amesbury Elementary S. Hampton Road	1-4	WSW 5m (WSW 4-5)	527	21	43	591

TOWN	SCHOOL	GRADES	DISTANCE & DIRECTION (SECTOR)	(B) NO. OF STUDENTS	(C) NO. OF TEACHERS	(D) NO. OF STAFF	(A) TOTAL FALL 1979 POPULATION (B+C+D=A)
Amesbury	Amesbury Middle Sch. Main Street	6-8	SW 5½m (SW 5-6)	701	43	26	770
Amesbury	Amesbury High School Highland Street	9-12	SW 5½m (SW 5-6)	840	5	22	927
Amesbury	Horace Mann School Congress Street	K	SW 4½m (SW 4-5)	207 (divided into 2 shifts)	12	4	223
Amesbury	Amesbury Country Day School 186 Market	Pre K & K	WSW 4½m (WSW 4-5)	125 (divided into 3 shifts)	4	2	131
Amesbury	Seventh Day Advent-ists School Monroe Street	1-8	SW 5m (SW 4-5)	14	1	3	18
Amesbury	Charles C. Cashman Friend Street	1-5	WSW 5¾m (WSW 5-6)	641	32	24	697
Salisbury	Kiddie Corner Nursery 16 John Street	N	SW 2½m (SW 2-3)	32	2	0	34

TOWN	SCHOOL	GRADES	DISTANCE & DIRECTION (SECTOR)	(B) NO. OF STUDENTS	(C) NO. OF TEACHERS	(D) NO. OF STAFF	(A) TOTAL FALL 1979 POPULATION (B+C+D=A)
Amesbury	Harbor Schools, Inc. (2 units) Pleasant Valley Rd.	Special	SW -7¼m (SW 7-8)	20	5	0	25
Amesbury	Miss Rose's Child Care Center Rte. 110 & Main Street	N-K + Daycare	SW 5½m (SW 5-6)	68	6	0	74
Salisbury	Salisbury Memorial Sch. (also called Jacob F. Spalding) Maple Street	K-6	S 4m (S 3-4)	654	28	19	701
Salisbury	Salisbury Plains School Main Street	K	SW 3m (SW 2-3)	90	3	0	93
Merrimac	Helen R. Donaghue School Union Street	3-6	WSW 9¼m (WSW 9-10)	323	23	3	347
Merrimac	Merrimac Child Care Center High Street	K	SW 8¾m (SW 8-9)	24	3		27
Merrimac	Red Oak School Church Street	K-3	SW 9¼m (WSW 9-10)	211	29		240

TOWN	SCHOOL	GRADES	DISTANCE & DIRECTION (SECTOR)	(B) NO. OF STUDENTS	(C) NO. OF TEACHERS	(D) NO. OF STAFF	(A) TOTAL FALL 1979 POPULATION (B+C+D=A)
Haverhill	Rocks Village School	K-3	SW 10m (SW 9-10)	27	2	1	30
Newbury	Newbury Elementary 63 Hanover Street	K-6	S 7½m (S 7-8)	382	15	8	405
Newbury	Woodbridge School Graham Avenue	1-2	S-7m (S 7-8)	91	4	3	98
Newbury	Harbor School 24 Rolfe's Lane	Special	S 6¾m (S 6-7)	22	6		28
Newbury	Harbor School 28 Rolfe's Lane	Special	S 6¾m (S 6-7)	24	7		31
Newburyport	Belleville School 333 High Street	K-4	SSW 6½m (SSW 6-7)	577	28	2	607
Newburyport	George W. Brown School Milk Street	K-4	S 6½m (S 6-7)	322	17	2	341
Newburyport	Davenport School Congress Street	K-4	SSW 6m (SSW 6-7)	105	5	2	112
Newburyport	Kelley School 149 High Street	K-4	SSW 6¼m (SSW 6-7)	116	7	2	125

TOWN	SCHOOL	GRADES	DISTANCE & DIRECTION (SECTOR)	(B) NO. OF STUDENTS	(C) NO. OF TEACHERS	(D) NO. OF STAFF	(A) TOTAL FALL 1979 POPULATION (B+C+D=A)
Newburyport	Ruppert A. Nook Middle Sch. Low Street	5-8	SSW 6½m (SSW 6-7)	966	68	17	1,051
Newburyport	Newburyport High School 241 High Street	9-12	SSW 6m (SSW 6-7)	871	56	15	942
Newburyport	Immaculate Conception Green & Washington Streets	1-8	S 6½m (S 6-7)	182	NA	Approx.---20	202
Newburyport	Living & Learning School 151 Low Street	N-K	SSW 6½m (SSW 6-7)	90	13		103
Newburyport	Mrs. Haley's Pre-school 29 Marlboro Street	N-K	S 6½m (S 6-7)	NA	NA		NA
Newburyport	My School YMCA - State Street	N-K	S 6½m (S 6-7)	24	4		28
Newburyport	Spring Street School 6 Parsons Street	N-3	S 6½m (S 6-7)	39	6		45
Newburyport	The First School 893 Main St., W. Newbury	K-3	SW 7½m (SW 7-8)	11		2 (will expand 3-4 per year)	13

TOWN	SCHOOL	GRADES	DISTANCE & DIRECTION (SECTOR)	(B) NO. OF STUDENTS	(C) NO. OF TEACHERS	(D) NO. OF STAFF	(A) TOTAL FALL 1979 POPULATION (B+C+D=A)
Newburyport	The Children's House 23 Chapel Street	N-K	SSW 5¾m (SSW 5-6)	24	3		27
Newburyport	Mrs. Murray's Nursery Sch. 13 Federal Street	N-K	S 6¼m (S 6-7)	60	4		34
Greenland	Central School Post Road	1-8	N 9¼m (N 9-10)	312	20	6	338
W. Newbury	Central Grammar School 381 Main Street	1, 2	SW-10m (SW 9-10)	141	7	5	153
W. Newbury	Dr. Page School 694 Main Street	3-6	SW 8¼m (SW 8-9)	228	12	7	247
Rye	Elementary School 461 Sagamore Road	1-5	NNE 10m (NNE 9-10)	200	20		220
Rye	Rye Junior High School 501 Washington Road	6-8	NNE 9¼m (NNE 9-10)	300	20		320
Stratham	Memorial School Bunker Hill Avenue	1-6	NNW 8½m (NNW 8-9)	251	17	7	275
Stratham	Acorn School Winnicut Road	K-3	N 8½m (N 8-9)		4	(May Expand)	55

TOWN	SCHOOL	GRADES	DISTANCE & DIRECTION (SECTOR)	(B) NO. OF STUDENTS	(C) NO. OF TEACHERS	(D) NO. OF STAFF	(A) TOTAL FALL 1979 POPULATION (B+C+D=A)
E. Kingston	Elementary School Andrews Lane	3-6	WNW 8½m (WNW 8-9)	92	9		101
E. Kingston	Brown's Academy	1, 2	WNW 8½m (WNW 8-9)	35	2		37
Newton	Teddy Bear Nursery School 40 Highland Road	N	W 10m (W 9-10)	54	4		58
Exeter	Montessori School of Exeter 8 Center Street	N-K	NW 7 ⅔m (NW 7-8)	40	4	(May Expand)	44
Exeter	Rockingham School for Special Children 40 Lincoln Street	Special	NW 7¼m (NW 7-8)	41	NA	NA	NA
Exeter	Richie-McFarland Children's Center, II Prospect Ave.	2-6 yrs.	NW 7¼m (NW 7-8)	20	4	3	27
Exeter	Philips-Exeter Academy	9-12	NW 7¼m (NW 7-8)	970	125	NA approx. 100	1,195
Exeter	Exeter Day Care Center 261 Water Street	N	NW 8m (NW 7-8)	45	9		54

TOWN	SCHOOL	GRADES	DISTANCE & DIRECTION (SECTOR)	(B) NO. OF STUDENTS	(C) NO. OF TEACHERS	(D) NO. OF STAFF	(A) TOTAL FALL 1979 POPULATION (B+C+D=A)
Exeter	Exeter Day School 6 Marlboro Street	3-5 yrs.	NW 7¼m (NW 7-8)	125	8		132
Exeter	Exeter Elementary School Lincoln Street	1 & 2 3 - 6	NW 7½m (NW 7-8)	672 271	75 50		742 321
Exeter	High School Linden Street	9-12	NW 7½m (NW 7-8)	1,305	90		1,395
	Vocational High School (Planned Opening 9/80)	11 & 12	NW 7½m (NW 7-8)	690 max.- 2 sessions	20-40	12	742
Exeter	Jr. High School	6-8	NW 7½m	630	50		680
Exeter	Main Street School Main Street	1,2 special	NW 7½m (NW 7-8)	291	20		311
Exeter	School Street School School Street		NW 7¾m (NW 7-8)	Current Use	Office Space		
Exeter	ABC Preschool 16 Ridgecrest Drive	3-6 yrs.	NW 7¾m (NW 7-8)	32	2		34
Exeter	Child Garden Country Day 9 Chestnut Street	K-3	NW 7¾m (NW 7-8)	NA	NA		NA

TABLE 2.1-15 MEDICAL RELATED FACILITIES WITHIN 10 MILES OF THE SEABROOK PLANT

Name and Type	Location (Sector)	Bed Capacity (Estimated Population)	Planned Expansion
		NEW HAMPSHIRE	
		Hampton	
Odyssey House (Medical and Therapeutic Treatment)	30 Winnacunnett (NNE 2-3)	40 adolescents, 15 staff (55 persons)	None
Seacoast Health Center Inc. (nursing home)	22(NNE 3-4) Tuck Rd	76 beds, staff estimate NA (77 persons)	None
		Exeter	
Exeter Hospital	10 Buzell Ave. (NW 7-8)	100 beds (110 beds), 400 staff (510 persons)	None
Court Street Unit	131 Court St. (NW 7-8)	100 beds, 100 staff (200 persons)	None
Eventide Home	81 High St. (NW 7-8)	21 beds, 12 nurses, 13 staff (46 persons)	None
Goodwin's of Exeter	Hampton Rd. (NW 7-8)	75 beds, 75 staff (150 persons)	None
		MASSACHUSETTS	
		Salisbury	
Greenleaf House Nursing Home	335 Elm St. (SSW 4-5)	60 beds, 60 staff (120 persons)	None
		Amesbury	
Amesbury Hospital	Highland Ave. (SW 5-6)	63 bed max., 47 avg., 230 staff (Total 293)	None
Amesbury Nursing and Retirement Home	22 Maple St. (WSW 5-6)	124 beds, 110 staff (234 persons)	None
Hillside Nursing Home	29 Hillside (SW 5-6)	26 beds, 8 staff (34 persons)	None
Maplewood Manor Nursing Home	Morrill Pl. (SW 5-6)	120 beds, 100 staff (220 persons)	None
Eastwood Rest Home	39 High St. (SW 5-6)	33 beds, 12 staff (45 persons)	None
North Eastwood Rest Home	276 Main (SW 5-6)	20 beds, 10 staff (26 persons)	Possible expansion of 17

Name and Type	Location (Sector)	Bed Capacity (Estimated Population)	Planned Expansion
			beds, 10 staff
Parkside Rest Home	56 Sparhawk (SW 5-6)	30 beds, 8 staff (38 persons)	None
	Newburyport		
Anna Jacques Hospital	Highland Ave. (SSW 6-7)	104 beds, 520 staff (624 persons)	33 bed med/ surg. addition sched.to begin 3/ 1/80
Brigham Manor Nursing Home	77 High St. (S 6-7)	64 beds, 60 staff (124 persons)	None
Country Manor Convalescent Home, Inc	Low St. (SSW 6-7)	123 beds, 100 staff (223 persons)	None
Newburyport Manor Chronic Hospital	Low & Hale St. (SSW 6-7)	102 beds, 100 staff (202 persons)	None
Worcester Park Nursing Home	351 High St. (SSW 5-6)	68 beds, 56 staff (124 persons)	None
Home for Aged Men (Newburyport Society)	361 High St. (SSW 5-6)	9 residents, 8 full and part-time staff (17 persons)	None
Line House Treatment Center	37 Washington (SSW 6-7)	12 beds, 5 staff (17 persons)	None
Home for Aged Women (Newburyport Society)	75 High St. (S 6-7)	10 residents, 9 full and part-time staff (19 persons)	None

TABLE 2.1-16 **ESTIMATE OF 1970 SEASONAL UNITS FOR COUNTIES WITHIN 50 MILES**

County	Population	All Housing Units	Vacant-- Seasonal & Migratory
MASSACHUSETTS			
Essex	637,887	216,201	5,540
Middlesex	1,397,268	431,012	766
Suffolk	735,190	264,471	249
Norfolk	605,051	181,192	561
Plymouth	333,314	110,662	9,182
Worcester	637,969	204,083	2,931
NEW HAMPSHIRE			
Carrol	18,548	14,838	5,830
Belknap	32,367	16,230	3,604
Merrimack	80,925	29,250	2,607
Hillsborough	223,941	74,666	2,126
Strafford	70,431	23,874	1,810
Rockingham	138,951	53,132	8,274
MAINE			
York	111,576	48,530	9,373

Source: 1970 U.S. Census of Housing, Maine, Massachusetts, and New Hampshire (Table 60).

TABLE 2.1-17 **EMPLOYMENT STATISTICS BY PLACE OF RESIDENCE**

County	1940 Total Employment	1970 Total Employment	1970 Total Employment Inside 50-Mile Radius
MAINE			
York	31,191	44,780	33,585
MASSACHUSETTS			
Essex	181,438	263,698	263,698
Middlesex	337,385	586,238	480,715
Norfolk	116,659	250,241	77,574
Plymouth	58,289	127,539	8,928
Suffolk	300,915	309,594	309,594
Worcester	181,080	266,609	2,666
NEW HAMPSHIRE			
Belknap	8,247	13,235	5,426
Carroll	4,805	7,298	365
Hillsborough	54,132	92,818	44,553
Merrimack	20,935	33,137	15,574
Rockingham	20,616	56,820	56,820
Strafford	16,489	28,823	28,823
		TOTAL	1,328,321

Source: Regional Employment by Industry, 1940-1970,
U.S. Department of Commerce

TABLE 2.1-18 **SUMMARY OF 1980 PEAK TRANSIENT POPULATION ESTIMATES WITHIN 0 TO 10 MILES OF THE SITE**

			0-5 Miles	5-10 Miles	0-10 Miles
(1)	Seasonal Resident				
	(a)	Weekend Day	30,500	12,512	43,012
	(b)	Weekday	(21,669)	(8,886)	(30,555)
(2)	Overnight Visitors				
	(a)	Hotels, Motels and Guest houses	10,019	1,005	11,024
	(b)	Camping	3,160	4,488	7,648
(3)	Daily Transient				
	(a)	Fee and Free Lots and Metered On- Street Parking	30,441	12,233	42,674
	(b)	"On-Street" Parking	10,246	2,384	12,630
Total Seasonal Resident, Overnight Visitors and Daily Transients			84,366	32,622	116,988
Total Permanent Population 1980			32,060	68,660	100,720
Total Peak Transient & Permanent Population			116,426	101,282	217,708

TABLE 2.1-19 PROJECTED RESIDENT POPULATION WITHIN THE LOW POPULATION ZONE

	Year	0-1 Mile	1-1.25 Miles	Totals 0-1.25 Miles
N	1980	20	0	20
	1983	20	0	20
	1990	20	0	20
	2000	20	0	20
	2010	20	0	20
	2020	20	0	20
	2025	20	0	20
NNE	1980	0	0	0
	1983	0	0	0
	1990	0	0	0
	2000	0	0	0
	2010	0	0	0
	2020	0	0	0
	2025	0	0	0
NE	1980	0	0	0
	1983	0	0	0
	1990	0	0	0
	2000	0	0	0
	2010	0	0	0
	2020	0	0	0
	2025	0	0	0
ENE	1980	0	0	0
	1983	0	0	0
	1990	0	0	0
	2000	0	0	0
	2010	0	0	0
	2020	0	0	0
	2025	0	0	0
E	1980	0	0	0
	1983	0	0	0
	1990	0	0	0
	2000	0	0	0
	2010	0	0	0
	2020	0	0	0
	2025	0	0	0
ESE	1980	0	0	0
	1983	0	0	0

	Year	0-1 Mile	1-1.25 Miles	Totals 0-1.25 Miles
	1990	0	0	0
	2000	0	0	0
	2010	0	0	0
	2020	0	0	0
	2025	0	0	0
SE	1980	0	0	0
	1983	0	0	0
	1990	0	0	0
	2000	0	0	0
	2010	0	0	0
	2020	0	0	0
	2025	0	0	0
SSE	1980	10	0	10
	1983	10	0	10
	1990	20	0	20
	2000	20	0	20
	2010	20	0	20
	2020	30	0	30
	2025	30	0	30
S	1980	120	70	190
	1983	140	80	220
	1990	170	100	270
	2000	210	120	330
	2010	250	140	390
	2020	310	170	480
	2025	340	190	530
SSW	1980	250	70	320
	1983	280	80	360
	1990	340	100	440
	2000	410	120	530
	2010	500	140	640
	2020	610	170	780
	2025	680	190	870
SW	1980	60	260	320
	1983	60	280	340
	1990	80	350	430
	2000	100	420	520
	2010	120	520	640
	2020	140	630	770

	Year	0-1 Mile	1-1.25 Miles	Totals 0-1.25 Miles
	2025	160	700	860
WSW	1980	0	70	70
	1983	0	80	80
	1990	0	100	100
	2000	0	120	120
	2010	0	150	150
	2020	0	180	180
	2025	0	200	200
W	1980	110	510	620
	1983	120	570	690
	1990	140	630	770
	2000	170	710	880
	2010	210	810	1,020
	2020	260	920	1,180
	2025	290	1,000	1,290
WNW	1980	170	90	260
	1983	180	90	270
	1990	190	110	300
	2000	210	110	320
	2010	240	110	350
	2020	270	110	380
	2025	280	110	390
NW	1980	20	40	60
	1983	30	40	70
	1990	30	50	80
	2000	30	50	80
	2010	30	50	80
	2020	30	50	80
	2025	30	50	80
NNW	1980	30	80	110
	1983	30	80	110
	1990	30	100	130
	2000	30	100	130
	2010	40	100	140
	2020	40	100	140
	2025	40	100	140

	Year	0-1 Mile	1-1.25 Miles	Totals 0-1.25 Miles
Totals	1980	790	1,190	1,980
	1983	870	1,300	2,170
	1990	1,020	1,540	2,560
	2000	1,200	1,750	2,950
	2010	1,430	2,020	3,450
	2020	1,710	2,330	4,040
	2025	1,870	2,540	4,410

TABLE 2.1-20 MAJOR TRANSIENT POPULATION ELEMENTS WITHIN THE LOW POPULATION ZONE

Facility Name or Description	Estimated Distance and Direction	Peak Occupancy* or Population
Bailey Division	1 mile--WSW	1,000
Shopping Center	1 mile--W	1,400
Shopping Center	1¼ miles--SW	1,800
Seabrook Elementary And Jr. High School	1¼ miles--S	739
Hawaiian Garden Motor Inn	1 mile--W	52
Sleepy Hollow Motel	1¼ miles--WSW	34
Village Motel	1¼ miles--WSW	22
Spruce Manor	1¼ miles--WSW	34

* 1978 Estimates

TABLE 2.1-21 **CUMULATIVE RESIDENT POPULATION DENSITY**

Distance (Miles)	Density (Persons Per Square Mile) 1983	2025
0-1	277	596
0-2	532	1155
0-3	525	1043
0-4	443	836
0-5	442	802
0-10	342	685
0-20	294	490
0-30	406	575
0-40	527	706
0-50	505	658

Table 2.2-1 SUMMARY OF HAZARDOUS MATERIALS USAGE

A.	Henkel Corporation	W 2	Adhesives, sealants, paintings and coatings manufacturer. For a complete list of chemicals used and company information, see Reference 85.
B.	US Foods	W 2	Foodservice distributor. For a complete list of chemicals used and company information, see Reference 85.
C.	Foss Manufacturing	N 2.8	Nonwoven fabrics manufacturer. For a complete list of chemicals used and company information, see Reference 85.
D.	Mackenzie Fuels	W 2.2	Supplier of home heating oil and propane. For a complete list of chemicals used and company information, see Reference 85.
E.	Brazonics, Inc.	NE 2.5	Manufacturer of aluminum brazed assemblies. For a complete list of chemicals used and company information, see Reference 85.
F.	Eastern Propane Gas, Inc.	N 1.6	Propane, see Reference 85.
G.	Aero Dynamics, Inc.	W 2	Electroplating and metal finishing of products. For a complete list of chemicals used and company information, see Reference 85.
H.	Giant Lift Equipment	N 7.4	Manufacturer of lifts and lifting devices. For a complete list of chemicals used and company information, see Reference 85. Included, although outside of 5 mile radius, due to large quantity.

Table 2.2-2 LIST OF MANUFACTURING FIRMS WITHIN FIVE MILES OF THE SITE

Deleted

TABLE 2.2-3 TABLE OF DISTANCES FOR STORAGE OF LOW EXPLOSIVES (CLASS B)

Pounds Over	Pounds Not Over	Inhabited Buildings Distance (Feet)	Public Rail- Road and Highway Distance (Feet)	Aboveground Magazine (Feet)
0	1,000	75	75	50
1,000	5,000	115	115	75
5,000	10,000	150	150	100
10,000	20,000	190	190	125
20,000	30,000	215	215	145
30,000	40,000	235	235	155
40,000	50,000	250	250	165
50,000	60,000	260	260	175
60,000	70,000	270	270	185
70,000	80,000	280	280	190
80,000	90,000	295	295	195
90,000	100,000	300	300	200
100,000	200,000	375	375	250
200,000	300,000	450	450	300

TABLE 2.2-4 TRAFFIC COUNT, PEASE AIR FORCE BASE 1979

Month	IFR Arrival	IFR Departure	Stg II	IFR Satellite	Over Flight	VFR Service	Total
Jan.	1,270	1,308	26	14	1,099	64	3,781
Feb.	1,431	1,620	25	7	1,179	60	4,322
Mar.	1,347	1,599	30	32	1,584	128	4,720
Apr.	1,303	1,579	31	26	1,510	283	4,732
May	1,429	1,647	46	41	1,623	348	5,134
June	1,340	1,626	86	34	2,660	377	6,123
July	1,171	1,373	32	49	2,470	365	5,460
Aug.	1,533	2,492	27	106	2,415	523	7,096
Sept.	1,113	1,465	48	60	2,413	438	5,537
Oct.	1,450	1,662	33	57	1,481	799	5,482
Nov.	1,195	1,291	21	27	1,322	492	4,348
Dec.	1,006	1,052	23	40	943	813	3,877
TOTAL	15,588	18,714	428	493	20,699	4,690	60,612

IFR Arrival - Any IFR arrival, practice or termination to Pease terminal control area.

IFR Departure - Any IFR departure from Pease terminal control area.

Stage II - VFR A/C landing or departing PAFB requesting spacing.

IFR Satellite - Pertains to instrument approach to satellite airports within the Pease terminal area.

Over Flight - Any VFR A/C requesting advisories and all IFR A/C below 5,000 feet within approximately 40 miles of PAFB (in general, A/C passing through the PAFB control zone).

VFR Service - A/C terminating or originating at the Pease terminal control area requesting advisories.

TABLE 2.2-5 INSTRUMENT APPROACHES AT PEASE AIR FORCE BASE

Type Aircraft	Sept(79)	Nov(79)	Dec(79)	Jan(80)	Feb(80)	Mar(80)	6 Month Total
FB-111A	221	366	298	386	406	413	2,090
KC-135	494	430	382	442	457	373	2,580
C-130	61	83	50	81	38	28	341
C-141	15	1	7	38	13	6	80
C-123, F4, C5A, P3, B57, B52	19	7	19	42	28	35	150
Other*	317	234	282	216	264	277	1,590
TOTAL	1,127	1,162	1,038	1,075	1,206	1,132	6,831

* Other indicates aircraft weighing equal to or less than 12,500 pounds.

TABLE 2.2-6 ANNUAL LANDINGS TO RUNWAY 34 (PEASE AIR FORCE BASE)

Type Aircraft	Total Approaches (6 months)	Annual Approaches	Expected Number of Approaches to Rwy. 34
FB-111A	2,090	4,180	2,352
KC-135	2,580	5,160	2,903
C-130	341	682	384
C-141	80	160	90
C-123, F4, C5A, P3, B57, B52	150	300	169

TABLE 2.2-7 ACCIDENT RATE BY AIRCRAFT TYPE - 1968 - 31 OCT 1979

YEAR	TOTAL HOURS FLOWN	NUMBER DESTROYED	ACCIDENT (DESTROYED) RATE/100,000 HRS
		FB-111A	
1968	0	0	0.0
1969	0	0	0.0
1970	3,973	1	25.2
1971	18,481	0	0.0
1972	23,186	0	0.0
1973	16,935	0	0.0
1974	18,821	0	0.0
1975	17,381	3	17.3
1976	17,822	1	5.6
1977	17,729	2	11.3
1978	16,469	0	0.0
1979	15,021	1	6.7
TOTAL	165,818	8	4.8
		C-130	
1968	593,976	6	1.0
1969	537,126	4	.7
1970	504,113	3	.6
1971	487,137	1	.2
1972	480,989	5	1.0
1973	399,605	1	.3
1974	360,549	3	.8
1975	365,181	2	.5
1976	336,592	0	0.0
1977	334,524	0	0.0
1978	348,168	5	1.4
1979	302,828	0	0.0
TOTAL	5,050,788	30	.6
		KC-135	
1968	502,467	5	1.0
1969	431,849	4	.9
1970	376,930	0	0.0
1971	372,410	2	.5
1972	438,029	1	.2
1973	329,410	1	.3
1974	296,320	1	.3
1975	266,522	1	.4
1976	259,785	2	.8
1977	262,304	2	.8
1978	271,819	0	0.0
1979	227,071	1	.4
TOTAL	4,034,916	20	.5

TABLE 2.2-8 DESTROYED AIRCRAFT BY PHASE OF OPERATION 1968 - 30 NOVEMBER 1979

YEAR	FB-111A	C-130	KC-135
1968		*Landing final	*Initial climb
		*Landing	Cruise
		*Landing roll	*Landing
		*Landing	*Takeoff roll
		*Landing	Descent
		*Landing	
1969		*Cruise	*Landing roll
		*Landing final	*Takeoff roll
		*Cruise	Cruise
		*Initial climb	Cruise
		*Initial climb	
1970	Landing approach	Prolonged climb	*Takeoff roll
		Cruise	
		Cruise	
1971		*Takeoff	Landing initial
			Cruise
1972		Landing	*Landing
		*Go-around	
		*Takeoff	
		*Takeoff	
1973		Prolonged climb	
1974		Initial climb	*Initial climb
		*Landing roll	
		Cruise	
1975	*Night formation (2 A/C)	*Initial climb	*Initial climb
	*Normal cruise	Cruise	
1976	Descent		Descent
			Descent
1977	Normal cruise		*Landing roll
	Normal cruise		Initial climb
1978		*Landing pattern	
		*Landing final	
		Landing pattern	
		Cruise	
		Landing pattern	
1979	*Normal cruise		Takeoff

* Indicates aircraft eliminated when calculating accident rate.

Note: A detailed description of the above accidents is available through Norton Air Force Base.

TABLE 2.2-9 GROSS WEIGHT OF FB-111A AT HIGH FIX (WARNI) (PEASE AIR FORCE BASE)

Minimum FB-111A Weight: 56,500 pounds

Maximum FB-111A Weight: 86,000 pounds

Average FB-111A Weight: 67,849 pounds

Number of FB-111A Weighting More Than 81,800 pounds = 4

Date and Weight of FB-111A Exceeding 81,800 pounds

21 Nov. 1979	86,000 pounds
27 Nov. 1979	83,700 pounds
15 July 1980	82,400 pounds
7 Aug. 1980	83,800 pounds

Annual Estimated Number Arriving at WARNI Weighing More Than 81,800 pounds = 12

Note: Flight Plans Examined (dates)

1 - 30 Nov. 1979

22 May - 31 May 1980

1 - 30 June 1980

1 - 31 July 1980

1 - 21 Aug. 1980

TABLE 2.2-10 **PEASE SECTOR PEAK DAY AIR TRAFFIC COUNT[1] MAXIMUM AIRCRAFT TAKEOFF WEIGHT LESS THAN 80,000 POUNDS**

AIRCRAFT TYPE	WEIGHT (LBS.)	CRUISING SPEED (MPH)	AVERAGE ALTITUDE (Ftx100)	NUMBER OBSERVED
BE99	4,717	254	83.0	20
AC69	9,400	278	250.0	1
PA31	6,500	248	66.6	25
DH6	12,500	210	78.0	10
MO20	2,740	185	60.0	1
LR25	15,000	534	350.0	6
PA2T	5,200	277	76.0	4
C172	2,550	153	70.0	3
PA28	2,325	146	63.3	4
H3	2,134	155	80.0	1
MU2	11,575	365	180.0	5
BE18	4,490	336	50.0	1
C500	4,309	647	90.0	2
AA5	2,400	160	70.0	1
N265	19,615	563	290.0	2
PA32	3,400	180	70.0	1
C337	4,630	195	65.0	2
C210	1,542	356	100.0	1
C206	3,600	169	50.0	2
K200	12,500	320	175.0	1
LR35	15,000	534	316.6	3
C340	5,990	267	190.0	1
BE60	6,775	275	150.0	1
MO21	2,575	169	110.0	1
C421	7,450	242	90.0	1
LR24	13,500	545	280.0	2
A4	24,500	646	155.0	2
F27	45,000	298	150.0	3
A6	60,400	477	230.0	1
T37	6,574	425	250.0	1
BE90	10,100	287	200.0	1
BE20	12,500	320	230.0	2
*L329	---	---	250.0	2
WW24	12,495	320	100.0	1
AC50	9,500	403	60.0	1
C501	11,850	403	70.0	1

* Weight is assumed to be more than 12,500 pounds.

(1) Date of peak air traffic count: July 27, 1979

TABLE 2.2-11 **PEASE SECTOR PEAK DAY AIR TRAFFIC COUNT[1] MAXIMUM AIRCRAFT TAKEOFF WEIGHT EQUAL TO OR GREATER THAN 80,000 POUNDS**

AIRCRAFT TYPE	WEIGHT(LBS.)	CRUISING SPEED (MPH)	AVERAGE ALTITUDE (Ftx100)	NUMBER OBSERVED
C141	344,900	350	362.7	13
B747	820,000	608	359.3	16
B727	209,500	599	252.0	18
B707	333,600	605	354.0	12
C5	764,500	541	336.6	4
DC10	572,000	573	350.0	10
DC8	350,000	565	318.5	9
IL62	363,760	560	376.6	3
L1011	496,000	500	276.6	3
VC15	335,000	581	390.0	1
P3	135,000	473	230.0	16
C130	155,000	375	260.0	1
DC9	121,000	564	190.0	1

[1] Date of peak air traffic count: July 27, 1979

TABLE 2.2-12 **BACKGROUND DATA – PROPANE SPILL PROBLEM**

[redacted]		[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	[redacted]	[redacted]	[redacted]	[redacted]
	[redacted]	[redacted]	[redacted]	[redacted]
	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]

[redacted] [redacted]

TABLE 2.2-13 CONCENTRATION AND METEOROLOGICAL DATA FOR PROPANE SPILL

						[redacted]		
[redacted]	[redacted]			[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	[redacted]	[redacted]	[redacted]					
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]	[redacted]

TABLE 2.2-14 **CONTINUOUS RELEASE ANALYSIS ASSUMPTIONS AND PARAMETERS FOR FLAMMABLE VAPOR CLOUDS**

[redacted]	
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted] [redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted] [redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]

[redacted]

TABLE 2.2-15 **PUFF RELEASE ANALYSIS PARAMETER VALUES**

[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]
[redacted]	[redacted]

[redacted] [redacted]

[redacted] [redacted]

[redacted] [redacted]

TABLE 2.2-16 PROBABILITY DATA FOR A WORST CASE DEFLAGRATION

[redacted]				
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]		[redacted]		
			[redacted]	[redacted]

TABLE 2.2-17 **PROBABILITY DATA FOR A WORST CASE DETONATION**

		[redacted]		
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	[redacted]	[redacted]	[redacted]	[redacted]
	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]		[redacted]		[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
	[redacted]	[redacted]	[redacted]	[redacted]
	[redacted]	[redacted]	[redacted]	[redacted]
			[redacted]	[redacted]
[redacted]			[redacted]	[redacted]

TABLE 2.3-1 **OFFSITE METEOROLOGICAL INSTRUMENTATION INFORMATION**

a. – Boston NWS

STATION LOCATION BOSTON, MASSACHUSETTS

						Sea Level	Elevation Above Ground									
Location	Occupied from	Occupied to	Airline distance and direction from previous location	Latitude North	Latitude West	Ground at temperature site	Wind instruments	Extreme thermometers	Psychrometer	Telepsychrometer	Tipping bucket rain gage	Weighing Rain Gage	8" Rain Gage	Hygrothermometer	*	Remarks
City																
Old State House, corner State & Devonshire Sts.	10/20/70	1/09/71		42º21′	71º04′	16										Ground elevation approximate.
103 Court Street	1/10/71	8/12/75	600 ft. NW	42º21′	71º04′	40										Ground elevation approximate.
Equitable Building Corner Milk & Devonshire Streets	8/12/75	10/01/84	1200 ft. SE	42º21′	71º04′	12	172	156	156				162			
Old U.S. Post Office and Courthouse Milk, Devonshire, Congress & Water Streets East Tower	10/01/84	6/07/29	300 ft. NE	42º21′	71º04′	17	188	115	115			154	174 154			8 inch rain gage moved from bad exposure atop east tower to west tower, 154 feet above ground on 7/1/91. Marvin Weighing Rain and Snow Gage installed
Young's Hotel Building Corner City Hall Avenue and Court Street	6/07/29	9/29/33	700 ft. NW	42º21′	71º04′	40	165	106	106		96		96			Anemometer atop City Hall Annex, across City Hall Avenue.
New U.S. Post Office and Courthouse Same site as old	9/29/33	6/06/64	700 ft. SE	42º21′	71º04′	20	360	337	336		329		328			Observation Program transferred to Airport 1/1/36.
Airport																
U.S. Army Hanger No. 1 Boston Airport East Boston	10/15/26	4/01/27		42º22′	71º02′	3										Pibal only.
Section F, Army Base South Boston	4/01/27	11/01/27	1-¾mi. S	42º21′	71º02′		143									Pibal only.
Shack 25 feet South of Commercial Hanger Boston AP, East Boston	11/01/27	7/01/29	1-¾mi. N	42º21′	71º02′	2	22		4							Pibal only

* Requests for information concerning solar radiation data or instrumentation should be made to the Director, National Climatic Center, Federal Building, Asheville, NC 28801

STATION LOCATION — BOSTON, MASSACHUSETTS

						Elevation Above										
						Sea Level	Ground									
Location	Occupied from	Occupied to	Airline distance and direction from previous location	Latitude North	Latitude West	Ground at temperature site	Wind instruments	Extreme thermometers	Psychrometer	Telepsychrometer	Tipping bucket rain gage	Weighing Rain Gage	8" Rain Gage	Hygrothermometer		Remarks
Shack 200 feet SW of East Coast Hanger Boston AP, East Boston	7/01/29	5/01/30	1/8 mi. SW	42º22′	71º02′	12	24		4							Pibal only to 2/16/30.
Administration Building Boston Municipal airport East Boston	5/01/30	11/01/45	3/8 mi. NW	42º22′	71º02′	12	50	31	31		a3	b3	3			a– Added 1/1/36. b – Added 2/1/38. Official synoptic records began 1/1/36.
Administration Building Boston Municipal Airport East Boston	11/01/45	11/22/51	Same	42º22′	71º02′	12	*62	*33	*33		#32	#32	#32			* - Installed on 30 foot instrument tower on roof 9/17/37. # - Gages moved to roof 3/10/44.
Gate No. 11, Boutwell Building, Logan Int'l. Airport, East Boston	11/22/51	12/05/63	5/8 mi. E	42º22′	71º01′	15	X33	20	20		19	19	18			x – 34 feet to 7/20/54 and 75 feet to 8/23/57.
General Aviation Admin. Building, West Wing, Logan International AP	12/05/63	Present	5/8 mi. W	42º22′	71º02′	d15	22	e33 f6	e33 f5		33 f5	33 f5	33 f5	c4		Instrument relocations completed 12/11/63. c – commissioned on field site 4/1/64. d – 12 FT TO 4/1/64. e – Standby status after 4/1/64. f – Effective 8/5/71.

b. Portland NWS

STATION LOCATION — PORTLAND, MAINE

Fort Preble	1-1820	12-1836		43º39′	70º14′	53										Surgeon General Station.
Fort Preble	10-1840	8-1845		43º39′	70º14′											Surgeon General Station.
Fort Preble	1-1849	12-1853		43º39′	70º14′											Surgeon General Station.
Portland	1856	1859		43º39′	70º15′											Cooperative Station.
Fort Preble	1865	1871		43º39′	70º14′											Cooperative Station.
City																
4 Exchange Street	7/15/71	9/30/73		43º39′	70º15′	30	?	40					7			
Boyd's Block, Middle & Exchange Streets	9/30/73	12/01/74	450 ft. NW	43º39′	70º15′	51	?	50					7			
Custom House, Fore St.	12/01/74	7/01/85	450 ft. E	43º39′	70º15′	32	82	28					71			
First National Bank 57 Exchange Street	7/01/85	12/04/40	350 ft. WNW	43º39′	70º15′	47	117*	81	#81		Ø75		77			Ø – 74 feet 1/94 to 4/97 * - 89 ft 5/22/93 to 11/28/99 # - Added January 1889.

STATION LOCATION PORTLAND, MAINE

						Sea Level	Elevation Above Ground									
Location	Occupied from	Occupied to	Airline distance and direction from previous location	Latitude North	Latitude West	Ground at temperature site	Wind instruments	Extreme thermometers	Psychrometer	Telepsychrometer	Tipping bucket rain gage	Weighing Rain Gage	8" Rain Gage	Hygrothermometer		Remarks
Airport																
Administration Building Portland City Airport General Aviation Terminal effective Oct. 1968 Portland International Jetport, Sept. 1969	12/03/39	Present	2.75 miles West of City Office	43º39′	70º19′	d43	a20 f20	e6	e6		b3	6	3	c4 f5		a – 36 ft to 6/41; 61 ft to 5/43; 43 ft to 8/48; and 55 ft to 10/6/64. b – 24 ft to 6/48 c – commissioned 1200 ft. ESE of thermometer 2/2/65. d – 61 ft to 2/2/65 and 47 ft to 12/10/69. e – Standby status after 2/2/65. f – moved 850′ SE 12/10/69

c. Concord NWS

STATION LOCATION CONCORD, NEW HAMPSHIRE

COOPERATIVE																
Number and identity of observers unknown	1-1828	9/30/56						-					-			
Hon. Wm L. Foster Winter & State Streets	10/01/56	8/13/97		43º13′	71º33′	*280		-					-			*Approximate
Prof. Wm. W. Flint St. Paul's School	8/14/97	10/31/02	2.5mi WSW	43º12′	71º35′	*350										*Approximate
City																
Smith Block, 28 North Main Street	11/01/02	7/31/33	0.3 mi. SE	43º12′	71º32′	272	80	71	70		62		62			First order station. Exposures fair.
Patriot Building 4 Park Street	8/01/33	12/33/37	¼ mi. NNW	43º12′	71º32′	276		61	60				55			Second order station.
First National Bankl Bldg., 18 N. Main St.	12/23/37	5/01/41	¼ mi. SSE	43º12′	71º32′	270	72	63	62		56		56			First order station re_opened.
AIRPORT																
NE Airlines, Inc. Old Hanger	10/27/33	3/28/39		43º12′	71º31′	339										On call station. No instruments
1st Floor, Admin. Bldg., Municipal Airport	3/01/39	11/19/42		43º12′	71º31′	339	37	5	5				3			Observations by CAA.

STATION LOCATION CONCORD, NEW HAMPSHIRE

						Sea Level	Elevation Above Ground									
Location	Occupied from	Occupied to	Airline distance and direction from previous location	Latitude North	Latitude West	Ground at temperature site	Wind instruments	Extreme thermometers	Psychrometer	Telepsychrometer	Tipping bucket rain gage	Weighing Rain Gage	8" Rain Gage	Hygrothermometer		Remarks
2nd Floor, Admin. Bldg., Municipal Airport	5/01/41	8/08/62	1.5 mi. E of City Office	43°12′	71°31′	339	47	5	5		3	4	3			Semi-rural locations. Exposures good to excellent.
1st Floor, Admin. Bldg., Municipal Airport	8/08/62	Present		43°12′	71°30′	b342	20	c5	%5		3	4	3	a4		a – commissioned 1600′ E of thermometer 3/15/65. b – 339′ to 3/15/65. c - Standby status after 3/15/65, discontinued 3/16/66. % - Standby equipment

d. Pease AFB

STATION NO. ON SUMMARY	STATION NAME	LATITUDE	LONGITUDE	STATION ELEV. (FT)	CALL SIGN	WMO NUMBER
0474	PEASE AFB NH/PORTSMOUTH	N 43 05	W 070 49	111	KPSM	

STATION LOCATION AND INSTRUMENTATION HISTORY

Number of Location	Geographical Location & Name	Type of Station	At This Location		Latitude	Longitude	Elevation Above MSL		OBS PER DAY
			From	To			Station (Ft)	Hgt Barometer	
1	Pease AFB, Portsmouth NH	AFB	Apr 56	Feb 57	N 43 05	W 070 46	104	N/A	24
2	No change	AFB	Mar 57	22 Feb 60	No chge	W 070 49	111	88 ft.	24
3	No change	AFB	23 Feb 60	Dec 70	No chge	No chge	No chge	127 ft.	24

SURFACE WIND EQUIPMENT INFORMATION

Number of location	Date of Change	Location	Type of Transmitter	Type of Recorder	Ht Above Ground	Remarks, Additional Equipment or Reason for Change
1	Apr 56 to 24 Apr 56	Located 50 ft. S of the weather station.	AN/GMQ-11	ML204B	15 ft.	
2	25 Apr 56 to Feb 47	Located on the ground	AN/GMQ-1	N/A	N/A	
3	Mar 57 to Feb 58	Located 750 ft. W of centerline of Rnwy 16/34.	AN/GMQ-11	RO-2	13 ft.	
4	Mar 58 to Feb 59	Located 500 ft. off center at N end of Rnwy 16	No chge	No chge	No chge	
5	Mar 59 to 22 Feb 60	Located 3350 ft. WnW of weather station	No chge	No chge	No chge	
6	23 Feb 60 to Feb 62	Located 1000 ft. from touchdown pt on W side of Rnwy 16.	AN/GMQ-11	RO-2	15 ft.	
7	Mar 62 to Feb 63	Located 500 ft. W of Rnwy 16/34 centerline, 3700 ft. S of N end of Rnwy	No chge	No chge	13 ft	
8	Mar 63 to 27 Mar 69	1. Located 1500 ft. S of N end of Rnwy 16, 500 ft E of center-line of the Rnwy.	No chge	No chge	No chge	
		2. Located 750 ft N of S end of Rnwy 34, 500 ft E of center-line of the Rnwy.	No chge.		No chge	
9	28 Mar 69 to Sep 70	1. No change	No chge	RO-362	No chge	
		2. No change	No chge		No chge	
10	Oct 70 to Dec 70	1. No change	AN/GMQ-20	No chge	No chge	
		2. No change	No chge		No chge	

TABLE 2.3-2 MEAN NUMBER OF DAYS WITH THUNDERSTORMS

	Boston	Portland	Concord	Pease AFB
Jan	*	*	*	0.1
Feb	*	*	*	0
Mar	1	*	*	0.1
Apr	1	1	1	0.9
May	2	2	2	2.3
Jun	4	4	5	4.1
Jul	5	4	6	5.5
Aug	4	4	4	3.8
Sep	2	2	2	1.4
Oct	1	1	1	0.6
Nov	*	*	*	0.4
Dec	*	*	*	0
Annual	19	18	20	19.0
Years	1936 – 1977	1941 – 1977	1942 – 1977	Apr. 1956 - 1970

References: 8, 9, 10, 11
* Less than one half

TABLE 2.3-3 ESTIMATED FREQUENCY OF CLOUD-TO-GROUND LIGHTNING – NUMBER PER YEAR (REFERENCE 12)

Period	**Height of Object Above Grade (ft)**			
	50	100	200	500
Dec – Feb	0.001	0.002	0.003	0.007
Mar – May	0.022	0.055	0.099	0.231
Jun – Aug	0.089	0.224	0.402	0.938
Sep – Nov	0.016	0.040	0.072	0.168
Annual	0.127	0.318	0.570	1.330

TABLE 2.3-4 **NUMBER OF DAYS WITH HAIL (REFERENCE 14)**

	Boston	Portland	Concord
Jan	0	0	1
Feb	0	0	0
Mar	1	2	2
Apr	3	4	3
May	5	4	9
Jun	5	3	3
Jul	8	6	4
Aug	3	6	2
Sep	1	2	1
Oct	0	6	2
Nov	2	0	1
Dec	0	0	1
Total	28	33	29
Average Per Year	0.7	0.83	0.83
Years of Data	1904 – 1943	1904 – 1943	1904 – 1933; 1939 - 1943

TABLE 2.3-5 **FASTEST MILE WIND SPEED (MPH)**

	Boston	**Portland**	**Concord**
Jan	66	50	44
Feb	61	58	42
Mar	73	76	71
Apr	63	57	52
May	55	49	48
Jun	46	45	38
Jul	52	44	45
Aug	52	69	56
Sep	87	62	61
Oct	63	45	39
Nov	80	76	72
Dec	73	62	52
Maximum	87	76	72
Years	1916 – 1978	1941 – 1978	1938 – 1978

References: (8, 18, 19), (9, 20), (1, 10)

TABLE 2.3-6 **FASTEST-MILE EXTREME WIND SPEEDS FOR SEABROOK AREA**

Return Interval (years)	Annual Probability of Exceedance	Wind Speed (mph)	
		10 Meters Above Grade	30 Meters Above Grade
10	0.1	61	72
25	0.04	72	84
50	0.02	81	94
100	0.01	90	105
200	0.005	98	115
400	0.0025	107	126
2000	0.0005	131	154

Derived from data in References, 50, 51

TABLE 2.3-7 **AVERAGE FREQUENCY OF OCCURRENCE (PERCENT OF HOURS) OF FREEZING RAIN AT PORTSMOUTH, NEW HAMPSHIRE (REFERENCE 24)**

Month	Percent of Hours
Nov	0.2
Dec	1.0
Jan	0.9
Feb	0.5
Mar	0.3
Apr	0.0
Annual	0.3
Years	1956 - 1961

TABLE 2.3-8 EPISODES WITH METEROLOGICAL CONDITIONS IN THE SITE AREA UNFAVORABLE FOR ATMOSPHERIC DISPERSION (REFERENCE 29)

Number Episodes in 5 years	Minimum Episode Duration (Days)	Maximum Mixing Height (Feet)	Maximum Wind Speed (mph)
0	2	1640	4.5
1	2	1640	9.0
2	2	1640	13.4
0	2	3280	4.5
3	2	3280	9.0
15	2	3280	13.4
0	5	1640	9.0
0	5	1640	13.4
0	5	3280	9.0
1	5	3280	13.4

TABLE 2.3-9 FREQUENCY OF OCCURRENCE OF SUMMERTIME 4-HOUR AVERAGE WET BULB TEMPERATURES

Four-Hour Average Wet Bulb Temperature (°F)	Frequency of Occurrence at Temperature in 25 Years (Summer)	Cumulative Frequency of Occurrence At or Above Temperature in 25 Years (Summer)
81	4	4
80	20	24
79	51	75
78	112	187
77	199	386
76	348	734
75	541	1,275
74	759	2,034
73	1,115	3,149
72	1,327	4,476
71	1,554	6,030
70	1,934	7,964
69	2,184	10,148
68	2,407	12,555
67	2,770	15,325
66	2,999	18,324
65	3,260	21,584

Data Base: Pease Air Force Base Hourly Observations
Summer Periods (June 1 through September 15)
for the Years 1956-1974 and 1976-1981.

TABLE 2.3-10 FREQUENCY OF OCCURRENCE OF 9-HOUR AVERAGE WET BULB TEMPERATURES FOR NIGHT-TIME HOURS DURING THE PERIOD JULY 16 THROUGH JULY 31

Nine-Hour Average Wet Bulb Temperature (°F)	Frequency of Occurrence at Temperature in 25 Years	Cumulative Frequency of Occurrence At or Above Temperature in 25 Years
76	7	7
75	2	9
74	15	24
73	42	66
72	63	129
71	56	185
70	86	271
69	117	388
68	111	499
67	95	594
66	149	743
65	221	964
64	236	1,200
63	191	1,391
62	150	1,541
61	164	1,705
60	170	1,875
59	170	2,045
58	203	2,248
57	122	2,370
56	80	2,450
55	83	2,533
54	68	2,601
53	59	2,660
52	13	2,673
51	12	2,685
50	16	2,701
49	16	2,717
48	7	2,724
47	2	2,726

Data Base: Pease Air Force Base Nighttime Hourly Observations Running
Nine-Hour Averages Ending in the Hours 0300 –0900 LST
Summer Period July 16 through July 31
Years 1956-1974 and 1976-1981

TABLE 2.3-11 FREQUENCY OF OCCURRENCE OF PERIODS OF 24-HOUR AVERAGE DRY BULB TEMPERATURES OF LESS THAN OR EQUAL TO 15°F BY LENGTH OF PERIOD

Episode Length (Whole Days)*	Number of Occurrences In 25 Years of Record
1	69
2	64
3	34
4	10
5	4
6	1
7	4
8	0
9	0
10	0
11	1
12	0
13	0
14	0
15	0
16	1`

Data Base: Pease Air Force Base Hourly Observations
April 1, 1956 – December 31, 1974 and
January 1, 1976 – December 31, 1981
Excluding the Summer Months June through August

*Fractional days discarded.

TABLE 2.3-12 MONTHLY AVERAGE TEMPERATURES (°F)

	Boston	Portland	Concord	Portsmouth
Jan	29.2	21.5	20.6	22.9
Feb	30.4	22.9	22.6	23.9
Mar	38.1	31.8	32.3	32.1
Apr	48.6	42.7	44.2	42.5
May	58.6	52.7	55.1	53.5
Jun	68.0	62.2	64.7	63.1
Jul	73.3	68.0	69.7	68.1
Aug	71.3	66.4	67.2	66.1
Sep	64.5	58.7	59.5	58.6
Oct	55.4	49.1	49.3	49.2
Nov	45.2	38.6	38.0	39.3
Dec	33.0	25.7	24.8	25.8
Annual	51.3	45.0	45.6	45.4
Years of Data	1941-1970	1941-1970	1941-1970	1954-1967

References : 8, 9, 10, 24

TABLE 2.3-13 MONTHLY MEAN OF DAILY MAXIMUM TEMPERATURES (°F)

	Boston	Portland	Concord	Portsmouth
Jan	36	31	31	32
Feb	38	33	34	35
Mar	45	41	42	42
Apr	56	53	57	53
May	67	64	69	66
Jun	77	73	78	75
Jul	81	79	83	80
Aug	79	78	80	78
Sep	72	70	72	70
Oct	63	60	62	61
Nov	52	48	48	49
Dec	39	35	35	35
Annual	58.7	55.3	57.5	56.4
Years of Data	1941-1970	1941-1970	1941-1970	1954-1967

References : 8, 9, 10, 24

TABLE 2.3-14 MONTHLY MEAN OF DAILY MINIMUM TEMPERATURES (°F)

	Boston	Portland	Concord	Portsmouth
Jan	23	12	10	13
Feb	23	13	11	13
Mar	32	23	22	22
Apr	41	33	32	32
May	50	42	42	41
Jun	59	51	52	51
Jul	65	57	57	56
Aug	63	55	54	54
Sep	57	47	47	47
Oct	48	38	36	37
Nov	39	30	28	30
Dec	27	16	15	17
Annual	43.8	34.7	33.7	34.4
Years of Data	1941-1970	1941-1970	1941-1970	1954-1967

References : 8, 9, 10, 24

TABLE 2.3-15 EXTREME HIGHEST TEMPERATURE (°F)

	Boston	Portland	Concord	Portsmouth
Jan	72 (1950)	65 (1906)	72 (1876)	58 (1966)
Feb	68 (1957)	64 (1957)	68 (1880)	64 (1957)
Mar	86 (1945)	86 (1946)	85 (1977)	76 (1962)
Apr	94 (1976)	89 (1927)	95 (1976)	92 (1962)
May	97 (1880)	96 (1937)	98 (1911)	94 (1964)
Jun	100 (1952)	97 (1941)	101 (1919)	96 (1956)
Jul	104 (1911)	103 (1911)	102 (1966)	99 (1963)
Aug	102 (1975)	103 (1975)	101 (1975)	98 (1955)
Sep	102 (1881)	96 (1939)	98 (1953)	92 (1965)
Oct	90 (1963)	88 (1963)	92 (1879)	88 (1963)
Nov	83 (1950)	74 (1974)	80 (1950)	76 (1959)
Dec	70 (1966)	65 (1911)	65 (1932)	60 (1966)
Record High	104 (1911)	103 (1911, 1975)	102 (1966)	99 (1963)
Years of Data	1872-1978	1872-1978	1871-1978	1954-1967

References : (8, 17, 18), (9, 17, 19), (1, 10, 17), (24)

TABLE 2.3-16 EXTREME LOWEST TEMPERATURE (°F)

	Boston	Portland	Concord	Portsmouth
Jan	-13 (1882)	-26 (1971)	-35 (1878)	-23 (1957)
Feb	-18 (1934)	-39 (1943)	-37 (1943)	-15 (1962)
Mar	-8 (1872)	-21 (1950)	-16 (1967)	-8 (1967)
Apr	11 (1874)	8 (1954)	7 (1874)	10 (1967)
May	31 (1882)	23 (1956)	21 (1966)	22 (1967)
Jun	41 (1945)	33 (1944)	30 (1972)	35 (1967)
Jul	50 (1879)	40 (1965)	35 (1965)	40 (1956)
Aug	46 (1940)	33 (1965)	29 (1965)	33 (1965)
Sep	34 (1914)	23 (1941)	20 (1941)	26 (1962)
Oct	25 (1936)	15 (1976)	10 (1972)	14 (1966)
Nov	-2 (1875)	-6 (1875)	-17 (1875)	11 (1957)
Dec	-17 (1933)	-21 (1963)	-24 (1875)	-12 (1962)
Record Low	-18 (1934)	-39 (1943)	-37 (1943)	-23 (1957)
Years of Data	1872-1978	1872-1978	1871-1978	1954-1967

References : (8, 17, 18), (9, 17, 19), (1, 10, 17), (24)

TABLE 2.3-17 MEAN NUMBER OF DAYS WITH MINIMUM TEMPERATURE 0 (°F) OR BELOW

	Boston	Portland	Concord	Portsmouth
Jan	1	7	12	4
Feb	*	5	8	5
Mar	0	1	1	*
Apr	0	0	0	0
May	0	0	0	0
Jun	0	0	0	0
Jul	0	0	0	0
Aug	0	0	0	0
Sep	0	0	0	0
Oct	0	0	0	0
Nov	0	0	*	0
Dec	*	3	6	2
Years of Data	1965-1977	1941-1977	1966-1977	1954-1967

*Less than one half

References : 8, 9, 10, 24

TABLE 2.3-18 HOTTEST CONTIGUOUS 24-HOURS IN ASSOCIATION WITH THE HOTTEST ONE-HOUR TEMPERATURE* OBSERVED DURING 1957 THROUGH 1981 AT PEASE AFB

Year	Date	Hour	Temperature °F
1964	June 30	Hr 15	89
		16	89
		17	89
		18	85
		19	81
		20	80
		21	77
		22	76
		23	76
	July 1	Hr 00	74
		1	76
		2	75
		3	75
		4	74
		5	73
		6	76
		7	80
		8	88
		9	92
		10	93
		11	96
		12	98
		13	101*
		14	100

TABLE 2.3-19 FIVE WARMEST AND FIVE COLDEST 24-HOUR PERIODS OBSERVED DURING 1957 THROUGH 1981 AT PEASE AFB

FIVE COLDEST 24-HOUR PERIODS						FIVE WARMEST 24-HOUR PERIODS					
Average	-8.08	-7.12	-5.50	-2.70	-2.50	Average	86.62	85.87	85.25	84.25	83.91
Year Period	1968	1957	1980	1967	1981	Year Period	1975	1977	1964	1978	1981
Ends	Jan 9	Jan 15	Dec 26	Feb 13	Jan 5	Ends	Aug 3	Jul 21	Jul 19	Jul 22	Jul 9
Hour						Hour					
00						00					
01						01					
02						02					
03						03					
04						04	74				
05						05	77				
06			-2		-4	06	82				
07			-5		-2	07	85				
08			-7		-4	08	90				
09			-7		-4	09	92				
10			-7	0	-2	10	97				
11	-6		-7	1	-2	11	99			88	
12	-5		-5	2	1	12	100			90	
13	-4		-4	2	2	13	100		93	91	
14	-4		-4	3	3	14	100		94	93	
15	-4		-4	3	3	15	98		94	93	
16	-5		-5	3	2	16	97	94	94	93	
17	-6		-5	2	0	17	92	92	92	91	
18	-7		-7	0	0	18	87	89	89	86	
19	-8		-7	-1	-2	19	84	87	87	85	87
20	-10		-7	-1	-2	20	82	85	85	84	86
21	-10		-7	-2	-5	21	80	83	84	82	82
22	-10	-4	-7	-3	-7	22	78	82	84	82	80
23	-10	-7	-6	-4	-7	23	77	80	82	82	80
00	-10	-8	-5	-5	-7	00	80	79	81	82	79
01	-9	-10	-5	-6	-5	01	77	79	80	80	78
02	-9	-11	-5	-7	-5	02	75	78	79	79	77
03	-10	-13	-5	-8	-5	03	76	77	79	79	75
04	-9	-14	-5	-8	-4	04		77	78	79	74
05	-10	-15	-4	-8	-4	05		77	77	78	75
06	-10	-15		-9		06		77	76	78	77
07	-12	-16-		-9		07		80	78	79	79
08	-10	14		-7		08		84	80	81	80
09	-9	-12		-3		09		87	84	82	82
10	-7	-8				10		91	90	85	85
11		-5				11		95	92		90
12		-1				12		96	94		89
13		0				13		96			93
14		1				14		98			95
15		0				15		98			94
16		-3				16					94
17		-4				17					93
18		-4				18					90
19		-4				19					
20		-2				20					
21		-2				21					
22						22					
23						23					

TABLE 2.3-20 MEAN MONTHLY RELATIVE HUMIDITY (%) AT PEASE AFB, NEW HAMPSHIRE (REFERENCE 11)

Jan	65.9
Feb	64.7
Mar	65.0
Apr	64.9
May	65.7
Jun	70.9
Jul	72.2
Aug	72.5
Sep	74.4
Oct	70.4
Nov	71.6
Dec	69.0
Annual	68.9
Years	Apr. 1956-1970

TABLE 2.3-21 MEAN NUMBER OF DAYS WITH PRECIPITATION 0.01 INCH OR MORE*

	Boston	Portland	Concord
Jan	12	11	11
Feb	11	10	10
Mar	12	11	11
Apr	11	12	11
May	11	13	12
Jun	11	11	11
Jul	9	9	10
Aug	10	9	10
Sep	9	8	9
Oct	9	9	8
Nov	11	12	11
Dec	12	12	11
Annual	128	127	125
Years of Data	1952 - 1977	1941 – 1977	1942 - 1977

References : 8, 9, 10
*Portsmouth Annual 129 days (Reference 24)

TABLE 2.3-22 MEAN MONTHLY PRECIPITATION (INCHES OF WATER)

	Boston	Portland	Concord	Portsmouth
Jan	3.7	3.4	2.7	4.2
Feb	3.5	3.5	2.5	4.0
Mar	4.0	3.6	2.8	3.4
Apr	3.5	3.3	2.9	3.6
May	3.5	3.3	3.0	2.8
Jun	3.2	3.1	3.4	2.7
Jul	2.7	2.6	3.1	3.4
Aug	3.5	2.6	2.9	2.7
Sep	3.2	3.1	3.1	3.8
Oct	3.0	3.3	2.7	4.1
Nov	4.5	4.9	4.0	4.6
Dec	4.2	4.1	3.3	3.5
Annual	42.5	40.8	36.2	42.6
Years of Data	1941-1970	1941-1970	1941-1970	1954-1967

References : 8, 9, 10, 24

TABLE 2.3-23 PRECIPITATION EXTREMES: MAXIMUM 24-HOUR TOTAL (INCHES OF WATER)

	Boston		Portland		Concord		Portsmouth	
Jan	3.3	(1881)	3.6	(1977)	2.1	(1888)	2.6	(1958)
Feb	4.5	(1886)	3.2	(1965)	2.1	(1951)	3.4	(1965)
Mar	4.1	(1968)	3.7	(1937)	2.6	(1936)	1.8	(1964)
Apr	3.2	(1921)	5.3	(1973)	3.0	(1923)	1.7	(1962)
May	5.7	(1954)	4.9	(1916)	3.1	(1922)	1.8	(1960)
Jun	5.4	(1875)	5.6	(1967)	4.5	(1944)	2.4	(1955)
Jul	6.0	(1921)	4.3	(1939)	5.1	(1887)	2.5	(1961)
Aug	8.4	(1955)	4.2	(1946)	5.3	(1908)	2.3	(1954)
Sep	5.6	(1954)	7.5	(1954)	6.0	(1932)	6.6	(1954)
Oct	4.9	(1895)	7.7	(1962)	4.2	(1962)	5.6	(1962)
Nov	5.4	(1876)	3.8	(1877)	4.0	(1927)	2.8	(1963)
Dec	4.2	(1969)	3.8	(1969)	3.3	(1969)	2.0	(1962)
Years	1871-1978		1871-1978		1871-1978		1954-1967	

References : (8, 17, 18), (9, 19), (1, 10), (24)

TABLE 2.3-24 PRECIPITATION EXTREMES: MAXIMUM MONTHLY TOTAL (INCHES OF WATER)

	Boston		Portland		Concord		Portsmouth	
Jan	9.5	(1958)	12.3	(1935)	6.3	(1978)	13.8	(1958)
Feb	7.1	(1969)	9.3	(1900)	5.9	(1896)	5.8	(1965)
Mar	11.0	(1953)	10.0	(1953)	9.8	(1936)	6.2	(1956)
Apr	9.1	(1904)	9.9	(1973)	7.4	(1904)	6.5	(1961)
May	13.4	(1954)	7.7	(1948)	8.3	(1954)	6.4	(1967)
Jun	9.1	(1931)	10.9	(1917)	10.1	(1954)	6.3	(1959)
Jul	11.7	(1921)	10.8	(1915)	10.3	(1915)	5.4	(1959)
Aug	17.1	(1955)	8.3	(1946)	9.0	(1892)	6.7	(1955)
Sep	10.9	(1933)	9.8	(1954)	11.0	(1888)	9.1	(1954)
Oct	8.8	(1877)	12.3	(1962)	8.8	(1962)	10.8	(1962)
Nov	11.0	(1876)	9.8	(1963)	7.6	(1937)	9.7	(1963)
Dec	9.7	(1969)	9.7	(1969)	7.6	(1936)	6.4	(1954)
Years	1871-1978		1871-1978		1871-1978		1954-1967	

References : (8, 17, 18), (9, 19), (1, 10), (24)

TABLE 2.3-25 MAXIMUM RECORDED SHORT PERIOD RAINFALL (INCHES OF WATER) (REFERENCE 30)

	Boston	Portland	Concord
Time Period (Minutes)			
5	0.56	0.51	0.66
10	0.95	0.78	1.12
15	1.25	1.09	1.6
30	1.63	1.49	2.53
Years of Data	1896-1961	1896-1961	1905-1932; 1938-1961
(Hours)			
1	2.10	2.11	2.71
2	2.85	3.4	2.73
3	4.05	4.51	3.56
6	5.46	5.84	3.82
12	6.74	7.09	5.53
Years of Data	1892-1961	1893-1961	1902-1961

TABLE 2.3-26 PRECIPITATION EXTREMES: MINIMUM MONTHLY TOTAL (INCHES OF WATER)

	Boston		Portland		Concord		Portsmouth	
Jan	0.9	(1970)	0.8	(1970)	0.4	(1970)	0.9	(1955)
Feb	0.5	(1877)	0.4	(1872)	0.4	(1877)	1.3	(1957)
Mar	T	(1915)*	0.1	(1915)	T	(1915)*	1.7	(1965)
Apr	0.9	(1892)	0.7	(1941)	0.4	(1941)	1.4	(1966)
May	0.3	(1944)	0.5	(1965)	0.3	(1899)	1.0	(1964)
Jun	0.3	(1912)	0.5	(1908)	0.1	(1913)	0.8	(1964)
Jul	0.5	(1952)	0.6	(1965)	0.9	(1910)	1.3	(1955)
Aug	0.4	(1883)	0.3	(1947)	0.4	(1882)	1.4	(1956)
Sep	0.2	(1914)	0.3	(1948)	0.2	(1914)	1.5	(1964)
Oct	0.1	(1924)	0.1	(1924)	0.1	(1924)	1.9	(1963)
Nov	0.6	(1917)	0.6	(1939)	0.5	(1939)	2.4	(1965)
Dec	0.7	(1935)	0.9	(1874)	0.6	(1943)	1.0	(1955)
Years	1871-1978		1871-1978		1871-1978		1954-1967	

*T = Trace, an amount too small to measure (less than 0.01 inch)

References : (8, 17, 18), (9, 19), (1, 10), (24)

TABLE 2.3-27 MEAN MONTHLY SNOWFALL (INCHES OF SNOW)

	Boston	Portland	Concord	Portsmouth
Jan	12.2	18.1	17.1	17.7
Feb	11.8	19.6	15.3	18.9
Mar	8.1	13.9	11.6	16.3
Apr	0.7	3.1	2.2	1.9
May	T*	0.2	0.2	T*
Jun	0	0	0	0
Jul	0	0	0	0
Aug	0	0	0	0
Sep	0	T*	0	0
Oct	T*	0.3	0.1	T*
Nov	1.2	3.3	3.9	1.8
Dec	8.1	16.0	14.3	15.6
Annual	42.1	74.5	64.8	72.2
Years of Data	1938-1978	1938-1978	1938-1978	1954-1967

*T = Trace, less than 0.1 inches

References : 8, 9, 10, 24

TABLE 2.3-28 SNOWFALL EXTREMES: MAXIMUM 24-HOUR TOTAL (INCHES OF SNOW)

	Boston		Portland		Concord		Portsmouth	
Jan	21.0	(1978)	23.3	(1935)	19.0	(1944)	15.0	(1966)
Feb	23.6	(1978)	21.5	(1969)	15.0	(1929)	15.0	(1966)
Mar	17.7	(1960)	19.8	(1939)	13.6	(1959)	15.0	(1956)
Apr	9.1	(1917)	15.0	(1906)	18.3	(1933)	8.0	(1956)
May	0.5	(1977)	7.0	(1945)	5.0	(1945)	T	(1963)*
Jun	0.0		0.0		0.0		0.0	
Jul	0.0		0.0		0.0		0.0	
Aug	0.0		0.0		0.0		0.0	
Sep	0.0		T	(1959)*	0.0		0.0	
Oct	0.5	(1884)	3.6	(1969)	2.1	(1969)	T	(1963)*
Nov	12.0	(1898)	11.2	(1898)	13.3	(1938)	5.2	(1961)
Dec	13.0	(1960)	22.8	(1970)	14.6	(1946)	21.6	(1954)
Years	1872-1978		1882-1978		1871-1978		1954-1967	

*T = Trace (less than 0.1 inches)

References : (8, 17, 18), (9, 19), (1, 10), (24)

TABLE 2.3-29 SNOWFALL EXTREMES: MAXIMUM MONTHLY TOTAL (INCHES OF SNOW)

	Boston		Portland		Concord		Portsmouth	
Jan	35.9	(1978)	59.0	(1935)	46.7	(1935)	47.6	(1966)
Feb	41.3	(1969)	61.2	(1969)	59.0	(1893)	38.4	(1967)
Mar	33.0	(1916)	46.6	(1956)	38.3	(1956)	53.9	(1956)
Apr	28.3	(1874)	20.5	(1906)	35.0	(1874)	9.7	(1956)
May	0.5	(1977)	7.0	(1945)	5.0	(1945)	T	(1963)*
Jun	0.0		0.0		0.0		0.0	
Jul	0.0		0.0		0.0		0.0	
Aug	0.0		0.0		0.0		0.0	
Sep	0.0		T	(1959)*	0.0		0.0	
Oct	0.5	(1884)	3.8	(1969)	3.0	(1884)	T	(1963)*
Nov	17.8	(1898)	24.3	(1921)	25.0	(1873)	6.4	(1961)
Dec	27.9	(1970)	54.8	(1970)	43.0	(1876)	42.5	(1956)
Years	1872-1978		1882-1978		1871-1978		1954-1967	

*T = Trace (less than 0.1 inches)

References : (8, 17, 18), (9, 19), (1, 10), (24)

TABLE 2.3-30 MEAN NUMBER OF DAYS WITH HEAVY FOG*

	Boston	Portland	Concord
Jan	2	2	2
Feb	2	2	2
Mar	2	4	3
Apr	2	3	2
May	3	6	3
Jun	2	6	4
Jul	2	7	6
Aug	2	6	7
Sep	2	6	9
Oct	2	5	7
Nov	2	4	4
Dec	1	2	3
Annual	23	52	51
Years of Data	1936-1977	1941 – 1977	1942 - 1977

*Heavy fog = visibility of ¼ mile or less

References : 8, 9, 10

TABLE 2.3-31 ANNUAL FREQUENCY OF FOG AT PEASE AFB (REFERENCE 24)

Month	Percent of Hours
Jan	13.5
Feb	12.3
Mar	13.3
Apr	16.6
May	13.9
Jun	18.1
Jul	15.3
Aug	12.2
Sep	16.9
Oct	19.0
Nov	17.0
Dec	14.2
Annual	15.2
Years of Data	1956-1961

TABLE 2.3-32 MEAN NUMBER OF HOURS PER MONTH WITH VISIBILITY LESS THAN 0.5 MILES AT PEASE AFB (REFERENCE 11)

Jan	15
Feb	18
Mar	15
Apr	10
May	14
Jun	12
Jul	17
Aug	13
Sep	19
Oct	20
Nov	16
Dec	18
Annual	187
Years of Data	Apr. 1956-1970

TABLE 2.3-33 – FREQUENCY AND PERSISTENCE OF FOG AT PORTLAND (REFERENCE 32)

	Frequency	Persistence Summary (Number of Occurrences By Hours) *													
Month	**(Percent of Obs.)**	**3**	**6**	**9**	**12**	**15**	**18**	**21**	**24**	**27**	**30**	**33**	**36-39**	**42-45**	**45**
Jan	10	24	11	12	5	5	3	6	3	1	0	0	1	1	0
Feb	12	15	7	11	6	5	3	2	5	3	2	0	3	1	0
Mar	17	19	9	13	11	5	11	5	1	1	1	0	3	2	5
Apr	15	29	19	8	15	6	5	3	4	0	3	1	2	1	1
May	21	26	25	13	10	13	8	6	7	5	2	2	3	0	2
Jun	28	44	34	20	17	13	5	12	3	4	1	1	1	3	5
Jul	22	42	28	28	13	13	8	4	2	2	0	1	3	1	3
Aug	21	30	23	33	14	13	11	1	7	0	0	2	2	0	3
Sep	25	37	25	19	17	12	5	11	5	3	4	0	4	1	2
Oct	19	32	19	14	11	10	7	2	3	2	4	1	5	1	2
Nov	18	24	26	14	13	12	6	1	3	3	1	2	1	1	2
Dec	16	18	9	9	14	13	8	3	3	3	1	0	4	1	1
Annual	**19**	**326**	**225**	**192**	**147**	**120**	**80**	**54**	**48**	**26**	**18**	**8**	**33**	**14**	**28**

Years of Data	1968-1977
Reference	(29a)

* Number of occurrences are based on observations made once every 3 hours and each observation was assumed to persist for 3 hours.

TABLE 2.3-34 PRE-CONSTRUCTION SEABROOK METEOROLOGICAL MEASUREMENT SYSTEM

30 Foot Level

Winds

11/71 – 11/72	Six-bladed Bendix Aerovane with Bendix Model 141-2 dual recorder.
11/72 – 6/74	Bendix P/N 2414914 3-cup anemometer and P/N 2416970 vane system with Bendix Model 141-2 Dual strip chart recorder. Starting speed: less than 1 mph.

Ambient Air Temperature

11/71 – 6/74	REC platinum temperature sensor, 400A REC resistance bridge, and Esterline-Angus multipoint recorder.

Dew Point

3/72 – 6/74	Foxboro Dewcel H103AZ, Dewcel Weatherhood, REC 400A resistance bridge, Esterline-Angus multipoint recorder.

130 Foot Level

Winds

11/72 – 6/74	Bendix P/N 2414914 3-cup anemometer and P/N 2416970 vane system with Bendix Model 141-2 Dual strip chart recorder. Starting speed: less than 1 mph.

Delta T Temperature between 30 feet and 130 feet

11/71 – 6/74	REC platinum temperature sensor, 400D differential bridge, 421 BX-2X differential chassis, Esterline-Angus multipoint recorder.

TABLE 2.3-35 METEOROLOGICAL DATA RECOVERY

PRE-CONSTRUCTION PROGRAM			
Parameter	Possible Hours	Useable Hours	Recovery Rate
30 Foot Wind Direction*	8784	8782	100.0%
30 Foot Wind Speed*	8784	8684	98.9
30 Foot Air Temperature*	8784	8727	99.4
30 Foot Dew Point+	8760	8266	94.3
130-30 Foot Delta T*	8784	8628	98.2

PRE-OPERATIONAL PROGRAM		
	Recovery Date	
Parameters	Apr. 79 - Mar. 80	Jun. 80 - May 81
43 Foot Wind Speed	98.8%	99.9%
209 Foot Wind Speed	98.6%	99.9%
43 Foot Wind Direction	98.5%	99.4%
209 Foot Wind Direction	98.8%	99.9%
43 Foot Temperature	98.8%	99.9%
43-150 Foot Delta Temperature	98.1%	96.9%
43-209 Foot Delta Temperature	98.6%	99.7%
Composite (43' WS, 43' WD, 43'-150' DT)	97.7%	96.4%
Composite (209' WS, 209' WD, 43'-209' DT)	98.3%	99.6%

* Period November 1971 - October 1972

\+ Period April 1972 - March 1973

TABLE 2.3-36 – METEOROLOGICAL INSTRUMENTATION SPECIFICATIONS FOR THE PRE-OPERATIONAL MONITORING PROGRAM

<table>
<tr><th rowspan="2">Parameters</th><th rowspan="2">System Accuracy Root-Sum-Squared Time Averaged</th><th colspan="4">Sensors</th><th colspan="2">Translators</th><th colspan="2">Analog Recorder</th><th colspan="2">A/D Convertor</th></tr>
<tr><th>Manufacturer & Model</th><th>Range</th><th>Accuracy</th><th>Threshold or Sensitivity</th><th>Manufacturer & Model</th><th>Accuracy</th><th>Manufacturer & Model</th><th>Accuracy</th><th>Manufacturer & Model</th><th>Accuracy</th></tr>
<tr><td>Wind Speed</td><td><± 0.5 mph</td><td>Climatronics F460 Transmitter</td><td>0 to 100 mph</td><td>± 0.15 mph or 1%</td><td>0.58 mph Threshold</td><td>Climatronics 100078</td><td>± 0.2%</td><td rowspan="2">Esterline Angus L11S2S

4 ½” Chart</td><td rowspan="2">± 0.25%</td><td rowspan="6">Mod Comp

1400 Analog Input Subsystem</td><td rowspan="6">± 0.05%</td></tr>
<tr><td>Wind Direction</td><td><± 5.0º</td><td>Climatronics F460 Transmitter</td><td>0º to 540º</td><td>± 3º</td><td>0.58 mph Threshold</td><td>Climatronics 100077</td><td>± 0.05%</td></tr>
<tr><td>Temperature and Delta Temp.</td><td>Temp:
<± 0.9ºF
Delta T:
<± 0.18ºF</td><td>Teledyne Geotech 327 Asp. Shield
Rosemont 78 Platinum Sensors
Rosemont 414L Temp Bridge</td><td>Temp:
-30º to +110ºF

Delta T:
-10º to +18ºF</td><td>Sensor:
± 0.47ºF @ 0ºC
± 0.95ºF @ 100ºC
Bridge:
±0.1% of Span</td><td>Temp:
± 0.2ºF Maximum Radiation Effect</td><td>Climatronics 100142
100143</td><td>± 0.05%</td><td rowspan="4">Esterline Angus E1124E

8 Channel Multipoint

10” Chart</td><td rowspan="4">± 0.25%</td></tr>
<tr><td>Dew Point</td><td><± 0.9ºF</td><td>Gen. Eastern 1200 APS [(a)]</td><td>-30º to +110ºF</td><td>± 0.36ºF</td><td>N/A</td><td>Climatronics 100089</td><td>± 0.05%</td></tr>
<tr><td>Precipitation</td><td>± 0.01 inch (Instantaneous)</td><td>Belfort 5-405H Precip.</td><td>N/A</td><td>± 1% to ± 6% of Rainfall Rate</td><td>± 0.01 inch Sensitivity</td><td>Climatronics 100157</td><td>± 0.05%</td></tr>
<tr><td>Solar Radiation</td><td><± 0.1 Cal/cm^2-min</td><td>Eppley 8-48 Pyranometer</td><td>0 to 2 Cal/cm^2-min</td><td>± 2% to ± 5%</td><td>75 mV per Cal/cm^2-min Sensitivity</td><td>Climatronics 100144</td><td>± 0.05%</td></tr>
</table>

(a) The General Eastern dew point system was replaced in May 1981 with a Climatronics Model DP-10 lithium chloride dew point system with a range from -30ºF to 110ºF and an accuracy of ± 0.9ºF.

TABLE 2.3-37 METEOROLOGICAL INSTRUMENTATION SPECIFICATIONS FOR THE OPERATIONAL MONITORING PROGRAM (PRIMARY SYSTEM)

VARIABLE	RANGE	SYSTEM PERFORMANCE SPECIFICATIONS
Wind Speed	0 to 100 mph	≤ ±0.5 mph for speeds less than 5 mph; ≤ ±10% for speeds greater than 5 mph, starting speed < 1.0 mph
Wind Direction	0° to 540°	≤ ± 5°; starting speed < 1.0 mph
Delta-Temp and Temperature	Delta-Temp: -10° to +18°F Temp: -30° to +110°F	Delta-Temp: ≤ ±0.3°F per 164 feet Temp: ≤ ±0.9°F
Precip.	0 to 0.99 in/15-min	N/A
Solar Radiation	0 to 2 Langley/min	N/A

NOTES:

1. The system performance requirements listed for wind speed, wind direction and delta-temp are those delineated in Revision 3 of Regulatory Guide 1.97 (See Section 7.5 for additional information); the system performance requirements listed for temperature are delineated in Revision 0 to Regulatory Guide 1.23. All performance requirements are applicable to the digital system only.

2. The originally installed wind speed rotational cup anemometers and wind direction rotational vanes have been replaced with ultrasonic anemometers that provide equivalent measurement data. Ultrasonic anemometers have no moving parts that need to be actuated by wind force therefore their response to changes in wind speed and direction is much quicker and more stable than the response of the previously installed rotational instruments. The Reg. Guide 1.97 performance parameters characterizing the mechanical response of rotational wind speed and direction instruments; distance constant, damping ratio and delay distance are not applicable to ultrasonic anemometers, however the measurement output of the ultrasonic anemometers are still appropriate for determining average wind speed and direction.

TABLE 2.3-38 **SEABROOK SECTOR DEPENDENT ACCIDENT DILUTION FACTORS FOR THE EXCLUSION RADIUS**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-39 **SEABROOK SECTOR DEPENDENT ACCIDENT DILUTION FACTORS FOR THE LOW POPULATION ZONE**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-40 **SEABROOK OVERALL-SITE ACCIDENT DILUTION FACTOR PROBABILITY DISTRIBUTION -EXCLUSION RADIUS CONCENTRATION CHI/Q VALUES (SEC/M3)**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-41 **SEABROOK OVERALL-SITE ACCIDENT DILUTION FACTOR PROBABILITY DISTRIBUTION -EXCLUSION RADIUS EFFECTIVE GAMMA CHI/Q VALUES (SEC/M3)**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-42 **SEABROOK MAXIMUM SECTOR ACCIDENT DILUTION FACTOR PROBABILITY DISTRIBUTION-EXCLUSION RADIUS CONCENTRATION CHI/Q VALUES (SEC/M3)**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-43 **SEABROOK MAXIMUM SECTOR ACCIDENT DILUTION FACTOR PROBABILITY DISTRIBUTION -EXCLUSION RADIUS EFFECTIVE GAMMA CHI/Q VALUES (SEC/M3)**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-44 **SEABROOK OVERALL-SITE ACCIDENT DILUTION FACTOR PROBABILITY DISTRIBUTION -LOW POPULATION ZONE CONCENTRATION CHI/Q VALUES (SEC/M3)**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-45 **SEABROOK OVERALL-SITE ACCIDENT DILUTION FACTOR PROBABILITY DISTRIBUTION -LOW POPULATION ZONE EFFECTIVE GAMMA CHI/Q VALUES (SEC/M3)**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-46 **SEABROOK MAXIMUM SECTOR ACCIDENT DILUTION FACTOR PROBABILITY DISTRIBUTION -LOW POPULATION ZONE CONCENTRATION CHI/Q VALUES (SEC/M3)**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-47 **SEABROOK MAXIMUM SECTOR ACCIDENT DILUTION FACTOR PROBABILITY DISTRIBUTION -LOW POPULATION ZONE EFFECTIVE GAMMA CHI/Q VALUES (SEC/M3)**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-48 SUMMARY OF DILUTION FACTORS AT THE EXCLUSION RADIUS (SEC/M^3) 914 METERS, APR 79 – MAR 80 ONSITE METEOROLOGY

	Time Interval	Maximum (ESE) Sector Values [(a)]	Overall-Site Values [(b)]
I. Concentration CHI/Q Values			
A. Conservative Estimates	0-1 Hour	2.67×10^{-4}	2.32×10^{-4}
	1-2 Hours	1.88×10^{-4}	1.72×10^{-4}
	2-8 Hours	1.02×10^{-4}	9.35×10^{-5}
	8-24 Hours	2.58×10^{-5}	2.64×10^{-5}
	1-4 Days	1.43×10^{-5}	1.49×10^{-5}
	4-30 Days	7.78×10^{-6}	7.57×10^{-6}
B. Realistic Estimates	0-1 Hour	3.53×10^{-5}	3.78×10^{-5}
	1-2 Hours	2.66×10^{-5}	2.83×10^{-5}
	2-8 Hours	1.44×10^{-5}	2.26×10^{-5}
	8-24 Hours	5.97×10^{-6}	1.06×10^{-5}
	1-4 Days	5.21×10^{-6}	7.45×10^{-6}
	4-30 Days	5.74×10^{-6}	5.81×10^{-6}
II. Effective Gamma CHI/Q Values			
A. Conservative Estimates	0-1 Hour	2.98×10^{-5}	3.00×10^{-5}
	1-2 Hours	2.05×10^{-5}	2.13×10^{-5}
	2-8 Hours	1.14×10^{-5}	1.12×10^{-5}
	8-24 Hours	6.02×10^{-6}	6.21×10^{-6}
	1-4 Days	3.71×10^{-6}	3.74×10^{-6}
	4-30 Days	2.37×10^{-6}	2.31×10^{-6}
B. Realistic Estimates	0-1 Hour	6.19×10^{-6}	7.23×10^{-6}
	1-2 Hours	4.75×10^{-6}	5.73×10^{-6}
	2-8 Hours	2.66×10^{-6}	4.30×10^{-6}
	8-24 Hours	1.91×10^{-6}	3.39×10^{-6}
	1-4 Days	1.48×10^{-6}	2.21×10^{-6}
	4-30 Days	1.61×10^{-6}	1.63×10^{-6}

(a) The maximum sector conservative CHI/Q values represent the ESE sector's values which are exceeded 0.5% of the total time; the maximum sector realistic CHI/Q values represent the ESE sector's median values.

(b) The overall-site conservative CHI/Q values represent the overall-site values which are exceeded 5% of the total time; the overall-site realistic CHI/Q values represent the overall-site median values.

TABLE 2.3-49 SUMMARY OF DILUTION FACTORS AT THE LOW POPULATION ZONE (SEC/M^3) 2012 METERS, APR 79 – MAR 80 ONSITE METEOROLOGY

			Maximum (ESE) Sector Values [(a)]	Overall-Site Values [(b)]
I. Concentration CHI/Q Values				
	A. Conservative Estimates	0-1 Hour	1.31×10^{-4}	1.10×10^{-4}
		1-2 Hours	9.17×10^{-5}	8.31×10^{-5}
		2-8 Hours	4.82×10^{-5}	4.49×10^{-5}
		8-24 Hours	7.21×10^{-6}	7.50×10^{-6}
		1-4 Days	4.25×10^{-6}	4.32×10^{-6}
		4-30 Days	2.25×10^{-6}	2.20×10^{-6}
	B. Realistic Estimates	0-1 Hour	1.25×10^{-5}	1.35×10^{-5}
		1-2 Hours	9.68×10^{-6}	1.08×10^{-5}
		2-8 Hours	5.54×10^{-6}	8.53×10^{-6}
		8-24 Hours	1.80×10^{-6}	3.25×10^{-6}
		1-4 Days	1.52×10^{-6}	2.23×10^{-6}
		4-30 Days	1.70×10^{-6}	1.71×10^{-6}
II. Effective Gamma CHI/Q Values				
	A. Conservative Estimates	0-1 Hour	1.15×10^{-5}	1.20×10^{-5}
		1-2 Hours	8.19×10^{-6}	8.26×10^{-6}
		2-8 Hours	4.43×10^{-6}	4.39×10^{-6}
		8-24 Hours	2.33×10^{-6}	2.43×10^{-6}
		1-4 Days	1.44×10^{-6}	1.45×10^{-6}
		4-30 Days	9.05×10^{-7}	8.81×10^{-7}
	B. Realistic Estimates	0-1 Hour	2.35×10^{-6}	2.77×10^{-6}
		1-2 Hours	1.82×10^{-6}	2.19×10^{-6}
		2-8 Hours	9.99×10^{-7}	1.66×10^{-6}
		8-24 Hours	7.20×10^{-7}	1.30×10^{-6}
		1-4 Days	5.54×10^{-7}	8.53×10^{-7}
		4-30 Days	6.15×10^{-7}	$6.21 \times 10\text{-}^{7}$

(a) The maximum sector conservative CHI/Q values represent the ESE sector's values which are exceeded 0.5% of the total time; the maximum sector realistic CHI/Q values represent the ESE sector's median values.

(b) The overall-site conservative CHI/Q values represent the overall-site values which are exceeded 5% of the total time; the overall-site realistic CHI/Q values represent the overall-site median values.

TABLE 2.3-50 **SEABROOK ANNUAL AVERAGE CHI/Q BEFORE DEPLETION (SEC/M3) PRIMARY VENT STACK RELEASE**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-51 **SEABROOK ANNUAL AVERAGE CHI/Q AFTER DEPLETION (SEC/M3) PRIMARY VENT STACK RELEASE**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-52 **SEABROOK ANNUAL AVERAGE DEPOSITION RATES (1/M2) PRIMARY VENT STACK RELEASE**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-53 **SEABROOK ANNUAL AVERAGE EFFECTIVE GAMMA CHI/Q (SEC/M3) PRIMARY VENT STACK RELEASE**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-54 **SEABROOK ANNUAL AVERAGE CHI/Q BEFORE DEPLETION (SEC/M3) TURBINE BUILDING RELEASE**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-55 **SEABROOK ANNUAL AVERAGE CHI/Q AFTER DEPLETION (SEC/M3) TURBINE BUILDING RELEASE**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-56 **SEABROOK ANNUAL AVERAGE DEPLETION RATES (1/M2) TURBINE BUILDING RELEASE**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-57 **SEABROOK ANNUAL AVERAGE EFFECTIVE GAMMA CHI/Q (SEC/M3) TURBINE BUILDING RELEASE**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.3-58 ESTIMATED FREQUENCY OF TIBL FORMATION AND SEA BREEZE CONDITIONS AT THE SEABROOK SITE APRIL 1979 – SEPTEMBER 1979

		TIBL Formation			Sea Breeze Conditions		
Month	No. of Good Hourly Obs.	No. of Hours	No. of Days	% of Good Hourly Obs.	No. of Hours	No. of Days	% of Good Hourly Obs.
April	687	67	15	9.8	42	12	6.1
May	660	67	13	10.2	26	8	4.0
June	718	133	22	18.5	79	21	11.0
July	735	117	22	15.9	92	22	12.5
August	740	80	16	10.8	67	15	9.1
September	720	63	12	8.8	45	11	6.3
Total – (Apr-Sept)	4260	527	100	12.4	351	89	8.2

TABLE 2.3-59 ANNUAL AVERAGE TIBL TERRAIN CORRECTION FACTORS PRIMARY VENT STACK RELEASE CHI/Q (BEFORE DEPLETION)

Downwind Sector		Distance From Release Point (Miles)							
	No. OBS	.25	.50	.75	1.00	1.50	2.00	2.50	3.00
N	386	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NNE	525	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NE	670	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ENE	763	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
E	891	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ESE	1530	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SE	858	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSE	334	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
S	328	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSW	221	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SW	348	1.00	1.00	1.00	1.01	1.01	1.00	1.01	1.00
WSW	316	.99	1.02	1.07	1.07	1.05	1.03	1.03	1.01
W	385	.97	1.03	1.07	1.07	1.04	1.02	1.02	1.00
WNW	351	.99	1.04	1.08	1.09	1.07	1.07	1.02	1.02
NW	411	1.00	1.10	1.19	1.17	1.09	1.07	1.07	1.06
NNW	309	1.00	1.00	1.02	1.02	1.02	1.02	1.02	1.01
Average	8626	1.00	1.01	1.02	1.02	1.02	1.01	1.01	1.01

Downwind Sector		Distance From Release Point (Miles)							
	No. OBS	3.50	4.00	4.50	5.00	7.50	10.00	15.01	20.00
N	386	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NNE	525	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NE	670	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ENE	763	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
E	891	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ESE	1530	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SE	858	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSE	334	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
S	328	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSW	221	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SW	348	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
WSW	316	1.01	1.01	1.01	1.01	1.01	1.01	1.00	1.00
W	385	1.00	1.00	1.00	1.00	1.00	1.00	1.00	.99
WNW	351	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.01
NW	411	1.04	1.04	1.04	1.04	1.04	1.03	1.03	1.02
NNW	309	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
Average	8626	1.01	1.01	1.00	1.00	1.00	1.00	1.00	1.00

TABLE 2.3-60 ANNUAL AVERAGE TIBL TERRAIN CORRECTION FACTORS PRIMARY VENT STACK RELEASE CHI/Q (AFTER DEPLETION)

Downwind	Distance From Release Point (Miles)								
Sector	No. OBS	.25	.50	.75	1.00	1.50	2.00	2.50	3.00
N	386	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NNE	525	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NE	670	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ENE	763	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
E	891	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ESE	1530	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SE	858	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSE	334	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
S	328	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSW	221	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SW	348	1.00	1.00	1.00	1.01	1.01	1.00	1.00	1.00
WSW	316	.99	1.03	1.07	1.07	1.05	1.03	1.03	1.01
W	385	.98	1.03	1.07	1.07	1.04	1.02	1.02	1.00
WNW	351	.99	1.04	1.09	1.09	1.07	1.07	1.02	1.02
NW	411	1.00	1.11	1.20	1.17	1.09	1.07	1.07	1.06
NNW	309	1.00	1.00	1.02	1.02	1.02	1.02	1.02	1.01
Average	8626	1.00	1.01	1.02	1.02	1.02	1.01	1.01	1.01

Downwind	Distance From Release Point (Miles)								
Sector	No. OBS	3.50	4.00	4.50	5.00	7.50	10.00	15.01	20.00
N	386	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NNE	525	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NE	670	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ENE	763	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
E	891	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ESE	1530	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SE	858	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSE	334	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
S	328	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSW	221	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SW	348	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
WSW	316	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
W	385	1.00	1.00	1.00	1.00	1.01	1.01	1.00	1.00
WNW	351	1.02	1.02	1.02	1.02	1.03	1.03	1.02	1.02
NW	411	1.04	1.04	1.04	1.04	1.04	1.04	1.03	1.03
NNW	309	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
Average	8626	1.01	1.01	1.01	1.01	1.01	1.00	1.00	1.00

TABLE 2.3-61 ANNUAL AVERAGE TIBL TERRAIN CORRECTION FACTORS PRIMARY VENT STACK RELEASE DEPOSITION RATES

Downwind		Distance From Release Point (Miles)							
Sector	No. OBS	.25	.50	.75	1.00	1.50	2.00	2.50	3.00
N	386	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NNE	525	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NE	670	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ENE	763	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
E	891	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ESE	1530	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SE	858	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSE	334	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
S	328	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSW	221	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SW	348	1.00	1.00	1.00	1.01	1.01	1.01	1.01	1.01
WSW	316	1.00	1.02	1.06	1.07	1.06	1.05	1.05	1.02
W	385	.98	1.03	1.07	1.08	1.06	1.04	1.04	1.02
WNW	351	.99	1.04	1.10	1.11	1.10	1.10	1.04	1.04
NW	411	1.00	1.13	1.24	1.22	1.14	1.12	1.11	1.10
NNW	309	1.00	1.01	1.02	1.03	1.03	1.03	1.03	1.03
Average	8626	1.00	1.01	1.02	1.02	1.02	1.02	1.01	1.01

Downwind		Distance From Release Point (Miles)							
Sector	No. OBS	3.50	4.00	4.50	5.00	7.50	10.00	15.01	20.00
N	386	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NNE	525	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NE	670	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ENE	763	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
E	891	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ESE	1530	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SE	858	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSE	334	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
S	328	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSW	221	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SW	348	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
WSW	316	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02
W	385	1.01	1.01	1.01	1.02	1.02	1.02	1.02	1.01
WNW	351	1.05	1.05	1.05	1.05	1.06	1.06	1.06	1.05
NW	411	1.08	1.08	1.08	1.08	1.08	1.08	1.07	1.07
NNW	309	1.02	1.03	1.02	1.02	1.02	1.03	1.02	1.02
Average	8626	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01

TABLE 2.3-62 ANNUAL AVERAGE TIBL TERRAIN CORRECTION FACTORS PRIMARY VENT STACK RELEASE EFFECTIVE GAMMA CHI/Q

Downwind	Distance From Release Point (Miles)								
Sector	No. OBS	.25	.50	.75	1.00	1.50	2.00	2.50	3.00
N	386	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NNE	525	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NE	670	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ENE	763	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
E	891	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ESE	1530	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SE	858	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSE	334	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
S	328	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSW	221	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SW	348	1.00	1.00	1.01	1.01	1.01	1.01	1.01	1.01
WSW	316	1.01	1.03	1.05	1.05	1.05	1.05	1.05	1.03
W	385	1.02	1.06	1.08	1.08	1.08	1.08	1.08	1.06
WNW	351	1.02	1.07	1.11	1.12	1.13	1.13	1.08	1.07
NW	411	1.04	1.12	1.17	1.17	1.14	1.14	1.14	1.13
NNW	309	1.00	1.01	1.02	1.03	1.03	1.03	1.03	1.03
Average	8626	1.00	1.01	1.02	1.02	1.02	1.02	1.02	1.02

Downwind	Distance From Release Point (Miles)								
Sector	No. OBS	3.50	4.00	4.50	5.00	7.50	10.00	15.01	20.00
N	386	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NNE	525	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
NE	670	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ENE	763	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
E	891	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
ESE	1530	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SE	858	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSE	334	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
S	328	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SSW	221	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
SW	348	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
WSW	316	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.02
W	385	1.05	1.06	1.04	1.04	1.05	1.05	1.05	1.05
WNW	351	1.08	1.08	1.08	1.09	1.09	1.10	1.10	1.09
NW	411	1.11	1.11	1.11	1.11	1.11	1.11	1.10	1.10
NNW	309	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
Average	8626	1.02	1.02	1.02	1.02	1.02	1.01	1.01	1.01

TABLE 2.4-1 TIDAL FLOOD ELEVATIONS BOSTON, MASSACHUSETTS TO PORTLAND, MAINE

	TIDAL ELEVATIONS (FT. MSL)		
STORM	BOSTON (OBSERVED)	NEW HAMPSHIRE (ESTIMATED)	PORTLAND (OBSERVED)
9-21-38*	6.4	6.5	6.8
9-14-44*	6.6	5.4	5.0
11-30-44	8.8	8.4	8.7
11-20-45	7.9	8.0	8.7
8-31-54*	8.2	7.7°	7.9
12-29-59	9.3	8.4	8.5
1-20-61	9.3	8.3	8.3
11-30-63	7.4	7.5	7.9
2-7-78	10.3	9.1+	9.6

*Hurricane +Record High °Record Hurricane High

TABLE 2.4-2 ANNUAL FREQUENCY OF OCCURRENCE FOR TIDES* IN EXCESS OF MHW

Portsmouth Navy Yard, Maine

Height Above MHW	Average Annual Occurrence
1 Foot	107 times
2 Feet	12 times
3 Feet	0.51 times
3.5 Feet	0.17 times

* Astronomical tide combined with surge

TABLE 2.4-3 **TIDES EXCEEDING MHW AT PORTSMOUTH NAVY YARD, MAINE**

Feet in Excess of MHW	Number of Occurrences
2.0	54
2.1	42
2.2	40
2.3	18
2.4	18
2.5	14
2.6	10
2.7	3
2.8	2
2.9	5
3.0	2
3.1	1
3.2	1
3.3	2
3.4	0
3.5	1
3.6	1
3.7	0
3.8	0
3.9	1 (11-30-44 storm)

TABLE 2.4-4 COMPARISON OF HAMPTON HARBOR TIDAL PARAMETERS

	Hampton Harbor
Mean Tidal Range	8.3 feet
Spring Tidal Range	9.5 feet
Highest Predicted Astronomical Tide*	10.8 feet MLW
Mean High Water (MHW)	8.3 feet MLW
Mean Sea Level (MSL)	4.15 feet MLW
Mean Low Water (MLW)	0.00 feet MWL
Lowest Predicted Astronomical Tide*	-2.2 feet MLW

* Based on Portland, Maine tidal information and converted to Hampton Harbor.

TABLE 2.4-5 **PREDICTED ASTRONOMICAL TIDES FOR HAMPTON HARBOR**

Year	Highest Tide of Year Feet MLW	Number of Times High Tide of 10.6 Feet MLW is Exceeded
1978	10.6	0
1977	10.7	4
1976	10.7	6
1975	10.6	0
1974	10.5	0
1973	10.6	0
1972	10.6	0
1971	10.6	0
1970	10.3	0
1969	10.4	0
1968	10.5	0
1967	10.5	0
1966	10.3	0
1965	10.4	0
1964	10.6	0
1963	10.6	0
1962	10.4	0
1961	10.7	1
1960	10.8	4
1959	10.8	7
1958	10.7	1
1957	10.6	0
1956	10.6	0
1955	10.7	2
1954	10.6	0

Reference: 34

TABLE 2.4-6 OPEN COAST PMH STORM SURGE RESULTS

Run No.	Radius of Maximum Winds R(Nautical Miles)	Translational Velocity V_T (Knots)	Maximum Open Coast Stillwater Elevation		Wind Stress Coefficients X10^{-6}	
			Feet Above MLW	Feet Above MSL	Constant	Constant Multiplier
1	30	37	16.23	12.08*	1.1	1.6
2	56	37	16.48	12.33*	1.1	1.6
3	30	52	16.36	12.21*	1.1	1.6
4	56	52	16.59	12.44*	1.1	1.6
5	56	52	17.5	13.4**	1.1	1.6
6	56	52	18.6	14.5**	1.1	2.5

* Includes storm surge and astronomical tide of +10.6 feet MLW.

** Includes storm surge, astronomical tide of +10.6 feet (MLW) and initial surge of 0.9 feet (sea-level anomaly).

TABLE 2.4-7 HAMPTON HARBOR SURGE, CASE NO. 1 - COEFFICIENT OF DISCHARGE = 0.55

Surge Hydrograph in an Enclosed Bay Due to Inlet Flow

Time in Hours	Open Coast Surge (HO) in Feet*	Surge in Bay (H) in Feet*
0.00	0.000	0.000
0.20	0.200	0.076
0.40	0.500	0.225
0.60	0.800	0.424
0.80	1.000	0.641
1.00	1.300	0.874
1.20	1.600	1.133
1.40	2.000	1.426
1.60	2.400	1.759
1.79	2.800	2.122
2.00	3.700	2.585
2.20	4.100	3.119
2.40	4.800	3.685
2.59	5.600	4.336
2.80	6.000	4.999
3.00	6.600	5.641
3.20	7.600	6.372
3.40	8.000	7.119
3.59	8.700	7.828
3.80	9.300	8.533
4.00	10.000	9.172
4.19	10.800	9.802
4.40	11.600	10.447
4.60	12.600	11.133
4.80	13.400	11.843
5.00	14.400	12.574
5.19	15.400	13.345
5.40	16.000	14.100
5.60	16.300	14.771
5.80	16.600	15.351
6.00	16.400	15.798
6.19	16.200	16.059
6.40	15.600	15.941
6.60	14.800	16.624
6.80	14.000	15.218
7.00	13.400	14.770
7.19	12.800	14.300
7.39	11.800	13.793
7.60	11.000	13.248
7.80	10.200	12.679
8.00	9.500	12.089
8.20	9.000	11.486

* Feet above MLW

TABLE 2.4-8 HAMPTON HARBOR SURGE CASE NO. 2 – COEFFICIENT OF DISCHARGE = 0.60

SURGE HYDROGRAPH IN AN ENCLOSED BAY DUE TO INLET FLOW

TIME IN HOURS	OPEN COAST SURGE (HO) IN FEET*	SURGE IN BAY (H) IN FEET*
0.00	0.000	0.000
0.20	0.200	0.081
0.40	0.500	0.240
0.60	0.800	0.450
0.80	1.000	0.676
1.00	1.300	0.917
1.20	1.600	1.183
1.40	2.000	1.485
1.60	2.400	1.827
1.79	2.800	2.200
2.00	3.700	2.679
2.20	4.100	3.229
2.40	4.800	3.808
2.59	5.600	4.474
2.80	6.000	5.145
3.00	6.600	5.786
3.20	7.600	6.524
3.40	8.000	7.271
3.59	8.700	7.972
3.80	9.300	8.662
4.00	10.000	9.283
4.19	10.800	9.912
4.40	11.600	10.563
4.60	12.600	11.261
4.80	13.400	11.984
5.00	14.400	12.731
5.19	15.400	13.518
5.40	16.000	14.287
5.60	16.300	14.963
5.80	16.600	15.539
6.00	16.400	15.966
6.19	16.200	16.163
6.40	15.600	15.987
6.60	14.800	15.634
6.80	14.000	15.193
7.00	13.400	14.709
7.19	12.800	14.207
7.39	11.800	13.664
7.60	11.000	13.080
7.80	10.200	12.470
8.00	9.500	11.836
8.20	9.000	11.186

*Feet Above MLW

TABLE 2.4-9 HAMPTON HARBOR SURGE CASE NO. 2 – COEFFICIENT OF DISCHARGE = 0.65

SURGE HYDROGRAPH IN AN ENCLOSED BAY DUE TO INLET FLOW

TIME IN HOURS	OPEN COAST SURGE (HO) IN FEET*	SURGE IN BAY (H) IN FEET*
0.00	0.000	0.000
0.20	0.200	0.087
0.40	0.500	0.254
0.60	0.800	0.474
0.80	1.000	0.708
1.00	1.300	0.956
1.20	1.600	1.228
1.40	2.000	1.537
1.60	2.400	1.887
1.79	2.800	2.267
2.00	3.700	2.760
2.20	4.100	3.325
2.40	4.800	3.914
2.59	5.600	4.593
2.80	6.000	5.269
3.00	6.600	5.907
3.20	7.600	6.650
3.40	8.000	7.394
3.59	8.700	8.087
3.80	9.300	8.755
4.00	10.000	9.368
4.19	10.800	10.000
4.40	11.600	10.659
4.60	12.600	11.369
4.80	13.400	12.106
5.00	14.400	12.866
5.19	15.400	13.670
5.40	16.000	14.450
5.60	16.300	15.130
5.80	16.600	15.699
6.00	16.400	16.104
6.19	16.200	16.229
6.40	15.600	16.988
6.60	14.800	15.616
6.80	14.000	15.144
7.00	13.400	14.630
7.19	12.800	14.099
7.39	11.800	13.526
7.60	11.000	12.907
7.80	10.200	12.258
8.00	9.500	11.585
8.20	9.000	10.896

*Feet Above MLW

TABLE 2.4-10 **EFFECTIVE FETCHES IN HAMPTON HARBOR DURING COMBINED PMH – SPF EVENT**

TIME HRS	WIND DIRECTION	WIND SPEED FPS	EFFECTIVE FETCH LENGTH FT.	AVERAGE WATER DEPTH FT.	MAXIMUM WAVE HT. FT.	SIGNIFICANT WAVE HEIGHT FT.	GENERATED WAVE PER. SEC
-1.4	N44°E	143.6	11,800	8.2	6.4	4.2	3.7
-1.0	N74°E	155.4	10,750	13.0	8.6	5.7	4.3
- .80	N85°E	157.3	10,460	15.3	9.0	5.9	4.4
- .60	S85°E	159.3	10,160	16.2	9.6	6.3	4.5
- .40	S75°W	163.9	10,120	15.4	9.6	6.3	4.5
- .20	S64°E	167.1	11,400	16.6	10.3	6.8	4.7
0	S58°E	168.4	12,300	17.0	10.6	7.0	4.7
+ .5	S52°E	152.8	12,600	16.4	9.7	6.4	4.6
+1.0	S40°E	124.0	12,200	10.6	6.7	4.4	3.8
+1.6	S34°E	104.5	11,800	9.1	5.5	3.6	3.5

Time ‘0’ corresponds to arrival of peak stillwater level at site.

TABLE 2.4-11 WAVE PERIOD ANALYSIS FOR HAMPTON HARBOR DURING COMBINED PMH-SPF EVENT

Time Hrs.	Fetch Direction	Diffraction Coefficient at Site	Diffraction Wave Height Ft.	Maximum Wave Height Ft.	Significant Wave Height Ft.	Regenerated Wave per Ft.
-1.0	N74°E	0.2	3.2	8.0*		4.3
-0.8	N85°E	0.3	5.2	8.5*		4.7
-0.6	S85°E	0.75	13.6	9.8*		5.4
-0.4	S75°W	0.45	8.4	10.7*		5.2
-0.2	S64°E	0.2	3.8	10.5	6.9	4.8
0	S58°E	0.17	3.6	10.9	7.2	4.8
+0.6	S50°E	0.14	2.8	8.0	5.3	4.2

* Waves controlled by available water depth

TABLE 2.4-12 DESIGN BASIS WAVES AND COINCIDENT STILLWATER LEVEL ON CRITICAL SECTIONS OF PLANT

SECTION	STILLWATER LEVEL Ft.—MLW	DESIGN WAVE HGT., FT.	DESIGN WAVE PERIOD, SEC.
1	19.7	2.0	4.8
4	19.7	5.8	4.8
5	19.7	7.9	4.8
6	19.7	7.9	4.8
7	19.7	3.9	4.8
8	18.1	2.1	4.3

TABLE 2.4-13 Flow Pathway Discharges (CFS)

Water Elev. Behind Seawall (ft msl)	Flow Pathways					
	1	2	3	4	5	Combined
20.6	86	399	209	0	0	694
20.7	113	504	254	0	0	871
20.8	142	618	300	0	0	1060
20.9	173	739	348	0	3	1263
21.0	210	867	398	5	6	1486
21.1	248	1002	449	10	10	1719

TABLE 2.4-14 REVETMENT STONE SIZES

Revetment A

Class	Weight	Thickness, Ft.
Armor Stone*	1.5 T to 3.0 T	6
A Layer	300 lbs to 600 lbs	3
B Layer	15 lbs to 30 lbs	1.2

Revetment B

Class	Weight	Thickness, Ft.
B Layer	15 lbs to 30 lbs	1 – 2.5

Revetment C

Class	Weight	Thickness, Ft.
Armor Stone*	0.5 T to 1.0 T	4.8
A Layer	50 lbs to 200 lbs	2.2

* K_D = 3.0, Density of Rock = 165 lbs/cu. ft.

TABLE 2.4-15 **WAVE FORCES ON VERTICAL SEA WALL SOUTH OF CONTAINMENT BUILDING NO. 1**

WAVE FORCES \ WAVE CONDITIONS			BREAKING WAVE (MINIKIN METHOD)	BROKEN WAVE (WALL SEAWARD OF SHORELINE)
STATIC FORCES	Pressure (psf)		804	905
	Thrust (lbs/ft of wall		5,047	6,398
DYNAMIC FORCES	Pressure (psf)	10% slope	9,136	381
		5% slope	10,097	
	Thrust (lbs/ft of wall)	10% slope	24,089	2,111
		5% slope	26,622	

H = 7.9 ft. (Wave Height)

T = 4.8 sec. (Wave Period)

d_s = 8.6 ft. (Depth from stiliwater level to toe of vertical wall)

TABLE 2.4-16 WAVE FORCES ON RETAINING WALL NORTH SIDE OF SITE

WAVE FORCES \ WAVE CONDITIONS		NONBREAKING WAVES (Sainflou Method)
STATIC FORCES	Pressure (psf)	490
	Thrust (lbs/ft of wall	1,862
DYNAMIC FORCES	Pressure (psf)	104
	Thrust (lbs/ft of wall	1,078

H = 2.0 ft. (Wave Height)

T = 4.8 sec. (Wave Period)

d_s = 7.6 ft. (Depth from stiliwater level to toe of vertical wall)

TABLE 2.4-17 DRILLED WELL SUMMARY SHEET – SEABROOK STATION

Well No.	Mag. Grid Location Plant Grid Location N. Coord.	E. Coord.	Ground[1] Elev.	Depth (Feet)	Bottom Elev.	Inflow Depth(s)	Inflow Elev. (s)	Est. Yield (GPM)	Test[2] Yield (GPM)	Soil Depth (Ft)	Casing Depth (Ft)	Hole Diam. (In.)
1	MG 23698 PG 13143	79683 7000	22.50	143	-120.5	130	-107.5	80	70.8	6	21	6.0
2	MG 23466 PG 13029	79325 6589	16.00	205	-189	100 130 160	-84 –114 –144	15 20 45	90.0	20	42	6.5
3	MG 23480 PG 12910	79765 7013	13.00	175	-162	170	-157	80	100.2	27?	27	6.5
4	MG 23874 PG 13339	79589 6964	17.50	205	-187.5	?	?	12	---	21?	21	6.5
5	MG 21338 PG 12012	75974 2752	40.67	310	-269.3	160 160+	-119	15 5	27	32	43	6.5
6	MG 21035 PG 11405	77027 3664	31. 86	295	-263.1	85 265	-53 –233	2 to 3 25	33.3	11	21	6.5
7	MG 20676 PG 11083	76962 3494	36.76	400	-363.2	?	?	1	---	45+	68	6.5
8	MG 19665 PG 10428.2	75937 2212.3	43.51	400	-356.5	340	-296	45	40	62	92	6.0

Well No.	Mag. Grid Location Plant Grid Location N. Coord.	E. Coord.	Ground[1] Elev.	Depth (Feet)	Bottom Elev.	Inflow Depth(s)	Inflow Elev. (s)	Est. Yield (GPM)	Test[2] Yield (GPM)	Soil Depth (Ft)	Casing Depth (Ft)	Hole Diam. (In.)
9	MG19653 PG 10565.6	75446 1739.7	45.16	400	-354.8	220±	-175	2	---	78	127	6.0
10	MG 19644 PG 10269.9	76394 2641.0	39.89	400	-360.1	150± 150±	-110	2.0 1.5	---	8	30	6.0
11	PG 14968.2	7130.7	19.24	246	-220.8	75 115 135 230	-55.8 –95.8 –115.8 –210.8	2 3 5 10	---	41	51	6.0
12	PG 14973.2	7147.8	18.39	41	-22.6	?	?	1.5	---	41	Set screen @ 25.1	
13	PG 13671.3	6515.3	21.97	240	-218	68 102	-46 –80	7 5	---	34	41	6.0
14	PG 12761.5	7465.4	12.51	200	-187.5	60 80 125 140	-47.5 –67.5 –112.5 -127.5	0.5 12+ 10+ 10+	---	6	21	6.0
15	MG 23665 PG 13176	79467 6784	17.75	200	-182.3	115 152	-97.3 -134.3	---	---	33	35	6.5

1. Top of Casing
2. 48-Hour Specific Capacity Pump Test

Note: Wells 4, 13, and 14 for observational purposes only. Wells 7, 9,10, 11 and 12 not developed due to insufficient water.

TABLE 2.4-18 RECORDS OF SELECTED WELLS AND TEST HOLES IN SOUTHEASTERN NEW HAMPSHIRE

Well no.	Location	Owner or user	Year completed	Altitude of land-surface datum (feet)	Depth (feet)	Type of well	Diameter of well (inches)	Water-bearing material		Water level		Type of pump	Use	Remarks
								Character	Geologic unit	Depth	Date			
								SEABROOK						
1	W16-9	Town of Seabrook	1956	100	54	Dr	24	Sand and gravel	Ice-contact deposits	5.76	4-25-56	T	PS	Reported yield 450 gpm. T 47.
2	W16-9	do.	1956	105	49	Dr	24	do.	do.	5	-56	T	PS	Reported yield 350 gpm.
3	W16-9	L. R. Matthews	1930	105	110	Dr	6	-	Bedrock	35	-30	F	D	
4	W16-9	R. E. Bergeron	1952	100	26.5	Dg	36	Sand and gravel	Ice-contact deposits	17.92	5-9-56	J	D	
5	W16-9	Joseph Neves	1900	63	12.4	Dg	30	-	Till	3.00	5-9-56	S	D	Not in use in 1956.
6	X16-7	Parkman Clinic	-	45	13.3	Dg	36	Sand	Outwash and shore deposits	3.93	5-9-56	S	D	
7	X15-1	Lloyd Property	1900	63	22.0	Dg	36	do.	do.	16.08	4-25-56	S	D	
8	X16-8	Town of Seabrook	1955	14	15.0	Dg	96	do.	Beach deposits	9.26	4-25-56	None	PS	Fire protection well. Reported yield 60 gpm
9	X15-1	Carroll F. Randall	1946	65	150	Dr	6	-	Bedrock	-	-	J	D	
10	X16-7	T. L. Boyd	1930	25	9.0	Dg	18	Sand	Outwash and shore deposits	2.35	5-9-56	S	D	
11	X16-7	Dearbon Academy Assoc.	1900	58	10.8	Dg	36	do.	do.	7.15	5-9-56	None	U	
12	W16-9	Town of Seabrook	1955	45	16.5	Dn	2 ½	-	-	-	-	None	T	
13	W16-9	do.	1955	47	28.7	Dn	2 ½	-	-	4	-55	None	T	
14	W16-9	do.	1955	50	43.0	Dn	2 ½	-	-	+2	-55	None	T	
15	W16-9	do.	1955	54	45.0	Dn	2 ½	-	-	1.9	-55	None	T	
16	X16-7	do.	1955	45	94.3	Dn	2 ½	-	-	2.8	-55	None	T	
17	W16-9	do.	1955	40	33.5	Dn	2 ½	-	-	+2.8	-55	None	T	Reported natural flow 7 gpm.
18	X16-7	do.	1955	43	37.3	Dn	2 ½	-	-	9.3	-55	None	T	
19	X15-1	do.	1955	48	41.5	Dn	2 ½	-	-	2	-55	None	T	
20	X15-1	do.	1955	18	53.5	Dn	2 ½	-	-	4.3	-55	None	T	
21	W16-9	do.	1955	100	54.8	Dn	2 ½	-	-	6.7	-55	None	T	At same location as Seabrook 1.
22	W16-9	do.	1955	50	41.0	Dn	2 ½	-	-	Flowing	-55	None	T	Reported natural flow 6 gpm.
23	X15-1	do.	1955	60	61.3	Dn	2 ½	-	-	2	-55	None	T	
24	W16-9	do.	1955	65	38.9	Dn	2 ½	-	-	+4	-55	None	T	
25	W16-9	do.	1955	70	52.7	Dn	2 ½	-	-	2.8	-55	None	T	

Well no.	Location	Owner or user	Year completed	Altitude of land-surface datum (feet)	Depth (feet)	Type of well	Diameter of well (inches)	Water-bearing material		Water Level		Type of pump	Use	Remarks
								Character	Geologic unit	Depth	Date			
SOUTH HAMPTON														
1	W16-7	Guy E. Kenerson	1938	188	75	Dr	6	-	Bedrock	-	-	F	D	Reported yield 7 gpm.
2	W16-7	do.	1900	190	19.0	Dg	36	Sand and gravel	Ice-contact deposits	4.92	5-14-56	None	U	
3	W15-1	R. E. Lowry	1935	87	53	Dr	6	-	Bedrock	-	-	F	D	
4	W16-8	Edith M. Spurr	1945	250	200	Dr	6	-	do.	17	-45	J	D	Reported yield 12 gpm.
5	W16-7	Albert E. Gray	1948	130	123	Dr	6	-	do.	Flowing	-48	S	D	Reported yield 10 gpm. T 50.
6	W16-7	Edmund Roy	1955	110	19.8	Dg	24	Sand	Outwash and shore deposits	4.33	5-18-56	S	D	
7	W16-8	Adam J. Mazur	1900	185	13.0	Dg	36	-	Till	5.30	5-18-56	S	D	Not in use in 1956.
NORTH HAMPTON														
1	X17-8	Paul Kelley	1954	110	138	Dr	8	-	Bedrock	20	-54	J	D	Reported yield 3 gpm.
2	X17-7	Lora Booker	1900	100	42.3	Dg	36	-	Till	35.31	4-13-56	L	D	Not in use in 1956.
3	X16-1	Charles Black	1900	70	23.8	Dg	36	-	do.	17.61	4-12-56	S	D	
4	X16-1	K. D. Bowers	1956	105	105	Dr	6	-	Bedrock	12	-56	J	D	Reported yield 4 gpm.
5	X16-1	R. A. Wright	1935	100	32.9	Dg	24	Sand and gravel	Ice-contact deposits	25.22	4-12-56	L	D	
6	X16-2	Wallace P. Hale	1954	122	100	Dr	8	-	Bedrock	45	-54	J	D	
7	X16-2	Hampton Water Works	1919	65	22	Dg	240	Sand and gravel	Ice-contact deposits	4.07	4-11-56	C	PS	Reported yield 450 gpm.
8	X16-2	do.	1919-1937	65	42	Dg-Dr	240-18	do.	do.	3.27	4-11-56	T	PS	Reported yield 450 gpm; 18-inch casing Installed inside 240-inch dug well in 1937.
9	X16-2	Hinckle Property	1900	30	18.5	Dg	36	-	Till	3.11	4-17-56	S	D	
10	X16-2	Mrs. J. Marshall	1953	83	175	Dr	6	-	Bedrock	Flowing	-53	J	D	Reported yield 8 gpm.
11	X16-1	Mrs. Irving Marsten	1948	85	74	Dr	6	Sand and gravel	Ice-contact deposits	32	-48	F	D	Reported yield 20 gpm.
12	X16-1	F. S. Snow	1947	95	179	Dr	6	-	Bedrock	23	-47	J	D	Reported yield 5 ½ gpm. T 55.
13	X16-1	Kenneth S. Ellingwood	1947	65	50	Dr	6	-	do.	12	-47	J	D	Reported yield 6 gpm.
14	X16-1	Abraham Lampert	1948	108	170	Dr	6	-	do.	26	-48	J	D	L. Reported yield 10 gpm. T 48.
HAMPTON														
1	W16-3	Charles Mathews	1900	142	28.0	Dg	42	-	Till	18.82	12-8-53	S	U	T 51
2	X16-5	Otis Garland	1900	47	30.0	Dg	36	Sand and gravel	Ice-contact deposits	27.95	12-11-53	L	U	T 51
3	X16-5	Hampton Water Works	1937	75	54	Dr	18	do.	do.	8	-37	T	PS	Reported yield 460 gpm. T 52.
4	X16-1	Mrs. O. D. Colvin	1900	100	15.0	Dg	36	-	Till	5.89	4-11-56	S	D	
5	X16-4	Robert F. Walker	1952	60	32	Dg	42	Sand	Ice-contact deposits	7.04	4-12-56	S	D	
6	X16-5	Hampton Water Works	1956	75	45.0	Dn	2 ½	Sand and gravel	do.	1.77	4-11-56	None	O	Reported yield 100 gpm.
7	X16-5	do.	1950	45	54	Dr	36	do.	do.	2	-50	T	PS	Reported yield 720 gpm.
8	X16-4	Godfrey Dearbon	1926	85	19.0	Dg	24	do.	do.	4.05	4-17-56	S	D	
9	X16-4	Ernest Woodburn	1900	60	14.0	Dg	24	-	Till	4.18	4-19-56	S	D	
10	X16-3	Deborah G. Bryer	1937	125	160	Dr	6	-	Bedrock	36	-37	F	D	Reported yield 15 gpm.
11	X16-1	Edwin L. Batchelder	1940	70	232	Dr	6	-	do.	11	-40	T	D	Reported yield 4 gpm.
12	X16-5	Gordon Yeaton	1913	77	90	Dr	6	-	do.	20	-13	J	D	Reported yield 5 ½ gpm.

Well no.	Location	Owner or user	Year completed	Altitude of land-surface datum (feet)	Depth (feet)	Type of well	Diameter of well (inches)	Water-bearing material		Water Level		Type of pump	Use	Remarks
								Character	Geologic Unit	Depth	Date			
							HAMPTON FALLS							
1	W16-9	J. M. Goodwin	1900	85	8.5	Dg	48	Sand	Ice-contact deposits	1.91	12-4-53	S	D	T 49
2	X16-7	R. P. Merrill	1945	68	20.0	Dg	18	do.	Outwash and shore deposits	11.45	4-17-56	S	D	
3	X16-7	E. J. Payne	1955	67	101	Dr	6	-	Bedrock	9	-55	J	D	Reported yield 16 gpm.
4	W16-6	Oscar McKenney	1900	90	25.3	Dg	28	-	Till	4.47	4-19-56	S	U	
5	W16-6	do.	1951	90	95	Dg	6	-	Bedrock	-	-	J	D	Reported yield 7 ½ gpm.
6	W16-6	Mark Kelly	1900	112	38.2	Dg	42	Sand and gravel	Ice-contact deposits	21.00	4-17-56	F	D	
7	W16-6	Ralph M. Farley	1954	60	120	Dr	6	-	Bedrock	5	-54	J	D	Reported yield 25 gpm.
8	W16-6	Donald Merchant	1955	65	17	Dg	48	Sand and gravel	Ice-contact deposits	15	8- -55	S	D	
9	W16-3	Alfred L. Binnette	1900	115	21.0	Dg	36	-	Till	5.65	4-19-56	S	D	
10	W16-6	V. L. Yeaton	1947	103	100	Dr	6	-	Bedrock	19	-47	F	D	L. Reported yield 2 ½ gpm.
11	W16-6	J. W. Elton	1942	110	150	Dr	6	-	do.	45	-42	J	D	T. Reported yield 45 gpm. Additional use, orchard.
12	X16-4	Eugene Whittemore	1948	65	140	Dr	6	-	do.	13	-48	J	D	Reported yield 25 gpm.
13	X16-7	C. M. Wellington	1948	60	120	Dr	6	-	do.	8	-48	F	D	Reported yield 15 gpm.
14	X16-7	Nicholas A. Natale	1947	30	94	Dr	6	-	do.	15	-47	J	D	Reported yield 20 gpm.
15	W16-6	William H. Coburn	1952	115	250	Dr	6	-	do.	-	-	L	D	Reported yield 35 gpm.
							KENSINGTON							
1	W16-4	Gordon Swift	1953	217	13.0	Dg	40	-	Till	9.56	7-13-54	None	U	
2	W16-4	Betsy J. Monahan	1900	270	25.0	Dg	30	-	do.	10.52	7-13-54	None	U	
3	W16-5	J. W. York	1953	75	108	Dr	6	-	Bedrock	-	-	J	D	Reported yield 6 gpm.
4	W16-5	F. E. Toothacre	1926	107	84	Dr	6	-	do.	22	-26	J	D	Reported yield 2 ½ gpm.
5	W16-4	Charles Matthews	1954	225	120	Dr	6	-	Till and Bedrock	28	-54	L	D	Reported yield 5 gpm.
6	W16-4	C. R. Hutchinson	1952	250	220	Dr	6	-	do.	30	-52	J	D	Reported yield 20 gpm.
7	W16-5	Kensington School	1952	128	50	Dr	6	Sand and gravel	Ice contact deposits	17	-52	J	PS	Reported yield 30 gpm.
8	W16-5	Mrs. Alice E. Bragg	1931	123	60	Dr	6	do.	Ice contact deposits and Bedrock	-	-	L	D	Reported yield 20 gpm.
9	W16-4	Amos S. Gove	1900	223	25.0	Dg	36	-	Till	7.53	5-21-56	H	D	
10	W16-8	A. Mertinooke	1915	170	23.0	Dg	36	-	do.	10.95	5-21-56	S	D	T 52.
11	W16-8	Leavitt Brown	1910	90	23.5	Dg	36	Sand and gravel	Ice-contact deposits	6.15	5-21-56	S	D	

RECORDS OF SELECTED WELLS AND

TEST HOLES IN SOUTHEASTERN NEW HAMPSHIRE

Owner or user: Name of present owner or agency responsible for installation or operation of well.

Year completed: Year when well was completed, if known. Wells completed prior to 1900 are not specifically dated unless exact year is known.

Altitude: Altitude expressed in feet and tenths, or in feet, tenths, and hundredths are instrumentally determined; those in whole feet are interpolated from topographic maps. Datum is mean sea level.

Depth: Depths expressed in feet and tenths are measured; those in whole feet are reported. Depths are below land-surface datum.

Type of well and diameter of well: Dg, dug; Dn, Driven; Dr, drilled; Dg-Dr, dug and drilled.

Water-bearing material: For explanation of geologic units from which water is drawn, see table 1 and accompanying test.

Water level: Water levels expressed in feet and tenths, or in feet, tenths, and hundredths are measured; those in whole feet are reported. Depths are below land-surface datum, except when proceeded by + indicating they are above land- surface datum.

Type of pump: C, centrifugal pump; F, force pump; H, hand drawn; J, jet pump; L, lift pump; 5, suction; Sb, submersible pump; T, turbine pump.

Use: C, use in cemetery; D, domestic or domestic and farm; I, Industrial or commercial; Ir, irrigation; 0, well installed as an observation or test well; PS, public supply; S, use for stock only; T, test hole or test well, now abandoned and casing removed; U, unused.

Remarks: Other available data are indicated as follows: D, destroyed; dd, drawdown; gpm, gallons per minute; T, temperature in degrees Fahrenheit.

SEABROOK UPDATED FSAR

TABLE 2.4-19

BORING LOGS OF SELECTED WELLS AND TEST HOLES

	Thickness (Feet)	Depth (Feet)
SEABROOK 14. W16-9. Alt. 50 ft.		
Lat. 42°53'29". Long. 70°52'41".		
Outwash and shore deposits:		
Peat	1.7	1.7
Sand, light brown; some gravel, fine	7.3	9.0
Marine deposits:		
Clay, hard, brown	4.0	13.0
Sand, fine, brown	1.3	14.3
Sand, fine, and clay, gray; changing to clay, gray	8.2	22.5
Clay, soft, gray	18.5	41.0
Till:		
Sand and gravel, sharp, and clay, gray	2.0	43.0
Refusal		43.0
SEABROOK 15. W16-9. Alt. 54 ft.		
Lat. 42°53'34". Long. 70°52'35".		
Outwash and shore deposits:		
Sand and gravel, brown	14.0	14.0
Marine deposits:		
Sand, fine, and clay, gray	9.8	23.8
Sand, fine, and clay, gray; changing to clay	10.2	34.0
Till:		
Sand and gravel, sharp, and clay, blue	10.3	44.3
Unreported	0.7	45.0
Refusal		45.0
SEABROOK 16. X16-7. Alt. 45 ft.		
Lat. 42°53'37". Long. 70°51'54".		
Outwash and shore deposits:		
Gravel, small	8.0	8.0
Sand, fine, brown	5.0	13.0
Sand, fine, and clay, brown	5.3	18.3
Sand and clay, red-brown	1.5	19.8
Marine deposits:		
Clay, light gray, and sand, fine	12.2	32.0
Clay, dark brown	3.5	35.5
Sand, medium-fine, brown	1.5	37.0
Sand, fine, and clay, gray	5.0	42.0
Sand, fine, and clay in layers, gray	28.0	70.0
Till:		
Sand, gravel, sharp, and clay, gray	7.9	77.9
Gravel, sharp, gray, and boulders; some clay	9.6	87.5
Sand and gravel, sharp, and clay, gray	6.8	94.3
Refusal		94.3
SEABROOK 17. W16-9. Alt. 40 ft.		
Lat. 42°53'01". Long. 70°52'48".		
Outwash and shore deposits:		
Sand and gravel, light brown	3.5	3.5
Marine deposits:		
Clay, gray	18.2	21.7
Sand, fine, and clay, gray	11.8	33.5
SEABROOK 18. X16-7. Alt. 43 ft.		
Lat. 42°52'50". Long. 70°51'48".		
Outwash and shore deposits:		
Sand, fine, brown	10.1	10.1
Marine deposits:		
Sand, fine, brown, and clay, gray	5.2	15.3
Clay, gray	14.8	30.1
Clay, gray, and sand	7.2	37.3
Refusal		37.3
SEABROOK 19. X15-1. Alt. 48 ft.		
Lat. 42°52'07". Long. 70°51'35".		
Outwash and shore deposits:		
Loam	1.0	1.0
Sand, brown	2.0	3.0
Sand and gravel	3.0	6.0
SEABROOK 17.--Continued		
Marine deposits:		
Sand, fine, and clay, brown	2.0	8.0
Sand, gray, changing to sand, fine, and clay	14.6	22.6
Clay, hard, brown	3.4	26.0
Ice-contact deposits:		
Sand, medium, gray; some gravel, fine	15.0	41.0
Till:		
Sand, gravel, and clay, gray	2.5	43.5
Refusal		43.5
SEABROOK 20. X15-1. Alt. 18 ft.		
Lat. 42°52'19". Long. 70°50'37"		
Outwash and shore deposits:		
Sand, medium-coarse	10.0	10.0
Sand and gravel; cemented layer at 19 feet	9.0	19.0
Marine deposits:		
Sand, fine, and clay, gray	28.5	47.5
Till:		
Sand, fine, and gravel, gray	6.0	53.5
SEABROOK 21. W16-9. Alt. 100 ft.		
Lat. 42°53'42". Long. 70°54'44".		
Ice-contact deposits:		
Sand and gravel	23.3	23.3
Sand and gravel, fine	10.6	33.9
Sand, coarse, and gravel	5.0	38.9
Sand and gravel, fine	10.2	49.1
Till:		
Sand and gravel, some clay	5.7	54.8
Refusal		54.8
SEABROOK 22. W16-9. Alt. 50 ft.		
Lat. 42°53'52". Long. 70°54'07".		
Marine deposits:		
Clay, brown	8.0	8.0
Clay, gray	27.9	35.9
Ice-contact deposits:		
Sand and gravel, blue	5.1	41.0
SEABROOK 23. X15-1. Alt. 60 ft.		
Lat. 42°52'29". Long. 70°51'18".		
Ice-contact deposits:		
Sand and gravel, brown	7.1	7.1
Sand, fine, brown	5.2	12.3
Sand, fine to medium, tight, brown	44.0	56.3
Till:		
Sand, fine to medium, and clay	5.0	61.3
Bedrock		61.3
SEABROOK 24. W16-9. Alt. 65 ft.		
Lat. 42°53'58". Long. 70°54'31".		
Marine deposits:		
Clay	7.1	7.1
Clay, gray	10.8	17.9
Clay, gray, and sand, fine	16.6	34.5
Ice-contact deposits:		
Gravel, blue, sand, fine, and clay, gray	4.4	38.9
SEABROOK 25. W16-9. Alt. 70 ft.		
Lat. 42°53'33". Long. 70°54'28".		
Outwash and shore deposits:		
Sand, fine, brown	19.5	19.5
Marine deposits:		
Sand, fine; a little clay, brown	10.3	29.8
Ice-contact deposits:		
Sand, fine, brown	10.7	40.5
Till:		
Sand, gravel, and clay, gray	12.2	52.7
Refusal		52.7
SOUTH HAMPTON 1. W16-7. Alt. 188 ft.		
Lat. 42°52'56". Long. 70°57'52".		
Unconsolidated deposits, undifferentiated:		
Sand, fine, yellow, and gravel	40	40
Bedrock	35	75
SOUTH HAMPTON 3. W15-1. Alt. 87 ft.		
Lat. 42°52'25". Long. 70°57'41".		
Marine deposits:		
Silt and clay	52	52
Bedrock	1	53
SOUTH HAMPTON 4. W16-8. Alt. 250 ft.		
Lat. 42°53'58". Long. 70°56'06".		
Till:		
Clay and boulders	100	100
Bedrock	100	200
SOUTH HAMPTON 5. W16-7. Alt. 130 ft.		
Lat. 42°53'29". Long. 70°58'56".		
Till:		
Gravel and clay	56	56
Bedrock	67	123
HAMPTON 3. X16-5. Lat. 42°57'29".		
Long. 70°49'35". Alt. 75 ft.		
Ice-contact deposits:		
Sand and gravel	9.0	9.0
Clay and sand, fine	0.5	9.5
Sand and gravel	48.5	58.0
Refusal		58.0
HAMPTON 7. X16-5. Lat. 42°56'11".		
Long. 70°48'46". Alt. 45 ft.		
Ice-contact deposits:		
Sand and gravel	54	54
HAMPTON 10. W16-3. Lat. 42°57'43".		
Long. 70°53'12". Alt. 125 ft.		
Unconsolidated deposits, undifferentiated	60	60
Bedrock:		
Gneiss	100	160
HAMPTON 11. X16-1. Lat. 42°57'32".		
Long. 70°51'55". Alt. 70 ft.		
Till:		
Sand, gravel, and clay	30	30
Bedrock	202	232
HAMPTON 12. X16-5. Lat. 42°56'42".		
Long. 70°49'07". Alt. 77 ft.		
Ice-contact deposits:		
Sand	30	30
Bedrock:		
Gneiss	60	90
HAMPTON FALLS 3. X16-7. Alt. 67 ft.		
Lat. 42°54'55". Long. 70°51'48".		
Ice-contact deposits:		
Sand	35	35
Bedrock	66	101
HAMPTON FALLS 5. W16-6. Alt. 90 ft.		
Lat. 42°55'28". Long. 70°52'47".		
Till	35	35
Bedrock	60	95
HAMPTON FALLS 7. W16-6. Alt. 60 ft.		
Lat. 42°55'07". Long. 70°53'35".		
Outwash and shore deposits:		
Sand	30	30
Till	30	60
Bedrock	60	120
HAMPTON FALLS 8. W16-6. Alt. 65 ft.		
Lat. 42°56'06". Long. 70°54'42".		
Ice-contact deposits:		
Sand and gravel	17	17
Bedrock		17
HAMPTON FALLS 10. W16-6. Alt. 103 ft.		
Lat. 42°57'21". Long. 70°54'18".		
Till:		
Gravel	11	11
Bedrock	89	100
HAMPTON FALLS 11. W16-6. Alt. 110 ft.		
Lat. 42°55'48". Long. 70°54'23".		
Till:		
Sand and gravel	6	6
Clay, yellow; gravel and rocks	33	39
Clay, blue; gravel and rocks	38	77
Bedrock	73	150
HAMPTON FALLS 15. W16-6. Alt. 115 ft.		
Lat. 42°55'38". Long. 70°53'57".		
Till	120	120
Bedrock	130	250
NORTH HAMPTON 1. X17-8. Alt. 110 ft.		
Lat. 43°00'02". Long. 70°48'41".		
Outwash and shore deposits:		
Gravel	20	20
Marine deposits:		
Clay and silt, gray	60	80
Bedrock	58	138
NORTH HAMPTON 4. X16-1. Alt. 105 ft.		
Lat. 42°57'56". Long. 70°50'21".		
Ice-contact deposits:		
Sand and gravel, silty	40	40
Bedrock	65	105
NORTH HAMPTON 6. X16-2. Alt. 122 ft.		
Lat. 42°59'22". Long. 70°49'16".		
Unconsolidated deposits, undifferentiated:		
Gravel	45	45
Clay	29	74
Not recorded	6	80
Bedrock	20	100
NORTH HAMPTON 8. X16-2. Alt. 65 ft.		
Lat. 42°57'32". Long. 70°49'24".		
Unconsolidated deposits, undifferentiated	7	7
Ice-contact deposits:		
Sand and gravel	35	42
NORTH HAMPTON 10. X16-2. Alt. 83 ft.		
Lat. 42°45'26". Long. 70°49'49".		
Unconsolidated deposits, undifferentiated	1	1
Bedrock:		
Rock	174	175
NORTH HAMPTON 11. X16-1. Alt. 85 ft.		
Lat. 42°57'23". Long. 70°50'42".		
Ice-contact deposits:		
Sand and gravel	74	74
NORTH HAMPTON 12. X16-1. Alt. 95 ft.		
Lat. 42°58'17". Long. 70°50'43".		
Unconsolidated deposits, undifferentiated:		
Gravel and bouldery material	29	29
Bedrock	150	179
NORTH HAMPTON 13. X16-1. Alt. 65 ft.		
Lat. 42°50'27". Long. 70°50'11".		
Unconsolidated deposits, undifferentiated:		
Clay and boulders	10	10
Bedrock	40	50
NORTH HAMPTON 14. X16-1. Alt. 108 ft.		
Lat. 42°52'51". Long. 70°50'10".		
Ice-contact deposits:		
Sand and boulders	20	20
Bedrock	150	170
NORTH HAMPTON 15. X16-2. Alt. 85 ft.		
Lat. 42°53'07". Long. 70°49'26".		
Till	19	19
Bedrock	53	72
NORTH HAMPTON 16. X16-2. Alt. 85 ft.		
Lat. 42°57'36". Long. 70°53'20".		
Outwash and shore deposits:		
Sand and gravel	12	12
Marine deposits:		
Clay and gravel	7	19
Refusal		19
NORTH HAMPTON 17. X16-2. Alt. 85 ft.		
Lat. 42°59'33". Long. 70°53'24".		
Outwash and shore deposits:		
Sand and gravel	6	6
Marine deposits:		
Clay	19	25
Refusal		25
NORTH HAMPTON 18. X16-2. Alt. 87 ft.		
Lat. 42°5?'42". Long. 70°53'28".		
Outwash and shore deposits:		
Sand and gravel	12	12
Till:		
Clay, sand, and gravel	17	29
Refusal		29
NORTH HAMPTON 19. X16-2. Alt. 95 ft.		
Lat. 42°59'46". Long. 70°53'31".		
Outwash and shore deposits:		
Sand and gravel	12.3	12.3
Marine deposits:		
Clay	5.5	17.8
Till:		
Clay, sand, gravel, and disintegrated bedrock	15.0	32.8
Refusal		32.8
NORTH HAMPTON 20. X16-2. Alt. 105 ft.		
Lat. 42°59'54". Long. 70°53'31".		
Outwash and shore deposits:		
Sand	12.3	12.3
Marine deposits:		
Clay	5.0	17.3
Clay and gravel	4.5	21.8
Refusal		21.8
NORTH HAMPTON 21. X16-2. Alt. 95 ft.		
Lat. 42°59'52". Long. 70°53'16".		
Ice-contact deposits:		
Clay and gravel	11.3	11.3
Till:		
Sand, gravel, and broken ledge	6.9	18.2
Refusal		18.2
SEABROOK 9. X15-1. Alt. 65 ft.		
Lat. 42°52'14". Long. 70°51'49".		
Ice-contact deposits:		
Gravel	12	12
Bedrock	138	150
SEABROOK 12. W16-9. Alt. 45 ft.		
Lat. 42°53'26". Long. 70°52'36".		
Outwash and shore deposits:		
Loam and subsoil	4.0	4.0
Sand, coarse, brown	0.5	4.5
Sand and clay, light brown	6.0	10.5
Marine deposits:		
Clay, hard, gray	5.2	15.7
Clay, gray, and broken stones	0.8	16.5
Bedrock		16.5
SEABROOK 13. W16-9. Alt. 47 ft.		
Lat. 42°53'27". Long. 70°52'39".		
Outwash and shore deposits:		
Loam	0.5	0.5
Sand, coarse; some gravel, fine	4.0	4.5
Sand and clay, gray	1.2	5.7
Sand and gravel, light brown	2.8	8.5
Marine deposits:		
Clay, brown	2.0	10.5
Sand, fine, and clay, gray	14.5	25.0
Till:		
Sand, gravel, and clay, gray	3.7	28.7
Refusal		28.7

TABLE 2.4-20 SUMMARY OF CHEMICAL ANALYSES OF GROUNDWATER SEACOAST REGION

Characteristic or Constituent	Seacoast Values Range PPM	U.S. Public Health Service Drinking Water Standards PPM
Silica	6.6 – 18	--
Iron	.01 – 3	.3
Manganese	00 - .04	.05
Calcium	1.4 – 39	--
Magnesium	.5 – 15	125.
Sodium	2.5 – 28	--
Potassium	0.6 – 11.0	--
Bicarbonate	4. – 110.	--
Sulfate	1.6 – 54	250
Chloride	1.2 – 96	250
Fluoride	0 – 1.0	.7 – 1.2
Nitrate	0.05 – 26	45
Dissolved solids	36 – 197	500
Hardness	13 – 188	--
pH	5.6 – 8.5	--
Color	Generally free of color in objectional amounts	15

Reference: Report on the Water Supply of Southeastern New Hampshire prepared by Southeastern New Hampshire Regional Planning Commission, August, 1972.

TABLE 2.4-21 **GALLONS OF WATER PUMPED ANNUALLY FOR DOMESTIC, INDUSTRIAL AND COMMERCIAL USES FROM SEABROOK**

Year	Municipal Water Supply
1960	56,433,660 gallons
1961	63,417,190 gallons
1962	71,399,710 gallons
1963	93,947,720 gallons
1964	105,581,720 gallons
1965	114,037,320 gallons
1966	111,838,520 gallons
1967	97,317,820 gallons
1968	139,859,380 gallons
1969	133,115,780 gallons
1970	150,915,940 gallons
1971	221,141,340 gallons
1972	256,128,900 gallons
1973	265,492,380 gallons
1974	227,106,080 gallons
1975	235,177,950 gallons
1976	239,772,680 gallons
1977	362,904,375 gallons
1978	381,636,800 gallons

Reference: Comprehensive Town Plan for Seabrook, New Hampshire by Hans Klunder and personal communications with the Seabrook Water Department

Note: Values for the years 1973 through 1976 are due to faulty meters (Reference 30).

TABLE 2.4-22 PAST, PRESENT AND PROJECTED WATER USE SOUTHEASTERN NEW HAMPSHIRE REGION AND SALISBURY, MASSACHUSETTS*

Community	1964	1971	1980	1990	2000	2010	2020
Exeter	0.5		1.4	1.7	2.3	2.6	3.3
Hampton	1.0		1.6	1.9	2.2	2.8	3.2
Hampton Falls	--	--	--	0.3	0.7	1.3	2.2
Kensington	--	--	0.1	0.5	0.8	1.3	2.3
North Hampton			0.4	1.1	2.0	2.9	3.6
Rye			0.9	1.3	2.0	3.2	4.8
Salisbury (Mass.)		0.7	2.2	4.0	--	--	--
Seabrook	0.3	0.6	1.2	1.8	2.4	3.1	3.7
South Hampton	--	--	--	--	0.1	0.4	1.3
Stratham	--	--	--	0.3	0.9	2.1	3.7

*These figures are for groundwater and surface water requirements in million gallons per day.

References:

Water Supply by The Southeastern New Hampshire Regional Planning Commission, August 31, 1972.

Water Supply and Sewerage Planning in Central Merrimack Valley Region by Metcalf & Eddy, Inc.

Engineering Investigation of New and Existing Sources of Water Supply for the Town of Seabrook, New Hampshire by Coffin & Richardson, November 1979.

TABLE 2.4-23 **SUMMARY OF FIELD PERMEABILITY FOR GLACIAL AND BEDROCK MATERIALS IN THE SEABROOK AREA**

Type of Material	Number of Samples	Permeability in gpd/sq. ft. Range	Mean
Outwash	6	17—130	50
Marine (silty phase)	2	0.3 – 0.6	0.4
Till	21	0.3 - 25	5
Bedrock	9	1 – 51*	4

*Large fracture, not used in Mean

Reference: Groundwater Hydrology for the Proposed Seabrook Nuclear Station, by Weston Geophysical Research, Inc., 1969.

TABLE 2.5-1 **K-AR AGE DATES FOR ROCK SAMPLES FROM SEABROOK STATION SITE STUDIES**

(Reference: Figure 2.5-8)

A. Samples from Site Excavations, Site Borings or Borings Along Cooling Water Tunnels Adjacent to the Site

Rock Unit	Rock Types	Location	Material Dated	K-Ar Age (millions of years)
Diabase dike d5	Diabase	Trench W Excavation, Unit II	Whole Rock	212±9
Diabase dike d12	Sheared Diabase	Trench W Excavation, Unit II, along Fault CII-1	Whole Rock	213±14
Diabase dike	Bleached (Tan) Diabase	Tunnel Boring ADT-16	Whole Rock	213±10
Diabase dike	Diabase	Tunnel Boring ADT-16D	Whole Rock	221±9
Diabase dike	Diabase	Tunnel Boring F-4	Whole Rock	224±10
Diabase dike	Diabase	Site Boring B-42	Whole Rock	225±10
Diabase dike d12	Diabase	Trench W Excavation, Unit II	Whole Rock	236±13
Diabase dike d1	Diabase	Fuel Storage Bldg. Excavation, Unit I	Whole Rock	255±13
Diabase dike d3	Diabase	Equipment Vault Excavation, Unit I	Whole Rock	295±14
Newburyport	Welded Quartz Diorite Breccia	Fuel Storage Bldg. Excavation, Unit I along Fault CI-2	Sericite	246±13
Kittery	Schist	Site Boring B-4	Biotite	254±9
Kittery	Sheared Quartzite	Tunnel Boring AIT-39	Sericite	261±10

Rock Unit	Rock Types	Location	Material Dated	K-Ar Age (millions of years)
Kittery	Sheared Chlorite Phyllite	Reactor Pit Excavation, Unit II	Micas	269±11
Newburyport	Biotite Diorite	Site Boring B-9	Biotite	284±9
Newburyport	Quartz Diorite	Site Boring B-2	Biotite	294±9
Undetermined	Hornblende Diorite	Reactor Pit Excavation, Unit II	Chlorite	1181±119
B. Samples From Outcrops and Borings for the Investigation of the Scotland Road Fault				
Diabase Dike	Diabase	Boring SRF-5	Whole Rock	199±9
Newburyport	Mylonitized Schist	Boring SRF-8	Sericite-Feldspar	248±9
Newburyport	Mylonitized Quartz-Muscovite Schist	Boring SRF-2	Whole Rock	256±10
Newburyport	Muscovite Mylonite	Boring SRF-3	Whole Rock	269±10
Newburyport	Altered Granodiorite	Boring SRF-5	Whole Rock	272±10
Newburyport	Granodiorite	Parker Street, Little River Area, Newburyport, MA	Chloritized Bioitite	294±20
Undetermined	Schist	Highfield Road, Abandoned RR Grade, Newbury, MA, South of Scotland Road Fault	Chlorite-Amphibole	304±15
Undetermined	Diorite	Boring SRF-1	Amphibole	324±14
Newburyport	Diorite	Interstate Rt. 95 at Scotland Road, Newbury, MA	Amphibole	422±17

C. Samples from Outcrops for the Investigation of the Portsmouth Fault

Rock Unit	Rock Types	Location	Material Dated	K-Ar Age (millions of years)
Kittery	Feldspathic Quartzite	U.S. Rt. 1 Bypass, Greenland Road, Portsmouth, NH	Mica-Quartz	262±11
Kittery	Quartzite	Towle Road, Exeter-Hampton Expressway, Hampton, NH	Chloritized Biotite	268±10
Rye	Feldspathic Gneiss	U.S. Rt. 1 Bypass, Lafayette Road, Portsmouth, NH	Muscovite	294±10
Kittery	Feldspathic Quartzite	Winnicut Rd., NH Rt. 151, Hampton, NH	Amphibole	308±14

TABLE 2.5-2 SUMMARY OF DATA FOR FAULTS IN SITE EXCAVATIONS

This table records for each fault the:

- Number,
- Location (See Table 2.5-4 for abbreviations),
- Length with a "+" added if both ends of a fault are not visible on the site,
- Range of strike and dip with important deviations noted,
- Trend and plunge of slickensides, if present, with multiple sets noted,
- Movement sense determined from offsets, slickensides,
- Amount of displacement reported as computed net slip (NS) if determinable or as measured horizontal separation (HS), or dip separation (DS),
- Ratio of amounts of strike-slip movement to dip-slip movement based on slickensides and/or cross-cutting relationships,
- Summary of cross-cutting relationships,
- Range of weathering along a fault:

Sl -	slight -	minor or very minor rock decomposition
Mod -	moderate -	some significant decomposition
Sev -	severe -	much or most of rock softened and decomposed
Ext -	extreme -	rock completely decomposed,

- Mineralization,
- Remarks concerning other significant observable data or comments on data already listed.
-

Set No. 1 NNW - Striking, NE - Dipping												
Fault No.	Location	Length (Ft)	Width (Unmineralized) & Fault Character	Strike, Dip	Slickensides Trend, Plunge	Movement Sense	Displacement	Movement Component Ratio Str. Slip/Dip Slip	Crosscutting Relationships	Weathering	Mineralization	Remarks
EI-1	From CWT opp. ESFPCI to SWT SE of CI	203	1/16 in discrete fracture	N20°-30°W, 50°-55°N; N end N35°E, 45°SE	None	Normal, E side down	NS=29-41 in	___	Displaces Nqd; cross-cut by dikes d1 and d3; displaced by fault CI-2B	Sl to sev	Quartz, Calcite	Quartz is locally vuggy with unbroken prismatic crystals. Visible dying out with depth in profile near S end of fault.
NI-1	From SE corner of the AB to NW corner of the TFI	257	1/16 in discrete fracture	N20°-45°W, 55°-65° NE	S80°-85°E, 60°-65° and N85°E, 52°	Normal, E side down	NS=10-14 in	0.23-0.48	Displaces Nqd, dikes d5 and d6; apparently cross-cut by dike d3; displaced by fault NI-4	C to mod	Quartz, Calcite	Quartz is locally vuggy with unbroken prismatic crystals.
NI-2	From SE corner of the AB to just S of CBI	125	1/16 in discrete fracture	N15°-20°W, 55°-60° NE	N75°E, 50°	Normal, E side down	NS=10 in	0.08	Displaces Nqd and dike d5, but not dike d6.	Sl to mod	Quartz, Calcite	Quartz is locally vuggy with unbroken prismatic crystals on fault surface. Fault ends to N against NI-1.
NI-3	From SW corner TBI to CBI	135	1/16 in discrete fracture	N25°W, 71°E	S70°-90°E, 50°-64°	Normal, E side down	NS=3/4 in	0.47-1.0	Displaces Nqd and dike d6	Sl	Quartz, minor Calcite	Visible dying out with depth in profile near S end of fault.
T-1	NW corner of TFI	109	1/16 in discrete fracture	N20°-27°W, 55°-63° NE	S70°-90°E, 54°-63°	Normal, E side down	NS=22 in	0.45-1.0	Displaces Nqd and diabase dike d3	Clean to Sl	Quartz, minor Calcite	Calcite mineralization contains angular pieces of Nqd - a fused breccia.
T-3	N end of TFI	60	1/8-1/16 in discrete fracture	N15°-20°W, 55°-60°E; S end N0°E, 90°	S85°E, 47°	Normal, E side down	None visible	0.41	Displaces Nqd, apparently cross-cut by dike d3	Clean to Sl	Patchy Quartz and Calcite	No displaced features seen; movement apparently minute.
T-3A	Just N of TFI	42	1/16 in discrete fracture	N20°W, 90°	None	____	____	____	None seen	Clean to Sl	Calcite	Feature may not be a fault, but it is mineralized with calcite and is on strike with T-3 and NI-2.
W-1	From W side of DI to just W of WPB	222	Mostly 1/16 in small segments open to 1/4 in; discrete fracture	N12°-30°W, 60°-65°E; N40°W strike in d5	S60°-90°E, 55°-65°	Normal, E side down	NS=13-27 in	0.50-1.1	Displaces Nqd, dike d3 and apparently dike d5	Clean to Sl	Calcite and patchy Quartz	Calcite mineralization forms the matrix at a fused breccia containing pieces of dike d5. Fault changes strike and character in dike d5. Quartz is slickensided
DI-2	From N end of DI to center of DI	45	1/16 in discrete fracture	N30°-50°W 46°-90°E	N45°E, 44°	Normal, E side down	HS=2 in	0.88	Displaces Nqd, dikelet d5	Sl to mod	Minor Calcite	Calcite mineralization forms the matrix of a fused breccia containing pieces of dikelet d5

Set No. 1 NNW - Striking, NE - Dipping												
Fault No.	Location	Length (Ft)	Width (Unmineralized) & Fault Character	Strike, Dip	Slickensides Trend, Plunge	Movement Sense	Displacement	Movement Component Ratio Str. Slip/Dip Slip	Crosscutting Relationships	Weathering	Mineralization	Remarks
W-2	NW corner WPB	58	1/16 in discrete fracture	N25°W, 54°NE; at S end N20°E, 50°SE	N80°E, 49°	Normal, E side down	NS=5 in	0.28	Displaces Nqd, dike d3	Sl	Quartz, Calcite	
W-2A	From S side of DI to just N of WPB	45	1/8 - 1/16 in discrete fracture	N24°W, 63°E		Normal, E side down	NS=5 in	____	Displaces Nqd and S contact of dike d5, but not N contact	Sl	Calcite	Ends with dike d5. On strike with but not connected with W-2.
W-3	From S end of CBI to center of WPB	158	1/16 in discrete fracture	N10°-25°W, 50°-63°E; at S end N0°-5°W, 65°E	S72°-90°E, 45°-55°	Normal, E side down	None visible	0.45-0.90	Displaces Nqd, dike d3, but not dike d5	Sl	Calcite	Ends at S contact of dike d5.
W-4	From just NE of WPB to E side of WPB	102	1/16 in discrete fracture	N15°-35°W, 54°-64°E	S58°W, 58°	Normal, E side down	NS=5-14 in	0.31-0.55	Displaces Nqd, but not dike d3	Sl	Quartz, Calcite	Ends at NI-4. Passes through dike d3 as quartz vein; that dike's contacts are irregular near this fault, but not displaced.
W-5	From center of DI to N half of WPB	124	1/16 in discrete fracture	N10°-35°W, 53°-64°E	S75°E, 32°	Normal, E side down	NS=3 in	0.47	Displaces Nqd, and apparently dike d5	Clean to Sl	Calcite, Quartz	Calcite mineralization locally consists of undisturbed prismatic crystals, and forms the matrix of a fused breccia containing pieces of dike d5. A small aphophysis of dike 5d was channeled along the fused calcite vein which represents this vault within dike d5.
W-6	From N end of WPB to just S of DI	65	1/16 in discrete fracture	N7°-28°W, 50°-66°E	S80°E, 51°	Normal, E side down	DS-6 in	0.44	Displaces Nqd	Clean to Sl	Calcite	
RII-1	SW of AB	187	1/4 in discrete fracture or locally a 3 in zone	N10°-25°W, 65°-80°E	N65°E, 58°	Normal, E side down	HS=6 in	0.88	Displaces Nqd; ends to S against dike d5	Clean to Mod	Minor Calcite	The longest of a group of 4 faults on strike with each other. Has minor dip-reversal near S end.
RII-1A	SW of AB	47	1/4 in discrete fracture to 3 in zone	N15°-25°W, 77°E	____	Normal, E side down	HS=3 in	____	Displaces Nqd; ends to N against A-1	Mod	Minor Calcite	
RII-1B	SW of AB	14	1/4 in discrete fracture	N33°W, 76°E	____	Normal, E side down	VS=5 in	____	Displaces Nqd	Sl to mod	Quartz	

Set No. 1 NNW - Striking, NE - Dipping												
Fault No.	Location	Length (Ft)	Width (Unmineralized) & Fault Character	Strike, Dip	Slickensides Trend, Plunge	Movement Sense	Displacement	Movement Component Ratio Str. Slip/Dip Slip	Crosscutting Relationships	Weathering	Mineralization	Remarks
RII-1C	SW of AB	25	1/4 in discrete fracture	N29°W, 65°E	____	Normal, E side down	None visible	____	Displaces Nqd?	Sl	Minor Calcite	May not be a fault. Ends to the N just S of dike d5.
RII-2	SW corner of AB	37	1/4 - 1/2 in discrete fracture or zone	N20°W, 70°E	N60°W, 70°E	Normal, E side down	NS=1.5 in	0.16	Displaces Nqd	Sl	None	
HI-1	NE corner of WPB	34	1/4 in discrete fracture	N10°W, 64°W; N end N35°E, 75°-90°W	N80°W, 58° N65°W, 90°	Reverse, E side down	NS=13 in	0.36	Displaces Nqd	Sl to sev	Calcite	On strike with W-2. Occurs among several E-dipping normal faults. Reverse motion is consistent with normal movement sense on E-dipping faults.

Set No. 2 NE - Striking, NW - Dipping

Fault No.	Location	Length (Ft)	Width (Unmineralized) & Fault Character	Strike, Dip	Slickensides Trend, Plunge	Movement Sense	Displacement	Movement Component Ratio Str. Slip/Dip Slip	Crosscutting Relationships	Weathering	Mineralization	Remarks
CI-2	From E wall of CP to SWT S of WPB	575	2-12 in zone	N20°-58°E, 30°-41°N	N32°-60°W, 34°-40°	Normal, NW side down	4 in-15 ft	0.0-0.41	Displaces Nqd, dike d20 and apparently dike d1; ends against dike d1	Mod-Sl with some sev and loc ext	None	Contains fused breccia.
CI-2A	From E wall of CP to SE of EHI	198	2-6 in zone	N35°-47°E, 35°-40°N	None	Normal, NW side down	NS=5 ft 2 in	0.05-0.10	Displaces Nqd; ends against dike d1	Mod to sev	None	
CI-2B	From E wall of CP to SE of EHI	144	2-6 in zone	N35°E, 36°N	None	Normal, NW side down	NS-4 ft 7 in	0.05-0.10	Displaced Nqd; ends against dike d1	Mod to sev	None	
CI-2C	From IS to W wall of CP	238+	2-6 in zone	N30°-35°E, 37°-50°N; in Sk on CP floor; N5°W, 22°W	N44°-50°W, 30°-39°	Normal, NW side down	NS=6-11 ft	0.19	Displaces Nqd, Sk, and apparently dike d1	Sl, loc mod, sev and ext	None	
CI-2D	N end of CP	77	2-6 in zone	N15°-46°E, 25°-45°N	N41°W, 24°	Normal, NW side down	NS=7 ft	0.05	Displaces Nqd, Sk, and apparently dike d1	Sl, loc mod, sev and ext	None	Ends to W within Sk xenolith; ends to E within dike d1, displaces one dike contact but not the other.
NI-4	From S of CBI to SE corner of CBI	128	1/8-6 in	N29°-40°E, 38°-42°N	None	Normal, NW side down	NS=5 ft 6 in	____	Displaces Nqd, NI-1; cross-cut by dike d5; displaced by T-1	Mod, loc sev	Minor quartz	Quartz is locally druzy with undisturbed, prismatic crystals on fault surface.
SIII-1	W wall DI to NE quad CII	130	1/16 in discrete fracture to 2 in zone	N12°-14°E, 36°N; in Sk on CII floor N8°W, 48°W	None	Normal, NW side down	NS=1 ft 10 in	0.19	Displaces Nqd, and apparently Sk; cross-cut by dike d5;	Sl-mod with loc sev	None	Ends to W within Sk xenolith.
CP-1	W wall CP	10	1/8-1/16 in discrete fracture	N35°E, 38°N	None	Normal, NW side down	DS=1 ft	____	Displaces Nqd	Sl	Minor quartz	Cross-cut by a NE-striking, steep S-dipping joint. In profile, 44 ft long.
TII-1	W wall of SCLTII SW portion TBII	12	1/16 in discrete fracture	N35°E, 38°N	None	Normal, NW side down	NS=6 in	____	Displaces Nqd	Sl	Minor quartz	In profile about 35 ft long; except for displacement is indistinguishable from joints.
DII-1	W wall DII	20	1/16 in discrete fracture	N35°E, 33°N	None	Normal, NW side down	NS=11 in	____	Displaces Nqd	Sl to sev	None	In profile about 15 ft long; except for displacement is indistinguishable from joints.

Set No. 3 NE - Striking, SE - Dipping												
Fault No.	Location	Length (Ft)	Width (Unmineralized) & Fault Character	Strike, Dip	Slickensides Trend, Plunge	Movement Sense	Displacement	Movement Component Ratio Str. Slip/Dip Slip	Crosscutting Relationships	Weathering	Mineralization	Remarks
CI-1	From EFPBI to WPB	390	1/8 in discrete fracture to 4 ft zone	N Segment N35°E 45°S; Middle Segment N 10°-20°W, 50°-80°E S Segment N20°-35°E, 50°-80°E	None	Normal, SE side down	NS=19 in	____	Displaces Nqd, dikelets d4; cross-cut by dikes d21 and apparently by d5	Sl to sev	Quartz, Calcite	Significant differences in fault width, character, and strike occur along this fault's length.
CI-1A	From E of TBI to EFPBI	175	1/16-1/2 in, discrete fracture or fractures	N22°W, 57°NE; S end N43°E, 56°S	N65°E, 42°	Normal, SE side down	NS=3/4 in	0.05	Displaces Nqd, cross-cut by dike d7	Sl to mod	Quartz, Calcite	En echelon with CI-1 and CI-1B.
CI-1B	From E of TBI to CWT	190	1/8 to 1/4 in, discrete fracture or fractures	N25°E, 60°SE; N end N15°-20°W, 62°NE	None	Normal, SE side down	____	____	Displaces Nqd?	Sl to mod	Quartz Calcite	No displacement apparent.
CT-1A	From FII to SWT S of PII	128	1/8 in discrete fracture to 3 in zone	N30°-45°E, 60°-80°S; locally N60°-65°E, 90°	S65°E, 76°	Normal, SE side down	NS=2 in	0.38	Displaces Nqd, Sk	Sl with loc mod to sev	None	Splays off CT-1 to W, Ends to E at FII-1.
CT-3A	W end CT	64+	1/16 in discrete fracture to 3 in zone	N45°-72°E, 72°S	S85°E, 45°	Normal, SE side down	____	1.5	Displaces Nqd	Sl to mod	Minor Calcite	Silkensided surface but no displacement visible.
CT-3B	W half CT	135+	1/16 in discrete fracture to 1 1/2 in zone	N45°-60°E, 71°S-70°N	S85°E, 70°-73°	Normal, SE side down	NS=5 in	1.5	Displaces Nqd	C to mod	Clacite, Minor Chlorite (?)	May join CT-3A to W.
A-2	S end of TBII W of HB	35	less than 1/2 in	N35°E, 75°S	S60°E, 60°	Normal, SE side down	____	____	Displaces Nqd?	Sl to mod	Calcite	No displacement apparent; on strike with A-2A.
A-2A	S end of TBII E of HB	53	less than 1/2 in	N40°E, 65°S	S25°E, 60°	Normal, SE side down	NS=5 in	0.33	Displaces Nqd and a calcite vein	Sl to mod	Calcite	
SP-1	NE side of SP	35	1/16 in discrete fracture	N34°-42°E, 59°-67°S	S0°E, 63° (vague)	Normal, SE side down	DS=4 in	0.58	Displaces Nqd	Sl	Calcite, Quartz	Generally a closed, tight or fused quartz fracture.
TW-1	TWII	17	1/16 in discrete fracture	N47°E, 72°-90°S	None	Normal, SE side down	NS=1 in	____	Displaces Nqd, Sk, dike d5	Mod	None	Displacement visible at only one contract of dike d5.
CI-1D	From NE quad of CI to CWT	285	1/8 in discrete fracture	N40°E, 67-81°S	None	Normal, SE side down	DS=4 in	____	Displaces Nqd	Sl to Mod, loc sev	Quartz	Splays off CI-1 to W.

Set No. 4 NE - Striking, Vertical

Fault No.	Location	Length (Ft)	Width (Unmineralized) & Fault Character	Strike, Dip	Slickensides Trend, Plunge	Movement Sense	Displacement	Movement Component Ratio Str. Slip/Dip Slip	Crosscutting Relationships	Weathering	Mineralization	Remarks
SI-1	N end of ESFPCI	54	1/16 in discrete fracture	N37°-40°E, 70°N-90°	None	Normal, NW side down	HS=1 ft 10 in	High	Displaces Nqd; cross-cut by dike d4	Sl to mod, loc sev	Minor Calcite	Locally tightly fused.
DI-3	W portion of DI	44	1/16 in discrete fracture	N30°E, 87°E-90°	None	Normal, SE side down	NS=5 in	____	Displaces Nqd; cuts into but not across d5 and does not displace d5	Clean to Sl	Calcite	Changes to calcite vein(s) to E before ending. Calcite forms matrix of a fused breccia containing small angular pieces of country rock.
A-1	From center of AB to just S of TBII	400	2-5 in wide zone	N40°-45°E, 80°N-90°	None	Left-lateral?	____	____	Displaces Nqd, Sk(?)	Mod, loc sev	Calcite, Quartz	Quartz and calcite crystals in vugs within fault are undisturbed. Ends to W against fault CII-2, and may cross-cut a portion of that fault.
A-1A	From W of TBII to E side of TBII	294+	1/4 - 2 in wide zone or 0-18 in wide fused breccia	N30°E, 80°NW-90°	None	None measurable	____	____	Displaces Nqd, diabase?	Clean to sev	Calcite	Consists generally of a vein of fused breccia with a matrix of calcite containing angular pieces of country rock. Contains pods of diabase 1-3 in across at N end.

Set No. 5 ENE-Striking, S-Dipping

Fault No.	Location	Length (Ft)	Width (Unmineralized) & Fault Character	Strike, Dip	Slickensides Trend, Plunge	Movement Sense	Displacement	Movement Component Ratio Str. Slip/Dip Slip	Crosscutting Relationships	Weathering	Mineralization	Remarks
CT-1	SWT N of CT to area W of SWT	342+	1/8 in discrete fracture to 4-6 in zone	N70°-85°E, 71°S-80°N	N90°E, 6°-16°	Left Lateral Normal, S side down	NS=8 in	13	Displaces Nqd, Sk	Mod to Sl, loc sev	Calcite	Ends to E just beyond PII-IA, displacing that fault slightly.
CT-2	S of CT to W end of CT	255+	2-12 in wide zone	N75°-90°E, 77°-87°S	S80°E, 35°; N85°E, 19°; N90°W, 3°	Left Lateral Normal, S side down	NS=1-7 in	3.0	Displaces Nqd, dike d1	Mod to sev	Calcite	Fault plane forms S wall of CT for some distance. Minor bleaching associated.

Set No. 6 WNW- Steep Dipping												
Fault No.	Location	Length (Ft)	Width (Unmineralized) & Fault Character	Strike, Dip	Slickensides Trend, Plunge	Movement Sense	Displacement	Movement Component Ratio Str. Slip/Dip Slip	Crosscutting Relationships	Weathering	Mineralization	Remarks
CII-1	From TWII to area SE of CII	500+	1 1/2 in zone to 14 ft diffuse zone	N80°-90°W, 80°-85°N	N65°W, 44°; N75°E, 44°	Left Lateral Reverse, N side up; Left Lateral Normal, N side down	NS=6-12 ft	1.5-11	Displaces Sk, hd, dikes d5, d11, d12, and d13; partially cross-cut by dike. d3	Sl to sev, loc ext	Chlorite, Quartz, Minor Pyrite, Calcite, Graphite (?)	Dike 5d is displaced considerably less than the other three. Small Quartz-lined vugs with unsheared crystal occur within the fault. Cross-cuts and may displace EII-1. Overlying till is undisturbed.
CII-2	From TWII to E wall of RPII	324+	1 in to 3 ft zone	N80°-90°W, 72°N-85°S	N49°W, 41°; S73°E, 23°; N60°E, 29°	Left Lateral Reverse, N side up; Left Lateral Normal, N side down; Right Lateral	HS=4-12 ft	1.1-2.5	Displaces Sk, dikes d11, d12, and d13; cross-cut by dike d5	Sl to sev, loc ext	Chlorite, Minor Quartz, Pyrite	Acts as a conduit for a portion of dike d5. Ends to the E against fault EII-1. Overlying till is undisturbed.
CII-2A	From E side of CII to area SE of CII	181	1 in to 2 ft zone	N64°-80°W, 70°-90°N	None	Left Lateral Reverse?	Small	----	Displaces Sk; cross-cut by dike d3	Sl, loc mod	Minor Pyrite, Chlorite	On strike with fault CII-2. Splays into EII-2, and for about 20 ft is indistinguishable from fault.
CII-3	From N side of CBII to area SE of CII	288	1/8 in discrete fracture to 4-6 ft diffuse zone	N70°-80°W, 60°N-77°S	S80°W, 17°s	Left Lateral Reverse	Small	6.0	Displaces Sk; Nqd	Sl, loc mod	Minor Quartz, Calcite Chlorite	Mostly a zone of closely-spaced, tight joints.
FII-1	From just NW of FII to E end of CT	188	1/8 in discrete fracture to 6 ft, diffuse zone	N55°-80°W, 80°S-80°N	None	Left Lateral	Small	____	Displaces Sk; cross-cut by dike d1	Sl, loc mod	Chlorite, Minor Calcite	

Set No. 4 NE - Striking, Vertical

Fault No.	Location	Length (Ft)	Width (Unmineralized) & Fault Character	Strike, Dip	Slickensides Trend, Plunge	Movement Sense	Displacement	Movement Component Ratio Str. Slip/Dip Slip	Crosscutting Relationships	Weathering	Mineralization	Remarks
PII-1	From N end of DII to W side of FII	381	1/16in discrete fracture to 6ft, diffuse zone	N65°-88°W, 75°N-75°S	N90°W, 5°	Left Lateral	HS=5 in	25.0	Displaces Sk, Nqd	C to mod, loc sev	Calcite, Chlorite, Minor Quartz, Pyrite	Contains stringers of Nqd which appear to have been injected lit-par-lit, post-faulting. Calcite locally forms the matrix of a fused breccia which contains angular fragments of Sk.
NII-1	From NW corner of CBII to S wall of EVII	189+	1/8in discrete fracture to 1/2-8in zone	N65°-75°W, 72°S-80°N	N62°W, 49°	Right Lateral Reverse?	NS=2 ft 6 in	9.0	Displaces Sk	C to Sl, loc mod	Minor Calcite, Chlorite	Calcite locally forms matrix of a fused breccia which contains angular fragments of Sk. Movement sense suggested by slickensides is opposite to that observed for other faults in this set.
NII-2	From NW to SE in CBII	122+	1/2in to 1ft zone	N75°W-N85°E, 55°-90°N	None	Left Lateral	HS=6 in	____	Displaces Sk	C to Sl	Minor Chlorite	May join NII-1 to NW.
PII-1A	From SWT S of FII to E end of CT	89	1/4 discrete fracture to 6 ft diffuse zone	N75°-88°N, 77°-90°N	None	Left Lateral?	Small	____	Displaces Sk, Nqd; cross-cut by dike d1.	Mod to Sl	Chlorite, Minor Pyrite	On strike with PII-1.
EII-1	From SE corner of TBII to E end of CT	378	1/16-1 in zone in Nqd and db; 2-8 in zone in Sk	N25°-66°W, 30°-65°SW	S75°W, 40°; N65°W, 12°	Left-Lateral Reverse, SW side up; Right-Lateral Normal, SW side down	NS=5-6ft	Very high; .047-1.2	Displaces Sk, hd and Nqd; apparently cuts dikes d1 and d5	Mod to sev, loc ext	Chlorite, Minor Pyrite	A particularly wary, bumpy surface. Fault has apparently experienced two ancient movements. Displaced by CII-1.
EII-1A	E end of CT	33+	2-6 in wide zone	N25°W, 38°W	None	Right-Lateral Normal, SW side down (?)	Small	____	Displaces Sk	Mod with loc ext	Minor Chlorite	On-strike with EII-1. Drag Indicates right-lateral movement.
EII-2	From between ESFPCII and WPB to SWT NE of CT	188+	1-12 in wide zone	N10°-20°W, 68°W	None	Right-Lateral Normal, SW side down	NS=1 ft 2 in	1.6	Displaces Sk	Mod with loc sev	Minor Chlorite, Minor Quartz	Quartz crystals in small vugs within fault are undisturbed. Splays into fault CII-2A. Drag indicates some Left-lateral movement.
EII-3	Just SE of CII	49	1-4 in wide zone	N25°-40°W, 78°-90°W	None	Right-Lateral	HS=8 ft	____	Displaces Sk	SL	Chlorite	Splays into CII-1. Drag indicates right-lateral movement.

TABLE 2.5-3 **JOINTING IN SITE BEDROCK**

Rock Unit	Average Set Orientation	Degree of Development	Spacing	Occurrence – Distribution	Remarks
Newburyport	N35°-45°E, 35°-40°NW	Very Good	6in-8ft	Very common	Local strike variations for this set range from N10°-50°E.
	N35°-45°E, 50°-75°SE	Good to fair	1-10ft	Fairly common	Conjugate with above set.
	N10°-30°W, 55°-65°SE	Good	6-12ft	Common	
	N45°E, 80°-90°NW	Good to fair	Very wide	Occasional	Parallel to diabase dikes, often calcite-coated or filled.
	N90°E, 80°S-80°N	Fair to poor	Very wide	Occasional	
	N50°-60°W, 40°-50°NE	Fair	Very wide	Occasional	
Kittery	N70°-90°W, 70°S-60°N	Very good	2in-6ft	Very common	Parallel to foliation
	N35°-55°E, 35°-40°NW	Fair to very good	1-3ft	Very common to occasional	More common within 50-100 ft of contacts with Newburyport.
	N25°-40°E, 55°-65°SE	Fair	5ft	Common to occasional	More common within 50-100ft of contacts with Newburyport.
	N10°-25°W, 50°-70°SW	Fair to poor	1-10ft	Occasional	

Rock Unit	Average Set Orientation	Degree of Development	Spacing	Occurrence – Distribution	Remarks
Kittery	N10°-50°W, 30°-50°SW	Fair to poor	1-10ft	Occasional	
	N0°-15°E, 55°-75°W	Fair to poor	1-10ft	Occasional	
	N25°-55°W, 30°-35°NE	Fair to poor	Very wide	Occasional	
	N40°-55°W, 60°-80°SW	Fair to poor	Very wide	Occasional	Locally well-developed.

TABLE 2.5-4 FOUNDATION EXCAVATIONS AND ABBREVIATIONS USED

(Unit No. I or former Unit II indicated as necessary)

Administration Building	AB
Containment Building	CI or CII
Control Building	CBI or CBII
Circulating Water Pumphouse	CP
Cooling Tower	CT
Circulating Water Trench	CWT
Diesel Generator Building	DI or DII
Discharge Tunnel Shaft	DS
Emergency Feedwater Pump Building	EFPBI or EFPBII
Equipment Hatch	EHI or EHII
East Steam & Feedwater Pipe Chase	ESFPCI or ESFPCII
Equipment Vault	EVI or EVII
Fuel Storage Building	FI or FII
Ringer Crane	GII
Haul Road	HI
Heater Bay (in Turbine Building)	HBI or HBII
Intake Tunnel Shaft	IS
Primary Auxiliary Building	PI or PII
Reactor Pit (in Containment Building)	RI or RII
RCA Tunnel	RCAII
S-Column Line Trench	
(in Turbine Building)	SCLTI or SCLTII
Service Water Pumphouse	SP
Service Water Trench	SWT
Turbine Building	TI or TII
Tank Farm	TFI or TFII
Trench W	TWII
Waste Processing Building	WPB

TABLE 2.5-5 **EARTHQUAKES IN THE NORTHEAST LATITUDE 40.0N TO 48.0N LONGITUDE 64.0W TO 75.0W**

The information contained in this table is historical information and is not acceptable for electronic format. A copy of this information may be obtained through the Records Management Department.

TABLE 2.5-6 **SEISMIC EVENTS WITH DUBIOUS LOCATION OR ORIGIN**

Date	Origin Time	Latitude*	Longitude*	Intensity	Remarks
1730 Dec. 30		42.7	70.6	IV-V	Location Uncertain Cape Ann Region
1734 Nov. 23		42.7	70.6	IV-V	Location Uncertain Cape Ann Region
1737 Feb. 17		42.4	71.0	IV	Location Uncertain Felt in Boston and Vicinity
1739 Aug. 13		42.7	70.6	IV-V	Location Uncertain Cape Ann Region
1761 Mar. 12	0215 L.	42.7	70.6	V	Location Uncertain Cape Ann Region
1766 Jan. 23	0500	43.65	70.28	IV	Location Uncertain Portland, Maine Region
1766 Aug. 25	0105	41.5	71.3	F	Dubious seismic origin – probably a meteor
1785 Jan. 2	1215	46.0	67.0	VIII	Location Uncertain Reported from Portland, Maine to Baltimore, Maryland
1808 June 26	1951	44.0	70.0	V	Location Uncertain Coastal Maine

Date	Origin Time	Latitude*	Longitude*	Intensity	Remarks
1860 Mar. 16		42.25	71.17	IV	Location Uncertain Reported from Eastern and Southeastern Massachusetts
1892 July 25				IV	Georges Shoals – location uncertain, reported by pilot boat
1903 Jan. 21	1000	42.1	70.9	V	Dubious seismic origin – probable frost action
1903 Jan. 22		41.95	71.3	IV	Dubious seismic origin – frost action
1921 July 29	1614	42.22	71.07	IV	Location Uncertain – isolated felt report from Cambridge, Massachusetts
1925 July 1		46.0	68.0	IV	Dubious seismic origin
1954 Feb. 13		42.15	72.60	III-IV	Dubious seismic origin – probable frost action (2 shocks)
1973 Feb. 3		41.5	71.7	V	Location and origin dubious; not recorded by seismograph stations in the area (U.S. Earthquakes, 1973)

* For computer retrieval only.

TABLE 2.5-7 **JESUIT SEISMOLOGICAL ASSOCIATION STATIONS**

Station	Latitude	Longitude	Elevation Meters	Date Opened Day	Month	Year	Date Closed Day	Month	Year	Location
BUF	42.9333N	78.8500W	195	01	01	1912		--		Buffalo, NY
CHI	41.9000N	87.6333W	183	01	09	1912		--		Chicago, IL
CNN	39.1450N	84.4967W	203	01	01	1927	01	01	1963	Cincinnati, OH
CLE	41.4888N	81.5321W	328	01	01	1904		--		Cleveland, OH
FOR	40.8631N	73.8856W	24	01	01	1910		--		Fordham, NY
GEO	38.9000N	77.0667W	29	01	01	1911		--		Georgetown, DC
MLW	43.0333N	87.9167W	194	01	01	1909		--		Milwaukee, WI
NOL	29.9483N	90.1200W	2	01	01	1910		--		New Orleans, LA
SHA	30.6944N	88.1428W	61	01	12	1910		--		Spring Hill, AL
WES	42.3847N	71.3221W	60	01	01	1929		--		Weston, MA
FLO	38.8017N	90.3700W	160	09	07	1961	08	03	1971	Florisant, MO
--	38.6364N	90.2333W	--	01	01	1910		early 60's		St. Louis, MO
--	38.6361N	90.2361W	--			1927		--		St. Louis, MO
--	37.3167N	89.5333W	--			1938		--		Cape Girardeau, MO
--	34.7833N	92.3500W	--			1930		mid 60's		Little Rock, AR

*Macelwane (1925-1950)

TABLE 2.5-8 WORLD-WIDE STATIONS IN EASTERN UNITED STATES*

Station	Latitude	Longitude	Elevation Meters	Date Opened Day	Date Opened Month	Date Opened Year	Date Closed Day	Date Closed Month	Date Closed Year	Location
AAM	42.2997N	83.6561W	249	01	01	1940		--		Ann Arbor, MI
ATL	33.4333N	84.3375W	273	21	06	1963		--		Atlanta, GA
BLA	37.2112N	80.4205W	634	04	09	1962		--		Blacksburg, VA
FLO	38.8017N	90.3700W	160	09	07	1961	08	31	1971	Florisant, MO
FVM	37.9840N	90.4260W	--	10	05	1974		--		French Village, MO
GEO	38.9000N	77.0667W	43	07	12	1961		--		Georgetown, DC
MDS	43.3722N	89.7600W	278	16	01	1962	10	06	1968	Madison, WI
MNN	44.9145N	93.1900W	--	07	05	1962	04	11	1965	Minneapolis, MN
OGD	41.0875N	74.5958W	367	01	01	1960		--		Ogdensburg, NJ
OXF	34.5118N	89.4092W	101	01	08	1963	01	05	1976	Oxford, MS
SCP	40.8098N	77.8694W	353	26	01	1962		--		State College, PA
SHA	30.6944N	88.1428W	61	01	12	1910		--		Spring Hill, AL
WES	42.3874N	71.3221W	60	01	01	1929		--		Weston, MA

* Institute of Science and Technology, University of Michigan (see References).

TABLE 2.5-9 SEISMOGRAPH STATIONS IN EASTERN CANADA*

Station	Latitude	Longitude	Elevation Meters	Date Opened Day	Month	Year	Date Closed Day	Month	Year	Location
TNT	43.6670N	79.3990W		01	09	1897	01	01	1942	Toronto, Ont.
SHF	46.3300N	72.4500W				1928	08	12	1965	Shawinigan Falls, Que.
SFA	47.1200N	70.8200W	232			1928	31	07	1975	Seven Falls, Que.
HAL	44.6300N	63.6000W	56			1915				Halifax, N.S.
KLC	48.0900N	80.0200W		19	12	1939	30	06	1957	Kirkland Lake, Ont.
MNT	45.5000N	73.6200W	112	01	04	1956				Montreal, Que.
OTT	45.3900N	75.7200W	83	01	01	1906				Ottawa, Ont.
LND	42.5900N	81.1400W		01	01	1961	31	05	1967	London, Ont.
CHQ	46.8900N	71.3000W	145	11	11	1971				Charlesbourg, Que.
LHC	48.4200N	89.2700W	196	28	02	1969				Thunder Bay, Ont.
PBQ	55.2800N	77.7400W	20	14	09	1972				Poste-De-La-Baleine, Que.
POC	47.3600N	70.0400W	61	20	01	1972				La Pocatiere, Que.
QCQ	46.7800N	71.2800W	91	24	09	1971				Quebec, Que.
SCB	43.7200N	79.2300W	153	01	01	1962		01	1974	Scarborough, Ont.
SCH	54.8200N	66.7800W	540	22	07	1962				Schefferville, Que.

Station	Latitude	Longitude	Elevation Meters	Date Opened Day	Date Opened Month	Date Opened Year	Date Closed Day	Date Closed Month	Date Closed Year	Location
SIC	50.1900N	66.7400W	283	01	01	1962				Seven Islands, Que.
STJ	47.5700N	52.7300W	62	01	06	1964				St. John’s, Nfld.
SUD	46.4700N	80.9700W	267	22	11	1967				Sudbury, Ont.
UNB	45.9500N	66.6300W	56	01	09	1971				Fredericton, N. B.
GWC	55.2910N	77.7520W	8	29	09	1965	01	07	1972	Great Whale R., Que.
MNQ	50.5333N	68.7744W	487	01	01	1974				Manicouagan, Que.
MIQ	46.2300N	75.5800W		01	01	1974				Maniwaki, Que.
HV	49.1100N	68.1600W		01	04	1974	01	12	1974	Hauterive, Que.
LGQ	53.6900N	77.7300W	190	04	08	1976				La Grande, Que.
LMQ	47.5500N	70.3300W	419	03	11	1976				La Malbaie, Que.
GNT	46.3630N	72.3720W	10	04	24	1978				Gentilly, Que.

*United States Department of Commerce (1972)
Wetmiller and Horner (1978)

TABLE 2.5-10 LIST OF OPERATING N.E.U.S.S.N. SEISMIC STATIONS BY STATE JULY-SEPTEMBER 1978

STA	Latitude DG MN SEC	Longitude DG MN SEC	Elevation Meters		Operator
Connecticut					
BCT	412936.0N	732301.9W	69	Brookfield	WES
ECT	415004.7N	732440.8W	342	Ellsworth	WES
HDM	412908.7N	723123.6W	24	Haddam	WES
NSC	412850.7N	715105.7W	110	N. Stonington	WES
UCT	414954.0N	721502.0W	149	Storrs	WES
Delaware					
BBD	392046.0N	754036.0W	18	Blackbird	DGS
GTD	384429.0N	752452.0W	15	Georgetown	DGS
NED	394215.2N	754229.5W	46	Newark	DGS
Maine					
AGM	470454.0N	690124.0W	240	Allagash	WES
BPM	443754.0N	684721.6W	80	Bucksport	WES
CBM	465557.0N	680714.8W	250	Caribou	WES
D1A	470330.8N	690556.2W	304	Dickey	WES
D2A	470749.3N	690908.8W	402	Dickey (Kelly Mtn)	WES
D3A	470515.2N	691007.4W	259	Dickey (Carter Brook)	WES
D4A	471117.2N	691636.1W	490	Dickey (Rocky Mtn)	WES
D5A	470040.7N	691554.0W	365	Dickey (Browns Brook)	WES
D6A	470520.4N	692944.5W	430	Dickey (Two Mile Stream)	WES
EMM	444421.0N	672922.0W	20	East Machias	WES
HKM	443923.0N	693826.9W	79	Hinckley	WES
JKM	453919.8N	701433.4W	378	Jackman	WES
MIM	451437.0N	690225.0W	140	Milo	WES
TRM	441534.9N	701518.3W	113	Turner	WES

STA	Latitude DG MN SEC	Longitude DG MN SEC	Elevation Meters		Operator
Massachusetts					
COD	412438.9N	700450.1W	-85	Cape Cod	MIT
DUX	420407.0N	704604.0W	27	Duxbury	MIT
FLR	414260.0N	710717.5W	52	Fall River	WES
GLO	423825.0N	704338.0W	15	Gloucester	MIT
HRV	423023.0N	713330.0W	180	Harvard	MIT
LNX	422020.0N	731620.6W	345	Lenox	WES
QUA	422723.8N	722225.8W	201	Quabbin	WES
WES	422304.9N	711919.5W	60	Weston	WES
WFM	423638.0N	712926.0W	87	Westford	MIT
WGMA	421720.4N	813506.0W	130	Westboro	WGE*
New Hampshire					
BNH	443526.1N	711523.0W	472	Berlin	WES
CSNH	434857.6N	712743.2W	200	Center Sandwich	WGE
DNH	430721.0N	705341.2W	24	Durham	MIT
GHNH	435212.0N	710708.4W	200	Goe Hill	WGE
HNH	434219.0N	721708.0W	180	Hanover	WES
LANH	433527.6N	712924.0W	200	Laconia	WGE
MBNH	434337.2N	711919.2W	200	Moultonborough	WGE
ONH	431645.0N	713019.9W	280	Oakhill	MIT
PNH	430343.0N	720808.9W	659	Pitcher Mtn	MIT
WBNH	433614.4N	710556.4W	200	Wolfeboro	WGE
WNH	435206.0N	712359.0W	220	Whiteface	MIT
New Jersey					
GMTN	405257.0N	741104.2W	165	Garret Mountain	LDO
GPD	410103.6N	742739.0W	360	Green Pond	LDO
LVNJ	404834.2N	744505.4W	201	Long Valley	LDO
OGD	410400.0N	743659.9W	363	Ogdensburg	LDO
PQN	410026.3N	750509.0W	229	Pahaquarry	LDO

STA	Latitude DG MN SEC	Longitude DG MN SEC	Elevation Meters		Operator
PRIN	402200.6N	744304.2W	110	Princeton	LDO
TABN	395151.0N	743946.2W	30	Tabernacle	LDO
New York					
ALF	421331.2N	774749.7W	671	Alfred	LDO
ALX	441921.0N	755540.8W	122	Alexander Bay	LDO
APH	435028.7N	742949.1W	564	Airport Hangar	LDO
BGR	444943.8N	742227.0W	329	Bangor	LDO
BLM	411946.9N	735718.0W	134	Blum	CON*
CLIN	415230.0N	735056.4W	168	Clinton	LDO
CLY	435104.7N	742656.4W	579	Crystal Lake	LDO
CROG	435418.0N	752445.0W	244	Groghan	LDO
CTR	435226.9N	742735.9W	585	Castle Rock	LDO
DANY	444530.0N	735008.4W	507	Dannemora	LDO
DBM	411739.9N	735829.9W	27	Dunderburg Mtn	CON*
DNY	425010.7N	781007.7W	381	Dersam	LDO
DPL	411510.0N	735439.0W	67	Delli Paoli	CON*
EGN	435134.6N	742854.6W	549	Eagles Nest	LDO
GCB	411946.0N	735519.0W	150	Gobbelet	CON*
GSC	411558.0N	740014.0W	110	Girl Scout Camp	CON*
HNY	424954.6N	753053.4W	500	Hamilton	LDO
IPS	411602.0N	735654.0W	0		CON*
AMNH	404651.0N	735825.8W	0	Manhattan	LDO
MSNY	445954.0N	745143.2W	55	Massena	LDO
OCN	435305.4N	743145.6W	701	Over Castle Rock	LDO
OSB	412137.0N	725525.9W	212	Osborn	CON*
PAL	410015.0N	735432.9W	91	Palisades	LDO
PNY	445002.9N	733317.9W	177	Plattsburg	LDO
PTN	443420.9N	745858.1W	238	Potsdam	LDO
SANY	431025.8N	785213.2W	172	Sanborn	LDO

STA	Latitude DG MN SEC	Longitude DG MN SEC	Elevation Meters		Operator
SNP	411427.0N	735816.0W	30	Stoney Pt	CON*
SPS	411806.9N	735326.0W	168	St. Peters School	CON*
SRM	411342.0N	740050.0W	165	Scherman	CON*
STL	411119.0N	740012.9W	125	Stiles	CON*
TBR	410829.9N	741319.9W	261	Table Rock	LDO
UWL	435016.2N	743236.0W	561	Utowana Lake	LDO
WGL	412132.0N	735357.9W	152	Wegel	CON*
WND	422015.0N	740909.0W	602	Windham	LDO
WNY	442327.5N	735134.2W	598	Wilmington	LDO
WPNY	414810.8N	735814.4W	76	Westpark	LDO
WPR	411516.7N	733508.3W	152	Ward Pound Ridge	LDO
WVLY	422815.0N	783406.0W	600	West Valley	LDO
Pennsylvania					
BVR	404200.N	801960.0W	0	Beaver	PSU
ERI	420760.0N	795859.9W	0	Erie	PSU
MVL	395957.0N	762102.0W	0	Millersville	MSC
PHI	400659.9N	750760.0W	0	Abington	PSU
SCP	404742.0N	775153.9W	352	State College	PSU
Rhode Island					
None					
Vermont					
BVT	432055.8N	723506.9W	300	Baltimore	WES
COV	443439.6N	730845.0W	85	Colchester	LDO
DVT	445743.2N	721015.2W	370	Derby	WES
FLET	444322.0N	725706.0W	366	Fletcher	LDO
MARL	425018.0N	724803.0W	580	Marlboro	LDO
MDV	435956.9N	731052.1W	134	Middlebury	LDO
MGVT	445449.0N	723740.0W	262	Montgomery	LDO
MPVT	441642.0N	723624.0W	240	Montpelier	LDO

* Station codes not all cleared through NEIC.

Operator Code

CON – Consolidated Edison, Indian Point, New York
DGS – Delaware Geological Survey
EPB – Earth Physics Branch, Dept. of Energy, Mines, and Resources, Can.
LDO – Lamont-Doherty Geological Observatory of Columbia University
MIT – Massachusetts Institute of Technology
MSC – Millersville State College
PSU – Pennsylvania State University
WES – Weston Observatory, Boston College
WGE – Weston Geophysical Engineers, Inc.

TABLE 2.5-11 SITE INTENSITIES OF HISTORICAL EARTHQUAKES

Date Yr	Mo	Da	(GMT) Hr	Mn	Sec	L*	Lat. (N)	Long. (W)	Epicentral Intensity	Magnitude m_b	m_{bLg}	M_L	Dist. To Site MI	KM	Site Intensity A[1]	B[2]
1534							47.60	70.10	IX-X				326.2	524.9	4.8-5.8	
1638	06	11	20				47.65	70.17	IX				329.2	429.8	4.8	
1643	06	11	13	00			42.80	70.80	IV				7.2	11.6	4.0	
1661	02	10	12				45.50	73.00	VII				208.7	335.9	3.5	
1663	02	05	17	30			47.60	70.10	X				326.2	524.9	5.8	
1665	02	24					47.80	70.00	VIII				340.4	547.8	3.7	
1685	02	18					42.70	70.80	IV				13.9	22.4	4.0	
1727	11	09	22	40		L	42.80	70.60	VII				14.0	22.5	7.0	VII
1727	11	09	23	35		L	42.80	70.60	IV				14.0	22.5	4.0	
1727	11	10	02	15		L	42.80	70.60	IV				14.0	22.5	4.0	
1727	11	14	17	00		L	42.80	70.60	IV-V				14.0	22.5	4.0-5.0	
1727	11	18	11	20		L	42.80	70.60	IV				14.0	22.5	4.0	
1727	11	24	04	00		L	42.80	70.60	II-III				14.0	22.5	2.0-3.0	
1727	12	01					42.80	70.60	IV				14.0	22.5	4.0	
1727	12	16					42.80	70.60	IV				14.0	22.5	4.0	
1727	12	19	10	00		L	42.80	70.60	IV				14.0	22.5	4.0	

*Denotes local time.

[1] Site intensity calculated using: $I_{site} = I_o + 3.7 - .0011\ (\Delta km) - 2.7 Log_{10}\ (\Delta km)$ (Gupta and Nuttli, 1976). Arabic numerals used instead of Roman to denote intensity.

[2] Site intensity observed from isoseismal maps. Parentheses indicate an estimated value.

Date Yr	Mo	Da	(GMT) Hr	Mn	Sec	L*	Lat. (N)	Long. (W)	Epicentral Intensity	Magnitude m_b	m_{bLg}	M_L	Dist. To Site MI	KM	SiteIntensity A[1]	B[2]
1727	12	28	22	30		L	42.80	70.60	IV				14.0	22.5	4.0	
1727	12	29	04	00		L	42.80	70.60	II-III				14.0	22.5	2.0-3.0	
1728	01	04	23	00		L	42.80	70.60	IV-V				14.0	22.5	5.0	
1728	01	12	14	00		L	42.80	70.60	II-III				14.0	22.5	2.0-3.0	
1728	02	04	21	30		L	42.80	70.60	IV				14.0	22.5	4.0	
1728	02	08	06	30		L	42.80	70.60	IV				14.0	22.5	4.0	
1728	02	10	15	30		L	42.80	70.60	V				14.0	22.5	5.0	
1728	05	16					42.80	70.60	IV				14.0	22.5	4.0	
1728	07	30	10	00		L	42.80	70.60	IV				14.0	22.5	4.0	
1728	08	02	03	15		L	42.80	70.60	IV				14.0	22.5	4.0	
1729	11	25	08	00		L	42.80	70.60	IV				14.0	22.5	4.0	
1729	12	08	20	00		L	42.80	70.60	IV				14.0	22.5	4.0	
1730	03	09	01	45		L	42.80	70.60	IV				14.0	22.5	4.0	
1730	04	23	20	00		L	42.80	70.60	IV				14.0	22.5	4.0	
1730	12	22	18	45		L	42.80	70.60	III				14.0	22.5	3.0	
1731	01	12	19	00		L	42.80	70.60	IV				14.0	22.5	4.0	
1731	01	22	24	00		L	42.80	70.60	IV				14.0	22.5	4.0	

* Denotes local time.

1 Site intensity calculated using: $I_{site} = I_o + 3.7 - .0011 (\Delta km) - 2.7 Log_{10} (\Delta km)$ (Gupta and Nuttli, 1976). Arabic numerals used instead of Roman to denote intensity.

2 Site intensity observed from isoseismal maps. Parentheses indicate an estimated value.

Yr	Date Mo	Da	Hr	(GMT) Mn	Sec	L*	Lat. (N)	Long. (W)	Epicentral Intensity	Magnitude m_b	m_{bLg}	M_L	Dist. To Site MI	KM	Site Intensity A[1]	B[2]
1731	10	12	23	00		L	42.80	70.60	IV				14.0	22.5	4.0	
1732	09	16	16	00			45.50	73.60	VIII				225.3	362.6	4.4	(IV)
1734	07	10	03	15		L	42.80	70.60	II-III				14.0	22.5	2.0-3.0	
1736	11	23	02	00		L	42.80	70.60	IV				14.0	22.5	4.0	
1736	11	23	06	00		L	42.80	70.60	II-III				14.0	22.5	2.0-3.0	
1737	09	20	10	20		L	42.80	70.60	IV				14.0	22.5	4.0	
1737	12	18					40.80	74.00	VII				217.5	350.1	3.4	
1744	06	03					42.80	70.60	II-III				14.0	22.5	2.0-3.0	
1744	06	14	10	15		L	42.50	70.90	VI				27.8	44.7	5.2	(IV)
1744	06	14	17	00			42.52	70.92	IV				26.5	42.7	3.3	
1745	01	03					42.80	70.60	II-III				14.0	22.5	2.0-3.0	
1755	11	18	04	12		L	42.70	70.30	VIII				30.6	49.3	7.1	VI-VII
1755	11	18	05	29			42.70	70.30	IV				30.6	49.3	3.1	
1755	11	22	20	27		L	42.70	70.30	V				30.6	49.3	4.1	IV
1755	12	19	20	15			42.70	70.30	IV				30.6	49.3	3.1	
1766	06	14					42.70	70.90	III				14.1	22.7	3.0	
1766	12	13	18	40			43.10	70.80	IV				13.9	22.4	4.0	

* Denotes local time.

1 Site intensity calculated using: $I_{site} = I_o + 3.7 - .0011\,(\Delta km) - 2.7\log_{10}(\Delta km)$ (Gupta and Nuttli, 1976). Arabic numerals used instead of Roman to denote intensity.

2 Site intensity observed from isoseismal maps. Parentheses indicate an estimated value.

Date			(GMT)				Lat.	Long.	Epicentral	Magnitude			Dist. To Site		Site Intensity	
Yr	Mo	Da	Hr	Mn	Sec	L*	(N)	(W)	Intensity	m_b	m_{bLg}	M_L	MI	KM	A[1]	B[2]
1791	05	06	08	00		L	41.50	72.50	VI-VII				128.5	206.8	3.2-4.2	F
1791	12	06	23				47.40	70.50	VIII				310.8	500.2	3.9	
1801	03	01	15	30		L	43.07	70.77	IV				12.2	19.7	4.0	
1805	04	25	18	20			42.50	70.90	IV				27.8	44.7	3.2	
1807	01	13	23	00		L	43.00	71.00	IV				10.6	17.1	4.0	
1810	11	09	21	15		L	43.00	70.80	V				7.2	11.6	5.0	V
1811	12	16	08	00			36.60	89.60	XII				1085.3	1746.5	5.0	<IV
1812	01	23	15				36.60	89.60	XII				1085.3	1746.5	5.0	
1812	02	07	09	45			36.60	89.60	XII				1085.3	1746.5	5.0	
1814	11	28	19	14		L	43.70	70.30	IV-V				61.5	98.9	2.2-3.2	F
1816	09	09					45.50	73.60	VII				225.3	362.6	3.4	
1817	10	05	11	45		L	42.50	71.20	V-VI				33.1	53.2	4.0-5.0	(III)
1823	07	23	06	55		L	42.90	70.60	IV-V				12.1	19.5	4.0-5.0	IV
1837	01	15	07				42.50	70.95	IV				28.1	45.3	3.2	
1846	05	30	13	30			42.70	70.30	IV				30.6	49.3	3.1	
1846	08	25	04	45		L	42.50	70.80	V				27.7	44.5	4.2	(IV)
1847	08	08	10	00		L	41.70	70.10	V-VI				91.0	146.4	2.7-3.7	NF

* Denotes local time.

1 Site intensity calculated using: $I_{site} = I_o + 3.7 - .0011\,(\Delta km) - 2.7Log_{10}\,(\Delta km)$ (Gupta and Nuttli, 1976). Arabic numerals used instead of Roman to denote intensity.

2 Site intensity observed from isoseismal maps. Parentheses indicate an estimated value.

Date			(GMT)				Lat.	Long.	Epicentral	Magnitude			Dist. To Site		SiteIntensity	
Yr	Mo	Da	Hr	Mn	Sec	L*	(N)	(W)	Intensity	m_b	m_{bLg}	M_L	MI	KM	A[1]	B[2]
1852	11	27	23	45		L	43.00	70.90	V				7.5	12.1	5.0	(IV)
1854	12	11	00	30		L	43.00	70.80	IV-V				7.2	11.6	4.0-5.0	(IV)
1857	12	23	13	30		L	44.10	70.20	VI				88.7	142.8	3.7	NF
1860	10	17	11	15			47.50	70.10	VIII-IX				319.3	513.9	3.8-4.8	
1869	10	22	11	00			45.00	66.20	VIII				272.1	437.9	4.1	
1870	10	20	16	30			47.40	70.50	IX				310.8	500.2	4.9	
1872	11	18	14	00		L	43.20	71.60	IV-V				43.5	70.0	2.6-3.6	NF
1874	11	24					42.70	70.90	IV				14.1	22.7	4.0	
1879	10	25	22	30		L	42.98	71.47	IV				32.3	51.9	3.0	
1880	05	12	07	45		L	42.70	71.00	IV-V				16.0	25.7	3.9-4.9	(III-IV)
1880	07	20	19	00		L	42.98	71.47	IV				32.3	51.9	3.0	
1881	06	19					42.80	70.90	III				7.5	12.1	3.0	
1882	12	19	17	24		L	43.20	71.40	V				35.0	56.3	3.9	(NF)
1884	08	10	19	07			40.60	74.00	VII				227.1	365.5	3.4	F
1884	11	23	12	30		L	43.20	71.70	V				48.0	77.3	3.5	NF
1886	01	17	17	14		L	42.77	71.45	IV				32.1	51.7	3.0	
1886	09	01	02	51			32.90	80.00	X				851.0	1369.4	3.7	NF
1891	05	01	19	10		L	43.20	71.60	V				43.5	70.0	3.6	(NF)
1893	11	27	16	50			45.50	73.30	VII				216.7	348.7	3.5	

* Denotes local time.

1 Site intensity calculated using: $I_{site} = I_o + 3.7 - .0011\ (\Delta km) - 2.7 Log_{10}\ (\Delta km)$ (Gupta and Nuttli, 1976). Arabic numerals used instead of Roman to denote intensity.

2 Site intensity observed from isoseismal maps. Parentheses indicate an estimated value.

Date Yr	Mo	Da	(GMT) Hr	Mn	Sec	L*	Lat. (N)	Long. (W)	Epicentral Intensity	Magnitude m_b	m_{bLg}	M_L	Dist. To Site MI	KM	Site Intensity A[1]	B[2]
1897	03	23	18	07		L	45.50	73.60	VII				225.3	362.6	3.4	
1903	04	24	12	30			42.70	71.00	IV				16.0	25.7	3.9	
1905	07	15	05	10			44.20	70.00	V-VI				99.0	159.3	2.6-3.6	(III)
1905	08	30	10	40			43.10	70.70	V				15.5	24.9	4.9	(III)
1907	10	16	00	10			42.80	71.00	V				10.6	17.1	5.0	(III-IV)
1910	08	21	18	45	00.		42.70	71.10	IV				19.1	30.7	3.7	
1915	02	21	02	03			42.80	71.10	IV				14.8	23.9	4.0	
1918	08	21	05	15			44.20	70.50	VI				91.3	146.9	3.7	(NF)
1924	09	30	08	52	30		47.60	69.70	VII-VIII		5.5		328.9	529.2	2.8-3.8	
1925	01	07	13	07			42.60	70.60	V				24.0	38.6	4.4	(IV)
1925	03	01	02	19	20.		47.60	70.10	IX		6.6	7.0	326.2	524.9	4.8	IV
1925	03	09					42.93	71.47	IV				31.9	51.3	3.0	
1925	10	09	13	55			43.70	71.10	VI				56.7	91.2	4.3	NF
1926	03	18	21	09			42.80	71.80	V				49.0	78.9	3.5	(NF)
1927	03	09	04	08			43.30	71.40	IV-V				39.4	63.5	2.8-3.8	NF
1927	06	01	12	23			40.30	74.00	VII				242.3	389.9	3.3	
1929	08	12	11	24	48.		42.87	78.35	VIII		5.2	5.8	379.4	610.6	3.5	NF
1929	11	18	20	32	00.7		44.50	56.30	X			7.2	733.1	1179.8	4.1	III
1931	04	20	19	54			43.40	73.70	VII		4.7	5.0	147.9	238.1	4.0	NF
1934	08	02	14	58			42.60	70.70	IV				21.9	35.2	3.5	
1936	11	10	02	46			43.55	71.43	V				53.7	86.5	3.4	

* Denotes local time.

1 Site intensity calculated using: $I_{site} = I_o + 3.7 - .0011\,(\Delta km) - 2.7\mathrm{Log}_{10}\,(\Delta km)$ (Gupta and Nuttli, 1976). Arabic numerals used instead of Roman to denote intensity.

2 Site intensity observed from isoseismal maps. Parentheses indicate an estimated value.

Date			(GMT)				Lat.	Long.	Epicentral	Magnitude			Dist. To Site		Site Intensity	
Yr	Mo	Da	Hr	Mn	Sec	L*	(N)	(W)	Intensity	m_{bLg}	M_n	M_L	MI	KM	A[1]	B[2]
1940	12	20	07	27	26.		43.80	71.30	VII		5.4	5.8	66.2	106.6	5.1	IV
1940	12	24	13	43	44.		43.80	71.30	VII		5.4	5.8	66.2	106.6	5.1	IV
1940	12	27	19	56	09.		43.80	71.30			3.8	3.9	66.2	106.6	3.1	
1943	03	14	14	02	27.5		43.70	71.57				3.9	66.2	106.6	3.8	
1944	09	05	04	38	45.		44.97	74.90	VIII		5.8	5.9	247.0	397.5	4.2	III
1954	07	29	19	57	06.		42.70	70.70	V			4.0	15.5	25.0	4.9	
1957	04	26	11	40	06.		43.60	69.80	VI		4.8	4.7	71.1	114.5	4.0	I-IV
1958	09	19	17	45			43.60	70.20	V				58.0	93.3	3.3	
1962	12	29	06	19	10.		42.80	71.70	V			4.3	44.0	70.8	3.6	
1963	10	16	15	31	01.8		42.50	70.80	V		3.9	4.2	27.7	44.5	4.2	IV
1963	10	30	17	36	57.9		42.70	70.80	IV-V		2.4	5.0	13.9	22.4	4.0-5.0	
1963	12	04	21	32	34.9		43.60	71.60	IV-V			3.7	61.5	99.0	2.2-3.2	
1964	06	26	11	04	46.		43.30	71.90	V	2.6		3.6	60.1	96.7	3.2	
1966	10	23	23	05	34.		43.00	71.80	IV-V			3.1	48.9	78.8	2.5-3.5	
1969	08	06	16	03			43.80	71.40	V				68.1	109.6	3.1	
1971	10	21	00	54	46.2		42.70	71.15	V				20.9	33.6	4.5	
1973	06	15	01	09	05.		45.39	71.03			4.9		172.0	276.8	3.7	(III)
1975	08	03	01	03	22.		42.67	70.85				2.4	15.9	25.5	3.5	
1978	08	25	20	01	30.5		42.87	70.83			2.3		2.1	3.4	2.3	

* Denotes local time.

1 Site intensity calculated using: $I_{site} = I_o + 3.7 - .0011\,(\Delta km) - 2.7 Log_{10}\,(\Delta km)$ (Gupta and Nuttli, 1976). Arabic numerals used instead of Roman to denote intensity.

2 Site intensity observed from isoseismal maps. Parentheses indicate an estimated value.

TABLE 2.5-12 SUMMARY OF ROCK PROPERTIES

Property	Rock Type (2)	Range of Values	Average Value
1. Permeability of Rock Mass (cm/sec)			
a. Borehole water pressure tests (20 ft test zones)	D, Q	0 to 7 x 10^{-3}	(1)
b. Pumping test, Boring F-5 (200 ft. thickness of rock)	D, Q	10^{-3}	1 x 10^{-3}
2. Compression (P) Wave Velocity (ft/sec)			
a. Seismic	D, Q	13,000 – 16,000	(1)
b. Uphole and crosshole geophysical tests	D	16,500 – 18,500	(1)
c. Laboratory sonic tests (no confining pressure)			
Dry specimens	D, Q	14,682 – 17,687	16,240
Saturated Specimens	D, Q	16,960 – 20,050	18,870
3. Shear (S) Wave Velocity (ft/sec)			
a. Uphole and crosshole geophysical tests	D	8,000 – 10,000	(1)
4. Density (g/cm^3)	D, Q	2.63 – 3.01	2.80
5. Unconfined Compressive Strength (psi)	D	6,000 – 34,000	18,300
	Q	5,900 – 19,200	12,100
6. Moduls of Elasticity – Initial Tangent Modulus E_i (10^6 psi)			
a. Uphole and crosshole geophysical tests	D	6.5 – 9.8	(1)
b. Laboratory unconfined compression tests	D, Q	0.2 – 13	(1)

TABLE 2.5-12 SUMMARY OF ROCK PROPERTIES

Property	Rock Type (2)	Range of Values	Average Value
7. Modulus of Elasticity at 50% of Ultimate Compressive Strength (10^6psi)			
a. Laboratory tangent modulus (E_{t50})	D, Q	1.3 – 6.3	(1)
b. Laboratory tangent modulus (E_{s50})	D, Q	7.4 - 12	(1)
8. Poisson's Ratio			
a. Unconfined compression tests; initial load	D, Q	0.17 – 0.36	(1)
b. Unconfined compression tests; 50% of ultimate compressive strength	D, Q	0.19 – 0.28	(1)
c. Uphole and crosshole geophysical tests (from P and S velocities)	D	0.29 – 0.35	(1)
9. In-Situ Rock Stresses (psi)			
a. Largest stress	D	150 – 2,150	1,240
b. Smallest stress	D	50 – 1,570	860

(1) Average value not computed

(2) D = Diorite
Q = Quartzite

TABLE 2.5-13 SUMMARY OF MATERIAL PROPERTIES

		Bedrock	Structural Backfill	CLSM Backfill (#5015)
1.	Material Description	Quartz diorite and quartzite	Widely graded sands, gravelly sands, trace fines (SW)	Flowable, excavatable sand, cement, fly ash 15% air entrainment
2.	Compression Wave Velocity (ft/sec)		(1)	4410
	a. Seismic Survey	13,000 – 16,000		-
	b. Uphole and Crosshole Geophysical Tests	16,500 – 18,500		-
	c. Laboratory Sonic Tests	14,682 – 20,050		-
3.	Shear Wave Velocity (ft/sec)	8,000 – 10,000	(1)	2260
	a. Uphole and Crosshole Geophysical Tests	164 - 188		-
4.	Bulk Density (lbs/ft^3)		123 – 140(2)	103
5.	Shear Modulus, G ($lb/in.^2$)			$1.12 – 1.23 \times 10^5$
	a. Calculated from shear wave velocity ($G – pv_s^2$)	$2.3 – 3.5 \times 10^6$		-
	b. From undrained triaxial tests, 95% Compaction, strain _0.1%		4,000(3)	4615 – 5384(4)
	c. From drained triaxial tests, 95% Compaction, strain _0.1%		5,250(3)	3173 – 3525(4)
	d. Plate Load Test, Initial Loading, 97% Compaction		3,800 – 3,950(3)	(5)
	e. Plate Load Test, Reloading, 97% Compaction		7,500 – 11,150(3)	(5)

NOTES:

(1) Structural backfill compacted to at least 95% of maximum dry density as determined by ASTM 1557-78. Measurements of compression and shear wave velocities were not performed.

(2) Calculated from range of dry densities measured in the field during August 1979 using estimated water contents of 8 to 12%.

(3) Calculated from measured Young's modulus using equation $G = \frac{E}{2(1+v)}$ with V = 0.33 for drained conditions; v = 0.50 for undrained conditions.

(4) Calculated from measured Young's Modulus using equation G = E / 2(1+v).

(5) Plate Load Tests were not conducted for CLSM backfill.

TABLE 2.5-14 INDEX OF LABORATORY TESTS ON SAMPLES OF OVERBURDEN SOILS

	Field Exploration Program	Laboratory Testing Performed	FSAR Appendix Containing Results
1.	Initial Site Area Borings (B, D, E Series)	Water Contents on Split-Spoon Samples Plastic Limit q_u (penetrometer) on Split-Spoon Samples	2J
2.	Circulating Water Tunnel Investigation (AIT, AAIT, ADT, F Series Borings)	Cyclic Triaxial Test on Undisturbed Sand Samples Sieve Analyses S_u (Torvane)	References 117 and 120
3.	Additional Plant Site Borings (G Series)	Sieve Analyses	2I
4.	Intake Tunnel Extension Borings (AIT Series, continued)	Atterberg Limits	Reference 118

TABLE 2.5-15 INDEX OF SUBSURFACE EXPLORATIONS

	Field Exploration Program	Boring Series	Date Performed	FSAR Subsection Summarizing Program	References Containing Logs and Report
1.	Initial Borings on Site and in Vicinity of Site	A, B, C, P	October 1968 to July 1969	2.5.4.3a	GEI-6[(3)] and FSAR Appendix 2D
2.	Additional Site Area Borings	D, E	July to December 1972	2.5.4.3b	FSAR Appendices 2D and 2J
3.	Seismic Surveys and Fathometer Surveys in tidal marsh, Hampton Harbor, barrier beach and off-shore	- -	March to April 1972 (also some previous surveys in Fall 1968)	2.5.4.3c and 2.5.4.4	FSAR Appendix 2K
4.	In-Situ Wave Velocity Measurements in Site Area	- -	June to July 1973	2.5.4.3d and 2.5.4.4	- -
5.	Circulating Water Tunnel Investigations	AIT, AAIT, ADT, F	April 1973 to May 1974	2.5.4.3e	GEI-1[(1)]
6.	In-Situ Rock Stress Measurements in Site Area	OC	June to July 1973	2.5.4.3f	FSAR Appendix 2H

(1) Geotechnical Engineers Inc, Report GEI-1, “Geotechnical Report, Circulating Water Tunnel”, June 1974.

(3) Geotechnical Engineers Inc., Report GEI-6, “Drillers Logs for A, B, C and P Borings”.

	Field Exploration Program	Boring Series	Date Performed	FSAR Subsection Summarizing Program	References Containing Logs and Report
7.	Scotland Road Fault Investigation	SRF	November 1973 to March 1974	2.5.4.3g	FSAR Appendix 2C
8.	Portsmouth Fault Investigation	PF	May 1974	2.5.4.3h	FSAR Appendix 2C
9.	Inclined Borings at Reactor Locations	E2	May to June 1974	2.5.4.3i	FSAR Appendix 2F
10.	Additional Plant Site Borings and Test Pit	G	September to October 1974	2.5.4.3j	FSAR Appendix 2I
11.	Intake Tunnel Extension Borings	AIT continued	June to August 1975	2.5.4.3k	GEI-3 (2)
12.	Exploratory Trenches at Reactor 2	- -	August to September 1974	2.5.4.3l	FSAR Appendix 2L
13.	Geologic Mapping of Construction Excavations	- -	1977 Through 1980	2.5.4.3m and 2.5.1.2.b.7	Reference 121

(2) Geotechnical Engineers Inc., Report GEI-3, “Geotechnical Report, Intake Tunnel Extension”, September 1975.

TABLE 2.5-16 SUMMARY OF PROPERTIES FOR OFFSITE BORROW[1]

Specific gravity	2.67
Maximum dry density, pcf	112-126
Optimum water content, %	12-14
Poisson's ration	
Drained, 90% compaction [2]	0.30
Drained, 95% compaction [2]	0.33
Undrained	0.5
Peak friction angle in degrees from triaxial tests	
Drained, 90% compaction	34
Drained, 95% compaction	39
Undrained, 90% compaction	34
Undrained, 95% compaction	36
Modulus from triaxial tests, 10^3 psi	
Drained, 90% compaction [2]	6
Drained, 95% compaction [2]	14
Undrained, 90% compaction [2]	6
Undrained, 95% compaction [2]	12
Modulus from plate load test on test fill, 10^3 psi	
Drained, 97% compaction, initial loading	10.1 – 10.5
Drained, 97% compaction, reloading	20.0 – 29.7

Notes:

(1) Based on laboratory and test fill studies described in Subsections 2.5.4.5.c.1 and 2.5.4.5.d and in Appendices 2M and 2N.

(2) For $\bar{\sigma}_{3c}$ = 7.1 psi, strain less than 0.1%. For values at larger stress levels and strain level, see Appendix 2M.

TABLE 2.5-17 SUMMARY OF PROPERTIES FOR SAND-CEMENT BACKFILL (1)

	Range	Average
Unit weight, pcf (after curing in humid room surface dry)	123.9 – 127.4	125
Drained friction angle in degrees from Triaxial Test – 28 days	-	46.6°
Drained cohesion intercept c in psi Triaxial Test - 28 days	-	17.5
Unconfined Compressive Strength, psi		
2-in. cubes, 7 days	72-85	75
2-in. cubes, 28 days	128-142	135
2.8-in.-dia. cylinders, 28 days	89-106	95
2-in. cubes, 90 days	118-139	130
Confined Compressive Strength, psi		
$\bar{\sigma}_c$ = 7.1 psi, 2.8-in.-dia. cylinders, 28 days	119-134	125
$\bar{\sigma}_c$ = 42.7 psi, 2.8-in.-dia. cylinders, 28 days	364-376	370
Modulus of Elasticity, 10^3 psi		
Unconfined, 2-in. cube, 7 days	10.6-13.7	12
Unconfined, 2-in. cube, 28 days	19.1-33.3	25
Unconfined, 2-in. cube, 90 days	26.3-31.3	28
Unconfined, 2.8-in.-dia. cylinder, 28 days	34.3-75.0	54
$\bar{\sigma}_c$ = 7.1 psi, 2.8-in.-dia. cylinder, 28 days	17.4-32.6	24
$\bar{\sigma}_c$ = 42.7 psi, 2.8-in.-dia. cylinders, 28 days	32.9-39.6	36

NOTE: (1) Design mix (by weight): 1 part cement, 16.2 parts sand, 2.8 parts water.

TABLE 2.5-18 SUMMARY OF SAND –CEMENT FIELD AND LABORATORY DATA

Date	Description & Other Details	Ref.	Cement Content %/WT/ Batch (%/WT/Sand)	Cement WT/Batch LBS	Adjusted Sand WT Per Batch LBS	Water/Batch In Lbs.			Water/ Cement Ratio (WT) (Gal/Sack)	Water Temp °F	Sand/ Cement Temp °F	Slump In.	Air Content In %	Laboratory Density PCF		Nuclear Densometer 6″ Dir. Transm.			Cube Uncon. Comp. Strength – PSI (2″x2″)				Cyl. Uncon. Comp. Strength – PSI (6″ ϕ x 12″)			
						In Agg.	Added	Total						Wet WT (%MC)	Proctor Max. γd PCF	Wet WT PCF	Dry WT PCF (% Comp)	% MC	3 Days	7 Days	28 Days	90 Days	3 Days	7 Days	28 Days	90 Days
																				8	29			8	29	
9/18/77	Trial Batch Design	MM-	(12)	30	226.0	6.0 lbs	35.0 lbs	41.0 lbs	1.32	73	73	5-1/4	5.0	126.6						Days	Days			Days	Days	
		2651A	10.1						(14.93)											450	700	1050		390	870	1000
			(10)	25	231.0	6.0 lbs	34.0 lbs	40.0 lbs	1.55	72	71	5	6.2	126.9						550	580	1230		390	810	1050
			8.45						(17.45)											230	450	650		190	570	680
			(8)	20	236.2	6.2 lbs	33.8 lbs	40.5 lbs	1.96	69	71	5	6.0	126.3						250	380	580		240	550	670
			6.75						(22.07)											80	250	400		140	320	370
			(6)	15	241.3	6.3 lbs	34.2 lbs	40.5 lbs	2.61	69	70	5-1/4	7.9	125.2						100	240	430		150	300	390
			5.05						(29.4)											50	70	180	40	57	130	210
			(4)	10	246.5	6.5 lbs	34.5 lbs	41.0 lbs	4.0	70	68	5-1/4	8.7	122.6						30	100	230	30	50	120	200
			3.36						(44.64)											25	25	30		10	50	70
			(4)																	25	25	30		NA	40	70
9/28/77	Stump Dump Test Pit	PTL	3.45	114	2,730	-	-	56.0 gal			84	2	4.1	125.6												
		reprt	(6)	175	2,735	-	-	56.6 gal			70	3-1/2	3.8	128.8										NA	NA	50
		YD76	5.18																					70	110	50
	rodded																							60	110	220
																									110	180
			(8)	235	2,700	-	-	57.1 gal			74	4	2.8	132.0												240
			6.88																					120	150	320
																								120	150	300
	vibrated		(6)	175	2,735	_	_	56.6 gal			62	2-3/4	5.0	124.8										42	120	170
			5.18																					35	110	170
																									110	
																								8 days		
11/1/77	Duct Bedding 19P	PTL	(6)	175	2,735	-	-	56.6 gal			66	1-1/2	6.2	124.8					30	50	130	230	40	100	210	330
	South Set #1	S/C-1	5.18																30	80	150	230	40	100	230	310
	(non-safety related)																		30	30	180	200	40	100	220	270
	Duct Bedding 19P Set #2		(6)	175	2735	-	-	56.6 gal			56	1/4	5.5	124.8						80	100		60	130	240	270
	South		5.18																	90	130		60	140	260	280
	(non-safety related)																			80	150		60	120	260	240
11/3/77	PW Line East to South	PTL																								
	Pipe Shop	S/C-2	(6)	175	2,735	-	-	56.6 gal			56	3	4.5	124.0					50	60	100	150	10	60	100	180
	(non-safety related)		5.18																50	50	80	150	NA	60	130	150
																			80	50	80	120	NA	60	100	160
11/3/77	PTL Pittsburgh Lab.	PTL	(4) 3.29	109	2,730	-	-	56.0 gal	4.28										22.5	40.3	59.7					
	Test Data on Mix	177	(5) 4.1	137	2,730	-	-	56.0 gal	3.40										41.5	70.1	94.2					
	Design		(6) 4.88	164	2,730	-	-	56.0 gal	2.84										61.0	87.3	157.8					

2 days	4 days	6 days
30 20	60 50	60 60

Note:

- All Cylinder samples were molded in a Single-Use Mold, using tamping rods.
- Stripping of molded cylindrical specimen was conducted at the time of testing.
- Cube samples were stripped from mold after 24 hours and placed in plastic bag for curing in the curing room.
- Actual production batch weights were not available. Therefore only the design mix was given.

NA – Data was not available (usually because the sample crumbled or the movement of the needle was not readable in case of cube specimen).

Date	Description & Other Details	Ref.	Cement Content %/WT/ Batch (%/WT/Sand)	Cement WT/Batch LBS	Adjusted Sand WT Per Batch LBS	Water/Batch In Lbs.			Water/ Cement Ratio (WT) (Gal/Sack)	Water Temp °F	Sand/ Cement Temp °F	Slump In.	Air Content In %	Laboratory Density PCF		Nuclear Densometer 6″ Dir. Transm.			Cube Uncon. Comp. Strength – PSI (2″x2″)				Cyl. Uncon. Comp. Strength – PSI (6″ ϕ x 12″)			
						In Agg.	Added	Total						Wet WT (%MC)	Proctor Max. γd PCF	Wet WT PCF	Dry WT PCF (% Comp)	% MC	3 Days	7 Days	28 Days	90 Days	3 Days	7 Days	28 Days	90 Days
11/7/77	Potable Water Line	PTL	(6)	175	2735	-	-	56.6 gal			57	0	8.2	122.6					25	50	80	180	40	80	120	160
	North of Pipe Shop	S/C-3	5.18																50	50	100	250	30	80	150	180
	Set #1																		25	50	100	NA	40	80	140	140
	(non-safety related)																									
	Set #2																								120	120
																									110	120
																									140	120
11/16/77	East & South of Elec-	PTL	(6)	175	2735	-	-	56.6 gal			52	2	5.7	126.6					40	30	90	80	30	90	170	190
	trical Shop	S/C-4	5.18																50	30	50	50	30	80	170	180
	(non-safety related)																		25	30	50	30	20	NA	170	190
11/22/77	Manhole 22P	PTL	(6)	175	2735	-	-	56.6 gal			50	2-1/2	5.5	126.8					30	30	130	210	NA	NA	130	190
	(non-safety related)	S/C-5	5.18																30	30	100	200	NA	NA	110	180
																			30	30	100	210	20	70	130	180
12/5/77	Electrical Manhole 29P	PTL	(6)	175	2735	-	-	56.6 gal			58	2-1/4	4.5	128.4					30	50	50		20	50	110	
	(non-safety related)	S/C-6	5.18																30	50	50		20	60	120	
																			50	60	75		NA	70	130	
1/4/78	Electrical Duct Man-	PTL	5	169	2735	-	-	56.5 gal			40	2-1/2	6.0	125.2					NA	50	150		NA	90	140	
	Hole 2E & 3E Set #1	S/C-7																	NA	30	150		NA	80	150	
	(non-safety related)																		NA	50	150		NA	90	150	
	Set #2		5	169	2735	-	-	56.5 gal			53	1-3/4	6.2	NA					30	100	180		NA	80	170	
																			30	60	150		40	90	160	
																			NA	90	150		NA	NA	150	
1/12/78	Exeter-Hampton Duct	PTL	5	169	2735	-	-	56.5 gal			65	1-1/4	5.5	126.8	124.0	123.3	112.5	9.5	NA	50	60			80	160	
	Bank Sta. 3+25 to 3+50	S/C-8													@		(90.3)		NA	50	80			NA	140	
	Set #1										70	1-3/4	5.3	NA	14.0%	121.7	111.4	9.2	$\boxed{\text{3 hours}}$							
	(non-safety related)														OMC		(89.8)		NA	50	80			NA	160	
														NA	Sand	121.0	111.4	8.6								
															Alone		(89.8)									
																122.5	112.0	9.4								
																	(90.3)									
	Set #2										68	6	2.6	NA		121.7	111.7	8.9	$\boxed{\text{3 days}}$							
																	(90.1)		NA	50	30			80	130	
																119.9	111.1	7.9	NA	30	80			NA	130	
																	(89.8)		NA	50	80			NA	150	

Note:

- All Cylinder samples were molded in a Single-Use Mold, using tamping rods.
- Stripping of molded cylindrical specimen was conducted at the time of testing.
- Cube samples were stripped from mold after 24 hours and placed in plastic bag for curing in the curing room.
- Actual production batch weights were not available. Therefore only the design mix was given.

NA – Data was not available (usually because the sample crumbled or the movement of the needle was not readable in case of cube specimen).

Date	Description & Other Details	Ref.	Cement Content %/WT/ Batch (%/WT/Sand)	Cement WT/Batch LBS	Adjusted Sand WT Per Batch LBS	Water/Batch In Lbs.			Water/ Cement Ratio (WT) (Gal/Sack)	Water Temp °F	Sand/ Cement Temp °F	Slump In.	Air Content In %	Laboratory Density PCF		Nuclear Densometer 6″ Dir. Transm.			Cube Uncon. Comp. Strength – PSI (2″x2″)				Cyl. Uncon. Comp. Strength – PSI* (6″ ϕ x 12″)			
						In Agg.	Added	Total						Wet WT (%MC)	Proctor Max. γd PCF	Wet WT PCF	Dry WT PCF (% Comp)	% MC	3 Days	7 Days	28 Days	90 Days	3 Days	7 Days	28 Days	90 Days
1/23/78	Stump Dump-Test Pit	PTL YD105	5	169	2735	-	-	56.6 gal				1/2	4.7	126.8					30	80	150			90	190	
																			50	100	190			100	170	
																			30	130	180			100	170	
2/2/78	Exeter-Hampton Duct Line Set 1 (non-safety related)	SC-9	5	169	2735			56.6 gal			56	4-1/2	3	126.6					80	50	100			90	150	
																			100	50	190			90	160	
																			80	30	160			-	170	
																			50	30	160			100	190	
																			80	10	110			90	210	
	Set 2		5	169	2735			56.6 gal			70	2-1/2	4.5	-					80	40	140			-	200	
2/3/78	Exeter-Hampton Duct Line Set 1 (non-safety related)	SC-10	5	169	2735			56.6 gal			70	3-1/4	4.6	127.6					30	30	110			80	180	
																			30	10	120			80	160	
																			30	30	110				150	
																			50	10	150			NA	160	
																			50	40	160			NA	170	
	Set 2		5	169	2735			56.6 gal			54	3-1/2	3.5						30	30	110					
2/16/78	Service Water Trench (safety-related)	SC-11									59	2-3/4	5.0	126.8					30	40	210	250		50	200	240
	Set 1		5	169	2735			56.6 gal											10	50	160	250		70	250	N/A
																			30	50	150	180		-	260	
																			50	100	160	230		40	260	270
																			50	80	150	230		70	240	270
	Set 2		5	169	2735			56.6 gal			58	5-1/2	2.8						30	130	140	180			230	
2/17/78	Service Water Trench (safety-related)	SC-12									58	2	5.5	127.7					30	50	190	250		110	200	310
	Set 1		5	169	2735			56.6 gal											10	50	210	250		100	220	330
																			30	30	210	300				
																			30	30	160	180		120	210	270
																			10	30	140	180		130	240	280
	Set 2		5	169	2735			56.6 gal			54	3-1/4	3.8						40	40	140	150			250	

Note:

- All Cylinder samples were molded in a Single-Use Mold, using tamping rods.
- Stripping of molded cylindrical specimen was conducted at the time of testing.
- Cube samples were stripped from mold after 24 hours and placed in plastic bag for curing in the curing room.
- Actual production batch weights were not available. Therefore only the design mix was given.

*Safety-related data used in preparing Figure 2.5-45

NA – Data was not available (usually because the sample crumbled or the movement of the needle was not readable in case of cube specimen).

Date	Description & Other Details	Ref.	Cement Content %/WT/ Batch (%/WT/Sand)	Cement WT/Batch LBS	Adjusted Sand WT Per Batch LBS	Water/Batch In Lbs.			Water/ Cement Ratio (WT) (Gal/Sack)	Water Temp °F	Sand/ Cement Temp °F	Slump In.	Air Content In %	Laboratory Density PCF		Nuclear Densometer 6″ Dir. Transm.			Cube Uncon. Comp. Strength – PSI (2″x2″)				Cyl. Uncon. Comp. Strength – PSI* (6″ ϕ x 12″)				
						In Agg.	Added	Total						Wet WT (%MC)	Proctor Max. γd PCF	Wet WT PCF	Dry WT PCF (% Comp)	% MC	3 Days	7 Days	28 Days	90 Days	3 Days	7 Days	28 Days	90 Days	
2/22/78	Service Water Trench	SC-13	5	169	2735			56.6 gal			54	2	5.5	126.2					50	50	90	230		80	140	240	
	(safety-related) Set 1																		50	50	110	220		90	160	250	
																			50	50	110	220			N/A		
																			60	50	100	230		90	150	250	
	Set 2		5	169	2735			56.6 gal			48	2	5.5						60	50	100	170		90	150	250	
																			50	50	100	230			N/A		
	Set 3		5	169	2735			56.6 gal			51	2-1/2	3.7						100	50	180	270		90	180	230	
																			100	50	150	240		80	180	230	
											51	2-1/2	3.7						60	80	130	240			N/A		
1/24/78	GEI Laboratory Tests	GEI	5	1	16.18	-	-	2.79 gal						124.0						66.7							
		prlim		(part by weight of batch)				)						123.9						72.5							
		reprt												126.2						85.3							
														127.4							141.6						
														126.2							133.8						
														126.8							130.0						
														124.4								117.9		2.8" ϕ Cylinders			
														124.5								139.4					
														125.0								133.7					
														126.2											91		
														124.8											89		
														124.1											106		
														124.6											119		@ conf stress/7.1 PSI
														123.9											134		@ conf stress/7.1 PSI
														124.3											122		@ conf stress/7.1 PSI
														124.4											372		@ conf stress/42.7 PSI
														124.1											376		@ conf stress/42.7 PSI
														124.8											364		@ conf stress/42.7 PSI
																											@ conf stress/42.7 PSI
3/7/78	Service Water Trench	SC-14	5	169	2735			56.6 gal			58	2	5.6	127.1										90	170	250	
	(safety-related) Set 1																							90	170		
																									220		
	Set 2	SC-14	5	169	2735			56.6 gal			52	1-1/4	5.2	-										110	170	200	
																								120	170		
																									160		

Note:

- All Cylinder samples were molded in a Single-Use Mold, using tamping rods.
- Stripping of molded cylindrical specimen was conducted at the time of testing.
- Cube samples were stripped from mold after 24 hours and placed in plastic bag for curing in the curing room.
- Actual production batch weights were not available. Therefore only the design mix was given.

*Safety-related data used in preparing Figure 2.5-45

NA – Data was not available (usually because the sample crumbled or the movement of the needle was not readable in case of cube specimen).

Date	Description & Other Details	Ref.	Cement Content %/WT/ Batch (%/WT/Sand)	Cement WT/Batch LBS	Adjusted Sand WT Per Batch LBS	Water/Batch In Lbs.			Water/ Cement Ratio (WT) (Gal/Sack)	Water Temp °F	Sand/ Cement Temp °F	Slump In.	Air Content In %	Laboratory Density PCF		Nuclear Densometer 6″ Dir. Transm.			Cube Uncon. Comp. Strength – PSI (2″x2″)				Cyl. Uncon. Comp. Strength – PSI* (6″ ϕ x 12″)			
						In Agg.	Added	Total						Wet WT (%MC)	Proctor Max. γd PCF	Wet WT PCF	Dry WT PCF (% Comp)	% MC	3 Days	7 Days	28 Days	90 Days	3 Days	7 Days	28 Days	90 Days
3/7/78	Service Water Trench N9774, E6250 to N9774, E6300 safety-related																		60 60 50 80 60 80	110 110 - 125 110 100	160 100 130 150 160 150	260 270 280 200 150 160				
3/8/78	Service Water Trench N9774, E6250 to N9774, E6300 safety-related																		100 80 80	160 140 130	220 280 200	180 190 200		130 140 140	180 190 -	270 350 -
3/29/78	Service Water Trench N9774, E6250 to N9774, E6340 safety-related																		50 80 80	150 160 150	290 380 -	320 300 340		90 90 -	300 280 280	330 320 -

Note:

- All Cylinder samples were molded in a Single-Use Mold, using tamping rods.
- Stripping of molded cylindrical specimen was conducted at the time of testing.
- Cube samples were stripped from mold after 24 hours and placed in plastic bag for curing in the curing room.

*Data used in preparing Figure 2.5-45

TABLE 2.5-19 SUMMARY OF PROPERTIES OF QUARTZITE TUNNEL CUTTINGS

Specific gravity	2.83
Maximum dry density, pcf (ASTM D1557)	146-153
Optimum water content, %	5-7
Modulus from plate load test, 10^3 psi	
Controlled placement, 95.3% compaction, initial loading	28.3–35.9
Controlled placement, 95.3% compaction, reloading	54.3-66.6
No special control, 93.3% compaction, initial load	7.3-7.7
No special control, 93.3% compaction, reload	25.2-40.3
No special control, drained for 2 weeks, initial load	13.2-21.2
No special control, drained for 2 weeks reload	43.1-49.2
Stratified with gravelly sand, no special control, initial load	17.0-26.1
Stratified with gravelly sand, no special control, reload	41.2-45.3

TABLE 2.5-20 TYPES OF ENGINEERED BACKFILL BENEATH CATEGORY I STRUCTURES

Category I Structure	Type of Engineered Backfill Between Bottom of Structure and Top of Sound Bedrock (1)			Allowable Bearing Pressure	Maximum Bearing Pressure
	Fill Concrete	Offsite Borrow	Tunnel Cuttings	tsf	tsf
Unit No. 1					
Containment Structure	X			60	12
Containment Enclosure Building	X			60	36
Containment Enclosure Ventilation Area	X			60	2.8
Control Building	X			60	
Diesel Generator Building	X			60	
Non-Essential Switchgear Room	X			60	
RHR Spray Equipment Vault	X			60	
Primary Auxiliary Building	X			60	
Fuel Storage Building	X			60	
Fuel Storage Facility Wall	X			60	
Condensate Storage Tank	X			60	5.2
Emergency Feedwater Pumphouse	X			60	14
Steam and Feedwater Pipe Chase (East)	X			60	4.0
Steam and Feedwater Pipe Chase (West)	X			60	18
Pre-Action Valve Building	X			60	4.5
Personnel Hatch Area	X			60	
Tank Farm Area	X			60	
Refueling Water Storage Tank	X			60	
Reactor Makeup Water Storage Tank	X			60	

Category I Structure	Type of Engineered Backfill Between Bottom of Structure and Top of Sound Bedrock (1)				Allowable Bearing Pressure	Maximum Bearing Pressure
	Fill Concrete	Offsite Borrow	Tunnel Cuttings	CLSM	tsf	tsf
Other Structures						
Circulating Water Pumphouse	X				60	
Service Water Pumphouse	X				60	
Electrical Control Room	X				60	
Intake Transition Structure	X				60	
Discharge Transition Structure	X				60	
Piping Tunnels	X				60	
Waste Processing Building	X				60	
Service Water Cooling Tower	X				60	
Service Water Pump House - Barrier 1 Missile Barrier	X				60	
Safety-Related Electrical Manholes		X (2)	X (2)	X (5) (6)	2.5	0.5
Safety-Related Electrical Duct Banks		X (3)		X (5) (6)	-	-
Safety-Related Service Water Pipes		X (4)		X (5) (6)	-	-

NOTES:

(1) Backfill Concrete and sand-cement were not used as engineered backfill beneath the foundations of any seismic Category I structures.

(2) Offsite borrow was used beneath all safety-related electrical manholes except Manhole W19/20. The maximum thickness of offsite borrow beneath safety-related manholes is approximately 18 ft. Manhole W19/20 is founded on tunnel cuttings with a few layers of offsite borrow included within the tunnel cuttings. The thickness of the combined tunnel cuttings and offsite borrow beneath this manhole is 15.3 ft. (See Fig. 2.5-42c.)

(3) The maximum thickness of offsite borrow beneath safety related electrical duct banks is approximately 18 ft.

(4) The thickness of offsite borrow beneath safety-related service water pipes is 15 ft. or less, except in the area between the Circulating Water/Service Water Pumphouse and the Intake/Discharge Transition Structures where the thickness of offsite borrow beneath the service water pipes is approximately 25 ft.

(5) Controlled Low Strength Material (CLSM) is a relatively new engineered backfill material mixture approved for use in lieu of traditional compacted granular backfills at both safety related and non-safety related applications in support of maintenance or inspection activities. There are no thickness limitations for CLSM backfill which provides the required support and protective properties as described in Subsection 2.5.4.5c.6 with no detrimental effect to the ability of buried piping to accommodate design loadings.

(6) The CLSM has a 7-day unconfined compressive strength of 35 psi or 2.5 tsf matching the stated allowable bearing capacity of the compacted offsite borrow. The CLSM has a (min.) unconfined compressive strength of 50 psi at 28-days which equates to 3.6 tsf or a 44 percent strength margin. The CLSM continues to gain strength over time with a 90-day unconfined compressive strength of 83 psi or 5.9 tsf or a 236 percent strength margin over the allowable of 2.5 tsf. CLSM with an unconfined compressive strength less than 150 psi is considered “excavatable”.

TABLE 2.5-21 PROPERTIES FOR SEISMIC DEFORMATION ANALYSIS OF REVETMENT

	Property	Revetment Stone	Offsite Borrow 90% or 95% Compaction	Glacial Till	Filter Cloth
1.	Unit Weight				
	Saturated – below water	140 pcf	136 pcf	140 pcf	-
	Moist – above water	126 pcf	126 pcf	-	-
2.	Shear Modulus Parameter, K_2 (1)	170	55	110	-
3.	Damping at low strain Level ($\leq 10^{-6}$ in. / in.)	0.5%	0.5%	0.5%	_
4.	Poisson's ratio, μ				
	Saturated – below water	0.3	0.48	0.48	-
	Above water table	0.3	0.3	0.3	-
5.	Friction angle	36°	34°	36°	32°

(1) Parameter K_2 used to compute shear modulus at low strain level ($\leq 10^{-6}$ in. /in.) with equation $G_{max} - 1000K_2(\bar{\sigma}_m)^{1/2}$ where $\bar{\sigma}_m$ is the octahedral effective stress.

TABLE 2.5-22 SUMMARY OF PROPERTIES FOR CLSM [(1)(2)]

	Range	Average
Unit weight, (28 day cure time), (pcf)	103.6 – 105.2	104.4
Bulk Density, (pcf)	-	104
Buoyant unit weight, (pcf)	-	41
Moisture content, (percent)	-	13.4
Void Ratio	-	0.355
Drained friction angle from Triaxial Test (28 days) (degrees)	-	25.7
Drained cohesion intercept, c, from Triaxial Test (28 days) (psi)	-	14.5
Poisson's Ratio	0.30 – 0.34	0.32
Unconfined Compressive Strength, (psi)		
7 days cure time	-	35
28 days cure time	-	50
90 days cure time	-	83.5
Confined Shear Strength, (psi)		
σ_c = 7.1 psi, 28-days, undrained test	-	20.2
σ_c = 7.1 psi, 28-days, drained test	-	32.7
σ_c = 42.7 psi, 28-days, undrained test	-	54.7
σ_c = 7.1 psi, 28-days, drained test	-	57.9
Shear Wave Velocity, (ft/sec)	2260 – 2370	2315
Compression Wave Velocity, (ft/sec)	4410 – 4580	4495
Shear Modulus, G, (very low strain), (psi)	112,000 – 123,000	117,500
Young's Modulus of Elasticity, E, (very low strain), (psi)	300,000 – 319,000	309,500
Young's Modulus of Elasticity, E, (large strains), (psi)		
7 days, unconfined	2650 – 2950	2800
28 days, unconfined	2950 - 3250	3100
90 days, unconfined	17,650 – 18,650	18,150
28-days, undrained test, σ_c = 7.1 psi	-	12,000
28-days, drained test, σ_c = 7.1 psi	-	9,167
28-days, undrained test, σ_c = 42.7 psi	-	14,000
28-days, drained test, σ_c = 42.7 psi	-	8,250
At Rest Earth Pressure Coefficient (from Poisson's ratio)	0.43 – 0.54	0.48

NOTES:

(1) Controlled Low Strength Material (CLSM), Aggregate Industries FlowFill Mix No. 5015 and Seabrook Specification S-S-1-E-0224. CLSM Design Mix (lbs/cy): 80 lbs cement, 70 lbs fly ash, 2210 lbs sand, 375 lbs water, 1 lb Sika Lightcrete air entrainment.

(2) CLSM properties from Altran Solution Test Report No. 12-1235-TR-001, Rev 0, "Laboratory Evaluation of CLSM for Use with Buried Pipe Construction"

Table 2R-1 Direction and Distance Data

Release Point	Receptor Point	Release Height (ft)	Release Height (m)	Receptor Height (ft)	Receptor Height (m)	Distance (ft)	Distance (m)	Direction with respect to true north (degrees)
Plant Vent	East Intake	185	56.4	6.5	2.0	352.34	107.3	196
Plant Vent	CR Fire Exit Door	185	56.4	5	1.5	215.31	65.6	67
Plant Vent	Diesel Building Intake	185	56.4	28.5	8.7	246.52	75.1	65
Closest Containment Surface Point	East Intake	6.5	2.0	6.5	2.0	272.09	82.9	196
Closest Containment Surface Point	CR Fire Exit Door	5	1.5	5	1.5	135.06	41.1	67
Closest Containment Surface Point	Diesel Building Intake	28.5	8.7	28.5	8.7	166.27	50.6	65
RWST	West Intake	50	15.2	8.25	2.5	315.3	96.1	7
RWST	CR Fire Exit Door	50	15.2	5	1.5	75.54	23.0	151

Release Point	Receptor Point	Release Height (ft)	Release Height (m)	Receptor Height (ft)	Receptor Height (m)	Distance (ft)	Distance (m)	Direction with respect to true north (degrees)
RWST	Diesel Building Intake	50	15.2	28.5	8.7	77.97	23.7	127
Containment Personnel Hatch	East Intake	9.5	2.9	6.5	2.0	372.69	113.5	210
Containment Personnel Hatch	CR Fire Exit Door	9.5	2.9	5	1.5	149.95	45.7	49
Containment Personnel Hatch	Diesel Building Intake	9.5	2.9	28.5	8.7	181.88	55.4	50
Main Steam Line Closest Point	East Intake	20.58	6.3	6.5	2.0	202.5	61.7	210
Main Steam Line Chase (West) Panel (North)	CR Fire Exit Door	38.38	11.7	5	1.5	112.26	34.2	55
Main Steam Line Chase (West) Panel (North)	Diesel Building Intake	38.38	11.7	28.5	8.7	144.26	43.9	55
Closest MSSV	East Intake	53.16	16.2	6.5	2.0	251.94	76.7	191

Release Point	Receptor Point	Release Height (ft)	Release Height (m)	Receptor Height (ft)	Receptor Height (m)	Distance (ft)	Distance (m)	Direction with respect to true north (degrees)
Closest ADSV	East Intake	54.5	16.6	6.5	2.0	282.71	86.1	185
Closest MSSV	CR Fire Exit Door	53.16	16.2	5	1.5	115.6	35.2	57
Closest ADSV	CR Fire Exit Door	54.5	16.6	5	1.5	125.91	38.3	75
Closest MSSV	Diesel Building Intake	53.16	16.2	28.5	8.7	147.55	44.9	56
Closest ADSV	Diesel Building Intake	54.5	16.6	28.5	8.7	156.05	47.5	71
Primary Auxiliary Building Louver PAH-L6D	West Intake	61	18.6	8.25	2.5	331.74	101.1	22
Primary Auxiliary Building Fan PAH-FN46A	CR Fire Exit Door	88	26.8	5	1.5	122.85	37.4	101

Release Point	Receptor Point	Release Height (ft)	Release Height (m)	Receptor Height (ft)	Receptor Height (m)	Distance (ft)	Distance (m)	Direction with respect to true north (degrees)
Primary Auxiliary Building Fan PAH-FN46A	Diesel Building Intake	88	26.8	28.5	8.7	146.41	44.6	92
Turbine Building Closest Point	East Intake	6.5	2.0	6.5	2.0	211.77	64.5	234
Turbine Building Closest Point	CR Fire Exit Door	5	1.5	5	1.5	117.6	35.8	1
Turbine Building Closest Point	Diesel Building Intake	28.5	8.7	28.5	8.7	102	31.0	54
Waste Process Building SW Corner Roll-Up Door	West Intake	8.25	2.5	8.25	2.5	164.03	49.9	4
Carbon Delay Bed (East)	Diesel Building Intake	41.42	12.6	28.5	8.7	80.5	24.5	144

Release Point	Receptor Point	Release Height (ft)	Release Height (m)	Receptor Height (ft)	Receptor Height (m)	Distance (ft)	Distance (m)	Direction with respect to true north (degrees)
BWST (West)	Diesel Building Intake	22.67	6.9	28.5	8.7	53.67	16.3	144

Notes:

1. Release heights are calculated as 20 feet less than the reference elevations to account for the plant grade elevation.
2. The closest/limiting MSSV is MSSV-V-40 for the East Intake and MSSV-V-54 for the control room fire exit door and Diesel building intakes. The closest/limiting ADSV off of main steam line MS-4002 for the East Intake and main steam line MS-4003 for the control room fire exit door and Diesel building intakes.
3. Release and receptor points are considered to be at the centerpoint or centerline of all openings.
4. The closest main steam line break point for the East Intake is off of main steam line MS-4002.

TABLE 2R-2 CONTROL ROOM CHI/Q

This table summarizes the results for CHI/Q factors for the control room intakes for the various accident scenarios. Values are presented for the release point to the unfavorable control room makeup air intake and the unfiltered inleakage point, which is a Diesel Building intake and/or the control room fire exit door. These values are not corrected for Control Room Occupancy Factors, but the control room makeup air intakes do include credit for dilution. Based on the layout of the site and the fact that both makeup air intakes have equal flow rates, the base χ/Q values may be reduced by a factor of 2. These reduced values are listed in the table below.

Some of the event analyses take credit for a factor of 5 reduction on the base χ/Q values to account for buoyant plume rise from the MSSVs and ASDVs in accordance with Section 6 of Regulatory Guide 1.194. This reduction factor is not reflected in the table below.

Release-Receptor Pair	Release Point	Receptor Point	0-2 hour CHI/Q	2-8 hour CHI/Q	8-24 hour CHI/Q	1-4 days CHI/Q	4-30 days CHI/Q
A	Plant Vent	East Intake	2.34E-04	1.85E-04	6.75E-05	4.62E-05	3.87E-05
B	Plant Vent	CR Fire Exit Door	7.54E-04	5.03E-04	2.00E-04	1.45E-04	9.89E-05
C	Plant Vent	Diesel Building Intake	7.01E-04	4.74E-04	1.89E-04	1.37E-04	8.97E-05
D	Closest Containment Surface Point	East Intake	4.40E-04	3.46E-04	1.29E-04	8.40E-05	6.80E-05
E	Closest Containment Surface Point	CR Fire Exit Door	3.08E-03	2.17E-03	8.48E-04	6.31E-04	4.64E-04
F	Closest Containment Surface Point	Diesel Building Intake	2.06E-03	1.48E-03	5.79E-04	4.29E-04	3.11E-04
G	RWST	West Intake	3.54E-04	2.75E-04	9.70E-05	6.90E-05	4.37E-05
H	RWST	CR Fire Exit Door	7.52E-03	3.85E-03	1.26E-03	9.29E-04	7.23E-04
I	RWST	Diesel Building Intake	5.06E-03	2.85E-03	9.00E-04	7.17E-04	6.17E-04

Release-Receptor Pair	Release Point	Receptor Point	0-2 hour CHI/Q	2-8 hour CHI/Q	8-24 hour CHI/Q	1-4 days CHI/Q	4-30 days CHI/Q
J	Containment Personnel Hatch	East Intake	2.84E-04	2.48E-04	1.04E-04	6.50E-05	5.10E-05
K	Containment Personnel Hatch	CR Fire Exit Door	2.84E-03	2.30E-03	8.67E-04	5.87E-04	3.70E-04
L	Containment Personnel Hatch	Diesel Building Intake	1.97E-03	1.60E-03	5.99E-04	4.04E-04	2.58E-04
M	Main Steam Line Closest Point	East Intake	8.70E-04	7.85E-04	3.22E-04	2.02E-04	1.61E-04
N	Main Steam Line Chase (West) Panel (North)	CR Fire Exit Door	4.55E-03	3.72E-03	1.38E-03	9.67E-04	6.35E-04
O	Main Steam Line Chase (West) Panel (North)	Diesel Building Intake	3.11E-03	2.50E-03	9.37E-04	6.53E-04	4.29E-04
P	Closest MSSV	East Intake	5.45E-04	4.50E-04	1.56E-04	9.85E-05	8.00E-05
Q	Closest ASDV	East Intake	4.44E-04	3.38E-04	1.16E-04	7.30E-05	6.05E-05
R	Closest MSSV	CR Fire Exit Door	4.11E-03	3.31E-03	1.24E-03	8.72E-04	5.86E-04
S	Closest ASDV	CR Fire Exit Door	3.49E-03	2.79E-03	1.02E-03	7.54E-04	5.45E-04
T	Closest MSSV	Diesel Building Intake	2.89E-03	2.39E-03	8.87E-04	6.17E-04	4.11E-04
U	Closest ASDV	Diesel Building Intake	2.64E-03	2.11E-03	7.82E-04	5.71E-04	4.07E-04

Release-Receptor Pair	Release Point	Receptor Point	0-2 hour CHI/Q	2-8 hour CHI/Q	8-24 hour CHI/Q	1-4 days CHI/Q	4-30 days CHI/Q
V	Primary Auxiliary Building Louver PAH-L6D	West Intake	3.21E-04	2.68E-04	1.02E-04	6.75E-05	3.72E-05
W	Primary Auxiliary Building Fan PAH-FN46A	CR Fire Exit Door	2.91E-03	1.98E-03	6.61E-04	5.09E-04	4.37E-04
X	Primary Auxiliary Building Fan PAH-FN46A	Diesel Building Intake	2.63E-03	1.81E-03	6.48E-04	4.86E-04	3.95E-04
Y	Turbine Building Closest Point	East Intake	8.40E-04	7.65E-04	3.44E-04	2.41E-04	1.91E-04
Z	Turbine Building Closest Point	CR Fire Exit Door	4.49E-03	3.22E-03	1.19E-03	8.27E-04	5.99E-04
AA	Turbine Building Closest Point	Diesel Building Intake	5.95E-03	4.80E-03	1.79E-03	1.24E-03	8.00E-04
BB	Waste Process Building SW Corner Roll-Up Door	West Intake	1.18E-03	8.85E-04	3.25E-04	2.28E-04	1.47E-04
CC	Carbon Delay Bed (East)	Diesel Building Intake	8.57E-03	4.46E-03	1.43E-03	1.11E-03	8.37E-04
DD	BWST (West)	Diesel Building Intake	1.86E-02	9.65E-03	3.08E-03	2.39E-03	1.84E-03

Table 2R-3 Release-Receptor Point Pairs Assumed For AST Analysis Events

Event	Filtered Makeup	Unfiltered Inleakage Through Diesel Building	Unfiltered Inleakage Through CR Fire Exit Door
LOCA			
Containment Leakage CEVA Release	A	C	B
Containment Leakage CEVA Bypass	D	F	E
ECCS Leakage CEVA Release	A	C	B
ECCS Leakage CEVA Bypass	D	F	E
RWST Backleakage	G	I	H
Containment Purge	A	N/A	B
FHA (bounding for Containment and FSB)	J	L	K
MSLB			
Break Release	M	O	N
MSSV/ASDV Release	P (prior to 2.5 hours, also applies plume rise factor of 5 reduction) Q (after 2.5 hours)	T (prior to 2.5 hours, also applies plume rise factor of 5 reduction) U (after 2.5 hours)	R (prior to 2.5 hours, also applies plume rise factor of 5 reduction) S (after 2.5 hours)
SGTR	P (prior to 0.5 hours for the iodine spike – also applies plume rise factor of 5 reduction; entire transient for the noble gas release) Q (after 0.5 hours for the iodine spike)	T (prior to 0.5 hours for the iodine spike – also applies plume rise factor of 5 reduction; entire transient for the noble gas release) U (after 0.5 hours for the iodine spike)	R (prior to 0.5 hours for the iodine spike – also applies plume rise factor of 5 reduction; entire transient for the noble gas release) S (after 0.5 hours for the iodine spike)
Locked Rotor	P (prior to 2.5 hours, also applies plume rise factor of 5 reduction) Q (after 2.5 hours)	T (prior to 2.5 hours, also applies plume rise factor of 5 reduction) U (after 2.5 hours)	R (prior to 2.5 hours, also applies plume rise factor of 5 reduction) S (after 2.5 hours)

Event	Filtered Makeup	Unfiltered Inleakage Through Diesel Building	Unfiltered Inleakage Through CR Fire Exit Door
RCCA Ejection			
Containment Leakage CEVA Release	A	C	B
Containment Leakage CEVA Bypass	D	F	E
Secondary Side Release	P (prior to 2.5 hours, also applies plume rise factor of 5 reduction) Q (after 2.5 hours)	T (prior to 2.5 hours, also applies plume rise factor of 5 reduction) U (after 2.5 hours)	R (prior to 2.5 hours, also applies plume rise factor of 5 reduction) S (after 2.5 hours)
Small Line Break			
Break Release	V	X	W
Condenser Release	Y	AA	Z
Radioactive Gaseous Waste System Failure	BB	CC	N/A*
Radioactive Liquid Waste System Failure	BB	DD	N/A*

*It is conservative for these release points to assume that all unfiltered Control Room inleakage is through the Diesel Building (i.e., no unfiltered inleakage through the Control Room Fire Exit door).