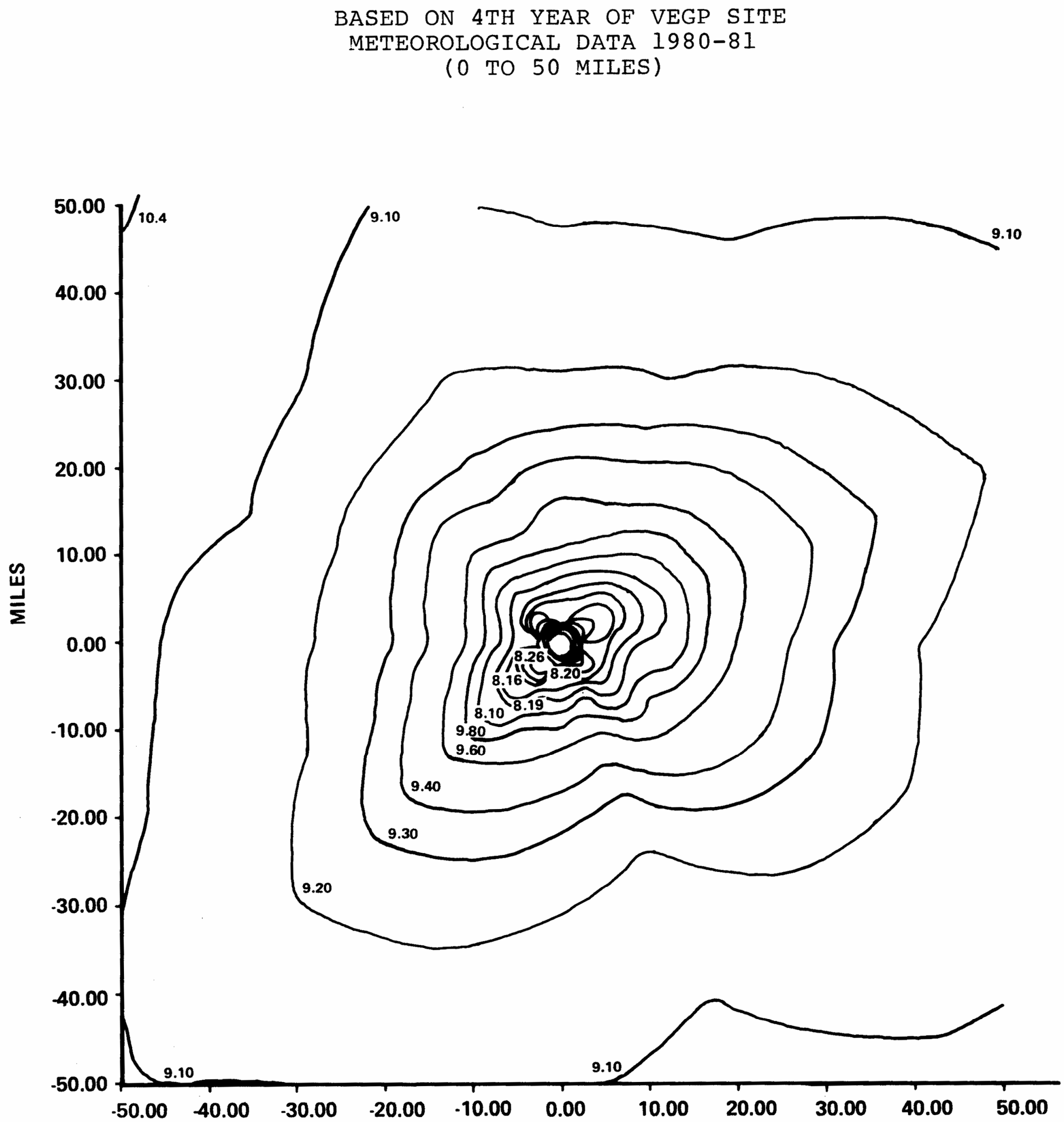


REV 14 10/07

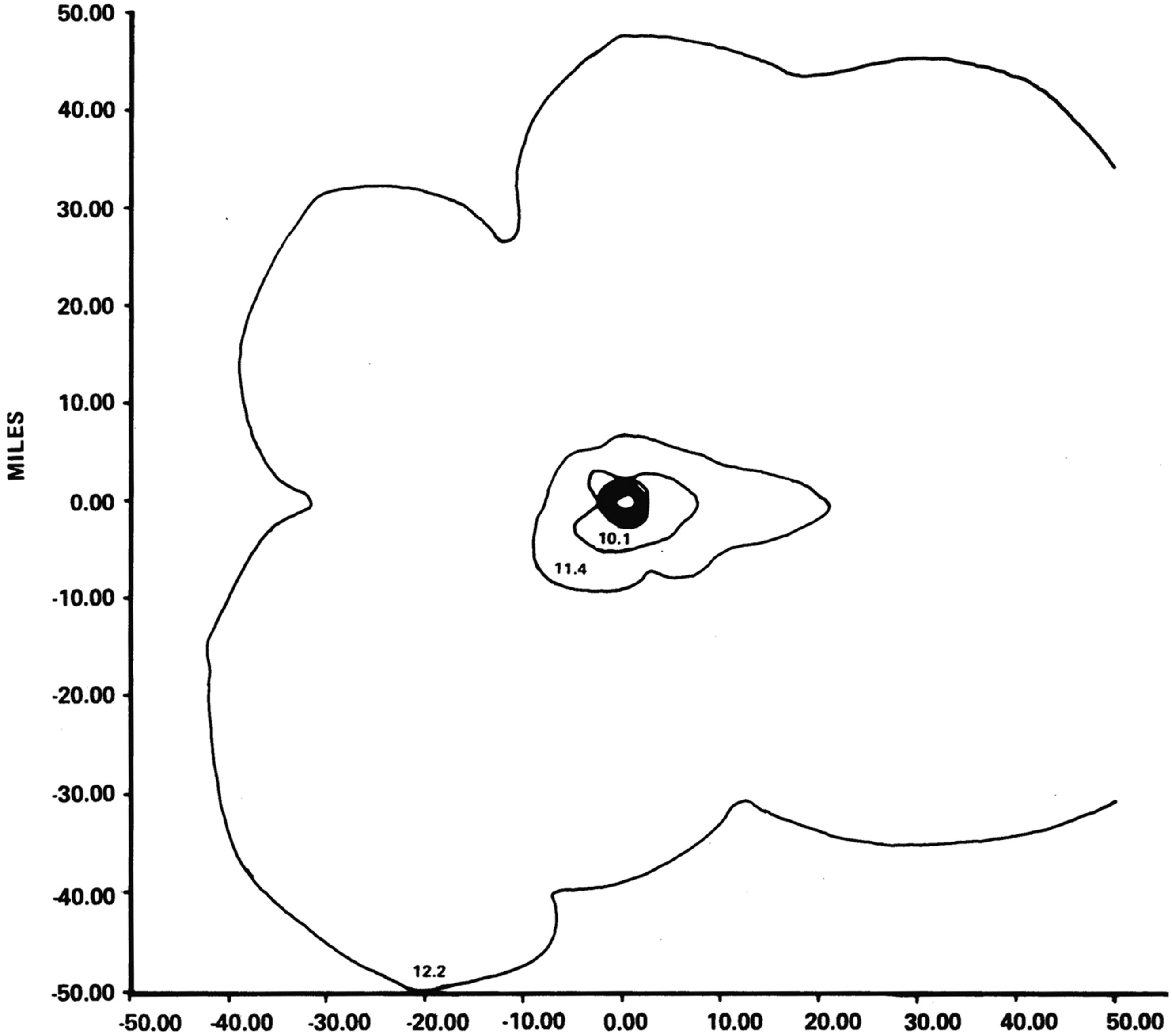
SOUTHERN COMPANY
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

ISOPLETH OF ANNUAL AVERAGE DEPOSITION (D/Q)

FIGURE 2.3.5–5

BASED ON 4TH YEAR OF VEGP SITE
METEOROLOGICAL DATA 1980-81
(0 TO 50 MILES)



REV 14 10/07

SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	ISOPLETH OF ANNUAL AVERAGE DEPOSITION (D/Q)
		FIGURE 2.3.5–6

2.4 HYDROLOGIC ENGINEERING

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

The site of the VEGP, which encompasses an approximate area of 3169 acres, is owned by Georgia Power Company and the co-owners. The plant is located about 26 air miles south-southeast of Augusta, Georgia, along the west bank of the Savannah River, and 15 air miles east-northeast of Waynesboro, Georgia, in the eastern sector of Burke County at river mile 151.1. The drainage area above the plant site is about 8015 mi^2 including the Beaverdam Creek area (about 35 mi^2). The Savannah River Basin and its subbasins above the plant are shown in figure 2.4.1-1. The drainage areas of the subbasins are given in table 2.4.1-1. The plant site and the approximate boundary of the property line topography and site environs are given in drawings CX2D46V003, CX2D46V004, CX2D46V005, and CX2D46V006.

The plant is on high ground with the entrance to the power block buildings at grade el 220 ft msl, approximately 140 ft above minimum river level, and about 80 ft above probable maximum flood (PMF) level. The PMF level is based on all-weather probable maximum precipitation (PMP) of Hydromet No. 51[1] and Hydromet No. 52[2] by the National Weather Service. As shown in drawings CX2D46V003, CX2D46V004, CX2D46V005, and CX2D46V006, nominal drainage changes will be made to divert runoff away from the buildings and to improve the safety and drainage of the plant area. During the construction of Units 3 and 4, contours and grading in the Units 3 and 4 construction impact area will be controlled such that the flooding analysis for Units 1 and 2 is not adversely affected. The grade elevation at the river intake structure is approximately 125 ft msl, and the grade elevation at the control building, containment buildings, and diesel generator buildings is approximately 220 ft msl. The access to major safety-related structures is well above the PMF level. The safety-related structures, their access elevations, and types of access openings to these structures are given in table 2.4.1-2.

2.4.1.2 Hydrosphere

The VEGP site is along the Savannah River about 50 river miles below Augusta, Georgia. At a minimum flow of 5800 ft^3/s, the river at this location is about 340 ft wide and from 9 to 16 ft deep and has an average velocity of about 2 mph. The Savannah River Basin has a drainage area of 10,577 mi^2, of which 4581 mi^2 are in western South Carolina, 5821 mi^2 in Georgia, and 175 mi^2 in southwestern North Carolina. The Tallulah and Chattooga Rivers, which form the Tugaloo River on the Georgia-South Carolina State line, and Whitewater and Toxaway Rivers, which form Keowee River in South Carolina, start in the mountains of North Carolina. Keowee River and Twelve Mile Creek join near Clemson to form the Seneca River. The two principal headwater streams, the Seneca and Tugaloo Rivers, join near Hartwell, Georgia, to form the Savannah River.[3]

From this point, the Savannah River flows about 300 miles south-southeasterly to discharge into the Atlantic Ocean near Savannah, Georgia. Its major downstream tributaries include Broad

River in Georgia, the two Little Rivers in Georgia and South Carolina, and Brier Creek in Georgia, as shown in figure 2.4.1-2. The topography of the basin varies from el 5500 ft at the headwaters of the Tallulah River to about 1000 ft in the rolling and hilly Piedmont province, descending to around 200 ft at Augusta, Georgia, and from there, gently rolling to the nearby Coastal province from Augusta to the Atlantic Ocean. Rainfall is generally abundant and is about 43 in. annually.

Snow cover is rare except in the mountains. Runoff average is about 15 in. annually for the entire drainage area, while runoff at Augusta, Georgia, averages about 19 in. Total stream flow varies considerably from year to year. Streams in the basin are typically high in the winter and early spring. During the summer, flows recede and remain low through autumn. Industry has settled along the Savannah River at Augusta, Georgia, where there is an inland port, and at Savannah, Georgia, where there is a deep draft harbor. Two upstream reservoirs, Clark Hill and Hartwell, along with certain channel improvements, ensure minimum water requirements. River regulation has increased the minimum daily flow from a record of 1105 ft^3/s before construction of the dams to 6100 ft^3/s after their construction.(3)

The Savannah River Basin is wider at upper reaches and has about uniform width from Burton Dam to Clark Hill Dam. From Clark Hill Dam to Augusta the width gradually narrows. Below Augusta the basin further narrows to the outlet with mild land slope.(3)

There are three major Corps of Engineers dams in the Savannah River Basin: namely, Hartwell, Richard B. Russell, and Clark Hill. The Richard B. Russell Dam is under construction and will be completed in 1984. After completion the reservoir, along with the Clark Hill Reservoir to the south and the Hartwell Reservoir to the north, will form a chain of reservoirs about 120 miles along the shoreline. The Hartwell Dam is located 89 miles above Augusta and 7 miles below the confluence of the Tugaloo and Seneca Rivers, which form the Savannah River. It is a multipurpose project with 5 ft of storage above the maximum power pool (660 ft msl) reserved for flood control. This is equivalent to a flood control storage capacity of 293,000 acre-ft.

The reservoir covers 55,950 acres at full power pool (660 ft msl). Surface area at the top of flood control pool (665 ft msl) is 61,350 acres. Minimum power pool elevation is 625 ft msl.(3)

Richard B. Russell Lake and Dam is another multipurpose project in the Savannah River Basin and is located on the Savannah River in Georgia and South Carolina, 275.1 miles above its mouth, 63 miles above Augusta, and about 16 miles southeast of Elberton, Georgia. At maximum power pool (el 475 ft msl) the reservoir has an area of 26,650 acres and has a stable lake with only 5 ft of drawdown. This project is scheduled for completion in 1984.(3)

Clark Hill Dam project was begun in August 1946 and completed in July 1954. It is a multipurpose project designed to reduce floods in the Savannah River and to ensure a required minimum river flow. The Clark Hill project is credited with reducing the sediment load in the Savannah River carried into the Savannah Harbor by 22 percent. At full power pool (330 ft msl), Clark Hill provides a total storage of 2,900,000 acre-ft. The flood control storage at Hartwell combined with the flood control storage at Clark Hill Dam (390,000 acre-ft at el 335 ft msl) reduces flood peak downstream. Due to Clark Hill Dam being in place, the flood stage at Augusta in March 1964 was reduced from 38 ft to 25 ft. The flood stage at Augusta is 32 ft.(3) The reservoir elevation is expected to recede to 326 ft msl from September to mid-December.

Flow regulation at Hartwell Dam regulates dependable power at Clark Hill Dam which, in turn, guarantees minimum flow downstream of Clark Hill Dam.(3)

A minimum flow of 5800 ft^3/s (based on the period of record) is required for navigation below Augusta; however, a discharge of 6300 ft^3/s is normally provided 80 percent of the time. Clark Hill Dam is designed for a maximum drawdown of 18 ft from the top of the power pool at el 330 ft msl to a minimum pool at el 312 ft msl. However, it is not anticipated that the minimum pool will be reached more often than once in 150 years.(3)

Heavy inflows into the lake begin generally in mid-December and continue through April with a full power pool by the first of May. For these major reservoirs and other water-controlling structures, the drainage areas, ownership, seismic design criteria, spillway design criteria, location, and type of structure are tabulated in table 2.4.1-3.

There are two domestic water users of surface water (the Savannah River) downstream of the plant. These users are Beaufort/Jasper County, 112 river miles downstream, and the Cherokee Hill (Port Wentworth) water treatment plant, 122 river miles downstream. Subsection 2.4.13 discusses the consequences of effluent releases to surface waters.

2.4.1.3 References

1. Schreiner, Louis C., and Riedel, John T., "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometeorological Report No. 51, National Weather Service, NOAA, U.S. Department of Commerce, Washington, D.C., 1978.

2. Hansen, E. M., and Schreiner, L. C., "Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometeorological Report No. 52 (preliminary), National Weather Service, Silver Springs, Maryland, 1979.

3. U.S. Army Corps of Engineers, South Atlantic Division, Water Resources Development in South Carolina, Atlanta, Georgia, March 1975.

2.4.2 FLOODS

2.4.2.1 Flood History

A partial list of annual peaks at Augusta, Georgia, from 1796 through 1979, covering periods before and after Clark Hill Dam started operations in 1954, is shown in table 2.4.2-1. These flood events were natural flood events before the upstream controlling structures were in place. No major flood has occurred at Augusta, Georgia, since 1951. The record flood at the site occurred in 1796 with an estimated water surface elevation of 116 ft msl. There have been no floods due to surges, seiches, or tsunami because the site is not located near a large body of water.

2.4.2.2 Flood Design Considerations

There are no safety-related structures that could be affected by floods and flood waves. For the derivation of probable maximum flood (PMF), the most recent National Oceanic and Atmospheric Administration, formerly Weather Bureau, publication Hydromet No. 51,(1) in conjunction with Hydromet No. 52,(2) has been used for developing probable maximum precipitation (PMP) envelopes for the Savannah River drainage basin above the VEGP. The PMP distribution is explained in paragraph 2.4.3.1. The PMF stage at the plant site is about 138 ft msl without wave runup, and with wave runup the water may reach as high as 165 ft msl. All safety-related structures have a grade elevation of 220 ft msl, which is well above the flood stage. If the valley storage effect between Clark Hill Dam and the VEGP site is taken into account, this results in lower flood peak and lower flood stage (drawing AX6DD319).

The makeup water river intake structure is nonsafety-related and has a deck elevation of 125 ft msl. This corresponds to the standard project flood stage elevation of 120 ft msl plus 5 ft freeboard allowance for waves. The river intake makeup water facilities are provided as a secondary backup source for the makeup water system. The makeup water for the nuclear service cooling water will be provided by wells drilled into the Tuscaloosa aquifer, which has enough water yield capacity to meet the makeup requirements for emergency conditions. In the unlikely event that the river intake structure is rendered inoperable, the safety of the plant operation or the plant safe shutdown will be unaffected, because the nuclear service cooling water tower basins are designed to provide 30-day inventory to permit safe shutdown (see subsection 9.2.5).

2.4.2.3 Effects of Local Intense Precipitation

The PMP, which is based on the world record envelope[3] and has a maximum intensity of 15 in. of rainfall in 1 h, is used to evaluate the effects of local intense precipitation. Runoff is directed away from the plant by the sloping ground surface to natural drainage channels and subsequently to the Savannah River. The site grading and drainage plan is shown in drawing CX2D46V003; pertinent elevations and additional slope arrows are shown in drawing CX2D46V006. It is assumed that all catch basins and roof drains are plugged for PMP flood analysis and that all flow is overland. The drainage system for the site precludes flooding of safety-related structures and equipment. The site drainage plan for local PMP storm is given in drawing CX2D46V004; the 100-year rainfall event is given in drawing CX2D46V005.

The normal yard drainage system, consisting of pipe culverts and open channels, is designed to drain the runoff from a 100-year rainfall event with a maximum intensity of 4 in. of rainfall in 1 h.

The rational formula is used to determine peak runoff rates from specified areas. Time of concentration is determined with the aid of the Kirpich nomograph.[4] Duration is assumed to be equal to time of concentration.

Corresponding intensity is taken from table 2.4.2-3 for 100-year storm flood analysis. Runoff coefficients for use with the rational method in determining 100-year peak rates of flow are as follows:

A. Unpaved Areas

Power block area	0.65
Adjacent to power block	0.50
All other	0.40

B. Paved Areas

Concrete	0.95
Asphalt	0.90

For the PMP flood event, the analysis is based on the following: The general power block area and plant are bounded by peripheral railroads and roads typically at or near el 218.50 ft. The ground surface is sloped from the power block with threshold el 220 ft to the railroads and roads. All ditches, inlets, and storm drains are considered to be plugged, allowing local ponding due to the PMP storm until the flow discharges over the railroad. The rational formula is used to determine peak runoff rates from specified areas (power block to the railroad). The time of concentration used is T_c = 5.0 min with corresponding intensity of 55.0 in./h. The resulting flow is considered to discharge over the peripheral railroad acting as a broad-crested weir. The formula $Q = CLH^{3/2}$ is then used to determine the water surface el at the railroad, with C = 2.60. The flow then follows the natural ground surface overland to drainage channels.

Corresponding intensity is taken from table 2.4.2-2 for PMP flood analysis.

Runoff coefficients for use with the rational method in determining PMP peak rates of flow are as follows:

A. Specific Areas

Paving	0.95
Grass	0.85
Gravel	0.75
Forest/shrub	0.65

B. Overall Areas

Power block	0.86
Adjacent to power block	0.77

All runoff coefficients are exclusive of structures. The runoff coefficient for all power block structures is 1.0.

Depth of flow in channels is determined using Manning's formula for open channel flow, with n = 0.014 for concrete lined channels and 0.020 for air blown mortar lined channels. Manning's n for pipe flow is taken as 0.024 for corrugated steel and 0.012 for concrete pipe. There are no unpaved channels. The roof drain systems for all safety-related structures are designed to pass the runoff from the PMP. The design includes measures to guard against wind-induced seepage through roof and wall penetrations and doors where safety-related equipment could be damaged.

Icing normally does not occur. The combination of icing followed by heavy local precipitation is not considered for determining the effects of local intense precipitation for site drainage.

Seismic Category 1 structure roofs are typically flat and surrounded by 2-ft-high parapet walls. With the exception of the control building part of the equipment building, which contains no Seismic Category 1 equipment, all roofs are drained by scuppers in the parapet walls.

For the 100-year storm the scuppers are located at the roof elevation, and the storm water is dumped into downspouts which run down the outside faces of structures onto adjacent roofs or below ground level to the yard drainage system. Since downspouts are located external to structures, there are no internal drain lines which could flood safety-related equipment.

Additional scuppers are provided to drain the PMP. These are located a maximum of 6 in. above the roof line, and are a minimum of 6 in. deep and 12 in. wide. Their size minimizes their susceptibility to clogging, and flow is allowed to fall directly to adjacent roofs or to grade. Downspouts, which would have the potential for clogging, are not provided to drain the PMP. The site is graded away from Seismic Category 1 structures so that flooding of safety-related structures and equipment will not occur.

The roofs of Seismic Category 1 structures are designed for 18 in. of ponded water corresponding to a load of 93.6 lb/ft^2. Flow is routed between adjacent structures and a sufficient number of PMP scuppers are provided to ensure that the 18-in. maximum ponded depth is not exceeded. It is assumed that during the PMP all 100-year storm scuppers and one-half the PMP scuppers could be plugged. A 30-lb/ft^2 load is considered in the design of the roofs of all Seismic Category 1 structures to account for the snow or 100-year rain load.

2.4.2.4 References

1. Schreiner, Louis C., and Riedel, John T., "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometeorological Report No. 51, National Weather Service, NOAA, U.S. Department of Commerce, Washington, D.C., 1978.

2. Hansen, E. M., and Schreiner, L. C., "Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometeorological Report No. 52 (preliminary), National Weather Service, Silver Springs, Maryland, 1979.

3. Chow, Ven Te, Handbook of Applied Hydrology, McGraw-Hill Book Company, New York, pp 9-46 through 9-48, 1964.

4. Kirpich, P. Z., "Time of Concentration of Small Agricultural Watersheds," Civil Engineering, Vol 10, No. 6, p 352, June 1940.

2.4.3 PROBABLE MAXIMUM FLOODING ON STREAMS AND RIVERS

The probable maximum flooding for the Savannah River Basin above the VEGP site is determined from probable maximum precipitation (PMP). The 72-h PMP depth-area-duration (DAD) envelope is prepared from Hydromet No. 51.[1] The runoff is routed through the streams and reservoirs. The resulting maximum stage at the VEGP site is 165 ft msl including allowance for wave action. The procedure for the development of DAD envelope, temporal distribution, and spatial distribution is described in paragraph 2.4.3.1. The procedures for flood hydrograph development and subsequent routing through the reservoirs and streams are described in paragraphs 2.4.3.3 and 2.4.3.4, respectively.

As shown in drawings CX2D46V003, CX2D46V004, CX2D46V005, and CX2D46V006, the plant is located on a ridge line, and access to it follows the high ground. The potential for flooding is minimal.

2.4.3.1 Probable Maximum Precipitation

The PMP values for 10, 200, 1000, 5000, 10,000, and 20,000 mi^2 drainage areas for 6, 12, 24, 48, and 72 h are read from all- season PMP charts.[1] These values are given in table 2.4.3-1 and are used to prepare the DAD curve as shown in figure 2.4.3-1. From the DAD curve, a cumulative PMP envelope for the drainage area above the VEGP site is prepared as given in figure 2.4.3-2. The PMP values at 6-h intervals are obtained from figure 2.4.3-2 and are given in table 2.4.3-2. For the spatial distribution of the storm over the drainage area, the storm is positioned at the center of the drainage basin above the VEGP site. Also, the storm is positioned at noncentral locations to see the effect of storm centering on the resulting hydrograph.

The elliptical isohyets are drawn to cover various drainage areas and are assigned the precipitation depth as read from DAD curves corresponding to the area covered by the isohyet. The areas between two isohyets are measured, and the weighted arithmetic average of PMP is obtained for the two positions as explained below.

Position No. 1: The storm is positioned at the center of the drainage basin above the plant site. The major axis of the isohyet is aligned with the major axis of the drainage basin as shown on drawing AX6DD315.

Position No. 2: This is the same as position No. 1 except that the center of the isohyet is offset to the west about 6 miles and laid over the drainage basin in such a way that the major axis of the isohyet is approximately parallel to the major axis of the drainage basin at the center, as shown on drawing AX6DD316.

In addition, the PMP for a storm centered at position No. 1 is determined by an alternate method identified below. The storm is positioned as in No. 1, but for the storm distribution the procedure given in Hydromet No. 52[2] is closely followed.

For positions No. 1 and 2, a two-to-one major to minor axis elliptical storm isohyetal pattern is used, while for Hydromet No. 52, a three-to-one major to minor axis pattern is used. The storm isohyets, as positioned on the drainage basin, are shown on drawings AX6DD315, AX6DD316, and AX6DD317. For positions No. 1 and 2, all 6-, 12-, 24-, 48-, and 72-h values are averaged over the subbasins. In the case of the Hydromet No. 52 procedure, only the three greatest 6-h PMP values are spatially and temporally adjusted. No adjustment is applied to the remaining 6-h PMP values. These values are given in tables 2.4.3-3 through 2.4.3-5. The PMP for positions No. 1 and 2 and that based on Hydromet No. 52 are given in table 2.4.3-6 for all subbasins above the VEGP site. In the case of positions No. 1 and 2, no spatial or temporal adjustment has been made. The critical sequence arrangement of PMP in all three procedures is the same.

The PMP computations for positions No. 1 and 2 are straightforward. The weighted arithmetic average PMP over the subbasins for 6-,12-, 24-, 48-, and 72-h durations is determined.

The DAD curve for each subbasin is prepared. The unit hydrograph for each subbasin and its duration is known.[3] The incremental PMP is read from the DAD curve at a duration equal to unit hydrograph duration. The Corps of Engineers' computer program HEC-1[4] is used to develop a flood hydrograph for each subbasin. It is routed through the stream or reservoir to a predetermined location to be combined with other hydrograph(s) at that location. The procedure is continued until the flood hydrograph is routed past the VEGP site.

In the procedure identified here as Hydromet No. 52, only the three greatest 6-h PMPs are spatially and temporally adjusted. The adjustment factor depends upon the basin orientation.[2] The isohyet adjustment factors for the three greatest 6-h PMPs are given in table 2.4.3-3, while the three greatest 6-h PMPs for isohyets enclosing various areas are given in table 2.4.3-4. The details are available in reference 2.

The spatially adjusted, idealized elliptical isohyets are laid over the drainage basin as in position No. 1. The first, second, and third greatest 6-h PMP values from table 2.4.3-4 are assigned to each isohyet enclosing the area given in table 2.4.3-4. The average PMP for the first, second, and third greatest 6-h period for each subbasin is computed by the weighted arithmetic average of the isohyets enclosing the subbasin area.

For each subbasin, as given in figure 2.4.1-1, the 72-h PMP at 6-h intervals is arranged in critical sequence and is given in table 2.4.3-5. The flood hydrograph for each subbasin is developed and routed as mentioned earlier. Identical initial rainfall loss and uniform infiltration loss are used in all the aforementioned cases (procedures) for the development of the flood hydrograph. Also, identical parameters and routing procedures are used to route the flood hydrograph resulting from the three procedures and are then used to compare them objectively.

2.4.3.2 Precipitation Losses

The Savannah River is predominantly forested. Rainfall is generally abundant, up to 80 in. annually. Snow cover is rare except in the mountains. Runoff averages about 15 in. annually for the entire drainage area.[5] The infiltration and retention losses are very high.

An initial loss of 0.5 in. and a nominal uniform rate of 0.05 in./h is used. These values are well below the losses that would be used in an ordinary situation.

2.4.3.3 Runoff and Stream Course Models

The River Forecasting Center in Atlanta has divided the Savannah River Basin into subbasins and has established gauging points for its forecasting purpose.[3] The center has developed unit hydrographs for each subbasin. For the purpose of this study the River Forecasting Center unit hydrographs and subbasin drainage areas are used. The subbasins are numbered as shown on drawing AX6DD315. The basins and their drainage areas along with unit hydrograph duration are shown in table 2.4.1-1. In the Savannah River Basin three major multipurpose dams, namely, the Hartwell Dam, the Richard B. Russell Dam, and the Clark Hill Dam are modeled.

The Corps of Engineers' HEC-1 computer program[4] is used to develop the inflow hydrograph and to route it to the desired location to be combined with other local or routed hydrograph(s). The unit hydrograph and lag time for each subbasin are provided by the River Forecasting Center in Atlanta. The lag time, as shown on drawing AX6DD320, is utilized to delay the peak. The Tatum method[4][6] with three steps is used to route the hydrograph through the streams, and the modified pulse method[4] is used to route through the reservoirs (Hartwell, Richard B. Russell, and Clark Hill). The reservoirs are assumed at maximum power pool levels to release more water and obtain a conservative hydrograph at the VEGP site. Channel storage was omitted in the HEC-1 routing procedure.[4] For comparison purposes, the National Weather Service (NWS) dam break model[7] is also used to route the probable maximum flood (PMF) outflow hydrograph from Clark Hill Dam to the VEGP site. The hydrographs on two schemes of routing are shown on drawings AX6DD318 and AX6DD319.

The following steps are followed to obtain the PMF hydrograph at the VEGP site:

1. Compute inflow into Burton Dam using unit hydrograph for subbasin 1.
2. Determine outflow from Burton Dam.
3. Route Burton Dam outflow to Hartwell Dam.
4. Compute local area hydrograph from Burton Dam to Hartwell Dam using unit hydrograph for subbasin 2.
5. Add results from steps 3 and 4 to obtain Hartwell inflow forecast.
6. Determine outflow from Hartwell Dam.
7. Route Hartwell Dam outflow to Calhoun Falls.
8. Compute local area hydrograph from Hartwell Dam to Calhoun Falls using unit hydrograph for subbasin 3.
9. Add results from steps 7 and 8 to obtain Calhoun Falls inflow forecast.
10. Route Calhoun Falls to Clark Hill Dam.
11. Compute Carlton Bridge forecast using unit hydrograph for subbasin 4.
12. a. Lag Carlton Bridge to Bell.
    b. Route lagged Carlton Bridge to Bell.
13. Compute local area hydrograph from Carlton Bridge to Bell using unit hydrograph for subbasin 5.
14. Add results from steps 12b and 13 to obtain Bell forecast.
15. a. Lag Bell to Clark Hill Dam.
    b. Route lagged Bell to Clark Hill Dam.
16. Compute local area hydrograph from Bell-Calhoun Falls to Clark Hill Dam using hydrograph for subbasin 6.
17. Add results from steps 10, 15b, and 16 to obtain Clark Hill inflow forecast.
18. Determine outflow from Clark Hill Dam.
19. Route Clark Hill outflow to Stevens Creek Dam.
20. Compute Modoc forecast using unit hydrograph for subbasin 7.

21. a. Lag Modoc to Stevens Creek Dam.

    b. Route lagged Stevens Creek outflow to Butler Creek.

22. Compute local area hydrograph from Clark Hill-Modoc to Stevens Creek Dam using unit hydrograph for subbasin 8.

23. Add results from steps 19, 21b, and 22 to obtain Stevens Creek inflow forecast.

24. Determine outflow from Stevens Creek Dam.

25. a. Lag Stevens Creek outflow to Butler Creek.

    b. Route lagged Stevens Creek outflow to Butler Creek.

26. Compute local area hydrograph from Stevens Creek to Butler Creek using unit hydrograph for subbasin 9.

27. Add results from steps 25b and 26 to obtain Butler Creek forecast.

28. a. Lag Butler Creek to plant site.

    b. Route lagged Butler Creek to plant site.

29. Compute local area hydrograph from Butler Creek to plant site using unit hydrograph for subbasin 10.

30. Route results obtained in step 29 to plant site.

31. Add results from steps 28b and 30 to obtain PMF hydrograph at plant site.

The hydrograph development scheme used in the HEC-1[4] computer program is given on drawing AX6DD320. The routing scheme used in the NWS[7] computer program is similar to the one shown on drawing AX6DD320, except that the outflow hydrograph from Clark Hill Dam is input to the NWS[7] computer model and routed through the downstream reach taking into consideration the downstream valley storage. The river reach between Clark Hill Dam and Burton's Ferry Bridge is divided into 42 subreaches, and the river reach between Clark Hill Dam and the VEGP site is divided into 27 subreaches, as shown on drawing AX6DD320. The runoff hydrographs from subbasins 7, 8, 9, and 10 are input as lateral inflows. The resulting hydrographs from both schemes are shown on drawings AX6DD318 and AX6DD319. It is obvious that the peaks are substantially reduced when the valley storage is considered in routing. The location of the river cross-sections used are shown on drawing AX6DD320.

2.4.3.4 Probable Maximum Flood Flow

The PMF is developed and routed through the stream course to the VEGP site as explained in paragraph 2.4.3.3. The routed hydrographs at the plant site are given on drawings AX6DD318 and AX6DD319. The three major dams upstream (Clark Hill, Richard B. Russell, and Hartwell) are all designed to withstand PMF combined with wind setup and wave runup from the wave action. It is unlikely that the dams will be overtopped. The dominotype upstream dam failure is discussed in paragraph 2.4.4.1. The Savannah River Basin embraces three distinct areas: the

Blue Ridge Mountains at the headwater; the Piedmont belt which ends around Augusta, Georgia; and the Coastal Plain from there to the Atlantic Ocean. The river reach between Clark Hill Dam and Augusta is narrow and has little valley storage. Below Augusta the river channel has wide flood plains which store flood flows and modulate the peak discharge as the flood progresses downstream. This causes the peak discharge by the VEGP site to be less than at Augusta.

The PMF peak discharge at the VEGP site, ignoring the valley storage effect, is 895,000 ft^3/s, which corresponds to a river stage of 136 ft msl read from a steady-state rating curve (figure 2.4.3-3). The PMF peak discharge at the same location, if valley storage of the upstream reach is taken into consideration, is 540,000 ft^3/s, and maximum flood wave elevation is 126 ft msl.

The Corps of Engineers has a PMF outflow hydrograph at Clark Hill Dam.(8) The NWS computer model(7) is used to route the hydrograph to the VEGP site. The river cross-sections used are those given on drawing AX6DD320.

The hydrographs from subbasins 7, 8, 9, and 10 are routed and combined with the main hydrograph at proper location as lateral hydrographs and are based on PMP from Hydromet No. 51 in conjunction with Hydromet No. 52. The PMF discharge based on this routing is 710,000 ft^3/s at the VEGP site, and the maximum flood wave elevation at the site is 138 ft msl. The routed hydrograph is shown in figure 2.4.3-4.

In a previous study done for the Preliminary Safety Analysis Report (PSAR), the floods of 1929, 1940, and 1948 were routed using the unsteady, nonuniform flow TVA computer program Simulated Open Channel Hydraulics (SOCH)(9) and assumed Manning's n values to produce observed stages. The Manning coefficient values, which are shown on drawing AX6DD406, are for left over bank, main channel, and right over bank, respectively, and are assumed the same at all elevations at the same cross-section. The river cross-sections used in this routing are shown on drawing AX6DD406. These Manning values reproduced the flood's stages close to the observed ones and therefore were assumed to be adequate and representative for use in other flood routings.

2.4.3.5 <u>Water Level Determinations</u>

A stage discharge relationship is developed for the VEGP site (river mile 151.1 as shown in figure 2.4.3-3), by using a computer program obtained from the Corps of Engineers, designated HEC-2 Water Surface Profiles.(10) In this study 29 cross-sections are taken from United States Geological Survey (USGS) topographical maps. The locations of these sections are shown on drawing AX6DD407. Three of these sections at the VEGP site are field checked and compared with the USGS maps as shown on drawing AX6DD407. In order to obtain reasonable n values, the floods of 1929, 1940, and 1948 are run to compare high-water marks with computer results. Drawing AX6DD407 shows the n values used in the comparison. Using these values are the steady flow conditions for various discharges, and the results are shown on drawing AX6DD407. These data do not include wind wave setup or wave runup effects.

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

A wave height analysis is made in accordance with standard procedure. (See references 11, 12, 13, 14, and 15.)

A reasonable sustained wind speed for this site is considered to be 50 mph.(13)(16)(17) This is about 40 percent more than the recorded maximum wind speed for a 1-h duration at Greenville, South Carolina. The longest fetch is for a location in the middle of the river opposite the plant site with the wind blowing in an upstream direction. The fetch diagrams(11)(14)(15) used to determine this longest fetch are shown on drawing AX6DD321. The effective fetch of 9.5 miles is used in the wind wave estimate for all conditions, because the water surface configuration would be about the same for each condition considered.

To generate the maximum wave the 50-mph wind must be from a southerly direction and must exist for at least 78 min. The wave study results are given below.

On a three-to-one riprapped slope:

A.	Significant wave height		8.4 ft
B.	Significant wave runup		9.1 ft
C.	Wind setup		1.9 ft
		Total runup (B + C)	11.0 ft
D.	Maximum wave height		14.0 ft
E.	Maximum wave runup		12.8 ft
F.	Wind setup		1.9 ft
		Total runup (E + F)	14.7 ft

On a three-to-one smooth slope:

G.	Maximum wave height		14.0 ft
H.	Maximum wave runup		25.3 ft
I.	Wind setup		1.9 ft
		Total runup (H + I)	27.2 ft

These total runups are added to standard project flood (SPF) pool el 119.9 ft and to SPF plus dam failure surge el 141 ft to obtain the maximum elevation the water would reach shown earlier in this section. When the total runup is added to the PMF level of 138 ft msl, the water level may reach 165 ft msl. The safety-related equipment is well above the flooding stage.

2.4.3.7 References

1. Schreiner, Louis C., and Riedel, John T., "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometeorological Report No. 51, National Weather Service, NOAA, U.S. Department of Commerce Washington D.C., 1978

2. Hansen, E. M., and Schreiner, L. C., "Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometeorological Report No. 52 (preliminary), National Weather Service, Silver Springs, Maryland, 1979.

3. Fox, William E., Hydrologist in Charge, River Forecast Center, Atlanta, Georgia, Letter of Correspondence, January 2, 1980.

4. U.S. Army Corps of Engineers, HEC-1 Computer Program User's Manual, 1975.

5. U.S. Army Corps of Engineers, South Atlantic Division, Water Resources Development in South Carolina, Atlanta, Georgia, March 1975.

6. Chow, Ven Te, Handbook of Applied Hydrology, McGraw-Hill Book Company, New York, New York, pp 9-46 through 9-48, 1964.

7. National Weather Service, The New Dam Break Flood Forecasting Model System User's Manual, October 1980.

8. U.S. Army Corps of Engineers, Reservoir Regulation Manual, Savannah, Georgia, December 1962 (Revised January 1968).

9. Tennessee Valley Authority, Computer Code Simulated Open Channel Hydraulics (SOCH), Transient Flow Mathematical Model, January 1972.

10. U.S. Army Corps of Engineers, HEC-2 Water Surface Profiles, February 1972.

11. Saville, T., Jr., McClendon, E. W., and Cochran, A. L., "Freeboard Allowances for Water in Inland Reservoirs," J. Waterways and Harbors Division, American Society of Civil Engineers, May 1962.

12. Saville, T., Jr., McClendon, E. W., and Cochran, A. L., "Freeboard Allowances for Water in Inland Reservoirs," J. Waterways and Harbors Division, American Society of Civil Engineers, p 81, May 1963.

13. McCartney, B. L., "Wave Runup and Wind Setup on Reservoir Embankments," U.S. Army Corps of Engineers, ETL 1110-2-221, November 29, 1976.

14. U.S. Army Corps of Engineers, "Computation of Freeboard Allowances for Waves in Reservoirs," ETL 1110-2-8, December 16, 1966.

15. U.S. Army Corps of Engineers, Coastal Engineering Research Center, Shore Protection Manual, Vol 2, 3rd Edition, 1977.

16. U.S. Army Corps of Engineers, "Freeboard Criteria for Dams and Levees," Concepts for Surface Wind Analysis and Record Velocities, Technical Bulletin No. 1, March 1959.

17. U.S. Army Corps of Engineers, "Freeboard Criteria for Dams and Levees," Severe Windstorms of Record, Technical Bulletin No. 2, January 1960.

2.4.4 POTENTIAL DAM FAILURES

The major dams upstream of the VEGP site are designed such that seismic events will not cause failure of these structures. However, domino-type failure of the upstream dams is assumed to evaluate the effect of the flood wave at the plant site. All the safety-related structures and equipment are above the probable maximum flooding coincident wave height elevation of 165 ft mean sea level (msl). The guidance provided in Regulatory Guide 1.59 was considered in the evaluation of potential dam failures.

2.4.4.1 Dam Failure Permutations

There are 13 dams on the Savannah River and its tributaries above the VEGP site.(1) The Hartwell and Clark Hill Dams are major dams immediately upstream of the plant site. Between Clark Hill and Hartwell Dams another major dam, Richard B. Russell, is under construction and is scheduled for completion in 1984. The existing and proposed dams are shown in figure 2.4.4-1.

A profile of the river is shown in figure 2.4.4-2 along with a list of locations for various dams. Data on the dams are listed in table 2.4.1-3. Plan and sections of Clark Hill Dam, a large dam 71.6 miles above VEGP and the closest upstream dam, are shown in figure 2.4.4-3. Area-capacity curves for five large reservoirs above VEGP are shown in figure 2.4.4-4.

The seismic design criteria and spillway design criteria for Jocassee and Keowee Dams are given in the Preliminary Safety Analysis Report (PSAR) for Oconee project, subsections 2.4.2 and 2.4.3.(2) Design criteria for Corps of Engineers' dams are given in the project report for each dam.(2)(3)(4)

The VEGP site is considered a dry site. Even if Clark Hill Dam fails due to a seismic event coincident with probable maximum flooding, the flood wave dissipates substantially due to valley storage before it reaches the plant site. In that event the intake structure and the river makeup water facility may go out of service. But, since the river makeup water facility is a secondary source of makeup water, it will not affect the safety of the plant, as discussed in paragraphs 2.4.11.5 and 2.4.11.6.

A study of the locations of the dams above VEGP and the Savannah River stream profile, as shown in figures 2.4.4-1 and 2.4.4-2, respectively, to determine the basic assumptions for the worst possible dam failure to affect VEGP, indicates that the worst reasonable failure that can be postulated would be to have Jocassee Dam fail during a standard project flood (SPF) with an earthquake and the chain reaction that would follow. A seismic failure of Jocassee Dam is not reasonably possible, since it has been designed to include earthquake forces (design basis earthquake - 0.1 g) as stated in the PSAR for the Oconee project referred to in the previous paragraph.(2) However, it is more conservative to postulate this failure in determining the maximum water level at VEGP.

Jocassee Dam is a rockfill dam 400 ft high, and it would be unlikely that the dam would fail in its entirety. For this analysis it is assumed that a breach would cause a wave about 30 ft high below the dam. This breach could be a slide at least 50 ft deep across the entire top of the dam caused by an earthquake at the peak of SPF, or it could be a deep, narrow section that would go out triggering a breach. This wave would be 16 ft(5) high at Keowee Dam, 15 miles downstream. Keowee Dam, during its SPF, would have a freeboard of 15 ft, so that the surge wave would not be contained but would overtop and breach the earth dike section. This could cause a 15-ft surge wave at Hartwell Dam 51 miles downstream. Since Hartwell Dam would have a freeboard of 11 ft during its SPF, it would be overtopped by the surge wave, starting a breach in the earth dam section. Assuming Richard B. Russell Dam has been built, which could cause a worse wave at VEGP, it would be overtopped. It is anticipated that a 20-ft surge wave could reach Richard B. Russell Dam from a Hartwell Dam failure and overtop the dam by about 9 ft during its SPF. From this failure a 15-ft surge wave would reach Clark Hill Dam, overtopping it by 6 ft during its SPF.

At this instant the spillway discharge at Clark Hill Dam would increase suddenly. The additional water pressure against the concrete structures would increase the horizontal load by 140 klb/linear ft and move the resultant about 10 ft. Thus, the concrete structures would remain stable. The 6-ft flow over the earth sections would start to erode them. The breaching of these sections would take some time; however, to be conservative, we assume their instantaneous removal. Then the flow elevation in the breach sections would be about el 290 ft based on the standard equations,(5) and the discharge through the combined east and west breaches would be about 1,600,000 ft^3/s. At this time the flow through the spillway would cease, because the reservoir level against the dam would be el 290 ft and below the spillway crest of el 300 ft. If the discharge through the breached dam is based on the submerged broad crested weir equations, the flow is estimated to be 2,400,000 ft^3/s.

Tailwater will be controlled by a river section 2.3 miles downstream of Clark Hill Dam. The Reservoir Regulation Manual, (Savannah River Basin, December 1962, Revised January 1968) shows a tailwater elevation of 255 ft msl for a spillway design flood of 1,055,000 ft^3/s (figure 2.4.4-5). Using the above discharge and tailwater elevation values, a channel n value of 0.065 is computed,(6) which is reasonable but low. For this value of n, the required elevation at the section to pass 2,400,000 ft^3/s is found to be el 292 ft msl.(6)(7) This value is higher than the stage at the breach and indicates that flow through the breach will be reduced by the tailwater effects. However, to be conservative, the flow of 2,400,000 ft^3/s is transferred to below Augusta undiminished in magnitude. At this point, the rise of 1,800,000 ft^3/s in discharge above the SPF is assumed to occur in 3 h and the peak of 2,400,000 ft^3/s lasts for 4 h. It should be noted that the SPF at Clark Hill is computed(3) by taking one-half of the ordinates of probable maximum flooding. The routing of the dam failure surge from below Augusta is described in paragraph 2.4.4.2.

It should be noted that stages at VEGP are not very sensitive to either peak flow magnitude or peak flow duration, probably due to the great width of flood plain in the vicinity of VEGP. Some initial studies were made, which show the relative insensitivity of VEGP stage to variation in peak flow and duration. At 2,400,000 ft^3/s an increase of peak duration from 4 h to 8 h increased the peak flow at VEGP about 30,000 ft^3/s. Increasing the flow from 2,400,000 ft^3/s to 3,400,000 ft^3/s, both at 4-h duration, increased the flow at VEGP about 100,000 ft^3/s. The corresponding steady flow stage changes at VEGP are about 0.5 ft and 2 ft, respectively.

The following is a description of conditions if an operating basis earthquake (OBE) is assumed during an SPF, which could affect both Clark Hill and Richard B. Russell Dams simultaneously. Richard B. Russell Dam is designed for a seismic factor of 0.1 g.(4) Since this closely corresponds to the OBE, the dam should not fail. At Clark Hill Dam the concrete structures were conservatively designed.(3) A review of the most critical section during maximum pool elevation of 346 ft with an OBE of 0.12 g shows the resultant falls within the base with a toe stress of 207 psi, which is within the allowable limit (figure 2.4.4-6). It is assumed that failure of the earth dike section will occur because of insufficient data for analysis. This is not the critical condition because the maximum water surface in the reservoir is 11 ft lower and the resulting flood wave is some 6 ft lower than that described above.

If a simultaneous earthquake at Hartwell and Clark Hill Dams is assumed, it is likely that the resulting conditions are less critical than those described initially in this section. If Hartwell Dam fails during an OBE, the resulting wave would be several feet less than that assumed from upstream domino-type failure. When this wave arrives at Clark Hill where the earth section had failed at the time of the OBE, the stage would be significantly lower than that assumed initially in this section.

The failure of War Woman Dam, shown in figure 2.4.4-2, has been considered, and it is found that the surge wave diminishes as it progresses downstream and the volume in War Woman Reservoir is contained in Hartwell Reservoir. Since the assumptions made for the failure of Clark Hill Dam and its routing to VEGP are conservative, no other mode of failure is assumed for determining effect at VEGP.

Since the flood plains at and below VEGP are wide, there is no likelihood that a landslide will occur to raise the flood level postulated. The effect of PMF concurrent with waves and the maximum stage reached is discussed in paragraph 2.4.3.6.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

The hydrograph for the failure of Clark Hill Dam, resulting from the failure of the dams further upstream as described in paragraph 2.4.4.1, is considered to be at New Savannah Bluff Lock and Dam, which is at the south end of Augusta, Georgia, and is routed to Burton's Ferry Bridge using the unsteady, nonuniform flow computer program SOCH(7) and roughness values stated in paragraph 2.4.3.4.

To determine the effect of the dam failure, the hydrograph for the SPF is routed past VEGP using the same computer program referred to above. The SPF hydrograph is computed by taking one-half of the ordinates of the Corps of Engineers' spillway design flood hydrograph (PMF hydrograph). The results at the plant site are shown in figure 2.4.4-7. The peak stage is el 119.9 ft msl. Then the dam failure hydrograph, as stated above, is routed using the same procedure. The results are shown in figure 2.4.4-8. This gives the peak stage an elevation of 141 ft msl and a discharge of 980,000 ft^3/s. Wind wave and runup effects are not shown, but the resultant water elevation discussed in paragraph 2.4.4.3 considers these phenomena.

2.4.4.3 Water Level at Plant Site

A wind velocity of 50 mph, as stated in paragraph 2.4.3.6, is also considered for this condition. The runup at the river pump station under the above conditions for the significant wave coincident with the dam failure surge wave by VEGP, as stated above, would reach el 152 ft msl. The wave runup would reach el 168 ft msl on the natural slope (3 to 1) along the plant side of the river.

2.4.4.4 References

1. U.S. Study Commission, Southeast River Basin, Plan for Development of the Land and Water Resources of the Southeast, Appendix 1, 1963.

2. Duke Power Company, Oconee Nuclear Plant Preliminary Safety Analysis Report, subsections 2.4.2 and 2.4.3.

3. U.S. Army Corps of Engineers, Reservoir Regulation Manual, Savannah River Basin, Savannah River, Clark Hill Lake.

4. U.S. Army Corps of Engineers, Design Memorandum II, Construction Procedure Diversion Plan, Hydraulic Data, Richard B. Russell Project, 1969.

5. Rouse, Hunter, ed, Engineering Hydraulics, John Wiley & Sons, Inc., New York, New York, 1950.

6. Chow, Ven Te, Open Channel Hydraulics, McGraw-Hill Book Company, Inc., New York, New York, 1959.

7. Tennessee Valley Authority, Computer Code Simulated Open Channel Hydraulics (SOCH), Transient Flow Mathematical Model, January 1972.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

Since the VEGP is not near a large body of water, this section does not apply.

2.4.6 PROBABLE MAXIMUM TSUNAMI FLOODING

Since VEGP is neither near a large body of water nor in a volcanically active seismic region, this section does not apply.

2.4.7 ICE EFFECTS

Icing does not occur on the lower reaches of the Savannah River. Based on records of minimum temperature as shown in table 2.4.7-1, the temperature is higher than 5°C most of the time.[1]

With Hartwell, Richard B. Russell, and Clark Hill Dams upstream of Augusta the ice jam is unlikely to occur at the VEGP river intake, since these upstream dams will modulate the water temperature.

If surface icing did occur at VEGP, the design of the river intake and the normal water depth of 14 ft ensure that an ice sheet across the entire river will not interfere with flow of water into the intake.

2.4.7.1 REFERENCE

1. Department of the Interior, U.S. Geological Survey, Atlanta, statistical data on river flow, temperature, and chemical analyses for the Savannah River (data retrieved from USGS computer on July 8, 1981, for the Hydro Projects Department, Southern Company Services).

2.4.8 COOLING WATER CANALS AND RESERVOIRS

2.4.8.1 Canals

There is a short canal less than 400 ft long formed by vertical sheet piling walls connecting the river intake structure and the Savannah River. As discussed in paragraph 2.4.11.5, the river makeup water is the secondary backup source for the nuclear service cooling water tower basins. The river intake structure as well as the intake canal have no safety function. Thus, failure of this river intake canal will not cause a threat to the safety of the plant operations (subsection 2.4.9).

2.4.8.2 Reservoirs

There are no reservoirs at VEGP.

2.4.9 CHANNEL DIVERSIONS

The river upstream from the site has bluffs and steep slopes along the west bank. If it is assumed that a bluff slid into the riverbed just upstream from the pump station and blocked the river flow for a few hours and if it occurred during low flow, the water would pile up above the obstruction and then divert over the flood plain. If this postulated event occurs, the plant can be safely shut down due to available storag3e onsite of adequate quantities of nuclear service cooling water (NSCW). The water supplies for the NSCW system and the mechanisms by which primary and secondary supplies are activated are discussed in subsection 9.2.5.

The Savannah River near the VEGP site has a relatively straight and stable channel, and also in this vicinity the river does not have an oxbow forming. It is very unlikely that the river will be diverted from the intake by natural causes. Any possible effect on water supply to the intake from channel changes should come from extremely slow changes, which can be remedied as they occur.

2.4.10 FLOOD PROTECTION REQUIREMENTS

None of the safety-related facilities at the VEGP is susceptible to flooding by the most severe flood at the site. One flood source is the river, and highest postulated flood level due to an upstream dam break is el 141 ft (paragraph 2.4.4.2). Waves resulting from such a flood and wind are postulated to reach maximum elevation of 168 ft, and since the plant structures are at finished grade elevation of 220 ft, flooding is not probable from this source.

Another source of flooding is severe rainfall (paragraph 2.4.3.1). The power block area for Units 1 and 2 is on a high plateau and is not in the path of any adjacent watershed. The topography is such that the runoff is directed away from the power block by a combined system of culverts and open ditches to natural drainage channels (drawings CX2D46V003, CX2D46V004, CX2D46V005, and CX2D46V006). The plant drainage system is designed for a maximum precipitation of 4 in./h and 4.8 in. of rain in 2 h (paragraph 2.4.2.3); the system has been checked to make sure that flooding of safety-related equipment does not occur as a result of the probable maximum precipitation (PMP).

The roofs of all safety-related structures are designed to pass the local PMP, corresponding to time of concentration of flow. The design includes measures to guard against wind-induced seepage through roof and wall penetrations or doors where safety-related equipment could be damaged. The design is based on PMP having an intensity of 15 in. of rainfall in 1 h and 22 in. of rainfall in 2 h (paragraph 2.4.2.3).

2.4.11 COOLING WATER SUPPLY

2.4.11.1 Low Flow in Streams

The low flow in the Savannah River by the VEGP site is regulated by Clark Hill Dam and the New Savannah Bluff Lock and Dam. A minimum flow of 5800 ft^3/s is required for navigation in the Savannah River downstream of Clark Hill Dam.(1) However, it should be noted that a discharge of 6300 ft^3/s is normally provided 80 percent of the time.(1) As shown in figure 2.4.11-1, a minimum required flow of 5800 ft^3/s is released from New Savannah Bluff Lock and Dam. The Clark Hill Dam project is designed for a maximum drawdown of 18 ft from the top of the power pool elevation of 330 ft msl to a minimum pool at 312 ft msl. However, it is not anticipated that the minimum pool will be reached more often than once in 150 years.(1) The Clark Hill Dam project also ensures a required minimum river flow at the Savannah River Plant of the former Atomic Energy Commission and helps to regulate the water temperature.(2)

A low flow stage at the VEGP site corresponding to minimum river flow of 5800 ft^3/s is el 80.4 ft msl.(3)

A low flow rating curve as given in figure 2.4.11-2 and an intermediate flow rating curve as given in figure 2.4.11-3 were developed at river mile 151.1 for the pumped intake structure. These curves are based on backwater profiles computed from actual river cross-sections near the site and from assumed sections based on river bottom profiles. The location of surveyed sections is shown in figure 2.4.11-4.

Echo soundings and probings were also made in this section of the river, and the results are shown in figure 2.4.11-5. For a distance of approximately 2 miles downstream, much of the river bottom is at el 70 ft. Studies have led to the conclusion that degradation of river, if any, will be slow and small. If a conservative allowance of a 2-ft decrease in stage were made for degradation, a dashed curve in figure 2.4.11-2 would result.

Flow records for Augusta, Georgia, for the periods 1884 through 1906 and 1926 through 1970 are examined. A hypothetical extreme drought flow of 957 ft^3/s is determined by a statistical analysis of the period 1926 through 1950 using a method developed by Gumbel.(4)(5) This period is used because it was a period during which no major dams were built on the river or its tributaries upstream of Augusta. It is then concluded that the hypothetical extreme drought would have a stage elevation of 74 ft msl.

From the flow records for the 62 years of examined data from the United States Geological Survey (USGS), it is concluded that a sustained minimum release of 5800 ft^3/s (the planned operation of Hartwell and Clark Hill Reservoirs) could have been maintained for this period. A flow of 500 ft^3/s is required under present conditions to provide water at el 73 ft at the pump intake. A flow of 1500 ft^3/s is required using the allowance for channel degradation. It is concluded that the regulated river flow is a highly dependable source of water for any makeup operation.

2.4.11.2 Low Water Resulting from Surges

This situation does not apply because VEGP does not withdraw water from a large body of water, nor is it located in a volcanic region.

2.4.11.3 Historical Low Water

The available flow records for Augusta, Georgia, for 62 years for the periods 1884 through 1906 and 1926 through 1970, as shown in Flow Characteristics of Georgia Streams,[6] are examined. The low flow of record for gauging station No. 2-1970, Savannah River at Augusta, Georgia, at New Savannah Bluff Lock and Dam (river mile 189.8) prior to construction of Clark Hill Dam, occurred on September 24, 1939. This was caused by the operation of gates at New Savannah River Lock and Dam. If the rating curve is extended below 1400 ft^3/s, an extreme minimum discharge of 648 ft^3/s is reached. This is an extrapolated instantaneous minimum. Water stage recorder graphs and discharge measurements were furnished by the Corps of Engineers. On the day this low flow was recorded, the average daily flow was 2940 ft^3/s. A copy of the recorded graph was obtained from the USGS, and figure 2.4.11-6 was prepared to show this hydrograph, which indicates that the lowest flow occurred for about 10 h, the daily flow being over 2000 ft^3/s. The lowest mean daily flow shown in the Augusta record was 1040 ft^3/s, which occurred on October 2, 1927.[6]

The minimum mean daily discharge for the period 1963 through 1970 (subsequent to the filling of both reservoirs) was 5130 ft^3/s in 1963. The storage for power and navigation releases (between normal and minimum pool levels) from Hartwell and Clark Hill Reservoirs is 2,445,000 acre-ft, which would provide an average release of 3350 ft^3/s for 1 year assuming no inflow.[1] The total storage (between top of gates and minimum pool level) from both reservoirs is 3,128,000 acre-ft, which would provide an average release of 4300 ft^3/s for 1 year assuming no inflow.[1]

2.4.11.4 Future Control

Since the minimum flow condition is controlled mainly by upstream dam releases and no large future users of water are known, the future minimum should be similar to past minimum.

2.4.11.5 Plant Requirements

The heat removal requirement is greater for Unit 2 than for Unit 1 due to the larger spent fuel storage capacity of the Unit 2 pool. This results in some differences in the system performance and available inventory for each unit. These differences, where they are significant, are outlined in the following paragraphs. When a single value is presented, it represents the upper bound for both units.

Evaporation losses from the safety-related nuclear service cooling water system (NSCWS) tower basins are presented in subsection 9.2.5. During normal plant operation, a flow of 435-gal/min makeup water is required to replace these evaporative losses and also losses due to drift, seepage, and cooling tower blowdown. During accident conditions, maximum evaporative losses are 1490 gal/min (for 2-train NSCWS operation) immediately after reactor shutdown, decreasing to 340 gal/min after approximately 1 day and 165 gal/min after 16 days. Tower blowdown is isolated during accident conditions, and therefore, total losses and makeup requirements during accident conditions comprise only evaporation, a seepage loss of 1 to 2 gal/min, and drift. However, as discussed in subsection 9.2.5, tower makeup is not required during the period subsequent to reactor shutdown under accident conditions concurrent with a loss of offsite power. The combined storage capacity of the two NSCWS cooling tower basins provided for each generating unit is sufficient to provide 30-day inventory for Unit 1 and 30-day inventory for Unit 2 to permit safe reactor shutdown and maintenance of the plant in a safe shutdown condition in conformance with the short-term recommendations of Regulatory Guide 1.27. Total NSCWS water required and available storage capacity of the NSCWS cooling tower basins are discussed in subsection 9.2.5.

As discussed in subsections 9.2.1 and 9.2.5, the NSCWS pumps take suction from the NSCWS cooling tower basins and circulate cooling water through the component cooling water heat exchangers, the auxiliary component cooling water heat exchangers, the containment coolers, and miscellaneous pump coolers. To ensure that the inventory of both NSCW basins is available in the event that only one NSCWS train is available, one safety-grade transfer pump is installed in each NSCWS cooling tower basin, with the train A transfer pump installed in the train B NSCWS tower basin and vice versa. The rated capacity of the transfer pumps is 600 gal/min, which is well in excess of the NSCWS makeup requirements during accident conditions, except during the first few hours of the transient, thus ensuring adequate makeup to the operating tower.

To ensure adequate NSCWS pump and transfer pump suction conditions at all times, all pumps are installed in pits, which extend 7 ft below the floor of the tower basin at el 137 ft.

The suction of the NSCW transfer pumps is located at elevation 132 ft 6 in., and the pumps are designed to operate with a minimum water level at elevation 141 ft 3 in. to provide adequate NPSH.

The suction of the NSCW transfer pumps is located at elevation 131 ft 4 in., and the pumps are designed to operate with a minimum water level at elevation 137 ft or as long as there is water in the tower basin.

Thus, the entire basin inventory of both NSCWS cooling towers, except for the last 4 ft 3 in., is available to satisfy the short term recommendations of Regulatory Guide 1.27 using only safety-related components.

To provide NSCWS makeup during normal operation and to provide long-term (after 30 days) cooling in conformance with Regulatory Guide 1.27, water from nonsafety-related systems (deep water wells and the Savannah River) is available. Two makeup wells provide the primary makeup to the NSCWS cooling towers for both generating units. Wells MU-1 and MU-2A are equipped with a 2000 gal/min and 1000 gal/min pump, respectively. The wells are approximately 2100 ft apart, and each is located approximately 1000 ft and 400 ft, respectively, from the immediate plant site boundary. These wells provide water for the following uses:

- Fire protection.
- Construction demands.
- Potable and sanitary uses.
- Makeup to the well water storage tank.

Water in the well water storage tank is utilized for utility purposes, for NSCW tower makeup, and for the makeup demineralizer. Plant operating procedures will be utilized by plant personnel to ensure proper operation of these water makeup systems.

The makeup wells have been sited on the basis of extensive investigations on the characteristics of the groundwater at plant site, which show the existence of two distinct aquifers, a shallow water aquifer and a deep, confined Tuscaloosa aquifer.

Estimated recoverable water quantity in the Tuscaloosa aquifer is approximately 21 billion acre-ft, thus providing a safe yield of 5 billion gal/day.

In the event that the well water is not available, river water can also be utilized for NSCW cooling towers makeup. The Savannah River provides the makeup water for the natural draft cooling towers and dilution water for the plant effluent discharge. The Savannah River, bordering the plant site on the east side, is approximately 340 ft wide and 13 ft deep at the site, with minimum and maximum flows of 5800 ft^3/s and 71,700 ft^3/s, respectively. The river temperature, recorded at the Burton's Ferry Bridge from January 1960 through September 1970, ranged from a minimum of 41°F (5°C) to a maximum of 84°F (28.9°C).

Four 22,000-gal/min capacity nonsafety-related river makeup water pumps provide makeup to the natural draft cooling towers. Provision is also made for these pumps to supply additional dilution water as required for the plant radwaste liquid discharge to the Savannah River. Piping is provided from the supply header of the river makeup water pumps to the basins of the NSCW cooling towers as a secondary source of makeup water. The plant makeup-water well system serves as the primary makeup source for the NSCW cooling towers.

To ensure adequate river makeup water pump suction conditions, the pump suction is installed 4 ft 10 in. below the minimum low river water surface elevation of 73 ft. The river intake pumps are designed to operate with a minimum submergence of 4 ft 6 in.

2.4.11.6 Heat Sink Dependability Requirements

As discussed in paragraph 2.4.11.5 and subsection 9.2.5, the ultimate heat sink for VEGP is provided by the water inventory in the NSCW cooling tower basins. As part of the power uprate, an analysis was performed to calculate the basin inventory, basin temperature, and the time-dependent evaporative loss. The combined volume of water in the two cooling tower basins is sufficient to provide greater than a 30-day cooling capacity for Unit 2 with two trains operating the first day and one train operating for the remaining days. The tower basin capacity for Unit 1 was not re-evaluated for the power uprate. The calculation was performed for Unit 2 since the higher heat loads from the reracked spent fuel make it the more conservative choice; therefore, the results will be bounding for Unit 1. As discussed in subsection 9.2.5, tower basin capacity and maximum evaporation losses are based upon the worst combination of recorded dry and wet bulb temperatures, which results in the greatest water loss over the postulated 30-day period following an accident. The total losses include tower drift and seepage losses but do not consider cooling tower blowdown, since blowdown is isolated during accident conditions.

Seepage into or out of the NSCW tower basins is minimized by the thick shell walls (3 ft thick above el 155 ft 5 1/2 in. and 5 ft thick below) and basemats (9 ft thick). In addition, level alarms are provided for each NSCW tower which will indicate when the water level in a tower basin drops below el 217 ft 3 in. (normal water level during plant operation is elevation 217 ft 9 in.). An automatic makeup valve will operate to maintain basin water level above el 217 ft 6 in. When a level alarm is activated, additional makeup water may be added and the basins will be monitored for any abnormal losses.

During normal operation, level instrumentation in the basins and pressure and flow instrumentation in the NSCWS piping give the operators a constant readout of system status with alarms as required to warn of abnormal system operation. As described in paragraph 2.4.11.5, there are two sources of makeup water: deep water wells and river water. During accident conditions concurrent with loss of offsite power, the safety-grade transfer pumps described in paragraph 2.4.11.5 can be manually operated to ensure full use of the basin inventory.

The safety-related portions of the NSCWS are designed and constructed as Nuclear Safety Class 3, Seismic Category 1, and in accordance with the ASME Boiler and Pressure Vessel Code, Section III, Class 3. Therefore, the system will remain functional during and after the safe shutdown earthquake and after any single failure. Thus, the system provides a dependable heat sink for the plant. Subsection 9.2.5 provides additional detail regarding the design of the ultimate heat sink.

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2.4.12 GROUND WATER

2.4.12.1 Description and Onsite Use

2.4.12.1.1 Regional Aquifers, Formations, Sources, and Sinks

There are two major regional aquifer systems recognized in the coastal plains of Georgia and South Carolina. The lower regimen is referred to as the Cretaceous aquifer system and consists primarily of the sands, gravels, and clays of the Tuscaloosa Formation. The upper regimen is variously referred to as the Tertiary aquifer system, the principal artesian aquifer, and the limestone aquifer. It consists primarily of the limestones and permeable sands of the Lisbon Formation or stratigraphic equivalents. In their outcrop areas, both systems are hydraulically connected and are under water table conditions. Both systems are under artesian conditions elsewhere throughout most of the region. The relatively impermeable clays and silts of the Huber and Ellenton Formations (Paleocene) overlie and confine the Cretaceous system, while the clays and clayey sands of the Barnwell Group (late Eocene), and other younger fine-grained sediments, overlie and confine the Tertiary system.

In addition to the major artesian aquifer systems, ground water also exists under water table conditions in the Late Eocene and younger formations, alluvial deposits, and Terrace deposits that are present throughout the region. Ground water is also present in the crystalline basement complex and the overlying Triassic Newark Group. In the paragraphs that follow, each of the regional systems is more fully described.

2.4.12.1.1.1 Pre-Cretaceous Formations. Pre-Cretaceous stratigraphy in the study region consists of Precambrian through Paleozoic igneous and metamorphic basement rocks overlain by Triassic fine- to coarse-grained clastic sediments and conglomerates. The rocks in the basement complex are dense and essentially impermeable, but ground water exists in secondary openings such as joints and fissures. Injection, pumping, and packer testing were performed in the crystalline basement in the early 1960s as part of an exploratory program to determine the feasibility of storing radioactive wastes in these rocks. This program was performed at the Savannah River Plant near Aiken, South Carolina, and indicated an average permeability of 0.0003 gal/d/ft^2 and an overall transmissivity of 160 gal/d/ft.[1]

These relative low values and correspondingly low yields to wells during pumping tests indicate that the crystalline basement complex is not a significant hydraulic unit, at least in the area tested. Hydraulic characteristics of the basement rocks in other areas are not as well known, but, based on lithologic descriptions, there are little data to indicate significantly different hydrogeologic properties elsewhere in the region.

Triassic rocks of the Newark Group overlie the basement complex, but their total areal extent has not been defined.[2] Where they are known to exist, the Triassic rocks consist of a variety of sedimentary grain sizes in various degrees of induration. Rocks of this lithologic character could conceivably include local permeable zones, but the highly productive and more shallow Cretaceous and Tertiary aquifer systems, which overlie the Triassic rocks, are available for use throughout the region, and, consequently, there has been little impetus to explore the deeper horizons for ground water potential. The very few wells that have been opened to the Triassic zones have yielded only meager quantities of ground water and this has also done little to encourage exploration. Until additional data are produced, the overall hydrogeologic character of the Triassic rocks will be obscure. Based on the sparse data presently available, the Triassic sequence does not appear to be a significant regional hydraulic unit, but it may have localized productive zones.

2.4.12.1.1.2 Cretaceous Aquifer System. Within the region, the Cretaceous ground water system is represented by the Tuscaloosa Formation, which consists of fluvial and estuarine deposits of cross-bedded quartzitic sand and gravel interbedded with silt, and clay. The coarse-grained sediments are mostly unconsolidated and are generally permeable, while the fine-grained sediments are partially consolidated and are generally permeable. In some areas, the fine-grained layers effectively confine the ground water to produce local artesian conditions that may differ from the overall system. In addition to the varying lithology, the formation also exhibits lateral facies changes, on-lap and off-lap relationships, and discontinuous lenses. The combined effects of these conditions have given rise to a complex multiaquifer system whose overall hydrogeologic character has not yet been fully defined.

The Cretaceous ground water system has not been extensively developed, primarily because the shallow Tertiary system is adequate for most ground water needs and is available for use throughout the region. Quantitative data from the limited number of test and production wells producing from the Cretaceous strata, and inferred data from geologic and stratigraphic studies, indicate clearly that the Cretaceous system is a highly transmissive regimen that is capable of yielding large quantities of good quality ground water. Callahan[12] estimates that the Cretaceous system has a safe yield of 5 billion gal/d throughout its known extent.

Recharge to the Cretaceous system is primarily by direct infiltration of rainfall in its outcrop areas, located north of VEGP in a 10- to 30-mile-wide belt extending from Augusta, Georgia, northeastward across South Carolina to the state line separating North and South Carolina. In the outcrop areas, precipitation penetrates the Cretaceous sediments and migrates downdip under a hydraulic gradient of 6 to 20 ft/mi.[2] Ground water in the outcrop areas is under water table conditions, but as it moves progressively downdip, it becomes confined beneath the Blue Bluff marl member of the Lisbon Formation in the vicinity of VEGP and beneath the Ellenton and Huber Formations farther downdip to the south and southeast. Hence, the Cretaceous system is under artesian conditions for most of its areal extent.

Discharge of the Cretaceous system ground water is primarily from subaqueous exposures of the aquifer that are presumed to occur along the Continental Shelf. Other discharge sources are to the Savannah River as described in section 2.14.12.1.2.2 and by pumping. Some discharge may also occur by upward leaking through the confining layers. This condition is inferred from the known variable permeability of the Ellenton and Huber Formations, but an area where such discharge is actually occurring has not been identified. Thickness of the Cretaceous system ranges from zero at the fall line to approximately 700 ft at the VEGP and 800 ft in east-central Georgia.

The quality of the ground water in the Cretaceous system has been satisfactory to good in the somewhat limited areas in which tests have been conducted. Such areas are mostly within the outcrop regions of the aquifer where it is shallow enough to develop economically. Several municipalities use water from the Cretaceous system and tests conducted by the United States Geological Survey (USGS) have shown that, except for marginally high concentrations of iron and nitrate, water from the Cretaceous aquifer meets all requirements of the U.S. Public Health Department.[2]

2.4.12.1.1.3 Tertiary Aquifer System. Underlying the southeastern United States is a highly productive aquifer system of Tertiary age that is referred to as the principal artesian aquifer in Georgia, Alabama, and South Carolina and as the Floridan aquifer in Florida[9]. It is the primary source of municipal, industrial and agricultural water supply on the coastal plain of Georgia. Within Georgia and South Carolina, the aquifer system includes the carbonate rocks (and associated interbedded sands) of Eocene and Oligocene age. Within a single stratigraphic time horizon, facies changes downdip result in a confining layer becoming an aquifer. For example, beneath VEGP the Tertiary aquifer is comprised only of the unnamed sands of the Lisbon Formation and the overlying Blue Bluff marl acts as a confining layer. However, downdip, the Blue Bluff interfingers with a permeable limestone that becomes part of the Tertiary aquifer system, and beds younger than the Lisbon Formation form the confining layer.

Recharge to the Tertiary aquifer is primarily by infiltration of rainfall in its outcrop area, which is a belt 20 to 60 miles wide extending northeastward across central Georgia and into portions of Alabama to the west and South Carolina to the east. The strata, which make up the aquifer, dip southeastward beneath younger confining sediments. They include nearly impermeable clays and semipermeable fine-grained sediments of Oligocene and younger formations.

Discharge from the Tertiary aquifer occurs from pumping, from natural springs in areas where topography is lower than the piezometric level of the aquifer, and from subaqueous outcrops that are presumed to occur offshore. Discharge also occurs to the Savannah River where the river has incised down through the marl confining layer allowing discharge from the aquifer to the riverbed, as described in section 2.4.12.1.2.2.

The regional direction of flow of ground water in the Tertiary aquifer is south by southeast. Local deviations from the general flow direction occur in response to structural features, cones of depression caused by pumping, and by the effects of the discharge to the Savannah River.

Transmissivities of up to 220,000 ft^2/day have been measured for the Tertiary aquifer in southern Georgia.[4] The overall transmissivity of the aquifer has not been fully defined, but known variations in both vertical and horizontal permeabilities suggest that the average transmissivity in Georgia is substantially less than 220,000 ft^2/day.

Thickness of the principal Tertiary aquifer ranges from near zero at its northwestern limit area to approximately 900 ft in southern Georgia, where it is represented by the Claiborne Group. In Florida the aquifer thickens to several thousand feet.

Generally, the water quality of the Tertiary aquifer is good, although high hydrogen sulfide content and other objectionable constituents have been reported in some areas.[4]

2.4.12.1.1.4 Water Table Aquifer. Ground water is present under water table conditions in the late Eocene and younger sediments throughout the region. These include the sands and carbonate rocks of the Barnwell Group (late Eocene), the clayey sand and gravel of the Hawthorne Formation (Miocene), and extensive Pleistocene to Holocene alluvial and terrace deposits. These sedimentary units are characterized by a wide spectrum of vertical and horizontal permeabilities, which have resulted in hydrogeologic conditions that are not favorable for significant development. Also, the water-bearing materials have been incised to various depths by the many streams throughout the region and are, therefore, not laterally continuous over large areas. Most wells open to the water table aquifer produce quantities that are adequate only for domestic and livestock use. A brief description of each stratigraphic unit and its water-bearing characteristics is given in table 2.4.12-1 and in the paragraphs that follow.

The Barnwell Group (Eocene) consists of fine- to coarse-grained clayey sand and sandy clay with subordinate lenses of limestone and sandstone. These deposits are semiconsolidated to consolidated and are present throughout the region. Limestone lenses in the Barnwell Group are prevalant in Georgia and occur as interbeds with the clayey sands that dominate the formation. The sands of the Barnwell Formation are relatively fine-grained, and the presence of silt and clay lenses reduces the overall permeability of the formation. Ground water is usually available only in quantities suitable for domestic or livestock use.

The Hawthorne Formation (Miocene) consists of tan, red, and purple, dense, sandy clay with interbedded lenses of gravel and numerous clastic dikes. Exposure of the Hawthorne Formation is extensive in the East Gulf Coastal Plain province. The fine-grained nature of the formation and clastic dike materials (silty to sandy clay) precludes significant ground water development. Ground water is locally available in more permeable lenses but only as small bodies of perched water.

Alluvial deposits ranging in age from Pliocene to Holocene, are present throughout the region. The Pliocene deposits occur principally as cappings on interstream divides, while the Pleistocene and Holocene deposits occur mainly in the tributary stream channels and flood plain of the Savannah River. All of the alluvial deposits consist of poorly sorted clay, sand, and gravel. Thickness rarely exceeds 30 ft. Areal extent is highly variable due to stream downcutting and erosional processes. In general, these deposits contain only small quantities of ground water.

Terrace deposits of Pleistocene age are also present in the Coastal Plain province. Cooke recognized seven such structures in South Carolina and postulated that they are of marine origin.(5) The deposits consist of sand, silty clay, and gravel; their thickness is not more than a few tens of feet. The water-bearing characteristics of the terrace deposits are not well known due to lack of development. Their principal hydraulic function is probably limited to acting as a recharge conduit to the lower aquifers.(2)

2.4.12.1.2 Local Aquifers, Formations, Sources, and Sinks

The initial geologic exploration program for VEGP included ground water field studies consisting of aquifer and permeability testing, well canvassing, chemical analyses, and observation well installation and monitoring. Additional hydrogeologic data were also acquired during the Millett fault study of 1982.(3) These data are the basis for establishing ground water conditions of the site area (figure 2.4.12-1)(3). In general, those conditions conform to the regional aquifer systems described in paragraph 2.4.12.1.1. The local characteristics are explained under the appropriate headings in the paragraphs that follow.

2.4.12.1.2.1 Pre-Cretaceous Formations. None of the exploratory borings for the VEGP penetrated the crystalline basement rocks, but an exploration program conducted in South Carolina to determine the feasibility of storing radioactive wastes in these rocks produced a considerable amount of hydrogeologic data. This program was conducted in the early 1960s by the USGS at the Savannah River Plant a few miles east of VEGP.[1] The data indicate that a nominal amount of ground water exists in the crystalline basement complex in secondary openings, i.e., joints and fissures. Although the igneous and metamorphic rocks making up the basement complex are essentially impermeable, those zones in which an adequate number of interlacing fractures exist behave as a hydraulic unit and have transmissivities of up to 165 gal/d/ft. Other tests, including measurement of recovery after swabbing and injection permeability tests confirmed the existence of water-bearing zones in the basement rocks. However, the test program indicated that these permeable zones are limited in areal extent. The significance of these findings with regard to the VEGP is that the basement complex does not transmit ground water over a large area and that basement rocks are not a potential source for ground water within the study area.

The Triassic sequence of sedimentary rocks which overlie the crystalline basement were not penetrated by any of the exploratory borings associated with VEGP investigations; hence, no hydrogeologic data are available for this zone. However, the presence of Triassic rocks in the study area have been demonstrated during investigations relating to the Savannah River Plant. Based on the limited amount of data available, the Triassic rocks do not appear capable of transmitting ground water over a large area nor of providing a large volume of water to wells. It is therefore not considered a significant hydrogeologic unit.

All of the units above the Triassic sequence have been penetrated by wells constructed at the VEGP site. The initial test well, TW-1, provided data with which a well field was designed (paragraph 2.4.12.1.3.3). The log of that well, shown on drawing AX6DD331, illustrates the sequence and thickness of the important hydrogeologic units beneath the main plant area and provides a reference for the following discussions of those units.

2.4.12.1.2.2 Cretaceous and Tertiary Aquifers. The Cretaceous ground water system is represented in the site area by the Tuscaloosa Formation. This unit is approximately 700 ft thick.

It consists primarily of cross-bedded sands and gravels with subordinate beds of silt and clay. It is a highly transmissive aquifer system (reference 3 and drawing AX6DD331). At the VEGP site, the Tertiary aquifer is represented by the unnamed sands in the lower part of the Lisbon Formation (drawing AX6DD331). It consists of fluvial sands and sandy clays for which formal stratigraphic nomenclature has not yet been established. These sediments are moderately permeable, as shown by field permeability tests and by the operation of the VEGP construction water supply wells. Total thickness of the Tertiary aquifer at the site is approximately 100 ft.

Recharge to the Cretaceous aquifer is primarily from infiltration of rainfall where the formation is exposed northwest of VEGP. In the same general area, the Tertiary aquifer is also exposed and off-laps the Cretaceous aquifer. In this outcrop area, the Cretaceous and Tertiary systems are in hydraulic contact and the ground water is under water table conditions. After the water infiltrates the sediments, it migrates downdip in a south by southeast direction.

Within a few miles downgradiant of the recharge/outcrop area, ground water of the Cretaceous and Tertiary aquifers is confined beneath the Blue Bluff marl, the upper member of the Lisbon

Formation. The marl consists of semiconsolidated glauconitic marl with subordinate lenses of dense, well-indurated, well-cemented limestone. The marl layer immediately overlies the unnamed sands which form the Tertiary aquifer. The marl is approximately 70 ft thick. The permeability of the marl layer is very low, and it effectively confines the aquifer underlying it. It is a barrier to ground water movement.

At the VEGP site both aquifers are confined beneath the marl, but are in apparent hydraulic contact with one another. At some distance downdip of the VEGP site the Cretaceous aquifer becomes hydraulically separated from the Tertiary aquifer by the intervening, relatively impermeable clays and silts of the Huber and Ellenton Formations (Paleocene). The point at which this occurs is not well defined, but it is probably a few miles downdip (south) of VEGP. The Huber Formation is present beneath the VEGP site but, based on the lithology, it probably is not a significant barrier to movement of ground water in this area. The Ellenton Formation was not recognized beneath the VEGP site, but was identified in exploratory borings 7 miles southeast of the site[3].

At some point more than 10 miles downdip of the VEGP, the Blue Bluff marl undergoes a facies change (transition) to permeable limestone. Combined with the moderately permeable underlying unnamed sands and permeable overlying sediments, this limestone/sand sequence is referred to as the principal artesian aquifer. Overlying clays and fine-grained sediments of the Barnwell and younger formations confine the aquifer in this area.

To summarize, the Tertiary aquifer overlies and off-laps the Cretaceous aquifer in the outcrop areas north of VEGP. Ground water is under water table conditons in these areas. Progressing downdip, the two acquifers become separated stratigraphically (but not hydraulically) by the Huber Formation. They also become confined beneath the Blue Bluff marl member of the Lisbon Formation. These conditions prevail beneath VEGP and to an unidentified point, south of the site. Beyond that point the Ellenton and Huber formations confine the Cretaceous aquifer, hydraulically separating it from the Tertiary aquifer.

The regional direction of ground water flow in the Cretaceous and Tertiary systems in south-by-southeast toward the sea. However, from the fall line to a point a few miles south of the VEGP, the Savannah River has downcut through the Blue Bluff marl confining layer and into the underlying strata. This allows both the Cretaceous and the Tertiary aquifers to discharge to the riverbed. This condition gives rise to a ground water sink, and flow directions in this limited area do not follow regional trends. (A more comprehensive description of the sink is given in reference 3).

2.4.12.1.2.3 Water Table Aquifer. Unconfined ground water beneath the VEGP site is present in deposits of the Barnwell Group overlying the Blue Bluff marl, and in Quaternary deposits along adjacent stream channels. Immediately beneath and adjacent to the VEGP power block structures, the water table aquifer is the backfill material placed in the power block excavation. The Barnwell formations are extensively dissected in the VEGP area. Pleistocene alluvial and terrace deposits, and Holocene flood plain deposits are present along the Savannah River.

In the general vicinity of VEGP, the basal unit of the Barnwell Group is the Utley limestone member of the Clinchfield Formation (drawing AX6DD331). This is a fossiliferous and cavernous limestone unit which is capable of transmitting ground water. However, the unit rarely exceeds a few tens of feet in thickness, and it is of limited areal extent.

The remaining Barnwell units overlying the marl that contain ground water consisting of unconsolidated clays, silts, and sands. Data from packer and permeameter tests performed during exploration and plant construction indicate that both lateral and vertical permeability of these sediments varies considerably. This variation is attributed mainly to the highly variable quantities of clay distributed throughout the aquifer.

Recharge to the water table aquifer is almost exclusively by infiltration of direct precipitation. The presence of porous surface sands and the moderate topographic relief in the site area indicate that there is no significant storm runoff; hence, virtually all precipitation infiltrates the ground. Lateral recharge from adjacent areas is insignificant because the plant area is situated on an interfluvial high; i.e., it is isolated by drainage channels which have down cut to or near the marl and act as interceptor drains to potential recharge sources moving laterally toward the plant site. Lateral recharge from the Savannah River would be possible only in the case of a very severe flood, one that raises the river level some 30 ft or more.

2.4.12.1.3 Onsite Use

2.4.12.1.3.1 Plant Operating Requirements. Ground water is the primary source of supply for reactor cooling water makeup, normal makeup to the nuclear service cooling towers, and fire protection. Two makeup wells (designated MU-1 and MU-2A on drawing AX6DD332) producing from the combined Cretaceous/ Tertiary aquifers supply water to storage tanks from which water is drawn as needed. Wells MU-1 and MU-2A are capable of supplying water at 2000 gal/min and 1000 gal/min, respectively. The two wells pumping simultaneously have sufficient capacity to supply expected process makeup requirements and completely fill the major plant tanks in one day. The plant water well system is not required for safe shutdown of the plant but is the normal source of water supplied as makeup to the ultimate heat sink. An independent, alternate source, the Savannah River, is also available for cooling water makeup. (See section 2.4.11.) Table 2.4.12-2 summarizes expected ground water use at the site.

There is one well located at the simulator building (SB on drawing AX6DD332) which supplies potable water for that facility. This well is producing from the Tertiary aquifer. Another well (PW on drawing AX6DD332) provides wash water for the combustion turbine plant (Plant Wilson) on site. It produces from the water table aquifer.

2.4.12.1.3.2 Construction Requirements. Initial construction requirements were approximately 350,000 gal/d or 240 gal/min of untreated water for concrete batch plant operation, dust suppression, and sanitary needs. These requirements gradually increased to 600,000 gal/d (420 gal/min) to accommodate peak construction activities. Makeup wells 1 and 2 originally supplied most of these needs. The wells were pumped intermittently to fill two temporary tanks, a 25,000-gal construction water tank and a 10,000-gal batch plant water storage tank. In August 1980, a new 400-gal/min well, CW-2, producing from the Tertiary aquifer, was installed west of the power block as a primary source of construction water. In addition, the two temporary tanks were replaced by a permanent 30,000-gal tank near the batch plant. During April 1990, well CW-2 was grouted from the bottom to the top of the well and then abandoned.

Potable water for the temporary construction office on site was provided by a well, CW-1, producing from the Tertiary aquifer. Yield of this well is 100 gal/min with a specific capacity of

25 gal/min/ft of drawdown. In July 1989, well CW-1 was filled with sand and concrete and then abandoned. A third well, CW-3, producing from the Tertiary aquifer provided potable water for the nuclear operations garage. The construction wells are shown on drawing AX6DD332. A summary of the well data are on table 2.4.12-7D.

2.4.12.1.3.3 Wells and Well Field Design. During normal operation of the plant, the ground water supply is provided by one makeup well, with other makeup wells on standby. There are two makeup wells (MU-1 and MU-2A) and one alternate makeup well (TW-1). Well MU-1 is capable of producing 2000 gal/min on a continuous basis for the life of the plant. Beginning in 2001, Well-2A, originally rated at 2000 gal/min, became limited to producing 1000 gal/min on a continuous basis for the life of the plant. The alternate well (TW-1) is capable of producing 1000 gal/min on a continuous basis for the life of the plant.

Wells TW-1 and MU-2 were constructed as a test well and a makeup well, respectively. Well MU-2 was utilized for construction water, but due to a design change, its location interfered with other facilities. Therefore, it was replaced by well MU-2A. The test well (TW-1) was originally drilled and tested to provide design data for the makeup well field. Wells TW-1 and MU-2 were capped and utilized as observation wells to monitor the Cretaceous aquifer until 1995. Beginning in 1999, well TW-1 became the alternate makeup water source, limited to 1000 gal/min. Initial use of this well, due to its location near Category 1 structures, includes reading of settlement markers for structures near the well to confirm that drawdown does not affect these structures' settlement.

The two plant makeup water wells (MU-1 and MU-2A) are constructed as gravel pack wells extending to depths of 830 ft and 865 ft, respectively, and are open to selected aquifer zones below a depth of 435 ft. Well casing to a depth of 250 ft in MU-1 is 16-in. nominal diameter; below that depth, the casing and screen are 10-in. nominal diameter. Well casing to a depth of 280 ft in MU-2A is 14-in. nominal diameter (the original 16-in. diameter casing has been sleeved with a 14 in. casing); below that depth the casing and screen are 10-in. nominal diameter. To prevent movement of water between the aquifers above and below the marl the well bores were enlarged to 34 in. diameter to a depth of 160 ft in MU-1 and 98 ft in MU-2A, and 26-in. nominal diameter well casings were set at these depths. The annular space between the hole wall and casing was filled with grout to provide the seal. The alternate plant makeup water well (TW-1) is constructed as a gravel pack well extending to a depth of 860 ft and is open to selected aquifer zones below a depth of 450 ft. Well casing to a depth of 91 ft in TW-1 is 28-in. nominal diameter. A second casing 16-in. nominal diameter extends to 250-ft depth. A third casing and screen is 10-in. nominal diameter, and extends to 850-ft depth. To prevent movement of water between the aquifers above and below the marl, the well bore was enlarged to 34-in. diameter to a depth of 91 ft. The annular space between the hole wall and upper casing was filled with grout to provide the seal.

Well design criteria, including the well screen openings, are based on data collected in the drilling and testing of the initial test well (TW-1). The basic design criteria for the wells are as follows:

A. Allowing for pumping well interference and fluctuations of water levels caused by increased use of ground water and climatic changes, wells MU-1 and MU-2A can be pumped continuously at their rated capacity for the life of the plant.

B. Minimum construction specification is the American Water Works Association standard for deep wells (standard AWWA A-100).

C. The wells are lined with steel casing of sufficient strength to carry the stress loads anticipated during operation of the plant.

D. The well screen is a wire-wound type fabricated of stainless steel stock and has sufficient strength to carry anticipated stress loads. Screen openings in MU-1 and MU-2A are 0.04 in. and 0.035 in., respectively, as determined from size analyses of the aquifer materials and design of the gravel filter pack. Length of screen provides sufficient open area to reduce entrance velocities to less than 4 ft/min and to avoid migration of fine material into the wells.

E. Gravel pack material is clean washed and graded sand and gravel consisting of sizes (selected with reference to the aquifer materials) which provide an efficient filter without restricting movement of water to the well.

F. Allowance is made for potential corrosion (electrolytic and chemical) to ensure sufficient strength and integrity of the casing and screen during the life of the wells.

G. A grout seal is placed in the annular space between the 34-in.-diameter hole wall and the 26-in.-diameter surface casing to a minimum depth of 160 ft in MU-1 and 98 ft in MU-2A. Grout was placed in one continuous operation by placing a tremie pipe inside the annulus and filling the annular space from the bottom up.

The two plant makeup wells provide a well field system that was designed to consider the factors of pumping well interference, fluctuations of water levels caused by climatic changes, and future increased use of ground water and to preclude possible subsidence of plant foundations from drawdown of piezometric levels caused by pumping cones of depression.

A well to supply water for irrigation purposes, IW-4, was added in 1989. This well was drilled to a depth of 370 ft and a 4-in. PVC screening was installed to a depth of 370 ft. The well produces water from the Tertiary aquifer with a maximum pump capacity of 120 gal/min. A well for potable water supply to the security tactical training facility, SW-5, was added in 1990. This well was drilled to a depth of 200 ft and a 2-in. PVC screening was installed to a depth of 200 ft. The well produces water from the Tertiary aquifer with a maximum pump capacity of 20 gal/min. The locations of the water supply wells are shown on drawing AX6DD332. A summary of the water supply well data is in table 2.4.12-7D.

2.4.12.1.3.4 Temporary Dewatering. Construction of the foundations at VEGP required excavation of the Eocene and younger sands, silts, and clays of the unconfined aquifer from about el 216 ft to el 130 ft. The portion of the excavation below the water table elevation of the unconfined aquifer (approximately el 160 ft) was dewatered during excavation by a series of ditches oriented in an east- west direction and connected by a north-south ditch, which drained to a sump in the southwest corner of the excavation. The sump was equipped with four pumps with a capacity of 500 gal/min each to remove inflows from ground water. Additional capacity was provided for the removal of inflows of storm water into the excavation.

The ditch and sump approach was successful when the invert elevation of the ditches was maintained at 15 to 20 ft below the adjacent grade. This permitted the use of conventional excavation procedures in reasonably dry materials.

Upon reaching the marl, the system of ditches and sump was replaced by a perimeter drainage system as shown on drawing AX6DD324. This consisted of a porous concrete pipe around the perimeter of the power block excavation feeding into three sumps at the toe of the south slope. Water pumped from the sumps was discharged to debris basin 1 southeast of the power block. The porous concrete pipe was encased in a granular filter material which was carried up the surface of the adjacent 2 to 1 slope to about el 160 ft. This filter blanket was placed so that there was a minimum of 4 ft of filter material measured horizontally from the face of the slope out to the face of the filter blanket as shown on drawing AX6DD324.

The construction dewatering system, i.e., the filter blanket and porous concrete perimeter drain, was designed to prevent ground water from entering the power block excavation during construction operations. In this regard, it performed properly throughout the construction period. However, the system was not adequate to drain excessive rainfall which accumulated in compacted fill. The Category 1 backfill program necessarily proceeded in steps and stages that were commensurate to the gradual rise of structure walls. Fill was placed and compacted to thicknesses ranging from a few feet to a few tens of feet above the marl and then left as a functional surface to await the next stage of structure wall construction. In the interim, rainfall penetrated the fill, percolated to the marl layer, and accumulated to a fairly constant level in the same manner that a perched water table develops over a clay layer. In some areas the water rose to a level very close to the top of the fill, precluding further backfilling on that surface. In addition to near-surface saturation at various locations, there was seepage from the face of the slopes of the compacted fill which rested on the marl bearing surface. This condition precluded backfilling against these slopes.

The conditions described above were relatively minor and caused no major delays or problems until November 2, 1979. On that date almost 6 in. of rain fell at the site, causing severe surface erosion of slopes and rapid saturation of the more shallow fill areas. As a result of the additional water, it was necessary to implement a supplemental dewatering system so that backfill operations could continue. Along with the dewatering system, slope repair and settlement monitoring plans were also implemented. The remedial program was completely successful and is described fully in reference 6.

2.4.12.1.3.5 Chemical Quality of Water. Several samples of ground water and surface water were taken for chemical analysis to determine the quality of water in the area, to identify characteristics of the different aquifer water, and to determine any correlation between ground water and surface water. Tables 2.4.12-3 through 2.4.12-6 list analyses of water samples from observation wells, domestic wells, plant water supply wells, springs, and surface waters. Locations of sample points are shown on drawings AX6DD323, AX6DD325, and AX6DD332.

Water samples were collected in October 1971 from both the water table aquifer and the confined aquifer from observation wells set in each system. Additional water samples were collected from the construction and makeup water wells at various times from 1977 to 1985.

Overall, the ground water of the area is calcium and calcium- sodium bicarbonate types, with total dissolved solids less than 200 ppm. Samples of ground water from the confined aquifer system contain from 110 to 194 ppm total dissolved solids, and sodium is the dominant cation.

Ground water from the water table aquifer system is more variable in total dissolved solids; analyzed samples for the system ranged from 20.0 ppm (spring 2, table 2.4.12-5) to 169.5 ppm (well 143, table 2.4.12-3). Sodium is again the dominant cation. The variation in total dissolved solids is probably related to the length of time the water has remained in the ground, since more time allows more leaching of solids.

The surface water is low in total dissolved solids (less than 100 ppm), with calcium and magnesium as the dominant cations. Bicarbonate is the dominant anion. The highest total dissolved solids were measured at Mathes Pond where solutioning of calcite in the shell zone is believed to be the major source of cations.

2.4.12.2 <u>Sources</u>

2.4.12.2.1 Present Ground Water Use

Large quantities of water are stored in the confined aquifers underlying the region of the plant site, and to date relatively small withdrawals have occurred. Although many small communities derive water from wells, the draft on the aquifers is low because of the low population density, limited industrial developments, only moderate need for crop irrigation (due to high rainfall), and abundant surface waters.

The largest withdrawal of ground water in the region is concentrated in the Savannah area, 80 miles southeast of the VEGP site, near the Atlantic Coast. Ground water is extracted principally from limestone beds of the Ocala Group of upper Eocene age. Heavy pumping from a relatively small area has caused a large, deep cone of depression to form. Recharge to the aquifer in the vicinity of Savannah may eventually stabilize the cone but the hydrograph of well CHA 84, drawing AX6DD326, indicates a continued decline through the year 1979.

Closer to the VEGP site, the principal withdrawal of ground water is from the Tuscaloosa aquifer (i.e., the Cretaceous system) at the Department of Energy Savannah River Project facilities. Pumping of ground water for these facilities began in the early 1950s to supply water for construction. Since then, ground water extractions have continued for operational purposes, although the amounts have been considerably less than was originally anticipated. As reported by Savannah River Project personnel, ground water extractions have remained relatively constant at about 5000 gal/min. These withdrawals will have no effect on ground water conditions at VEGP.

Sylvania is one of the few communities in the vicinity of the VEGP site that derives its water supply from ground water. Located approximately 30 miles south-southeast of the site, the Sylvania wells withdraw water from the Tertiary aquifer. Water level history is shown on the hydrograph for Screven well 3 (drawing AX6DD326). Although short-term trends are indicated, no aquifer dewatering or change in storage of ground water is evident over the period of record, except for a slight decline in the late 1970s.

The city of Augusta is the nearest large municipality. Small domestic wells are present there, and a few large industries have wells that extract water from the recharge area of the Tuscaloosa Formation. The main source of municipal supply, however, is drawn from the Savannah River.

The area of Richmond County south of Gordon Highway is served by the Richmond County Water and Sewer Authority. The service area for the authority is about 12 mi^2 and serves about 15,000 customers from 18 wells, most of which are in the southern part of the county. The wells average 200 ft in depth and can produce about 750 gal/min each from the Tuscaloosa aquifer. Approximately 9.4 Mgal/d are used throughout the service area.

The area around McBean (located about 13 miles northwest of the site) is served by the Pine Hill Water Authority. The authority has five wells, three of which are operable, that draw water from Cretaceous sands of the Tuscaloosa aquifer. Total depth of each of the wells is about 450 ft, and the wells are distributed throughout the service area, which has about 2200 customers. The USGS has begun monitoring one of the wells to provide a record of future trends of water levels in the area.

Ground water use in eastern Burke County was determined by an extensive well canvass performed during the site exploration phase and supplemented by the Millett fault study of 1982.(7)

The well survey data indicate ground water use is almost exclusively for domestic needs. Small amounts are used for stock, and there are a few small commercial buildings in the communities served by municipal wells. Except for VEGP there are no known industrial, irrigation, or similar activities requiring continuous withdrawals of large quantities of ground water.

The only incorporated community within 10 miles of the plant site is the town of Girard, with a population of about 250. Although 12 private wells were in use in the recent past, these were abandoned when city water service lines were installed. City water is supplied from two wells producing from the Tertiary aquifer.

Sardis is a community 12 miles due south of the site. It is larger than Girard (population of 830), and the community water supply is provided by three wells open to the Tertiary aquifer with pumping capacities from 300 to 500 gal/min. Two of the wells are for standby and fire protection purposes. The 1000 customers are predominantly domestic users.

The many private wells in eastern Burke County are small, with a maximum capacity of less than 10 gal/min. The average of each well is estimated to be less than 0.5 gal/min.

2.4.12.2.2 Projected Future Ground Water Use

In 1965 and 1966, the Central Savannah River Area Planning and Development Commission conducted a regional investigation of present and future water and waste disposal needs. According to this survey, the population of eastern Burke County is expected to reach approximately 2800 by the year 1986. That is when the Augusta-Savannah Industrial Corridor is scheduled to extend into Burke County with a concomitant increase in water requirements to 540,000 gal/d. A large portion of this increase will be for industrial uses, and the source will be primarily from the Savannah River.

There is little anticipation of any substantial change in the agricultural development of the area. Assuming that present water requirements are primarily for domestic use and are derived from ground water, future increases in ground water use can be assumed to be correlative to the population.

The Richmond County Water Authority currently has a capacity of about 13 Mgal/d. By the year 2000 they estimate that their capacity will be enlarged to 18.5 Mgal/d based on annual growth of about 500 customers per year.

2.4.12.2.3 Water Levels and Ground Water Movement

A ground water monitoring program to determine the direction of ground water movement and the piezometric levels of the aquifers was established at the VEGP site with the first exploration work in 1971. That program has included an array of observation wells open to the water table aquifer above the Blue Bluff marl, and an array of wells open to the confined Tertiary aquifer immediately below the marl (in the unnamed sands of the Lisbon Formation). Two wells open to the Cretaceous (Tuscaloosa) aquifer were added to the monitoring program in 1985.

Special observation wells were installed to provide data on the distribution of hydrostatic pressure across the marl. Other special wells include a series of short-lived construction "piezometers" that were installed in the backfill as it was placed around the power block complex. They were utilized to assure the water table in the backfill was far enough below the surface to achieve effective compaction.

The initial array of observation wells installed during the exploration period 1971 through 1972 included several wells located in areas of plant construction. These were destroyed and sealed as required by the construction schedule, and when possible, replaced after the construction was completed.

Additional wells have been installed to provide an effective system to monitor ground water during plant operation as needed. A complete list of all observation wells, both active and inactive, is on tables 2.4.12-7A through 7C.

2.4.12.2.3.1 Observation-Well Monitoring. The original observation wells installed in 1971-1972 included 16 open to the water table aquifer, 10 open to the confined aquifer, and two monitoring hydrostatic pressure in the marl. This array remained until July 1974 when site grading and excavation for the power block commenced. A majority of the wells were terminated at that time to make way for construction. All activity at the site was interrupted three months later, September 1974.

Resumption of construction, which began in 1976, required dewatering the power block excavation. The dewatering continued, uninterrupted, until March 1983. As construction progressed, more wells had to be terminated. Only 3 of the original 16 observation wells open to the water table aquifer remain intact November 1985. Of the original 10 wells open to the confined aquifer, 2 remain. Other wells have been installed periodically to replace those destroyed by construction.

During the period September 1971 through March 1972, water levels in observation wells at VEGP were monitored by Law Engineering Company at least biweekly and, commonly, more frequently. No water level measurements were made between April 1972 and April 1973. Georgia Power Company personnel commenced monitoring on a quarterly basis in April 1973. Monitoring was again stopped July 1974 when site grading and excavation for the power block began. Monitoring was not resumed until June 1979, at which time quarterly measurements of all existing wells was again initiated.

Dewatering of the power block excavation was in effect from June 1976 through March 1983. Hence, water levels in observation wells of the water table aquifer during this period were influenced by construction dewatering. Daily readings were made in observation wells 800 and 802 during the period December 11, 1980, through September 15, 1982, as part of the monitoring conducted for placement of the backfill around the power block structures. Temporary wells were installed to monitor the saturated level in the backfill as it was being placed to assure proper compaction.

In July 1985, a program of frequent measurement of water table wells was implemented. The purpose of this program was to provide more detailed information to support the basis for the hydrostatic loading design (see section 2.4.12.4). Locations of the wells in this program are shown on drawings AX6DD333, AX6DD334, and AX6DD335. Additional water-table observation wells were installed in the backfill adjacent to the power block structures and several previously destroyed wells were replaced. There were 17 wells monitoring the water-table aquifer. A series of 6 Casagrande-type piezometers have been set in the Blue Bluff marl to monitor the hydrostatic pressure. They were in two clusters of 3 piezometers each, located on the northwest and southeast side of the power block. The piezometers in each cluster monitor pore pressure in the upper, middle, and lower portions of the marl. In addition, monitoring of the 10 Tertiary aquifer wells immediately below the Blue Bluff marl and the two observation wells in the Cretaceous aquifer was also maintained.

Water-table aquifer wells were measured on a weekly interval for a minimum period of 6 months. With each water-level measurement, depth to the base of each well was also measured to determine if silting occurred.

A review of water-level and precipitation data was made after 6 months of monitoring to establish a correlation (amount and lag-time) of water table response to precipitation. The review established a correlation and indicated that frequency of measurements could be reduced to once a month. Measurements on a monthly basis continued to establish seasonable correlations with precipitation.

The six piezometers open to the Blue Bluff marl were also monitored weekly. These piezometers were measured the same day each week as the wells in the water table aquifer. The data from these piezometers was reviewed after 6 months to determine the degree of hydrostatic pore pressure fluctuations in the marl. Once equilibrium was accomplished and degree of fluctuation was determined, monitoring frequency was reduced.

The wells in both confined aquifers (Tertiary and Cretaceous) were monitored on a monthly basis for a period of 6 months. The measurements were made concurrently with measurements in the water table aquifer. After the 6-month review of the data, it was determined that quarterly measurements could be made.

In December 1988, the well monitoring program was modified. Quarterly measurements of the four water table wells within the area of backfill around the power block (LT1B, LT7A, LT-12, LT-13) are to be taken. The balance of wells and piezometers are to be measured semiannually.

The frequency of monitoring of the observation wells was evaluated in 1991. The evaluation concluded that the water table aquifer water levels consistently follow the same trends and fluctuations; therefore, the number of wells being read could be reduced. The evaluation also concluded that the Blue Marl readings can be represented by one cluster of piezometers.

In 1993, the well monitoring program was modified to reduce the number of wells being read quarterly and semiannually. The well monitoring program was modified in 1995 to only read levels in the surface water table wells. The current well program is shown on drawing AX6DD335 and in table 2.4.12-9. A summary of the observation well data is in tables 2.4.12-7A through 2.4.12-7C.

2.4.12.2.3.2 Tertiary Aquifer. Water levels measured in observation wells open to the Tertiary aquifer confined beneath the marl indicate the direction of ground water movement is toward the Savannah River in the vicinity of the site. Drawings AX6DD327 and AX6DD328 show the contours of the piezometric surface for October 1971 and for December 1984. A comparison of the two sets of contours indicates very little difference. These data demonstrate the marked gradient toward the river. Selected hydrographs of Tertiary aquifer wells are on drawing AX6DD336.

2.4.12.2.3.3 Water Table Aquifer. As previously noted, the site is on an interfluvial ridge. Ground water present in the materials overlying the marl is under water table conditions and isolated hydraulically from other aquifers by the marl. Replenishment is by infiltration of precipitation, and after precolating to the water table, ground water moves laterally to the bordering interceptor streams. Hydrographs of water-table aquifer observation wells are on drawing AX6DD337.

Contours of the water table for November 1971 and for December 1984 are shown on drawings AX6DD329 and AX6DD330. Contours for the 1971 data are based on measurements of water levels in observation wells and springs and seepage areas on the interceptor streams. The water table is, in general, a subdued reflection of the ground surface, and movement is from the central portions of the interfluve toward the bordering interceptor streams.

Construction dewatering at the site was completed in March 1983. Water levels and the flow pattern of the water table aquifer have returned to a preconstruction pattern. No permanent dewatering facilities have been provided nor are any necessary at the plant site.

2.4.12.2.4 Hydrogeologic Properties of Subsurface Materials

The hydrogeologic properties of the materials beneath the site were determined by extensive subsurface exploration at and near the site by in situ tests conducted in the field, and supplemented by laboratory tests of numerous samples collected throughout the drilling programs.

The hydraulic conductivity (permeability) of the subsurface materials was measured in the field by pumping tests, constant-head inflow tests (including open standpipe and permeameter tests), variable-head inflow tests, and slug tests.

2.4.12.2.4.1 Cretaceous and Tertiary Aquifers. As stated in paragraph 2.4.12.1.2.2, the Cretaceous and Tertiary aquifers are believed to be in hydraulic connection at the site. The hydrologeologic properties of the combined Cretaceous/Tertiary aquifer were determined at the plant site by pumping tests on test well TW-1 during early site investigations and by pumping tests on makeup wells MU-1 and MU-2 during plant construction. A brief description of the makeup well pumping test program is given in the following paragraphs. (A detailed description of the testing program for TW-1 was included in the Preliminary Safety Analysis Report). The transmissivities and storage coefficients for the wells are included in table 2.4.12-8.

The pumping test for MU-1 began on August 21, 1977, and concluded 72 h later. Recovery level readings began immediately following pump shutdown and continued for 24 h. The pump intake was placed at approximately 232 ft below ground level at approximately el -35 ft mean sea level (msl). Depth to water in the well before pump startup was 27.7 ft (from top of casing). The step-discharge test is summarized below:

Well Discharge (gal/min)	Step Duration (min)	Total Drawdown (ft)	Specific Capacity (gal/min/ft)
1150	720	32.7	35.2
2232	720	72.3	30.0
3334	2880	123.9	26.9

The transmissivity (T) of the aquifer was determined from the measurements of drawdown taken during the initial step-discharge test and from the measurements of recovery following the test.

The drawdown data were analyzed using the Jacob method (modified Theis nonequillibrium equation), and the recovery data were analyzed by the Theis recovery formula. The analyses show that the transmissivity is in the range of 110,400 to 116,600 gal/d/ft.

The pumping test for MU-2 began on December 15, 1977, and concluded 72 h later. Recovery level readings began immediately following pump shutdown and continued for 24 h. The pump intake was placed at 231 ft below top of 16-in diameter casing at approximate el -16 ft msl. Depth to water in the well before pump startup was 42.1 ft (from top of casing). The step discharge test is summarized below:

Well Discharge (gal/min)	Step Duration (min)	Total Drawdown (ft)	Specific Capacity (gal/min/ft)
1200	720	22.0	54.5
2175	720	47.5	45.8
3316	2880	81.3	40.8

The transmissivity of the aquifer was determined from the measurements of drawdown taken during the initial step- discharge test and from the measurements of recovery following the test. The drawdown data were analyzed using the Jacob method (modified Theis nonequilibrium equation), and the recovery data were analyzed by the Theis recovery formula. The analyses show that the transmissivity is in the range of 128,700 to 130,900 gal/d/ft.

Table 2.4.12-8 summarizes aquifer characteristics of the Cretaceous aquifer determined with test well TW-1 and with the makeup water wells. The moderately wide range of transmissivities and storage coefficients shown on the table suggests that the aquifer is not uniform in character and that permeability varies from place to place. No particular significance is attached to this condition because (1) the range of differences is not especially large and (2) the lowest of the values (110,400 gal/d/ft) is still indicative of a highly productive aquifer. It is noteworthy that the 158,000-gal/d/ft value, determined from a distance-drawdown analysis and considered more indicative of the average, falls very close to the median value between the highest and lowest transmissivities obtained. This implies that the value of 150,000 gal/d/ft is a realistic and conservative value to be used in evaluating the capability of the aquifer in this area to yield water to wells. The values of storage coefficient determined from the pumping tests indicate the aquifer is effectively confined.

2.4.12.2.4.2 Blue Bluff Marl. The Blue Bluff marl is approximately 70 ft thick. It extends over an area well beyond the limits of the plant site and the interfluvial ridge on which the plant site is located. The comprehensive exploration and testing that has been conducted demonstrates that the marl is an extensive and persistent unit. In particular, the marl's integrity as a barrier to ground-water movement has been demonstrated by (1) field permeability testing, (2) visual inspection of cored samples, the marl surface exposed during site excavation, and marl outcrops along the Savannah River, and (3) comparison of water levels in observation wells open to the water table aquifer with those observed in wells open to the confined aquifer immediately below the marl.

The continuity of this nearly impermeable material (i.e., the lack of voids, open joints or fractures) has also been demonstrated. Since 1971, there have been over 10,000 ft of marl penetrated at VEGP by drilling, coring, Standard Penetration Testing, and undisturbed sampling. When coring, the most revealing evidence for the occurrence of voids or permeable fractures is a loss of all or part of the drilling fluid and/or a sudden or rapid advance of the core barrel. At no time throughout this extensive testing was there any unaccountable fluid loss or abnormal tool advance in the marl. None of the borings encountered significantly fractured zones, nor was there evidence of leaching (removal of calcareous material by solution activity) of the marl.

Visual inspections and detailed logging of the many extracted samples of marl have likewise produced no indications of voids or extensive fracture zones. Over 1000 ft of the marl penetrated in drill holes have been either cored or sampled, and have been closely inspected and described. Very few joints or fractures were observed and those identified were consistently found to be tight, and without void space. More than 940,000 ft^2 of marl beneath the plant site was exposed during the excavation for the foundation and was directly examined and carefully logged. The results are discussed in detail in subsection 2.5.1.2.2.2.1.1. Additionally, marl outcroups along the Savannah River in the vicinity of VEGP have also been examined and mapped. These extensive and detailed mapping investigations of the marl formation at VEGP have found no evidence of voids, solution cavities, or systematic or extensive fractures in the marl.

The large and consistent hydraulic head differential between the water table aquifer above the marl and the confined aquifers immediately below the marl confirms that the marl is a barrier to ground water movement. The hydraulic head (energy potential) of ground water in an aquifer is commonly exprssed as ft (elevation) above sea level, and is determined from measuring the elevation of water in an observation well. In the vicinity of the plant, the hydraulic head in the water table aquifer is 45 to 55 ft greater than the hydraulic head in the aquifer immediately below the marl. To bring about such a marked difference in hydraulic head, the barrier must be extensive and without significant through-going openings (such as fractures or solution cavities).

This difference in hydraulic head can be seen by comparing the ground water (equipotential) contours shown on drawings AX6DD328 and AX6DD330. The contours are based on water levels measured in observation wells in December 1984. Similar conditions were observed prior to plant construction, as indicated by the contours of water levels measured in wells in October and November 1971 and shown on drawings AX6DD327 and AX6DD329.

A nest of observation wells constructed at the site of exploratory hole 42 provided a measure of this hydraulic head differential between the overlying water table aquifer and the confined aquifer sands beneath the marl. The observation wells were constructed in 1971 and included two, 42B, and 42C, open to the marl itself, one, 42A, open to the confined aquifer and one, 42D, open to the water table aquifer. At these wells, the marl is 65 ft thick. The wells were monitored for 4 years until construction of the plant required their closure, at which time they were sealed. Hydrographs of the measured levels are shown on drawing AX6DD408.

A general relationship between the water levels (head) of the observation wells and the zone each well monitors can be seen on drawing AX6DD408. That is, the differences in water levels (head) between the observation wells is generally proportionate to the thickness of marl between zones monitored by the wells. The zones monitored by each observation well are illustrated on drawing AX6DD408. For example, the difference in water levels of the two wells open to the two aquifers (42D and 42A) is about 55 ft, (the head in the water table aquifer is higher) and the thickness of marl between them is 65 ft. In comparison, well 42B is open to an interval of the marl that is near the bottom of the marl. The water levels measured in well 42B are from 15 to 20 ft different (higher) than those measured in well 42A, which is open to the underlying confined aquifer, and the thickness of marl between them is about 10 ft. Water levels measured in well 42C follows this general relationship. It is open to an interval of the marl that is within 3 ft of the top of the marl. The water levels measured in 42C are from 50 to 52 ft higher than those measured in 42A, and the thickness of marl between them is about 60 ft.

Two clusters of piezometers (A&B) were installed in the marl in June and July 1985. The clusters are located at opposite corners of the power block, as shown on drawings AX6DD333, AX6DD334, and AX6DD335. The piezometers provide a direct measurement of the hydraulic head over the full thickness of the marl. The differences in hydraulic head between the piezometers within a cluster show a progressive decline in head with depth as was observed in the 42 series (see drawing AX4DD408).

In situ permeability tests were performed in the marl during early site investigations, river facilities investigations, and during recent (1985) installation of observation wells. Results of the tests are summarized on table 2.4.12-10. Tests were conducted in 95 intervals at different depths in 28 exploratory holes. In 90 percent of the intervals tested, no measurable water inflow occurred. In only three holes was any measurable water inflow confirmed; in two of these the inflow occurred in near surface, weathered marl. (Water inflow measured in three other holes was due to leakage around the packers.)

Laboratory permeability tests were conducted on 10 samples of the core collected from the marl observation wells. The range of laboratory permeability measurements is from 5.0 x 10^3 ft/yr to 8.5 ft/yr. The results of the laboratory tests are summarized on table 2.4.12-10. The permeability tests indicate that the marl is nearly impermeable.

Porosity of the Blue Bluff marl was calculated from laboratory analyses of undisturbed samples taken in exploratory holes during the initial site studies. The porosities range from 24 percent to 62 percent, with a mean value of 47.5 percent. The calculated porosities are listed on table 2.4.12-11.

2.4.12.2.4.3 Water Table Aquifer. The permeability of the water table aquifer was measured by field-test methods and laboratory testing. Measurements were made of Barnwell sands and clayey sands, Utley limestone, and of backfill material. Preliminary estimates were made of the Barnwell sands and clay by correlation to grain-size analyses. Porosities of undisturbed samples of Barnwell sands, silty sands, and clay, as well as of samples of backfill material, were measured in the laboratory.

In-situ permeability of Barnwell sands and clayey sands was measured at two exploratory holes (183 and 184) at the plant site, and in the laboratory on four samples taken from hole 107A. Three of those samples were undisturbed; the fourth was a disturbed sample for which permeability was measured at three densities. Similarly, permeabilities were measured in the laboratory at four densities of two "grab" samples of backfill material. The backfill samples were selected for different amounts of material finer than the No. 200 sieve: one sample with 5.9 percent and one with 11 percent. Variation of permeability with density was determined by measuring each sample at four different densities. The testing procedure followed was to saturate the samples by the back-pressure technique, confine them at the effective overburden pressure, and then maintain a constant hydraulic gradient across the sample.

Preliminary estimates of permeability of the Barnwell sands as backfill material were made by the approximate relationship to grain size found by A. Hazen for filter sands [11]. These estimates are reported in reference 10.

The permeability of the backfill material was measured in situ by slug tests performed in four observation wells in the power block area [13].

The transmissivity values determined from the results of the insertion and extraction cycles of each test were averaged to obtain the transmissivity in the vicinity of each well. The average permeability of the backfill at each test site was then determined. The results are summarized on table 2.4.12-15.

Two test wells, each with an array of four observation wells, were constructed in the vicinity of the power block excavation to conduct aquifer pumping tests to measure the permeability of the Utley limestone. Beneath the site, the Utley limestone is composed predominantly of shells in a matrix of silt and clay, but includes thin and discontinuous beds of limestone, sand, and coquina. It was thought in the initial plans for dewatering that the limestone, which is at the base of the water table aquifer, might be very permeable and could act as a drain for dewatering.

The first well, W-1, was pumped at a rate of 36 gal/min for 97 h, and response was sufficient in all four observation wells for analysis. The second well, W-2, was in a much less permeable zone of the limestone than the first well, and response in the observation wells was negligible to small, precluding effective analysis. The average yield of W-2 was 12 gal/min, but it fluctuated considerably and the test was terminated after 27 h. The two tests indicated that transmissivity of the Utley limestone is relatively low and varies considerably from place to place. It was concluded it would not be an effective drain for dewatering.

In addition to the pumping tests described above, both falling-head and constant-head tests were conducted in some of the observation wells. During two of these tests, the water-level rise in an adjacent observation well was measured. Only the response of observation well 2A to the constant inflow of 74 gal/min at well 2B was adequate for analysis. The results of the tests of the Utley limestone are summarized in table 2.4.12-13.

Fifteen samples of undisturbed Barnwell sands and silty sands and clay were analyzed for porosity. Values measured range from 34 percent to 52 percent for the sands and silty or clayey sands. The mean of all tests is 43.9 percent. The porosity data are listed on table 2.4.12-14. In addition, porosity in relation to density of two recompacted samples of Barnwell sands used for backfill material is provided.

2.4.12.2.5 Potential Reversibility of Ground Water Flow

2.4.12.2.5.1 Confined Aquifer. On a regional basis, the general direction of ground water movement in the Tuscaloosa aquifer system is structure controlled toward south by the southeast under a gradient of 6 to 20 ft/mi.[2] Locally, piezometric surfaces take on discrete configurations under the influence of pumping, topography, recharge and discharge environments, geologic patterns, and other factors. Local ground water movement is therefore determined by the net effects of regional and local influences.

At the VEGP site, a local piezometric surface has been defined via observation wells set below the marl. Drawings AX6DD327 and AX6DD328 show clearly that the piezometric surface slopes to the northeast toward the Savannah River. This suggests that the aquifer is discharging to the river. Further, the marl layer is absent beneath the flood plain of the river; hence, the underlying Cretaceous and Tertiary aquifers are in hydraulic contact with the river. Therefore, as long as the hydraulic head of the aquifer is at a higher elevation than the riverbed, the river will receive water from the aquifer. Data from the Millett study substantiates these conclusions.[3]

Reversal of ground water movement in the confined Cretaceous or Tertiary aquifer would entail a change in conditions that would cause the Savannah River to discharge to the aquifer. The river would then be an influent stream. For this to occur, immense quantities of ground water would have to be withdrawn upgradient from the river, either at or in the vicinity of the VEGP. No significant new enterprises are anticipated in the vicinity of the plant. Plant needs are outlined on table 2.4.12-2 and discussed in paragraph 2.4.12.1.3.1. It can be seen that plant requirements are relatively small when compared to the large capacity and productivity of the aquifer. Therefore, the potential for reversing ground water flow is slight.

2.4.12.2.5.2 Water Table Aquifer. Ground water under water table conditions in the study area is available in quantities sufficient only for domestic use. Water for operation of VEGP is obtained from the confined Cretaceous aquifer and has no effect on the water table aquifer. Hence, the potential for change in flow direction as a result of operation of VEGP is negligible.

The power block construction dewatering system had a local effect on the ground water flow direction, i.e., flow was towards and into excavation. This was a temporary condition and existed only during construction. Dewatering has been discontinued, and the water table has recovered to near the preconstruction configuration (compare drawings AX6DD329 and AX6DD330).

There has been modification to that configuration due to site grading and plant excavation. These activities have lowered the topographic highs on site, with the effects that the general flow direction of ground water remains unchanged, but the water table is more subdued, and is lower in elevation.

2.4.12.3 Monitoring of Safeguard Requirements

Contamination of usable ground water by normal operation of VEGP, or by accidental spills, is unlikely. The potential for such an occurrence is very low. The marl underlying the site is an effective barrier to migration of fluids. Construction of the makeup water wells and observation wells includes a cement grout seal to prevent vertical movement of fluids. The pumping wells and observation wells extending beneath the marl provide a direct and available means of monitoring the confined ground water aquifer, if it is considered desirable or should a question arise. Samples may be taken immediately and analyses performed for prompt determination of any change in water quality.

Observation wells are also placed in the water table aquifer and provide a means to monitor and sample the ground water in the materials overlying the marl. Monitoring program observation wells are listed on table 2.4.12-9. Their locations are shown on drawings AX6DD333, AX6DD334, and AX6DD335.

2.4.12.4 Design Basis for Subsurface Hydrostatic Loading

The design basis for subsurface hydrostatic loading is elevation 165 ft msl. It is the estimated maximum probable level to which ground water could reasonably be expected to rise within the power block area during the life of the plant. The level was originally determined from a study of levels and fluctuations in observation wells in the water table aquifer in the vicinity of the power block site. Levels were measured during site exploration in 1971-1972. The wells nearest the power block during that period include 42D, 140, 143, and 245 (drawing AX6DD323).

Hydrographs of these four water table wells, and others, are shown on drawings AX6DD337 and AX6DD408. The highest level measured in the four wells nearest the power block site during 1971-72 was elevation 162 at well 140, located west and somewhat upgradient of the power block. The water table configuration during that period is illustrated by drawing AX6DD329. The largest fluctuation of water levels was 3.6 ft. From these records it was concluded that annual fluctuations could be expected to be less than 10 ft, and the maximum water table elevation at the power block would be 165.

Since that time, water level measurements in observation wells have continued. Several additional wells have been constructed, either to replace those wells that interfered with construction activities, or to supplement the observation network (see section 2.4.12.2.3.1 for details). Hydrographs of the levels measured through June 1985 are shown on drawing AX6DD337. The period of record extends over more than 14 years. The record demonstrates that natural fluctuations (not considering the response to dewatering of the power block excavation during construction) have been no greater than was expected, and the design level was not exceeded in the power block area, although it was approached in 1973.

Dewatering for the power block excavation was terminated in March 1983 and the water table in that area has recovered. A comparison with water table contours prior to construction, drawing

AX6DD329, with contours of the levels following recovery from dewatering, drawing AX6DD330, indicates that the water table has recovered to essentially the preconstruction levels and configuration. Excavation and grading have significantly reduced the topographic relief of the site. Because the water table is a subdued reflection of that relief, it follows that the post-construction water table can be expected to have lower maximum elevations than prior to construction.

Measurement of the water level in observation wells, as described in section 2.4.12.3, will be continued to verify that the design basis level is not exceeded.

2.4.12.5 References

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2.4.13 ACCIDENTAL RELEASES OF LIQUID EFFLUENTS IN GROUND AND SURFACE WATER

2.4.13.1 Consideration of Accidental Spill of Radioactive Material in Ground Water

In the very unlikely event that an accidental spill of radioactive fluid occurred at Plant Vogtle and infiltrated the ground without interception, it could percolate downward until it reached the water table (unconfined) aquifer. Because the Blue Bluff marl prevents significant vertical movement of ground water across it, continued migration of contaminants from an accidental spill at VEGP would be predominantly lateral in the direction of decreasing head in the water table aquifer. Accordingly, a spill would flow northwestward and after considerable travel time, would discharge into Mathes Pond and stream.

The impact of a postulated spill is assessed by assuming rupture of the recycle holdup tank (RHT), which is considered the worst-case release of radioactivity from the waste system. This tank would contain fluid with the highest concentration of critical isotopes. The migration of the spill in ground water is analyzed by a simplified, one-dimensional flow model in which a number of conservative assumptions are made. Extreme assumptions are imposed concerning the manner in which radioactivity is released to ground water. The analysis not only postulates tank failure, but also failure of the auxiliary building in which the tank is located (the floor on which the tank is located is below ground surface). It is assumed these failures are total, and the release to ground water occurs instantly, with no dilution or decay of the spilled waste. Analysis of the impact of the spill along the principal flow path (laterally, in the water-table aquifer) is first described. This is followed by a calculation of the possible rate of flow, vertically, across the marl to demonstrate the impact of a spill on the Tertiary and Cretaceous aquifers would be negligible.

2.4.13.1.1 Water Table Aquifer

Following these postulated events, the spilled waste would migrate in the ground water along a flow path northwestward to Mathes Pond. The flow path considered is a straight line between the auxiliary building and the south side of Mathes Pond, a distance of 3400 ft (drawing AX6DD330). The spill slug would first migrate an estimated distance of 550 ft through backfill material, and then would move through the Barnwell sands, eventually percolating through the underlying Utley limestone to reach Mathes Pond. Based on best estimates of the controlling parameters (i.e.; effective permeability, gradient) along the flow path, the time for ground water to migrate between the power block and Mathes Pond has been estimated to be as much as 350 years. Radionuclide decay during this long period is more than sufficient to reduce all radionuclide concentrations in a spill to below 10 CFR 20.1 - 20.601 limits.

An even more conservative analysis is to not consider the time of travel through the major portion of the flow path to Mathes Pond. Only that portion within the backfill material, a distance of 550 ft, is evaluated. Outside the backfill material, it is assumed that travel is rapid in an undefined conduit within the Utley limestone. Applying even this extreme assumption, analysis demonstrates that before any spill is discharged off site, concentrations of radioactivity would meet 10 CFR 20.1 - 20.601 limits.

The time required for ground water to migrate through the backfill is determined by the permeability and porosity of the materials, and the hydraulic gradient. The backfill is sand and silty sand compacted to an average of 97 percent of its maximum density (paragraph 2.5.4.5.2). The permeability assigned to the backfill is the maximum value measured in situ, 1220 ft/year (table 2.4.12-15). Total porosity measurements of backfill samples that meet the compaction criterion range from 31.6 to 38.8 percent (table 2.4.12-14) and average total porosity is 34 percent. For sand and silty sand, the total and effective porosity are essentially the same (1). The hydraulic gradient in the backfill along the Mathes Pond flow path is 3.5×10^{-3}, but, again for conservatism, is rounded off to 4.0×10^{-3}. The relationship between these parameters in determining ground water seepage velocity is expressed in a form of Darcy's Law[(1)]:

$$v = \frac{K_i}{n} \qquad (1)$$

where

v = ground water seepage velocity (L/T).

K = coefficient of hydraulic conductivity (permeability) (L/T).

i = hydraulic gradient (ratio).

n = effective porosity (ratio).

Applying the parameter values described, the calculated ground water velocity in the backfill is 14.4 ft/year. With a flow path length of 550 ft, the ground water travel time (t) in the backfill is 38.2 years.

The concentrations of spilled radionuclides that are ultimately transmitted through a ground water system to a discharge point (i.e., through the water table aquifer to Mathes Pond and stream, and, subsequently, discharged offsite to the Savannah River) is determined by the following factors:

- The source (tank) radionuclide inventory released to the ground water.
- The attenuation which takes place during transport through the system, caused principally by dispersion, dilution, adsorption, and radioactive decay.

Of the several radionuclides present in the liquid waste holding tanks, three are critical because of long half-lives. These include tritium (H^3), strontium-90 (Sr^{90}), and cesium-137 (Cs^{137}). Because they are chemically active and susceptible to adsorption, migration of Sr^{90} and Cs^{137} in the ground water will be retarded; they will move at a markedly slower rate than the water. Tritium is not adsorbed significantly, and tends to travel at the same rate as the ground water.

The degree of retardation is governed by various physical properties of the aquifer including bulk density and porosity, and the equilibrium distribution coefficients of the radionuclides. The relationship between ground water travel time (ground-water velocity and travel-path length), radionuclide adsorption, and the radionuclide fraction resulting from decay that is ultimately transmitted to Mathes Pond is given by the following expression (2):

$$\ln\left(C/C_o\right) = -\lambda t = \frac{-0.693\, t}{t_{1/2}} \text{ wA} \qquad (2)$$

where:

C/C_o = transmitted fraction of original concentration, C_o, (ratio),

λ = radioactive decay constant = $\frac{\ln 2}{t_{1/2}}\left(\frac{1}{T}\right)$

t = $t_w A$ = radionuclide travel time (T).

t_w = ground water travel time (T).

$t_{½}$ = radionuclide half-life (T).

A = adsorption retention factor (ratio).

The adsorption retention factor (also called retardation factor) is equal to (1 + p/n K_d)

where:

p = dry (bulk) density of the aquifer (M/L^3).

n = porosity of the aquifer (ratio).

and

K_d = equilibrium distribution coefficient which is defined as the mass of radionuclide adsorbed per gram of soil divided by the mass of radionuclide dissolved per milliliter of ground water.

The density of backfill at the required compaction was determined for 12 samples(3). The values range from 1.62 to 1.79 g/cm^3. Assuming a value of 1.6 g/cm^3, and a porosity of 34 percent, the ratio, p/n, is 4.71 g/cm^3.

The K_ds (equilibrium distribution coefficients) for Sr^{90} and Cs^{137} of 4 samples of backfill were measured by the batch method. The resulting values, shown on table 2.4.13-1, are all more than 5 times greater than conservative estimates of average values proposed by Isherwood(4). Therefore, using the estimated average values, 10 and 100 cm^3/g for Sr^{90} and Cs^{137}, respectively, will impose definite conservatism in this analysis. Because tritium is not adsorbed, the K_d is zero.

Using the values of the parameters above, and a ground water travel time of 38.26 years, the calculated reductions in concentration in the backfill along a northward flow path are summarized as follows:

Nuclides	Kd (cm^3/gm)	A	$t_{1/2}$ (yr)	C/C_o
H-3	0	1	12.2	1.10×10^{-1}
Sr-90	10	48.1	28	1.7×10^{-20}

Cs-137	100	472	30	6.8×10^{-182}

The concentration of radionuclides in the ground water after travel through the backfill is equal to the transmitted fraction times the initial concentration. The following summarizes the initial concentrations assumed to be present in the postulated worst-case spill, the reduced concentration after travel through the backfill due to radioactive decay and adsorption, and the maximum permissible concentration (MPC) for normal releases from Appendix B, Table II, Column 2 of 10 CFR 20.1 - 20.601.

Postulated RHT Rupture

Nuclides	Initial Concentration (μCi/cm^3)	Concentration After Travel In Backfill (μCi/cm^3)	MPC (μCi/cm^3)
H-3	1.0	1.2×10^{-1}	3.0×10^{-3}
Sr-90	1.0×10^{-5}	1.7×10^{-25}	3.0×10^{-7}
Cs-137	1.9×10^{-2}	1.3×10^{-183}	2.0×10^{-5}

It can be seen that under this very conservative scenario, the concentrations of both Sr^{90} and Cs^{137} in ground water would meet 10 CFR 20.1 - 20.601 limits after travel through the backfill. Parameters that would reduce the concentration further, such as dispersion and dilution, need not be considered. Because H^3 is not retarded and migrates with the ground water, the tritium concentration in ground water traveling through the backfill would exceed the maximum permissible concentrations limits (still ignoring any dilution or dispersion of the spill).

Contaminated ground water exiting the backfill would continue migration through the Barnwell Group before reaching Mathes Pond. The high permeability measurements in the Utley limestone (table 2.4.12-13) raise the remote possibility of a continuous conduit allowing very rapid flow to Mathes Pond (negligible decay in short travel time). However, even if this hypothesis were correct, the contaminated ground water subsequently reaching Mathes Pond would be further diluted in the pond and in the stream running from the pond to the Savannah River, reducing the concentration below 10 CFR 20.1 - 20.601 limits before it flows off site. Flow into Mathes Pond is continuous, and the pond level is held constant by a spillway. The rate of flow discharging from Mathes Pond has been measured to be at least 250 gal/min. Measurements of stream flow at points downstream of the pond indicate it progressively increases in magnitude before discharging to the Savannah River.

The ratio of Mathes Pond stream flow to the rate at which the postulated spill would discharge from the backfill (and into Mathes pond) (assumed to be the rate of discharge into Mathes Pond) is a measure of the potential for dilution of the spill within the stream. The discharge rate of the spill in the backfill is determined by the velocity of ground water (14.4 ft/year) and the assumed volume and dimensions of the spill slug.

The volume of the spill slug is assumed to be 80 percent of the total capacity (112,000 gal) of the RHT, and is assumed to be transferred instantly to the backfill. To achieve a large effective discharge rate from the backfill, the shape of the slug is assumed to be rectangular, and relatively equidimensional. The dimensions take into account the saturated thickness of the water table aquifer (estimated at 25 ft) and the effective porosity of the materials. With these parameters, the maximum spill discharge rate into Mathes Pond (discharge from backfill) is calculated to be 0.07 gal/min.

Contaminated ground water would first discharge to the pond, and would then flow into and down the stream below the pond for a distance of 4,000 ft before discharging off site. Mixing with the flow of Mathes Pond stream (a flow of at least 250 gal/min) would reduce the contaminant concentration by a factor of 2.8×10^{-4}. The concentration of tritium discharging from the backfill (0.12 $\mu Ci/cm^3$) would be reduced to 3.3×10^{-5} $\mu Ci/cm^3$, which is below the maximum permissible concentration.

As previously stated the analysis above is extremely conservative. It assumes an instantaneous release to the ground water, and thereby ignores any initial dilution or decay. It ignores any decay during the time to travel beyond the backfill. It ignores dispersion of the spill during its migration to Mathes Pond; and it ignores dilution due to the percolation of precipitation during the travel time. The effect of these additional parameters will be to further reduce the concentration of contaminants.

2.4.13.1.2 Blue Bluff Marl

Owing to the extent and very low permeability of the Blue Bluff marl, the impact of an accidental spill on the Tertiary and Cretaceous aquifers will be negligible. A calculation of the possible rate of flow across the marl demonstrates this conclusion.

The rate of flow is determined by the hydraulic gradient across the marl, and by the permeability and porosity of the materials (equation 1). The gradient is determined by the hydraulic head dissipated (the difference in piezometric levels of the water table and the Tertiary aquifers) over the travel path (the thickness of the marl). The difference in head beneath the power block can be determined from a comparison of piezometric surfaces of the two aquifers measured in December 1984 (drawings AX6DD328 and AX6DD330). These indicate a difference of about 50 to 55 ft. This is similar to the difference observed in a comparison of levels measured prior to construction November 1971. The minimum thickness of the marl is 38 ft, a result of excavation for structures beneath the power block. The maximum hydraulic gradient, then, is 55 ft of head over a distance of 38 ft, or 1.447.

The effective permeability of very-low permeable materials, such as the marl, is difficult to quantify. The most representative testing is by in-situ methods. However, the equipment and field conditions of these methods (such as packer and permeameter tests) limit the accuracy for measuring very low permeabilities. Under the field conditions and equipment used in these tests, no measurable water take (table 2.4.12-10) indicates that actual permeability is less than 0.1 ft/year.

Laboratory testing is capable of measuring very low permeabilities. However, the samples are small and may not be representative of in-situ conditions. The disturbance that occurs from collecting and handling the samples tends to increase the permeability of very low permeable materials.

The vertical permeability of the marl is heterogeneous, as is evidenced by the differences in head decline observed between the piezometers of well clusters A and B and well series 42 (drawing AX6DD408). The marl is composed of a series of beds, and a material comprised of such layers, each of different permeabilities, is described as having layered heterogeneity.[(1)] The downward migration of ground water across the marl, however minute in quantity, will dissipate more head traversing the layers of lowest permeability than in traversing those layers of relatively higher permeability. The average or effective permeability across such a material (vertical flow) has been found to be equal to the harmonic mean of the layer permeabilities.[(5)] Assuming the ten laboratory tests (table 2.4.12-10) are a representative sample of the layers present in the marl (each sample represents an equal proportion of the total marl thickness), the harmonic mean permeability would be 0.045 ft/year (4.3×10^{-8} cm/sec). Adopting an average, or effective vertical permeability for the marl of 0.1 ft/year is therefore reasonably conservative.

Total porosity of the marl has been calculated for 19 samples, and the average value of those samples is 47.5 percent. Recent studies at the University of Waterloo[(6)] show that for clays the effective porosity (the porosity affecting the rate of ground water movement) is essentially equal to total porosity.

Applying the values above for the three controlling parameters -- hydraulic gradient (1.447), average permeability (0.1 ft/year), and effective porosity (47.5 percent) -- the average vertical ground water velocity in the marl is calculated to be 0.31 ft/year, and the time required to traverse 38 ft of marl would be 123 years. Taking into account retardation (discussed previously in paragraph 2.4.13.1.1), this travel time is sufficient to reduce all radionuclides in a worst case spill below the maximum permissible concentrations set forth in Appendix B, Table II, Column 2 of 10 CFR 20.1 - 20.601 (which applies to routine, <u>continuous</u> releases).

2.4.13.2 Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents in Surface Waters

The only potentially radioactive tanks above grade are the refueling water storage tank, the reactor makeup water storage tank, and the condensate storage tanks. These tanks are designed and constructed to meet Seismic Category 1 requirements. Each tank has an approximately 2-ft-thick wall and 21-in.-thick roofs constructed of reinforced concrete. The tanks are lined with stainless steel liner plates. High level alarms are provided in the control room to alert the operator of a potential overflow condition. The tanks are surrounded by trenches to collect potential overflow. Provisions are made for sampling water collected in the trenches, and temporary connections are utilized to transfer potentially contaminated water to the liquid radwaste system. Thus no liquid effluent will be released to surface water from these tanks.

The radwaste processing facility described in subsection 1.2.2 is designed with a berm sufficient to contain the maximum liquid radwaste flow into the building for a period of greater than 30 minutes. A postulated leak or rupture of piping or components in the radwaste processing facility could potentially spray liquid over the berm and through gaps in the building wall around the door(s), resulting in a release outside the building. Bounding airborne and ground water releases are analyzed in paragraphs 15.7.2.5 and 15.7.3.4, respectively. Surface water runoff via the storm drain system (described in subsection 2.4.1) was analyzed and found to result in an exclusion area whole body dose less than the consquences of a liquid radwaste tank failure shown in table 15.7.2-2. Subsequent releases through the debris basin shown in drawing CX2D45V002 to the Savannah River were found to result in concentrations which would not exceed 10 CFR 20.1 - 20.601 limits at the nearest potable water intake.

An analysis of a radioactive release due to a failure of the most critical radwaste storage tank is provided in subsection 15.7.3.

The only direct discharge of radioactive liquids to surface water is from the waste monitor tanks, as discussed in subsection 11.2.3. The release point is shown in drawings AX4DB152-2 and AX4DB152-3. Normally, the discharge from the liquid waste processing system is combined with blowdown from the circulating water cooling system, the nuclear service cooling water system, and the steam generators as well as other station liquid wastes. If this flow is not sufficient to meet the Offsite Dose Calculation Manual limitations, additional river water is added to the discharge line to further dilute the radioactive wastes. Dilution factors are discussed in paragraph 11.2.3.4. An inadvertent release via the waste processing system or the steam generator blowdown system is prevented by interlocks between radiation monitors RE-018 and RE-021 (section 11.5) and associated downstream, air-operated valves. If the radioactivity in the streams exceeds a predetermined setpoint, then the streams are automatically isolated. In addition, locked closed valves in the waste stream ensure that administrative control of radioactive discharges is maintained.

Should an inadvertent discharge from the waste processing system occur, the release of one entire tank volume results in a release less than the normal annual releases discussed in subsection 11.2.3 and is within the limits found in the Offsite Dose Calculation Manual.

Discharge from the steam generator blowdown system (subsection 10.4.8) initially flows to the waste water retention basin. From there, it is pumped to the blowdown sump where it is combined with circulating water dilution flow and other blowdown flows before being discharged to the river. Upstream of the waste water retention basin, the steam generator blowdown is monitored for radioactivity by radiation monitor RE-021. If the monitor detects radioactivity exceeding a predetermined setpoint, the discharges to the waste water retention basin are automatically terminated.

2.4.13.3 References

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2.4.14 TECHNICAL SPECIFICATIONS AND EMERGENCY OPERATING REQUIREMENTS

There are no requirements for emergency protective measures designed to minimize the impact of hydrology-related events on safety-related facilities, and none will be incorporated into the Technical Specifications and emergency procedures.

TABLE 2.4.1-1

SAVANNAH RIVER SUBBASINS AND DRAINAGE AREAS ABOVE VEGP

Subbasin Number	Drainage Subbasin Gauging Station Description	Subbasin Hydrograph Area (mi^2)	Unit Duration (h)
1	Tallulah River at Burton Dam, Ga.	115.0	6
2	Savannah River at Hartwell Dam, Ga.	1850.0	6
3	Savannah River at Calhoun Falls, S.C.	788.0	6
4	Broad River at Carlton Bridge, Ga.	760.0	6
5	Broad River at Bell, Ga.	670.0	6
6	Savannah River at Clark Hill Dam	1740.0	6
7	Stevens Creek near Modoc, S.C.	545.0	6
8	Savannah River at Stevens Creek Dam, S.C.	484.0	6
9	Savannah River at Butler Creek, Augusta, Ga.	328.0	6
10	Savannah River at VEGP	735.0	6
TOTAL		8015.0	

TABLE 2.4.1-2 (SHEET 1 OF 5)

SAFETY-RELATED STRUCTURES AND ACCESS TO THEM

Building	Level	Elevation (ft)	Access Opening	Material
Control	1	220	Rollup door	Steel slats
Control	1	220	Emergency exit	Hollow metal
Control	1	220	Personnel access	Steel plate door
Control	1	220	TSC personnel access	Hollow metal
Control	1	220	TSC emergency exit	Hollow metal
Control	1	220	TSC equipment door	Hollow metal
Control	2	240	TSC roof access	Hollow metal
Control	3	260	Roof access door	Hollow metal
Control	4	280	Stairwell door	Hollow metal
Control	4	280	Elevator machine room	Hollow metal
Control	4	280	Unit 2 HVAC room	Hollow metal
Control	4	280	Unit 1 HVAC room	Hollow metal
Control	5	300	Roof access door	Hollow metal
North MSIV Control (Unit 1)	1	220	Access door	Hollow metal
North MSIV Control (Unit 2)	1	220	Access door	Hollow metal
Auxiliary	1	220	Emergency exit	Hollow metal
Auxiliary	1	220	Personnel missile door	Steel plate door

TABLE 2.4.1-2 (SHEET 2 OF 5)

Building	Level	Elevation (ft)	Access Opening	Material
Auxiliary	1	220	Railroad missile door	Steel plate door
Auxiliary	1	220	Emergency exit	Hollow metal
Auxiliary	1	220	Emergency exit	Hollow metal
Auxiliary	1	220	Emergency exit	Hollow metal
Auxiliary (Unit 2 South MSIV)	1	220	Access gate	Hollow metal
Auxiliary (Unit 1 South MSIV)	1	220	Access gate	Hollow metal
Auxiliary	3	260	Roof access(Unit 2)	Hollow metal
Auxiliary	3	260	Elevator machine room (Unit 2)	Hollow metal
Auxiliary	3	260	Roof access(Unit 1)	Hollow metal
Auxiliary	3	260	Elevator machine room (Unit 1)	Hollow metal
Containment (Unit 1)	B	183 ft 10 1/2 in.	Emergency escape door	Hollow metal
Containment (Unit 1)	1	220	Equipment building access	Hollow metal
Containment (Unit 1)	1	220	Containment equipment hatch	Steel hatch
Containment (Unit 2)	B	183 ft 10 1/2 in.	Emergency escape door	Hollow metal
Containment (Unit 2)	1	220	Equipment building access	Hollow metal

TABLE 2.4.1-2 (SHEET 3 OF 5)

Building	Level	Elevation (ft)	Access Opening	Material
Containment (Unit 2)	1	220	Containment equipment hatch	Steel hatch
Diesel generator (Unit 1)	1	220	Access door	Steel plate
Diesel generator (Unit 1)	1	220	Access door	Steel plate
Diesel generator (Unit 1)	1	220	Access door	Steel plate
Diesel generator (Unit 1)	1	220	Access door	Steel plate
Diesel generator (Unit 2)	1	220	Access door	Steel plate
Diesel generator (Unit 2)	1	220	Access door	Steel plate
Diesel generator (Unit 2)	1	220	Access door	Steel plate
Diesel generator (Unit 2)	1	220	Access door	Steel plate
Auxiliary feedwater pumphouse (Unit 1)	1	220	Access door	Steel plate
Auxiliary feedwater pumphouse (Unit 1)	1	220	Access door	Steel plate

TABLE 2.4.1-2 (SHEET 4 OF 5)

Building	Level	Elevation (ft)	Access Opening	Material
Auxiliary feedwater pumphouse (Unit 1)	1	220	Access door	Steel plate
Auxiliary feedwater pumphouse (Unit 1)	1	220	Access door	Steel plate
Auxiliary feedwater pumphouse (Unit 2)	1	220	Access door	Steel plate
Auxiliary feedwater pumphouse (Unit 2)	1	220	Access door	Steel plate
Auxiliary feedwater pumphouse (Unit 2)	1	220	Access door	Steel plate
Auxiliary feedwater pumphouse (Unit 2)	1	220	Access door	Steel plate
Train A NSCW pumphouse (Unit 1)	1	220	Access door	Steel plate
Train B NSCW pumphouse (Unit 1)	1	220	Access door	Steel plate
Train A NSCW pumphouse (Unit 2)	1	220	Access door	Steel plate

TABLE 2.4.1-2 (SHEET 5 OF 5)

Building	Level	Elevation (ft)	Access Opening	Material
Train A NSCW pumphouse (Unit 2)	1	220	Access door	Steel plate

TABLE 2.4.1-3 (SHEET 1 OF 2)

WATER CONTROL STRUCTURES SAVANNAH RIVER BASIN

Dam and Owner	River Mile	Storage (acre-ft)	Drainage Area (mi^2)	Earth Dike Length (ft)	Concrete Structure (Powerhouse Nonoverflow Wall) (ft)	Crest el (ft msl)	No. of Gates	Spillway Crest Length (ft)	Normal Pool el (ft msl)	Normal Head (ft)	Top of Dam el	Seismic Design Criteria	Spillway Design Criteria
New Savannah Bluff Lock and Dam Corps of Engineers	187.7	-	7508	-	-	-	-	-	13.1	152.6	-	-	-
Stevens Creek S.C. Elec. & Gas	209.7	-	-	-	-	-	-	-	-	29	187	-	-
Clark Hill Corps of Engineers	222.7	2,510,000 @ el 335	6144	3398	1186	300	23	1096	335	136	351	Yes	PMF
Richard B. Russell Corps of Engineers	260.2	600,000 @ el 475	2900	2640	1884	436	10	590	475	175	495	Yes	PMF
Hartwell Corps of Engineers	290.0	2,549,600 @ el 660	2088	15,952	1332	630	12	568	660	185	679	Yes	PMF
Yonah Georgia Power Co.	340.0	10,200 @ el 744.2	470	-	530	742.25	2'FB[(a)]	450	744.2	70.25	757	-	–
Tugaloo Georgia Power Co.	343.1	43,200 @ el 891.5	464	-	493	885	8'FB	357	891.5	144	905	-	–
Tallulah Falls Georgia Power Co.	346.7	2460 @ el 1500	186	-	110	1493	7'FB	316	1500	603.4	514	-	–
Mathis Georgia Power Co.	353.4	31,400 @ el 1689.6	151	370	312	1681.25	8.5'FB	285	1689.6	189.6	1704	-	–
Nacoochee Georgia Power Co.	362.1	8200 @ el 1752.5	136	-	350	1752.5	open	140	1752.5	62.2	1765	-	–

TABLE 2.4.1-3 (SHEET 2 OF 2)

Dam and Owner	River Mile	Storage (acre-ft)	Drainage Area (mi^2)	Earth Dike Length (ft)	Concrete Structure (Powerhouse Nonoverflow Wall) (ft)	Crest el (ft msl)	No. of Gates	Spillway Crest Length (ft)	Normal Pool el (ft msl)	Normal Head (ft)	Top of Dam el	Seismic Design Criteria	Spillway Design Criteria
Burton Georgia Power Co.	366.4	108,000 @ el 1866.6	118	-	845.5	1860	6.5'FB	197	1866.6	114.1	1873	-	-
Keowee Duke Power Co.	341.0+	940,000 @ el 800;	439	3500	-	765	4	176	800	140	815	-	-
Little River Duke Power Co.	366.0+	see Keowee	see Keowee	1750	- -	see Keowee	-	-	see Keowee	see Keowee	see Keowee	-	-
Jocassee Duke Power Co.	357.0+	1,100,000 @ el 1110	148	1800	-	1077	2	-	1100	310	1125	Yes	PMF

a. FB = flash board.

TABLE 2.4.2-1 (SHEET 1 OF 4)

GAUGING STATION RECORDS SAVANNAH RIVER BASIN
SAVANNAH RIVER AT AUGUSTA, GA.
ANNUAL FLOOD PEAKS[(a)]

Calendar Year	Month	Day	Discharge (ft^3/s)
1796	-	-	360,000
1840	May	28	270,000
1852	Aug.	29	250,000
1865	Jan.	11	240,000
1875	Dec.	30	86,400
1877	Apr.	14	119,000
1877	Nov.	23	51,500
1879	Aug.	3	44,000
1879	Dec.	16	102,000
1881	Mar.	18	130,000
1882	Sept.	12	93,300
1883	Jan.	22	111,000
1884	Apr.	16	81,000
1885	Jan.	26	77,000
1886	May	21	135,000
1887	July	31	173,000
1888	Sept.	11	303,000
1889	Feb.	19	149,000
1890	Feb.	27	48,500
1891	Mar.	10	197,000
1892	Jan.	20	140,000
1893	Feb.	14	60,000
1894	Aug.	7	54,000
1895	Jan.	11	106,000
1896	Jan.	10	107,000
1897	Apr.	6	93,300
1898	Sept.	2	117,000
1899	Feb.	8	112,000
1900	Feb.	15	138,000
1901	Apr.	4	124,000
1902	Mar.	1	175,000

TABLE 2.4.2-1 (SHEET 2 OF 4)

Calendar Year	Month	Day	Discharge (ft^3/s)
1903	Feb.	9	147,000
1904	Aug.	10	63,000
1905	Feb.	14	64,800
1906	Jan.	5	96,600
1906	Oct.	5	52,000
1908	Aug.	27	307,000
1909	June	5	87,300
1910	Mar.	2	69,800
1911	Apr.	14	32,800
1912	Mar.	17	234,000
1913	Mar.	16	156,000
1913	Dec.	31	48,000
1915	Jan.	20	61,000
1916	Feb.	3	82,400
1917	Mar.	6	68,000
1918	Jan.	30	45,500
1918	Dec.	24	128,000
1919	Dec.	11	133,000
1921	Feb.	11	129,000
1922	Mar.	12	92,000
1923	Feb.	28	59,700
1924	Apr.	6	56,400
1925	Jan.	20	150,000
1926	Jan.	20	55,300
1926	Dec.	30	39,000
1928	Aug.	17	226,000
1929	Mar.	6	191,000
1929	Oct.	2	350,000
1930	Nov.	17	26,100
1932	Jan.	9	93,800
1933	Feb.	9	48,200
1934	Mar.	5	73,200
1935	Mar.	14	63,700
1936	Apr.	8	258,000
1937	Jan.	4	90,200
1938	July	26	65,300

TABLE 2.4.2-1 (SHEET 3 OF 4)

Calendar Year	Month	Day	Discharge (ft^3/s)
1939	Mar.	2	82,400
1940	Aug.	15	252,000
1941	July	8	52,200
1942	Mar.	23	115,000
1943	Jan.	20	132,000
1944	Mar.	22	141,000
1945	Apr.	26	62,100
1946	Jan.	9	109,000
1947	Jan.	22	90,200
1948	Feb.	10	76,100
1949	Nov.	30	172,000
1950	Oct.	9	32,500
1951	Oct.	22	41,400
1952	Mar.	6	39,300
1953	May	8	35,200
1954	Mar.	30	25,500
1955	Apr.	15	23,900
1956	Apr.	12	18,600
1957	May	7	18,000
1958	Apr.	18	66,300
1959	June	8	28,500
1960	Feb.	14	34,900
1961	Apr.	2	34,800
1962	Jan.	9	32,500
1963	Mar.	23	31,300
1964	Apr.	9	87,100
1965	Dec.	27	34,600
1966	Mar.	6	39,300
1967	Aug.	25	26,500
1968	Jan.	12	35,900
1969	Apr.	21	45,600
1970	Apr.	1	25,200
1971	Mar.	5	63,900
1972	Jan.	20	33,700
1973	Apr.	8	40,200

TABLE 2.4.2-1 (SHEET 4 OF 4)

Calendar Year	Month	Day	Discharge (ft^3/s)
1974	Feb.	23	32,900
1975	Mar.	25	45,600
1976	June	5	33,300
1977	Apr.	7	34,200
1978	Jan.	26	43,100
1979	Feb.	27	37,300

a. U.S. Geological Survey, Surface Water Supply of the United States, Part 2, Vol 1, and Water Resources Yearly Data for the State of Georgia.

TABLE 2.4.2-2

PROBABLE MAXIMUM PRECIPITATION

Time (min)	Maximum Rainfall (in.)	Corresponding Intensity (in./h)
1	2.0	120
2	2.8	84
4	4.0	60
6	4.9	49
8	5.5	41
10	6.2	37
20	9.0	27
40	12.5	19
60	15.0	15
120	22.0	11
240	31.0	8

TABLE 2.4.2-3

YARD DRAINAGE
RECOMMENDED 100-YEAR RAINFALL CRITERIA[a]

Duration (min)	Intensity (in./h)
5	12.8
6	12.2
8	11.1
10	10.1
12	9.4
14	8.8
15	8.6
16	8.4
18	7.9
20	7.6
22	7.2
24	6.9
26	6.6
28	6.4
30	6.1
35	5.6
40	5.2
60	4.0
120	2.4
180	1.7
360	1.0

a. Hershfield, D. M., "Rainfall-Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years," U.S. Weather Bureau Technical Paper No. 40, 1961.

TABLE 2.4.3-1

PROBABLE MAXIMUM PRECIPITATION

Drainage Area (mi^2)	Probable Maximum Precipitation (in.)				
	6 h	12 h	24 h	48 h	72 h
10	31.0	37.2	43.8	48.2	51.0
200	23.0	28.0	34.7	38.8	42.0
1000	16.7	22.5	29.0	33.0	35.0
5000	9.7	14.0	19.5	24.0	27.0
10,000	7.5	11.2	16.0	20.0	23.5
20,000	5.4	8.8	12.5	16.4	19.5
For Drainage Area above VEGP					
8015	8.1	12.0	16.9	21.0	24.4

Location (approximate): latitude N 33°03'; longitude W 81°55'

TABLE 2.4.3-2

6-h SEQUENTIAL INCREMENTAL PROBABLE MAXIMUM PRECIPITATION (in.)

	6 h	12 h	18 h	24 h	30 h	36 h	42 h	48 h	54 h	60 h	66 h	72 h
Cumulative PMP	8.0	12.4	15.0	16.8	18.2	19.4	20.6	21.8	22.3	23.0	23.7	24.4
Incremental	8.0	4.4	2.6	1.8	1.4	1.2	1.2	1.2	0.5	0.7	0.3	0.7
Sequential	8.0	4.4	2.6	1.8	1.4	1.2	1.2	1.2	0.7	0.7	0.5	0.3
6-h duration interval	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0

TABLE 2.4.3-3

SPATIAL ISOHYETAL ADJUSTMENTS FOR
THE THREE GREATEST 6-h PROBABLE MAXIMUM PRECIPITATION
(percent)

		Isohyet Designation						
		P	A	B	C	D	E	F
Drainage area enclosed (mi^2)	10	35	270	800	3200	8700	19000	
Isohyet values in percent of PMP, greatest 6-h increase		258	227	174	137	95	55	32
Isohyet values in percent of PMP, second greatest 6-h increase		165	152	130	116	97	86	73
Isohyet values in percent of PMP, third greatest 6-h increase		113	111	107	104	100	97	93

TABLE 2.4.3-4

SPATIALLY ADJUSTED THREE GREATEST 6-h PROBABLE MAXIMUM PRECIPITATION

	Isohyet Designation						
	P	A	B	C	D	E	F
Drainage area enclosed (mi^2)	10	35	270	800	3200	8700	19700
Greatest 6-h PMP increment isohyet value (in.)	17.54	15.44	11.83	9.32	6.46	3.74	2.18
Second greatest 6-h PMP increment isohyet value (in.)	6.11	5.62	4.81	4.29	3.59	3.18	2.70
Third greatest 6-h PMP increment isohyet value (in.)	2.49	2.44	2.35	2.29	2.20	2.13	2.05

TABLE 2.4.3-5

6-h INCREMENTAL MAXIMUM PRECIPITATION ARRANGED IN CRITICAL SEQUENCE

Time from beginning of storm (h)	6	12	18	24	30	36	42	48	54	60	66	72
For critical sequence rank of 6-h increase, 1=highest, 12=lowest	9	7	8	3	2	1	5	4	6	10	11	12
Subbasin drainage identification number	6-h incremental PMP arranged in critical sequence (in.)											
Tallulah River at Burton Dam, Ga., No. 1	0.7	1.2	1.2	2.17	3.39	5.10	1.4	1.8	1.2	0.7	0.5	0.3
Savannah River at Hartwell Dam, Ga., No. 2	0.7	1.2	1.2	2.17	3.45	5.44	1.4	1.8	1.2	0.7	0.5	0.3
Savannah River at Calhoun Falls, S.C., No. 3	0.7	1.2	1.2	2.25	4.17	8.94	1.4	1.8	1.2	0.7	0.5	0.3
Broad River at Carlton Bridge, Ga., No. 4	0.7	1.2	1.2	2.18	3.49	5.66	1.4	1.8	1.2	0.7	0.5	0.3
Broad River at Bell, Ga., No. 5	0.7	1.2	1.2	2.18	5.73	3.51	1.4	1.8	1.2	0.7	0.5	0.3
Savannah River at Clark Hill Dam, No. 6	0.7	1.2	1.2	2.22	7.19	3.82	1.4	1.8	1.2	0.7	0.5	0.3
Stevens Creek near Modoc, S.C., No. 7	0.7	1.2	1.2	2.19	6.0	5.36	1.4	1.8	1.2	0.7	0.5	0.3
Savannah River at Stevens Creek Dam, No. 8	0.7	1.2	1.2	2.20	6.35	3.63	1.4	1.8	1.2	0.7	0.5	0.3
Savannah River at Butler Creek (Augusta, Ga.), No. 9	0.7	1.2	1.2	2.17	5.14	3.39	1.4	1.8	1.2	0.7	0.5	0.3
Savannah River at VEGP, No. 10	0.7	1.2	1.2	2.16	4.95	3.35	1.4	1.8	1.2	0.7	0.5	0.3

TABLE 2.4.3-6

CUMULATIVE PROBABLE MAXIMUM PRECIPITATION FOR VARIOUS STORM POSITIONS AND CALCULATION PROCEDURES

Subbasin number and description	Cumulative PMP for Storm Position No. 1 Based on Hydromet No. 51 Procedure (in.)					Cumulative PMP for Storm Position No. 2 Based on Hydromet No. 51 Procedure (in.)					Cumulative PMP for Storm Position No. 1 Based on Hydromet No. 52 Procedure (in.)				
	6 h	12 h	24 h	48 h	72 h	6 h	12 h	24 h	48 h	72 h	6 h	12 h	24 h	48 h	72 h
1. Tallulah River at Burton Dam, Ga.	0.52	1.92	3.65	7.65	10.74	1.09	3.52	5.16	11.21	13.57	1.6	2.2	4.0	7.3	10.7
2. Savannah River at Hartwell Dam, Ga.	1.53	4.27	6.22	11.37	14.3	3.71	8.07	10.72	15.11	18.35	1.8	3.4	6.2	11.6	14.3
3. Savannah River at Calhoun Falls, S.C.	6.32	12.4	15.74	19.26	22.36	13.36	19.28	25.7	28.93	31.42	7.6	10.8	14.8	19.4	22.4
4. Broad River at Carlton Bridge, Ga.	1.38	3.9	5.91	11.1	13.95	3.42	7.83	10.26	14.66	18.13	2.0	3.9	5.9	10.5	14.0
5. Broad River at Bell, Ga.	2.82	6.73	8.98	13.75	17.06	4.28	9.03	11.94	16.04	19.39	3.4	5.9	9.4	13.8	17.0
6. Savannah River at Clark Hill Dam	8.6	13.04	17.33	21.03	24.02	5.45	10.65	13.92	17.84	21.0	9.0	12.4	16.6	21.2	24.0
7. Stevens Creek near Modoc, S.C.	6.53	12.25	15.63	19.05	22.29	2.91	7.01	9.37	13.86	17.56	7.7	10.8	14.8	19.4	22.2
8. Savannah River at Stevens Creek Dam, S.C.	5.39	11.25	14.07	17.55	21.23	2.15	5.39	7.94	12.7	16.1	6.2	9.2	13.4	18.0	
9. Savannah River at Butler Creek, Ga.	3.92	9.21	11.4	15.4	19.67	1.26	3.78	5.64	11.41	14.06	4.8	7.8	11.7	16.1	19.6
10. Savannah River at VEGP, Ga.	2.04	5.18	7.66	12.54	15.85	0.73	2.67	4.26	9.64	12.11	1.5	5.0	8.2	12.6	15.8

TABLE 2.4.7-1

MINIMUM WATER TEMPERATURE OF THE SAVANNAH RIVER

Gauging Station	Period of Record	Statistical Parameter	Value Exceeded P Percent of Time	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
Savannah River at Augusta, Ga., river mile 187.4	1974 - 1980	Minimum temperature (°C)	V95	5.2	5.2	7.0	12.0	14.0	17.0	19.0	21.0	20.0	18.0	13.0	8.7
			V90	6.0	5.5	7.9	12.0	15.0	17.0	20.0	21.0	21.0	18.0	13.0	9.7
			V75	7.5	6.5	9.3	13.0	16.0	18.0	20.0	22.0	22.0	19.0	15.0	11.0
			V70	7.8	6.8	9.8	13.0	16.0	18.0	20.0	22.0	22.0	20.0	16.0	11.0
			V50	9.2	8.1	11.0	14.0	17.0	19.0	21.0	22.0	23.0	20.0	17.0	12.0
			V25	10.0	10.0	12.0	15.0	18.0	20.0	22.0	23.0	23.0	21.0	18.0	14.0
			V10	12.0	11.0	13.0	17.0	19.0	21.0	23.0	24.0	24.0	22.0	19.0	15.0
Savannah River at Jackson, S.C., river mile 156.8	1972 - 1980	Minimum temperature (°C)	V95	5.1	5.3	8.3	13.0	16.0	18.0	21.0	22.0	21.0	17.0	13.0	8.8
			V90	6.0	5.8	9.4	13.0	16.0	18.0	21.0	22.0	21.0	18.0	14.0	9.4
			V75	7.5	7.2	11.0	14.0	17.0	19.0	22.0	22.0	22.0	19.0	15.0	10.0
			V70	7.9	7.5	11.0	14.0	17.0	20.0	22.0	23.0	22.0	19.0	15.0	11.0
			V50	9.2	8.7	12.0	15.0	18.0	21.0	23.0	23.0	23.0	20.0	17.0	12.0
			V25	11.0	11.0	13.0	16.0	19.0	22.0	23.0	24.0	24.0	21.0	18.0	13.0
			V10	12.0	12.0	14.0	18.0	20.0	22.0	24.0	24.0	24.0	22.0	19.0	14.0
Savannah River below Steel Creek near Millet, S.C., river mile 138.8	1972 - 1980	Minimum temperature (°C)	V95	5.5	5.4	8.4	13.0	16.0	18.0	21.0	22.0	21.0	17.0	12.0	8.6
			V90	6.4	6.0	9.7	14.0	17.0	19.0	22.0	23.0	22.0	18.0	13.0	9.4
			V75	8.0	7.2	11.0	14.0	18.0	20.0	23.0	23.0	23.0	19.0	15.0	10.0
			V70	8.3	7.6	11.0	15.0	18.0	20.0	23.0	24.0	23.0	19.0	15.0	11.0
			V50	9.6	8.9	13.0	15.0	19.0	21.0	24.0	24.0	24.0	20.0	16.0	12.0
			V25	11.0	11.0	14.0	17.0	20.0	22.0	24.0	25.0	25.0	21.0	18.0	13.0
			V10	13.0	12.0	15.0	18.0	21.0	23.0	25.0	26.0	25.0	23.0	19.0	15.0
Savannah River at Burton's Ferry near Millhaven, Ga., river mile 118.7	1961 - 1969, 1973	Minimum temperature (°C)	V95	5.9	7.0	8.6	13.0	16.0	19.0	21.0	23.0	21.0	17.0	12.0	7.0
			V90	6.3	7.2	9.0	14.0	17.0	20.0	22.0	23.0	21.0	18.0	13.0	8.0
			V75	7.7	8.0	11.0	15.0	18.0	21.0	23.0	24.0	22.0	19.0	14.0	9.0
			V70	7.9	8.2	11.0	15.0	18.0	21.0	24.0	24.0	23.0	20.0	15.0	10.0
			V50	8.7	9.0	13.0	16.0	20.0	22.0	24.0	25.0	24.0	21.0	16.0	11.0
			V25	10.0	11.0	14.0	18.0	21.0	24.0	25.0	26.0	25.0	22.0	17.0	13.0
			V10	12.0	12.0	15.0	19.0	23.0	25.0	26.0	27.0	26.0	23.0	18.0	14.0

TABLE 2.4.12-1

WATER-BEARING PROPERTIES OF MATERIALS UNDERLYING VEGP AND VICINITY

Stratigraphic Unit	Thickness (ft)	Physical Characteristics	Water-Bearing Properties
Alluvium (Recent)	0-30	Tan to gray sand, silt, clay, and gravel	Very little water
Terraces (Pleistocene)	0-30	Sand, silt, clay, and gravel	Ground water supply moderate to none
Alluvium (Pliocene)	0-20	Gravel and sandy clay	Little or no water
Hawthorne Formation (Miocene)	0-80	Sandy clay, interbedded lenses of gravel, and numerous clastic dikes	Small to moderate amounts of ground water
Barnwell Group (Eocene-Jackson Age)	0-90	Red, brown, yellow, and buff sand and clay	Moderate amounts of ground water sufficient for domestic use
Lisbon Formation (Eocene-Claiborne Age)	0-250	Fine to coarse glauconitic sand interbedded with clay, sandy marl, or limestone, and siliceous limestone lenses	Ground water supply moderate to large in sands only; sufficient for limited industrial use
Huber/Ellenton Formation (Paleocene)	0-100	Dark, sandy lignitic micaceous clay containing disseminated crystals of gypsum; dark coarse sand and white kaolin	Ground water supply small in some areas
Tuscaloosa (Cretaceous)	0-700	Cross-bedded micaceous quartzite and arkosic sand and gravel, interbedded with clay and white kaolin	Large supplies of good quality ground water; yields up to 2000 gal/min from gravel packed wells
Newark Group (Triassic)	Unknown at VEGP	Sandstone, siltstone, graywacke, and claystone with sections of flaglomerate or conglomerate	Low yields to wells
Basement rocks (Paleozoic and Precambrian)	Unknown at VEGP	Granite, gneiss, schist, slate, and volcanic rocks	Limited supplies, ground water in fractures

Table derived from references 3 and 8.

TABLE 2.4.12-2

GROUND WATER USE

Service	No. Required	Maximum Water Demands/Capacity
Fire protection storage tank	2	300,000 gal/each
Construction water tank	1	30,000 gal (primary)
Batch plant water storage tank (plant construction only)	1	10,000 gal (backup)
Construction water tank	1	25,000 gal (backup)
Potable and sanitary water tank	1	25,000 gal
Potable and sanitary use	-	350 gal/min
Well water storage tank	1	300,000 gal
Utility water use	-	330 gal/min
Makeup demineralizer	1	440 gal/min process flow
Demineralizer water storage tank	1	250,000 gal
Reactor makeup water storage tanks	2	165,000 gal/each
Condensate storage tanks	4	480,000 gal/each
Demineralized water use	-	550 gal/min
Nuclear service cooling water towers	4	268 gal/min per tower at 4 cycles

TABLE 2.4.12-3 (SHEET 1 OF 2)

WATER QUALITY ANALYSES - VEGP WELLS

Observation Wells

(Values in parts per million unless otherwise designated)

Observation well	34[a]	121[a]	135[b]	124[c]	129[c]	142[c]	143[c]	24[b]	24[b]
Date sampled	10/06/71	09/22/71	10/31/71	10/12/71	10/13/71	10/13/71	10/13/71	07/21/71	10/06/71
Laboratory No.	17947	17840	17980	17978	17979	17981	17982		17944
Date of analysis	10/11/71	09/27/71	10/14/71	10/14/71	10/14/71	10/14/71	10/14/71		10/14/71
Constituents									
Sodium (Na)	13.3	15.9	16.3	6.8	18.2	5.2	21.6	6.75	4.6
Magnesium (Mg)	5.4	7.5	8.3	8.3	8.3	6.8	5.8	1.7	11.2
Potassium (K)	3.2	3.2	2.8	0.7	2.3	1.2	1.9	1.49	1.5
Calcium (Ca)	15.2	12.8	28.8	24.8	23.6	25.6	23.0	23.0	4.0
Carbonate (CO_3)	0.0	0.0	14.4	0.0	9.6	0.0	14.4	---	16.8
Bicarbonate (HCO_3)	95.2	84.2	114.7	96.4	93.9	103.7	96.4	---	45.1
Hydroxide (OH)	0.0	---	0.0	0.0	0.0	0.0	0.0	---	0.0
Sulfate (SO_4)	7.7	3.6	17.4	3.5	20.5	7.1	25.8	36.6	7.6
Chloride (Cl)	3.0	1.0	4.0	4.0	3.0	2.0	2.0	4.0	2.0
Nitrate (NO_3)	0.07	0.0	1.4	0.0	0.85	0.22	1.14	0.0	0.09
Fluoride (F)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	---	0.09
Total phosphate (PO_4)	0.15	0.66	0.26	0.26	0.28	0.26	0.63	---	0.5
Iron (Fe)	0.57	0.31	0.12	0.5	0.15	0.34	0.3	0.21	0.15
Silica (SiO_2)	7.9	8.0	9.2	5.2	5.4	7.7	8.4	6.75	7.4
Total dissolved solids	127.9	112.5	106.0	126.4	155.8	134.2	169.5	234.9	89.6
Conductivity (μmho)	140.0	130.0	180.0	130.0	170.0	120.0	150.0	390.0	115.0
pH	7.9	6.1	8.2	8.2	8.3	7.9	8.6	11.3	9.1
Free carbon dioxide	2.0	110.0	0.0	0.0	0.0	2.0	0.0	0.0	0.0
Total alkalinity as $CaCO_3$	78.0	69.0	106.0	79.0	85.0	85.0	91.0	119.0	51.0
Total hardness as $CaCO_3$	60.0	63.0	106.0	96.0	66.0	92.0	62.0	94.0	56.0
Temperature (°F)	---	65.0	---	---	---	---	68.0	---	68.0

TABLE 2.4.12-3 (SHEET 2 OF 2)

WATER QUALITY ANALYSES - VEGP WELLS

Supply Wells

(Values in parts per million unless otherwise designated)

Supply well	MU-1[d]	MU-1[d]	MU-2[d]	MU-2[d]	MU-2[d]	CW-2[e]	CW-3[e]	SB[e]
Date sampled	12/19/77	5/2/85	9/12/78	11/27/84	12/3/84	5/19/82	7/29/81	11/8/82
Date analyzed		5/2/85	9/19/78	3/1/85		6/14/82	7/30/81	11/9/82
Constituents								
Sodium (Na)	35.5	--	35.6	6.52	44.3	9.8	--	--
Magnesium (Mg)	0.63	--	0.4	0.35	0.36	1.48	--	--
Potassium (K)	1.4	--	1.6	0.47	0.81	2.3	--	--
Calcium (Ca)	3.8	3.8	2.2	2.0	1.96	20.7	--	--
Sulfate (SO_4)	4.8	3.12	3.0	6.7	6.0	--	--	6.7
Chloride (Cl)	2.2	2.67	3.13	2.1	2.4	4.74	3.58	198.4
Nitrate (NO_3)	--	0.2	0.04	--	--	0.10	0.014	1.15
Total phosphate ($P0_4$)	--	--	0.031	--	--	--	--	--
Iron (Fe)	--	0.13	0.21	0.06	--	1.28	0.14	3.07
Silica (SiO_2)	12.4	12.6	12.4	4.85	--	--	--	--
Total dissolved solids	114.0	--	201.0	112.0	101.0	96.0	--	568.0
Conductivity (µmho)	187.0	171.0	155.0	76.0	160.0	--	--	--
pH	7.9	7.52	8.03	7.8	7.4	7.74	7.61	10.49
Free carbon dioxide	--	--	--	--	--	--	3.3	--
Total alkalinity as $CaCO_3$	81.5	83.7	81.7	79.4	--	64.7	127.0	185.4
Total hardness as $CaCO_3$	12.1	4.6	7.3	7.0	--	57.8	116.0	18.8

a. Open to the Tertiary aquifer; flowing at the surface.

b. Open to the Tertiary aquifer; not flowing at the surface.

c. Open to the water table aquifer system (above the marl).

d. Open to the Cretaceous (confined) aquifer.

e. Open to the Tertiary (confined) aquifer.

TABLE 2.4.12-4

WATER QUALITY ANALYSES - DOMESTIC WELLS

(Values in parts per million unless otherwise designated)

Sample identification	Well 1	Well 3	Well 8	Well 10	Well 9	Well 6	Well 14	Well 14	Well 7
Date sampled	09/17/71	09/17/71	09/18/71	09/18/71	09/18/71	09/18/71	09/20/71	09/22/71	09/18/71
Laboratory No.	17803	17804	17806	17808	17807	17809	17812	17839	17805
Date of analysis	09/21/71	09/21/71	09/21/71	09/21/71	09/21/71	09/21/71	09/21/71	09/21/71	09/21/71
Constituents									
Sodium (Na)	2.0	2.2	1.9	2.2	3.3	6.0	3.8	3.6	2.9
Magnesium (Mg)	0.2	1.0	1.9	1.7	3.2	2.7	2.4	1.7	1.7
Potassium (K)	0.29	0.33	0.42	0.46	1.41	4.48	2.41	2.0	0.5
Calcium (Ca)	26.8	28.8	28.0	39.6	46.8	55.2	50.4	51.2	50.8
Bicarbonate (HCO_3)	78.1	95.2	98.8	118.3	152.5	128.1	156.2	158.6	159.8
Sulfate (SO_4)	0.6	0.0	0.0	7.5	4.8	5.2	4.9	4.9	5.3
Chloride (Cl)	0.0	0.0	0.0	0.0	2.0	6.0	0.0	3.0	0.0
Nitrate (NO_3)	0.42	0.0	0.0	0.0	0.0	7.8	0.0	0.0	0.0
Fluoride (F)	0.0	0.0	0.0	0.0	0.0	0.0	0.17	0.0	0.0
Total phosphate (PO_4)	0.6	0.66	0.79	0.46	0.44	1.7	1.0	0.35	0.92
Iron (Fe)	0.18	0.06	0.07	0.1	0.07	0.06	0.27	0.24	0.11
Silica (SiO_2)	6.9	7.3	8.6	15.5	14.8	6.1	13.4	15.5	11.8
Total dissolved solids	96.6	111.75	115.8	156.3	191.2	191.3	195.9	201.4	193.8
Conductivity (μmho)	115.0	120.0	130.0	165.0	200.0	260.0	205.0	220.0	200.0
pH	7.5	7.7	7.6	7.6	7.5	7.3	7.4	6.2	6.9
Free carbon dioxide	5.5	3.0	4.0	4.5	10.0	11.0	10.0	150.0	30.0
Total alkalinity as $CaCO_3$	64.9	78.0	81.0	97.0	125.0	105.0	128.0	130.0	131.0
Total hardness as $CaCO_3$	68.0	76.0	78.0	106.0	130.0	149.0	136.0	135.0	134.0
Temperature (°F)	72.0	---	---	---	69.0	---	67.0	68.0	68.0

TABLE 2.4.12-5

WATER QUALITY ANALYSES - SPRINGS

(Values in parts per million unless otherwise designated)

Sample identification	Spring 1	Spring 2	Spring3	Spring 5	Spring 6	Spring 7	Spring 4
Date sampled	09/17/71	09/22/71	09/22/71	09/20/71	09/22/72	09/18/72	09/18/72
Laboratory No.	17802	17835	17837	17811	17836	17838	17810
Date of analysis	09/21/71	09/21/71	09/21/71	09/21/71	09/21/71	09/21/71	09/21/71
Constituents							
Sodium (Na)	1.6	1.6	2.0	3.0	10.8	2.1	2.3
Magnesium (Mg)	0.0	2.2	2.4	0.0	4.6	1.9	0.2
Potassium (K)	0.29	0.25	0.0	0.54	0.33	0.0	0.5
Calcium (Ca)	2.0	0.8	2.8	37.2	29.2	2.8	22.0
Bicarbonate (HCO_3)	9.8	7.3	18.3	125.7	102.5	11.0	63.4
Sulfate (SO_4)	2.5	3.0	1.0	1.0	0.5	5.0	1.0
Chloride (Cl)	2.0	2.0	1.0	1.0	5.0	5.0	0.0
Nitrate (NO_3)	0.0	0.0	0.0	0.21	0.56	0.0	1.7
Fluoride (F)	0.0	0.0	0.09	0.0	0.0	0.09	0.26
Total phosphate (PO_4)	1.07	0.59	0.55	1.07	0.19	0.29	0.89
Iron (Fe)	0.12	0.11	0.08	0.06	0.04	0.96	0.06
Silica (SiO_2)	4.4	4.2	4.4	8.0	8.7	5.4	7.3
Total dissolved solids	21.8	20.0	28.1	147.9	136.8	32.3	83.8
Conductivity (µmho)	17.0	30.0	34.0	170.0	150.0	28.0	110.0
pH	6.4	6.1	6.0	7.2	6.0	6.1	7.1
Free carbon dioxide	6.0	12.0	25.0	11.0	160.0	15.0	8.0
Total alkalinity as $CaCO_3$	8.0	6.0	15.0	103.0	84.0	9.0	52.0
Total hardness as $CaCO_3$	7.0	11.0	17.0	99.0	92.0	15.0	56.0
Temperature (°F)	68.0	65.0	66.0	66.0	66.0	66.0	66.0

TABLE 2.4.12-6

WATER QUALITY ANALYSES - SURFACE WATER

(Values in parts per million unless otherwise designated)

Sample location[a]	Location 1	Location 2	Location 3	Location 3	Location 4
Date sampled	09/20/71			10/14/71	10/14/71
Laboratory No.	17813			17985	17987
Date of analysis		05/07/71	06/03/71	10/14/71	10/14/71
Constituents					
Sodium (Na)	1.8	2.9	1.9	1.9	2.2
Magnesium (Mg)	2.2	9.1	2.9	8.8	5.4
Potassium (K)	0.48	1.2	0.17	0.5	0.3
Calcium (Ca)	9.2	9.6	8.4	8.0	16.8
Carbonate (CO_3)	0.0	0.0	0.0	0.0	0.0
Bicarbonate (HCO_3)	18.3	---	---	31.7	29.3
Sulfate (SO_4)	0.77	5.2	2.6	2.8	1.5
Chloride (Cl)	2.0	4.0	2.0	1.0	2.0
Nitrate (NO_3)	0.0	0.79	0.75	0.39	0.44
Fluoride (F)	0.0	---	---	0.0	0.0
Total phosphate (PO_4)	1.05	---	---	0.16	0.26
Iron (Fe)	0.66	0.36	0.12	0.24	0.09
Silica (SiO_2)	6.5	10.1	5.6	5.4	4.9
Total dissolved solids	38.4	72.9	50.6	53.0	96.7
Conductivity (μmho)	37.0	60.0	57.0	50.0	90.0
pH	6.6	7.4	7.2	6.9	9.4
Free carbon dioxide	8.0	3.0	3.5	6.0	0.0
Total alkalinity as $CaCO_3$	15.0	30.0	28.0	26.0	58.0
Total hardness as $CaCO_3$	32.0	68.0	35.0	56.0	64.0
Temperature (°F)	76.0	---	---	69.0	---

a. Location 1 - Tributary to Daniels Branch.
Location 2 - Daniels Branch at road culvert.
Location 3 - Beaverdam Creek at River Road bridge.
Location 4 - Mathes Pond.

TABLE 2.4.12-7A (SHEET 1 OF 2)

OBSERVATION WELLS

Observation Wells in Water Table Aquifer

Well No.	History: Installed (YR)	History: Current Status	Coordinates N	Coordinates E	Ground Surface El (ft)	Top of PVC El (ft)	Depth Top of Marl (ft)	Screen Interval (ft)
129	1971	Inactive 1993	8856	9576	215.9	215.3	77	92 - 97
142	1971	Inactive 1993	8283	8262	231.2	224.5	92	85 - 95
179	1971	Inactive 1993	9059	7779	274.8	275.9	130	111 - 131
800	1979	Inactive 1993	8850	11011	213.7	215.3	83	69 - 89
801	1979	Inactive 1993	7656	10733	212.8	215.8	82	62.5- 82.5
802A	1985	Active	7196	10194	216.9	218.9	87.5	77 - 87
803A	1979	Grouted, 2013	7085	8898	218.3	220.3	82	57 - 77
804	1979	Inactive 1993	6597	8227	224.1	226.1	87	60 - 80
805A	1979	Active	6672	10403	232.7	236.95	124	95 - 115
806B	1980	Active	8821	9726	214.8	221.45	77[c]	55 - 65
807A	1980	Grouted 1988	9047	9835	213.6	218.0	77[c]	65 - 75
808	1985	Active	9625	9300	207.0	216.47	66.3	45.5 - 68
809	1985	Inactive 1993	8320	7860	222.8	224.23	89	69.3 - 90
LT-1B	1985	Active	8388	9304	(4)	221.75	83.3	65.2 - 84.7
LT-7A	1985	Active	8151	9317	(4)	222.24	87	65 - 87
LT-12	1985	Active	7775	9600	(4)	219.20	79	58 - 79
LT-13	1985	Active	8135	10110	219.0[d]	220.67	89	68 - 90
42D	1971	Grouted, 1974	8403	9571	209.7	212.7	72	60 - 70
42E	1971	Grouted, 1974	8408	9580	209.6	-	72	45 - 55
124	1971	Inactive, 1979 (buried)	6896	9527	260.2	259.9	128	160 - 170
138	1971	Grouted, 1985	8000	8500	225.2	225.1	87	5 - 82
140	1971	Grouted, 1985	7846	8702	222.4	223.5	89	81 - 96
141	1971	Grouted, 1985	7860	8293	230.4	223.6	97	90 - 100

TABLE 2.4.12-7A (SHEET 2 OF 2)

(Observation Wells in Water Table Aquifer)

Well No.	History Installed (YR)	Current Status	Coordinates N	E	Ground Surface El[(a)] (ft)	Top of PVC El[(b)] (ft)	Depth Top of Marl (ft)	Screen Interval (ft)
143	1971	Grouted, 1980	8283	8738	224.5	225.0	81	78.5 - 88.5
145G	1971	Inactive, 1974 (buried)	7792	7063	218.7	219.7	82	72 - 82
176	1971	Inactive, 1974 (buried)	7117	11423	196.4	196.9	77	65 - 75
177	1971	Grouted, 1980	8560	10865	213.0	213.0	79	60 - 80
178	1971	Grouted, 1978	9958	8994	240.4	240.5	89	71 - 91
243	1972	Grouted 1985	9154	8618	213.0	225.2	71	60 - 80
244	1972	Inactive, 1979 (buried)	8835	8859	212.6	213.7	72	51 - 71
245	1972	Grouted, 1978	8501	9917	207.6	209.0	71.5	52 - 92
247	1972	Inactive, 1972 (buried)	5750	5424	211.3	--	82	70 - 80
248	1972	Inactive, 1972 (buried)	7469	5111	166.8	--	70.3	60 - 70
249	1972	Inactive, 1979 (buried)	8826	10154	193.0	194.0	57.9	47 - 57
802	1979	Grouted, 1985	7201	10199	215.8	217.7	91	69 - 89
LT-1A	1979	Grouted, 1985	8388	9300	204.8	206.9	69	65.4 - 75.4
LT-7	1979	Grouted, 1985	8151	9323	197.2	200.4	63	58.2 - 68.2

a. Elevations determined at time of drilling.

b. Elevations are current (Nov 1985) or latest prior to well abandonment.

c. Depth based on log of well 129.

d. Observation wells in backfill; not completed to grade, November 1985.

TABLE 2.4.12-7B (SHEET 1 OF 2)

Observation Wells In Confined Aquifers

Well No.	History: Installed (YR)	History: Current Status	Coordinates N	Coordinates E	Ground Surface El[a] (ft)	Top of PVC El[b] (ft)	Depth Bottom Marl (ft)	Screen Interval (ft)
Unnamed sands of Lisbon Formation (Tertiary)								
27	1971	Inactive 1995	8622	13931	210.0	209.0	148	180 - 190
29	1971	Inactive 1995	9975	12392	193.0	193.4	126	200 - 210
34	1971	Inactive 1995	12180	10846	86.0	90.5	(3)	90 - 100
850A	1984	Inactive 1995	11723	10494	225.9	227.8	135	169 - 179
851A	1984	Inactive 1995	8868	7066	262.7	264.3	195	269 - 279
852	1984	Inactive 1995	5993	13380	200.7	202.1	153.5	199 - 209
853	1984	Inactive 1995	11020	9204	227.6	229.1	145	195 - 205
854	1984	Inactive 1995	9899	7917	236.8	238.3	153	197 - 207
855	1984	Inactive 1995	7159	13951	218.0	219.4	173	219 - 229
856	1984	Inactive 1995	4927	12558	186.7	188.1	155	176 - 186
24	1971	Grouted	7850	9092	216.0	216.4	145	210 - 220
26	1971	Grouted, 1984	5963	15197	203.0	203.8	158	190 - 200
31	1971	Grouted, 1984	8764	11237	211.0	216.8	151	200 - 210
32	1971	Grouted, 1984	9784	9572	214.0	217.4	139	200 - 210
33	1971	Grouted, 1984	11834	10864	238.0	238.6	157	210 - 220
42A	1971	Grouted, 1974	8380	9535	210.6	213.0	137	140 - 150

TABLE 2.4.12-7B (SHEET 2 OF 2)

Observation Wells In Confined Aquifers

Well No.	History: Installed (YR)	History: Current Status	Coordinates: N	Coordinates: E	Ground Surface El[(a)] (ft)	Top of PVC El[(b)] (ft)	Depth Bottom of Marl (ft)	Screen Interval (ft)
101A	1971	Grouted, 1974	7950	9515	210.6	211.7	138	190 - 200
121	1971	Grouted, 1985	10467	12195	88.8	--	(3)	78 - 88
135	1971	Grouted	8992	8742	200.5	201.3	124.8	160 - 170
144	1971	Grouted	10403	12124	103.2	103.2	38	38.5- 48.5
147	1971	Grouted, 1978	7965	8471	226.2	227.4	152	280 - 300
181	1971	Grouted, 1985	8744	6833	258.3	--	194.5	190 - 200
246	1972	Grouted, 1984	10532	6553	210.4	213.5	179.7	220 - 230
Tuscaloosa Formation (Cretaceous)								
TW-1	1972	Inactive 1995-1998	7738	9984	218.5	928	140	506 - 850
MU-2	1977	Inactive 1995	9500	9135	214.5	850	150	450 - 820

a. Elevations determined at time of drilling.

b. Elevations are current (Nov 1985) or latest prior to well abandonment.

c. Marl not present.

TABLE 2.4.12-7C

Observation Wells In Marl

Well No.	History Installed (YR)	Current Status	Coordinates N	Coordinates E	Ground Surface El[a] (ft)	Top of PVC El[b] (ft)	Marl Interval (ft)	Screen Interval (ft)
900	1985	Inactive 1995	7538	10119.5	216.3	218.05	92.6 - 148[c]	113.8 - 140.7
901	1985	Inactive 1995	7538	10104.5	215.58	220.75	91.6 - 148[c]	122.0 - 128.0
902	1985	Inactive 1995	7543.5	10110.5	215.97	221.11	91.0 - 148[c]	101.5 - 108.0
903	1985	Inactive 1993	8480	8900	215.75	216.73	78.0 - 148[c]	127.0 - 133.0
904B	1985	Inactive 1993	8464	8885	215.75	216.31	78.8 - 148[c]	90.0 - 96.0
905	1985	Inactive 1993	8450	8900	215.75	216.71	77.3 - 148[c]	109.8 - 116.0
42B	1971	Grouted, 1974	8386	9544	210.4	--	72 - 137	120 - 130
42C	1971	Grouted, 1974	8398	9563	210.0	--	72 - 137	80 - 90

a. Elevations determined at time of drilling.

b. Elevations are current (Nov 1985) or latest prior to well abandonment.

c. Bottom depth is interpolated from drawing AX6DD378.

TABLE 2.4.12-7D

Water Supply Wells

Well No.	Installed (YR)	Current Status	Coordinates N	Coordinates E	Ground Surface El[a] (ft)	Drilled Depth (ft)	Aquifer	Remarks
MU-1	1977	Active	9425	10531	196.9	851	Cretaceous	Cooling water makeup
MU-2A	1983	Active	8820	8400	225	884	Cretaceous	Cooling water makeup
TW-1	1972	Active	7738	9984	218.5	860	Cretaceous	Alternate cooling water makeup
CW-1	1976	Abandoned	6913	8919	255[b]	251	Tertiary	Construction supply
CW-2	1980	Abandoned	7452	6525	221	378	Tertiary	Construction supply
CW-3	1974	Active	3079	6645	210	220	Tertiary	Nuclear operations garage
SB	1981	Active	1666	15562	115[b]	340	Tertiary	Simulator building supply
PW	1973	Active	6880	14159	210[b]	100	Water table	Plant Wilson supply
IW-4	1989	Active	2979	6645	215[b]	370	Tertiary	Irrigation supply
SW-5	1990	Active	10704	8120	232[b]	200	Tertiary	Security training supply

a. Unless otherwise indicated, elevations shown are determined at time of drilling.

b. Estimated from topography, drawing AX6DD343.

TABLE 2.4.12-8

SUMMARY OF AQUIFER CHARACTERISTICS CALCULATIONS

Method of Analysis	Observation Point (s)	Calculated Transmissivity (gal/d/ft)	Storage Coefficient
Test Well Data (TW-1)			
Straight-line, distance drawdown	Pumping well observation points	158,000	(a)
Type-curve, time-drawdown	1	196,000	6.6×10^{-4}
Type-curve, time-drawdown	2	160,000	3.3×10^{-4}
Type-curve, time-drawdown	3	163,700	3.5×10^{-4}
Type-curve, time-drawdown	4	153,000	2.1×10^{-5}
Type-curve, time-drawdown	5	229,200	3.9×10^{-4}
Makeup Well Data MU-1 and MU-2)			
Type-curve, time-drawdown, MU-1	None	110,400	(a)
Type-curve, time recovery, MU-1	None	116,600	(a)
Type-curve, time drawdown, MU-2	MU-1	130,900	1.07×10^{-4}
Type-curve, time recovery, MU-2	MU-2	128,700	(a)

a. Storage coefficient calculated only from observation well data.

TABLE 2.4.12-9

OBSERVATION WELLS MONITORING PROGRAM 1995-PRESENT

Reading Frequency	Water Table Aquifer	Marl Aquiclude	Tertiary Aquifer	Cretaceous Aquifer
Quarterly	LT-1B LT-7A LT-12 LT-13	None	None	None
Semiannually	802A 805A 806B 808	None	None	None

Note: Details of well construction are in tables 2.4.12-7A, 7B, 7C, and 7D.

TABLE 2.4.12-10 (SHEET 1 OF 4)

PERMEABILITY TESTS
BLUE BLUFF MARL

Hole Number	Interval Tested (ft)	Permeability[a] (ft/yr)	Marl Interval (ft)	Remarks
Constant Head (Packer) Tests				
157	100.0-110.0	3.0	92.0-153.1	Tests rejected because of packer leakage. See results of tests in Hole 508.
	100.0-120.0	3.9		
	110.0-120.0	18.6-54.2		
	128.0-138.0	0		
	120.0-140.0	0		
170	104.5-124.5	0	92.0-152.0	
	110.0-130.0	0		
	120.0-140.0	0		
	130.5-150.5	0		
180	77.5-99.5	0	72.0-142.0	
	85.0-105.0	0		
	95.0-115.0	3.9		
	105.0-125.0	11.7		
245	80.0-100.0	0	71.5-135.5	
	82.0-102.0	0		
	86.0-106.0	0		
	103.0-123.0	0		
	110.0-130.0	0		
249	67.5-87.5	0	57.9-122.0	Tests questionable because of possible packer leakage.
	80.0-100.0	48.4		
	92.5-112.5	29		
501	76.5-96.5	0	74.0-150.0	
	88.0-113.0	0		
	114.0-130.0	0		
	135.0-150.0	0		
502	86.0-114.5	0	82.5-146.0	
	114.5-139.5	0		
	137.5-150.0	0		
503	63.5-82.0	0	58.0-121.5	
	66.0-102.0	0		
	81.0-102.0	0		
	100.0-122.0	0		

TABLE 2.4.12-10 (SHEET 2 OF 4)

Hole Number	Interval Tested (ft)	Permeability[a] (ft/yr)	Marl Interval (ft)	Remarks
Constant Head (Packer) Tests				
504	87.0-99.0	0	84.0-134.0	
	97.0-109.0	0		
	107.0-119.0	0		
	118.0-130.0	0		
	122.0-135.0	0		
505	148.0-160.0	0	147.0-187.0	
	157.0-167.0	0		
	166.0-178.0	0		
	175.0-187.0	0		
506	93.0-105.0	0	92.0-162.0	
	103.0-115.0	0		
	113.0-125.0	0		
	123.0-135.0	0		
	133.0-145.0	0		
	143.0-155.0	0		
	153.0-165.0	0		
507	112.0-124.0	0	111.0-180.5	
	125.0-137.0	0		
	135.0-147.0	0		
	140.0-152.0	0		
	150.0-162.0	0		
	160.0-172.0	0		
	165.0-177.0	0		
508	97.0-109.0	0	95.0-150.8	Drilled adjacent to Hole 157 to determine validity of original tests
	104.0-116.0	0		
	114.0-126.0	0		
	125.0-137.0	0		
	135.0-147.0	0		
	142.0-154.0	0		
510	95.0-107.0	0	93.0-154.0	
	105.0-117.0	0		
	115.0-127.0	0		
	125.0-137.0	0		
	135.0-147.0	0		
	141.0-153.0	0		

TABLE 2.4.12-10 (SHEET 3 OF 4)

Hole Number	Interval Tested (ft)	Permeability[a] (ft/yr)	Marl Interval (ft)	Remarks
Constant Head (Packer) Tests				
513	90.0-102.0	0	86.0-147.5	
	100.0-112.0	0		
	110.0-122.0	0		
	120.0-132.0	0		
	130.0-142.0	0		
518	124.0-129.0	0	77.5-139.7	
900	104.6-112.6	0	92.6-148	
	112.6-122.6	0		
	122.6-132.6	0		
	132.6-142.6	0		
	122.6-142.6	0		
901	118-128	0	91.6-148	
902	100-108	0	91-148	
903	85-96	0	78-148	
	96-106	0		
	106-116	0		
	116-126	0		
	126-133	0		
904B	85-96.7	0	78.8-148	
905	88.5-102.5	0	77.4-148	
	102.5-116	0		
P-1	11.0-31.0	0	4.0-33.0	
P-2	5.0-30.0	51	0.5-29.5	Analysis suggests leakage around packer.
P-3	7.1-17.0	0	7.0-39.5	Weathered marl
	17.0-37.0	0		
P-5	12.0-27.0	0	11.0-25.8	

TABLE 2.4.12-10 (SHEET 4 OF 4)

Hole Number	Interval Tested (ft)	Permeability[a] (ft/yr)	Marl Interval (ft)	Remarks
Well Parameter Tests				
P-1A	0.0-6.0	16	0.0-6.0	Weathered marl
P-3A	0.0-6.5	33	0.0-6.5	Weathered marl
Laboratory Permeability Tests[b]				
901	119.0	5.2×10^{-3}	91.6-148	Limestone
902	104.2	2.0×10^{0}	91 -148	Marl
903	108.2	2.0×10^{-1}	76 -148	Marl
903	112.7	5.0×10^{-1}		Marl and limestone Nodules
903	128.4	2.1×10^{0}		Marl
904B	92.3	2.5×10^{0}	78.8-148	Marl
905	91.6	1.5×10^{0}	77.3-148	Marl and limestone Nodules
905	96.7	8.8×10^{0}		Marl
905	107.5	1.4×10^{-1}		Marl
905	114.0	8.0×10^{-2}		Marl

a. Zero indicates no measurable water take.
b. Tests were performed by Harding Lawson Associates.

TABLE 2.4.12-11
POROSITY
(Blue Bluff Marl)

Boring No.	Sample Depth (ft)	Porosity Percent
102	125.8	48
	140.3	48
111	80.8	55
114	105.3	56
	80.8	54
	100.8	51
138A	97.0	56
	126.0	24
	134.0	36
	148.6	41
202	93.8-95.8	39
	134-136	41
203	82-84	62
	114.0	60
204	94.5	48
	132-134	42
216	84.5	47
	132.0	47

Note: Data from Law Engineering Testing Company, November 12, 1971.

TABLE 2.4.12-12 (SHEET 1 OF 2)

PERMEABILITY TEST RESULTS
(Barnwell Sands, Silts, and Clays; River Alluvial Sands, Silts, and Clays;
Unnamed Sands of Lisbon Formation)

Hole Number	Interval Tested (ft)	Permeability (ft/yr)	Material Tested and/or Remarks
Well Permeameter Tests			
183	50.0-60.0	200	Sand (Barnwell)
184	53.0-63.0	267	Sand (SW), clayey sand (SC) and clay (CL) (Barnwell)
P-4A	0 - 7.5	130	Silts/clay (river alluvium)
P-6A	0 - 4	260	Silts/clay (river alluvium)
Packer Tests			
P-6B	10-20	36,000[a]	Sands (river alluvium)
P-6C	20-30	21,000[a]	Sands (river alluvium)
P-6D	30-40	27,000[a]	Sands (river alluvium)
P-1	33-48	240	Unnamed sands Lisbon Formation
P-2	30-50	190	Unnamed sands Lisbon Formation
P-3	40-54.6	250	Unnamed sands Lisbon Formation
P-4	21-36	60	Unnamed sands Lisbon Formation
P-5	29-54	340	Unnamed sands Lisbon Formation
Laboratory Tests			
107A	13.8-14.4	302	Sand (SP); undisturbed sample
	34.0-36.0	9.8	Sand (SW); undisturbed sample
	49.0-51.0	19,973	Sand (SW); dry density = 83.1 per ft^3
		6,833	dry density = 84.0 per ft^3
		1,682	dry density = 91.0 per ft^3
	62.5-63.0	27.4	Sand (SW): undisturbed sample
S No. 10	Backfill	6,070	Percent compaction = 92.9
	(Grab	4,580	Percent compaction = 93.9
	sample)	4,400	Percent compaction = 95.7
		2,260	Percent compaction = 99.8

TABLE 2.4.12-12 (SHEET 2 OF 2)

Hole Number	Interval Tested (ft)	Permeability (ft/yr)	Material Tested and/or Remarks
Laboratory Tests			
S No. 11	Backfill (Grab sample)	4,110	Percent compaction = 91.2
		1,820	Percent compaction = 94.0
		1,430	Percent compaction = 97.0
		430	Percent compaction = 98.8

TABLE 2.4.12-13 (SHEET 1 OF 2)

PERMEABILITY TESTS
(Utley Limestone)

Observation Well No.	Tested Interval (ft)	Transmissivity (gpd/ft)	Permeability (ft/yr)	Remarks
1. Pumping Tests				
a. Well No. 1 pumped out at an average of 30 gal/min for 97 h				
1A	56-78	6,350	14,100	Theis curve match, maximum drawdown 1.92 ft
1B	68-78	25,700	125,400	Theis curve match, maximum drawdown 1.05 ft
1C	56-80	9,830	20,000	Theis curve match, maximum drawdown 2.49 ft
1D	56-80	21,700	44,100	Theis curve match, maximum drawdown 1.31 ft
1A,1B,1C,1D	59-70 (avg)	8,090	19,700	Distance-drawdown
b. Well 2B, pumped in at an average of 74 gal/min for 14 min				
2A	62-85	1,530	3,250	Semilog plot of recovery, maximum drawdown-6.22 ft
2. Falling Head (Variable Head) Tests				
Well No. 1	65-80	NA	5,800	Starting head = 36.7 ft
1A	63-78	NA	600	Starting head = 36.5 ft
Well No. 2	69-85	NA	980	Starting head = 44.1 and 10.6 ft (2 tests)
2A	70-85	NA	96	Starting head = 3.2 ft
2B	69-84	NA	360	Starting head = 0.6 and 0.9 (2 tests)
2C	65-85	NA	140	Starting head = 3.0 ft
2D	70-85	NA	2,100	Starting head = 1.8 ft

Table 2.4.12-13 (SHEET 2 OF 2)

Observation Well No.	Tested Interval (ft)	Transmissivity (gpd/ft)	Permeability (ft/yr)	Remarks
3. Constant Head Tests				
1A	56-78	NA	160	Total head = 61 ft
2A	56-85	NA	3,200	Total head = 64 ft
2B	56-84	NA	1,790	Total head = 75 ft
2D	56-85	NA	1,190	Total head = 77 ft

TABLE 2.4.12-14

POROSITY
Barnwell Sands, Silts, and Clays

Boring No. or Sample No.	Depth Interval (ft) or Sample Source	Porosity Percent	Remarks
102A	15.0-16.5	34	Silty sand-undisturbed sample
	35.0-36.5	47	Sand-undisturbed sample
	58.0-60.0	61	Clay-undisturbed sample
107A	13.2-15.2	45	Sand-undisturbed sample
	34.0-36.0	47	Sand-undisturbed sample
	62.0-64.0	52	Sand-undisturbed sample
138A	9-11	39	Silty sand-undisturbed sample
	14-16	40	Silty sand-undisturbed sample
	29-31	43	Silty, clayey sand-undisturbed sample
	34-36	38	Clayey sand-undisturbed sample
	49-51	49	Clayey sand-undisturbed sample
	59-60.5	44	Sand-undisturbed sample
204	18-19.3	40	Silty sand-undisturbed sample
226A	61-63	43	Sand-undisturbed sample
235	8-10	37	Sand-undisturbed sample
Sample No. 10	Backfill	39.4	@ 92.9 percent compaction
		38.8	@ 93.9 percent compaction
		37.6	@ 95.7 percent compaction
		35.0	@ 99.8 percent compaction
Sample No. 11	Backfill	36.9	@ 91.2 percent compaction
	Burrow area	34.9	@ 94 percent compaction
		32.9	@ 97 percent compaction
		31.6	@ 98.8 percent compaction

Note: Data from Law Engineering Testing Company, November 12, 1971 and August 31, 1984.

TABLE 2.4.12-15

PERMEABILITY OF BACKFILL

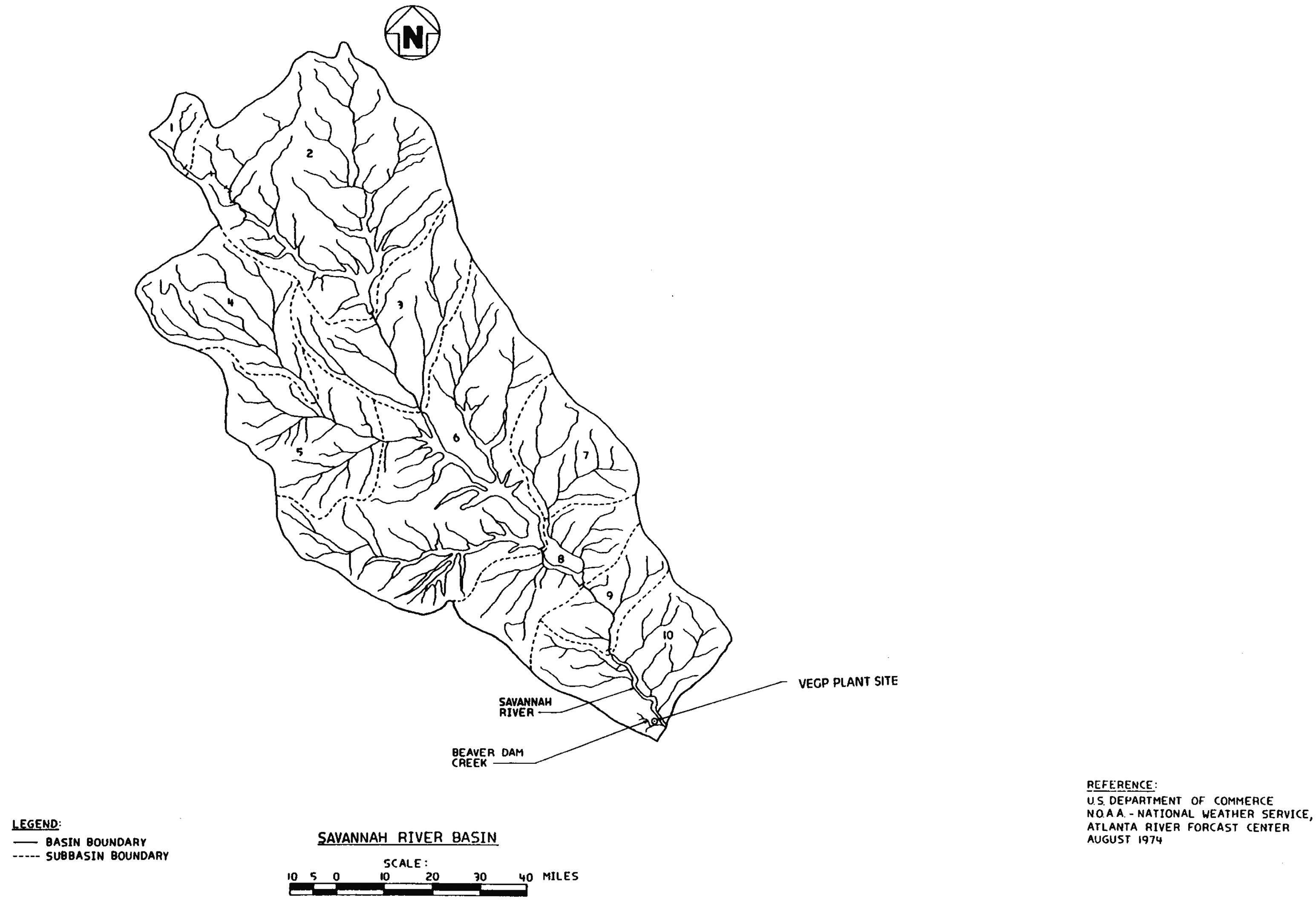
Well Number	Test Cycle	Transmissivity ft^2/d Test Cycle	Average	Open Interval ft	Permeability ft/yr
LT-1B	Insertion	59	65	19.5	1220
	Extraction	71			
LT-7A	Insertion	48	45	22	750
	Extraction	42			
LT-12	Insertion	29	28	21	480
	Extraction	26			
LT-13	Insertion	67	71	22	1180
	Extraction	75			

TABLE 2.4.13-1

Equilibrium Distribution Coefficient
Power Block Excavation Backfill
(ml/g)

Sample	Cesium^{137}	Strontium^{90}
1	385 $\pm$138	40.8 $\pm$6.0
2	1065 $\pm$160	94.7 $\pm$6.8
3	520 $\pm$31	76.0 $\pm$7.0
4	2134 $\pm$589	56.2 $\pm$20.3

NOTE: Values for each sample are the average of three replicates with the variability indicated. Testing conducted by Battelle Pacific Northwest Laboratories.



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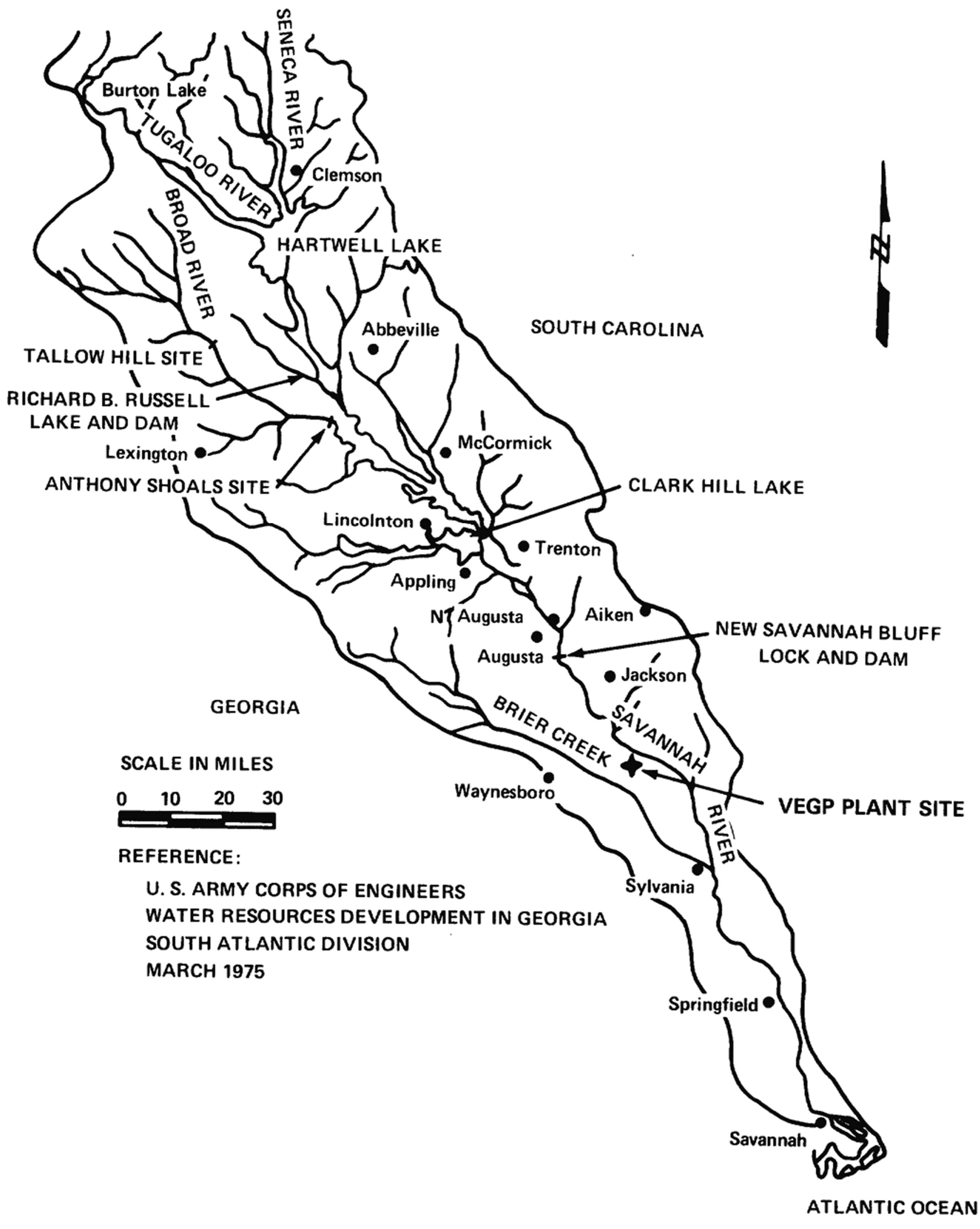
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

HYDROLOGIC MAP
SAVANNAH RIVER BASIN ABOVE SITE

FIGURE 2.4.1–1



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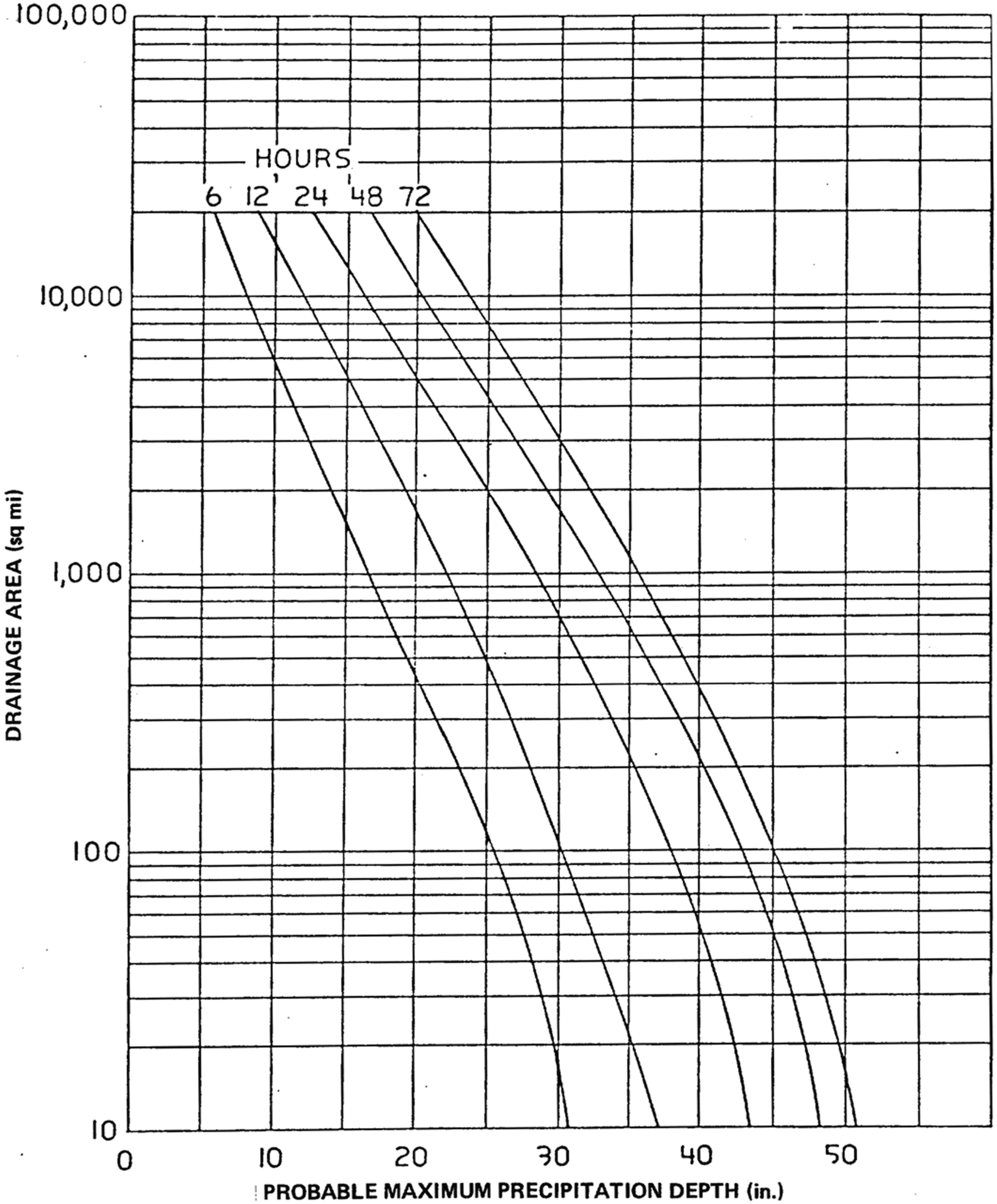
SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	SAVANNAH RIVER BASIN
		FIGURE 2.4.1–2



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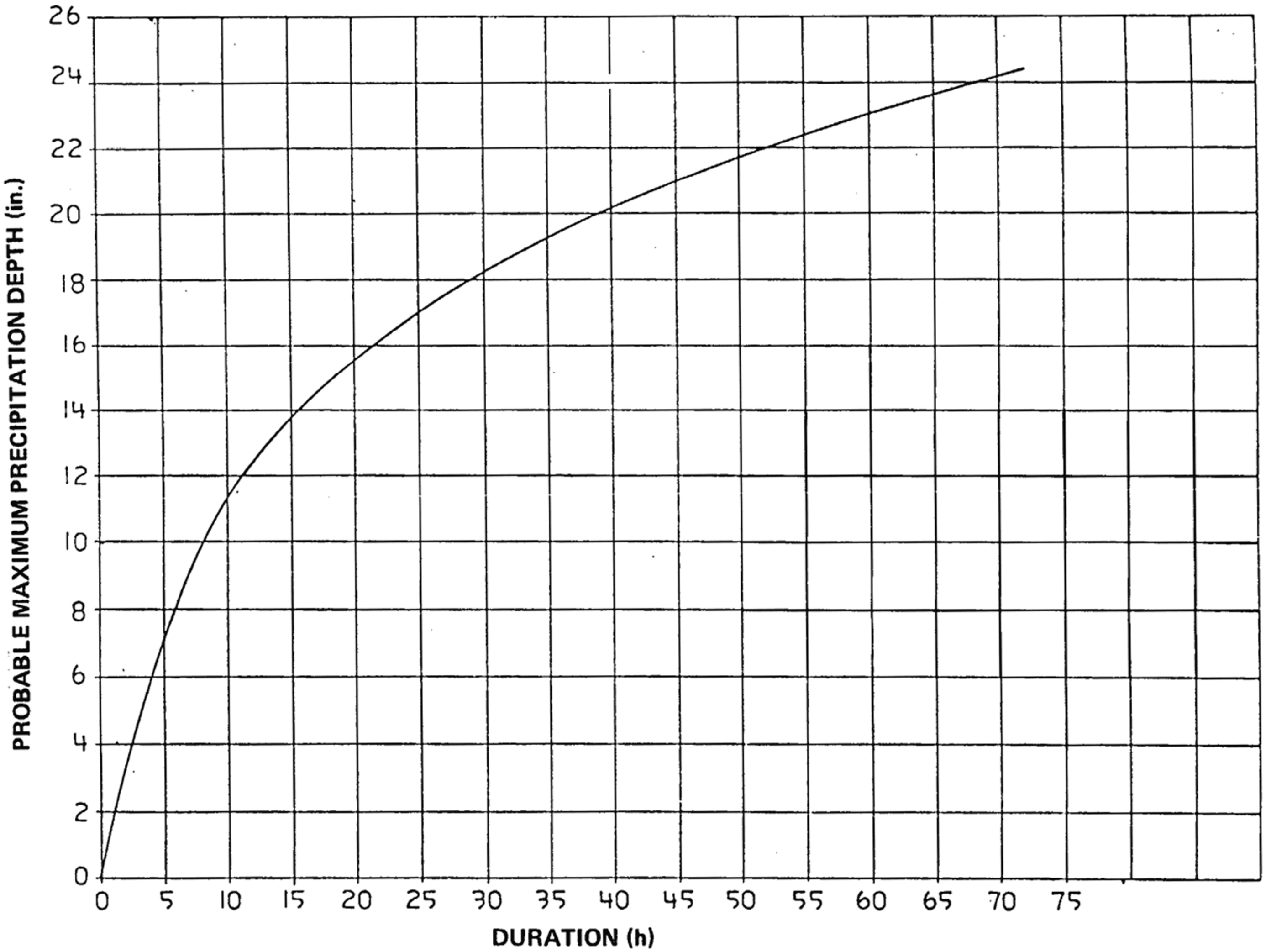
DEPTH–AREA–DURATION ENVELOPES

FIGURE 2.4.3–1



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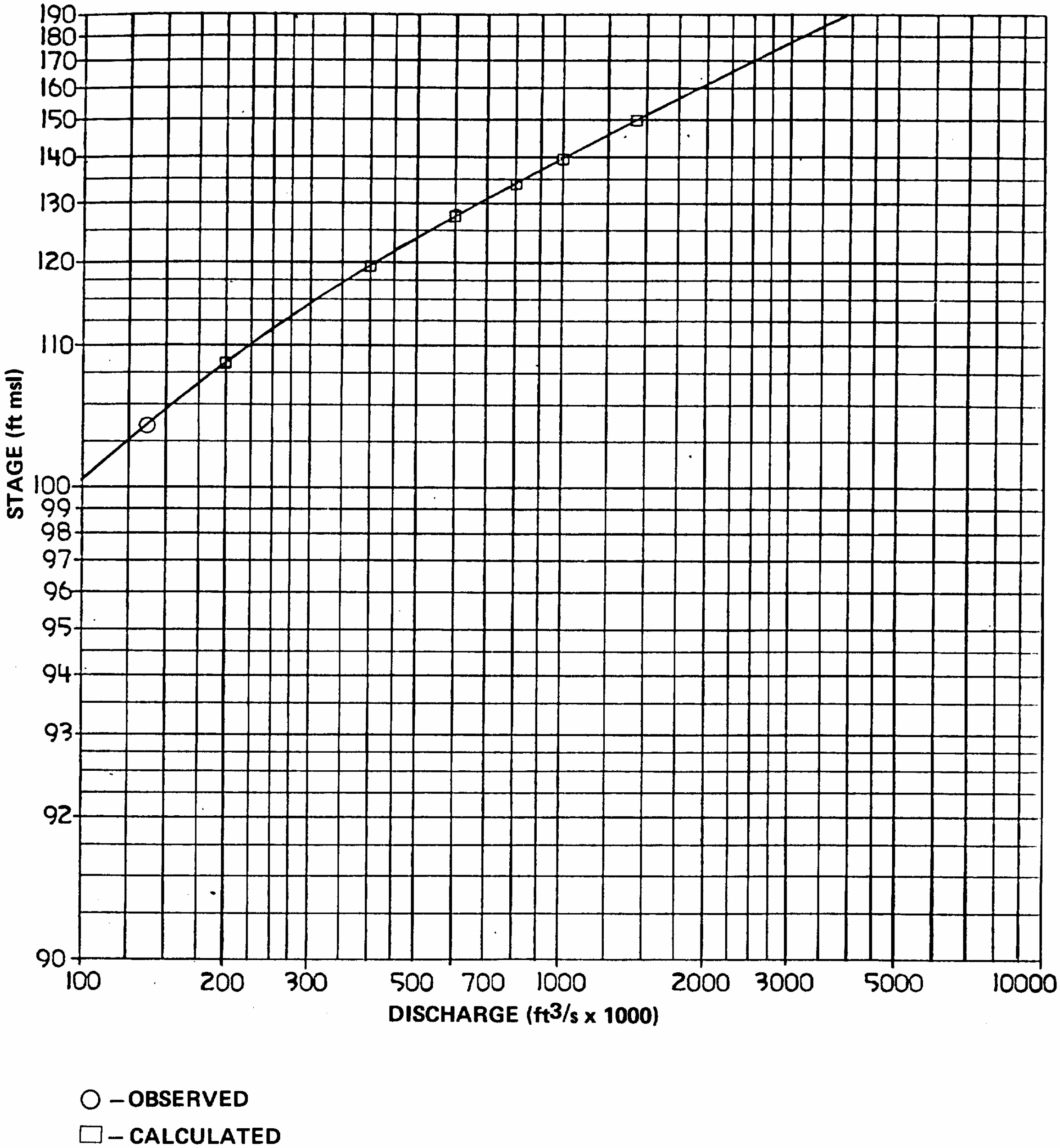
RMP DEPTH–DURATIN CURVE FOR
DRAINAGE (8015 MI^2) ABOVE THE PLANT

FIGURE 2.4.3–2



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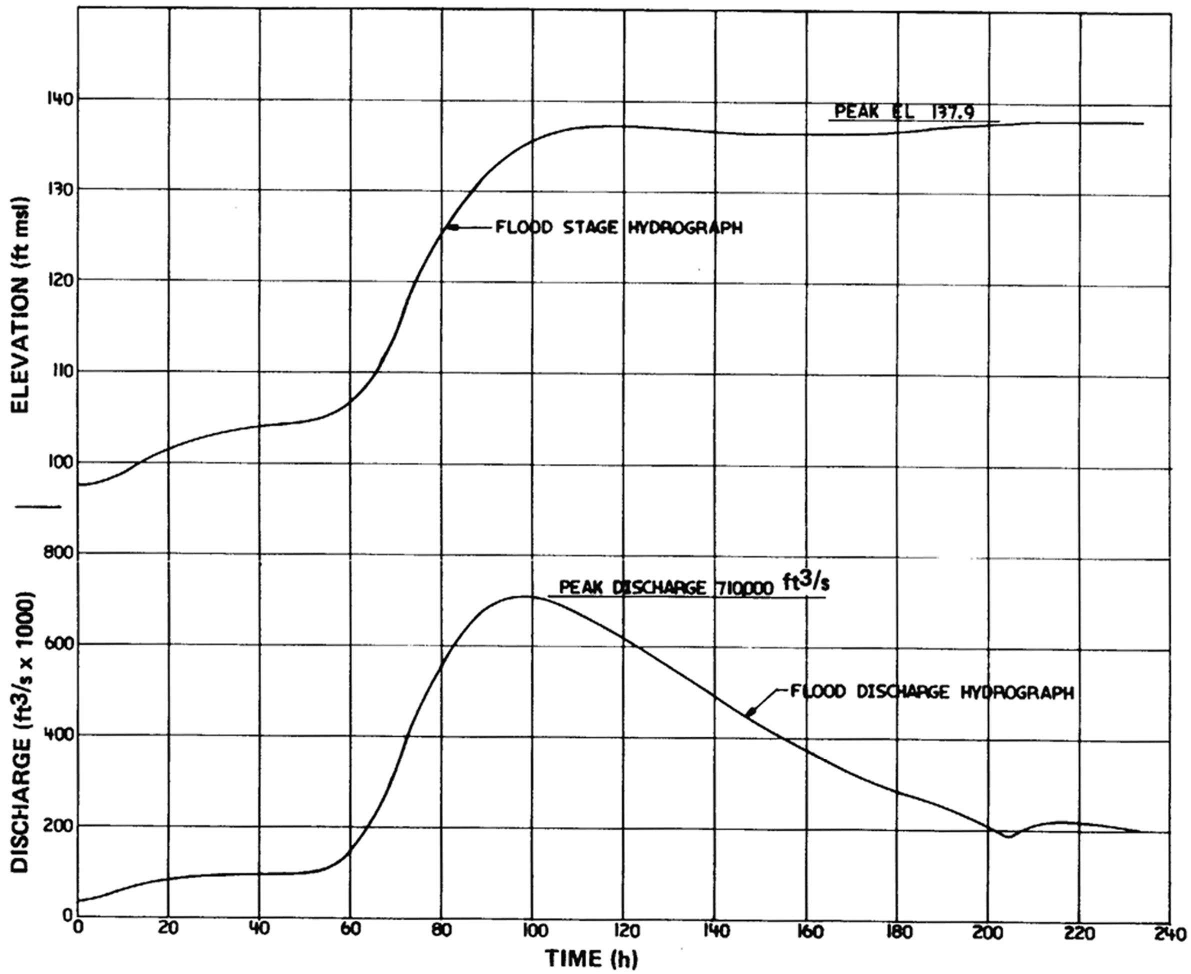
UNIT 1 AND UNIT 2

SAVANNAH RIVER STEADY FLOW CONDITION–DISCHARGE RELATION AT RIVER MILE 151.1

FIGURE 2.4.3–3



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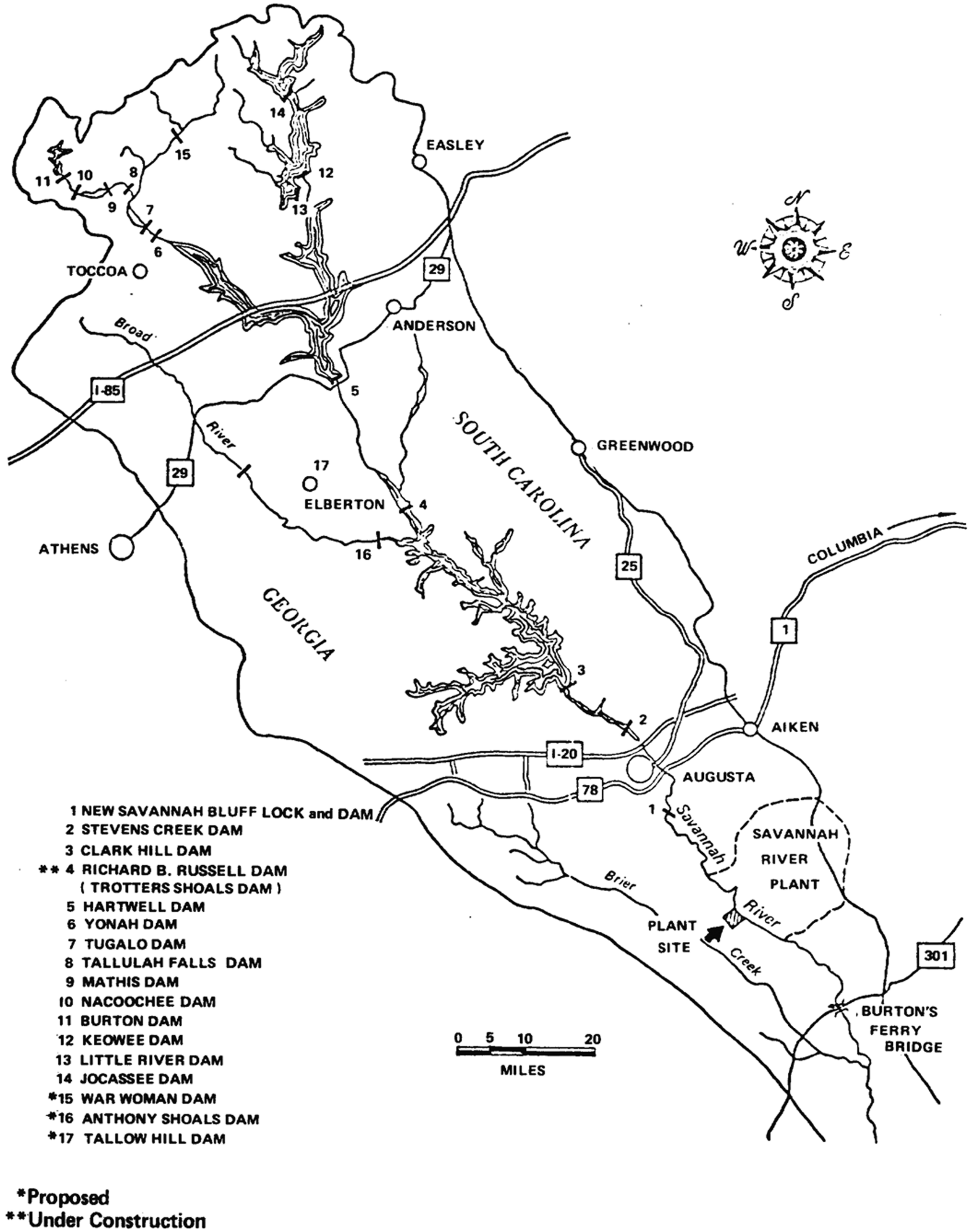
SOUTHERN COMPANY *Energy to Serve Your World®* VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	PROBABLE MAXIMUM FLOOD HYDROGRAPH AT RIVER MILE 151.1 BY ROUTING CORPS OF ENGINEERS PMF OUTFLOW HYDROGRAPH (VALLEY STORAGE TAKEN INTO ACCOUNT) AT CLARK HILL DAM THROUGH SAVANNAH RIVER
	FIGURE 2.4.3–4



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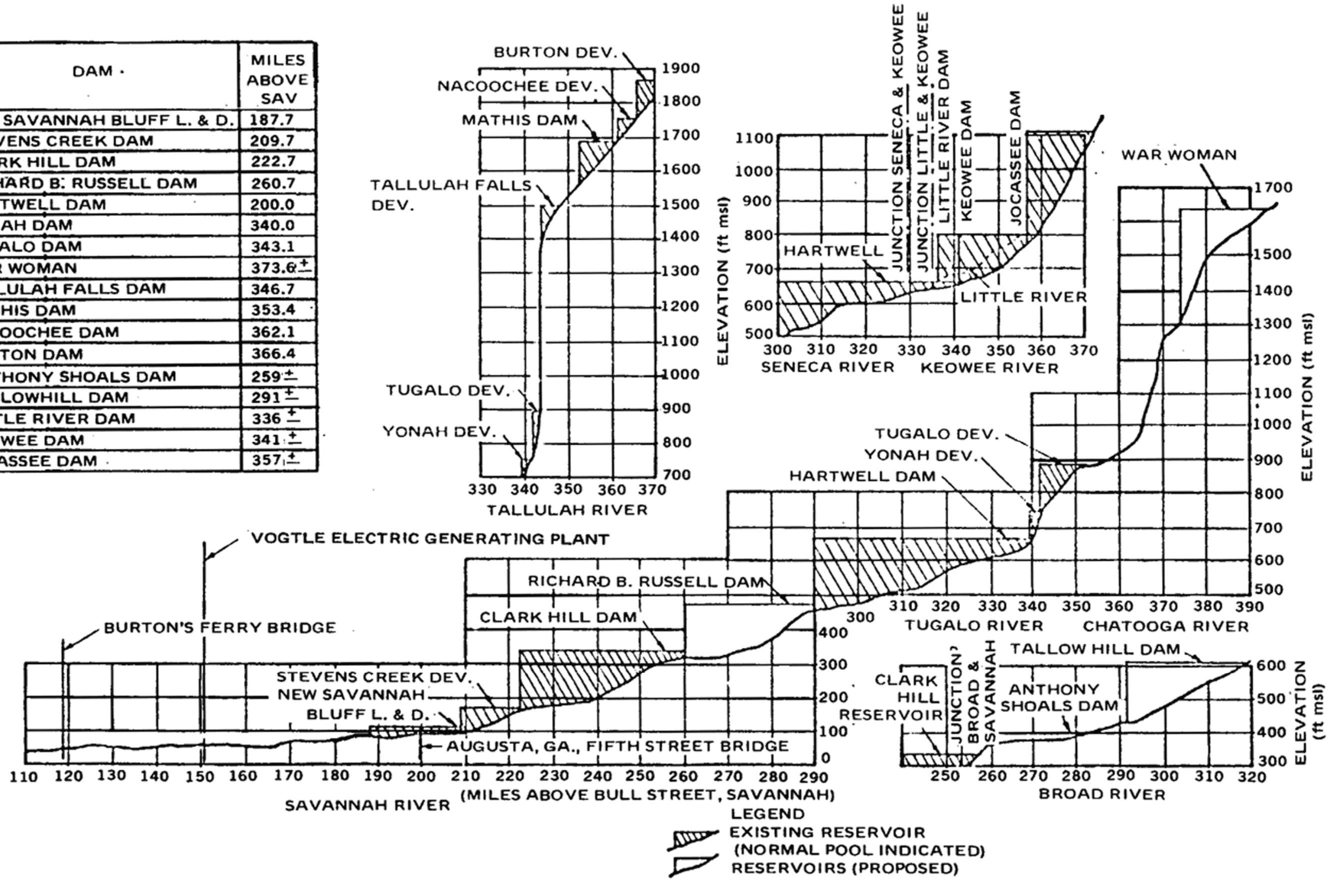
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

LOCATION OF DAMS
ON THE SAVANNAH RIVER

FIGURE 2.4.4–1

DAM	MILES ABOVE SAV
NEW SAVANNAH BLUFF L. & D.	187.7
STEVENS CREEK DAM	209.7
CLARK HILL DAM	222.7
RICHARD B. RUSSELL DAM	260.7
HARTWELL DAM	200.0
YONAH DAM	340.0
TUGALO DAM	343.1
WAR WOMAN	373.6±
TALLULAH FALLS DAM	346.7
MATHIS DAM	353.4
NACOOCHEE DAM	362.1
BURTON DAM	366.4
ANTHONY SHOALS DAM	259±
TALLOWHILL DAM	291±
LITTLE RIVER DAM	336±
KEOWEE DAM	341±
JOCASSEE DAM	357±



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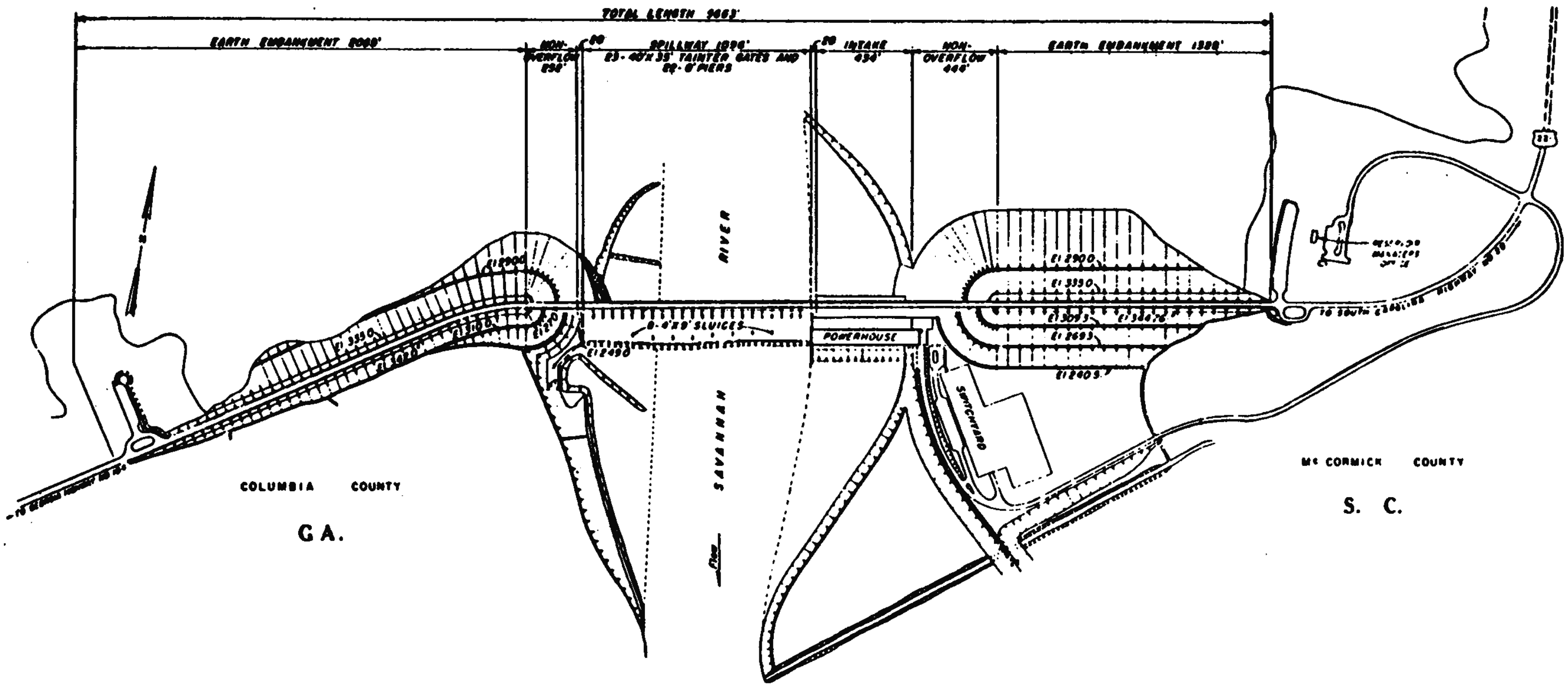
VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2

SAVANNAH RIVER STREAM PROFILE

FIGURE 2.4.4–2



PLAN

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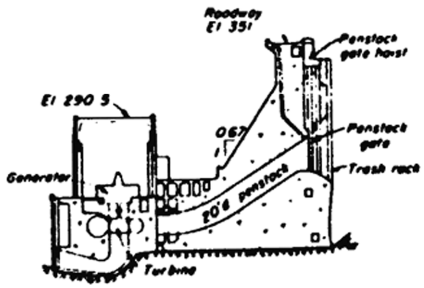
VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

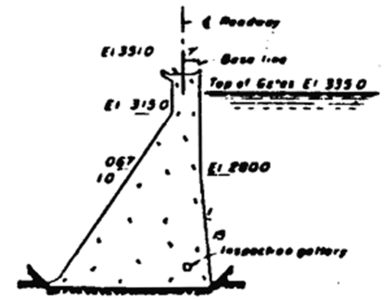
CLARK HILL DAM
PLAN AND SECTION

FIGURE 2.4.4–3 (SHEET 1 OF 2)



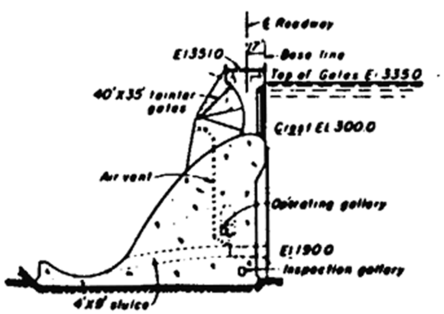
TYPICAL INTAKE SECTION

SCALE IN FEET



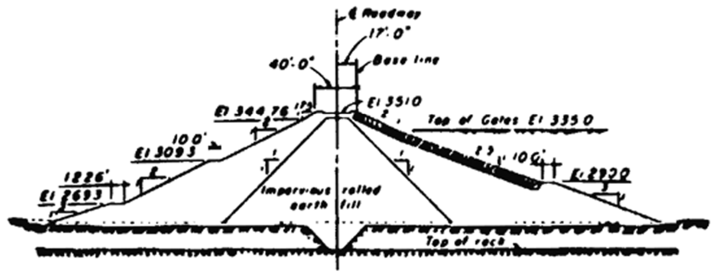
NON-OVERFLOW SECTION

SCALE IN FEET



SPILLWAY SECTION

SCALE IN FEET



EMBANKMENT SECTION

SCALE IN FEET

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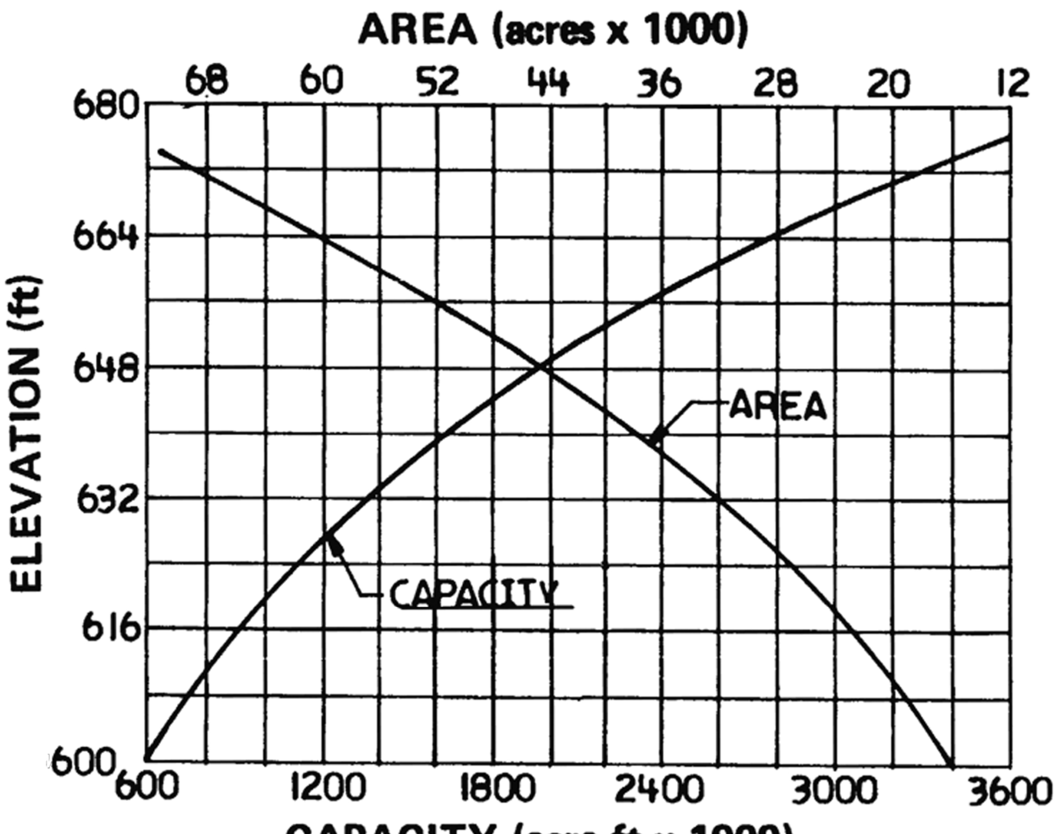
VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

CLARK HILL DAM
PLAN AND SECTION

FIGURE 2.4.4–3 (SHEET 2 OF 2)



HARTWELL RESERVOIR

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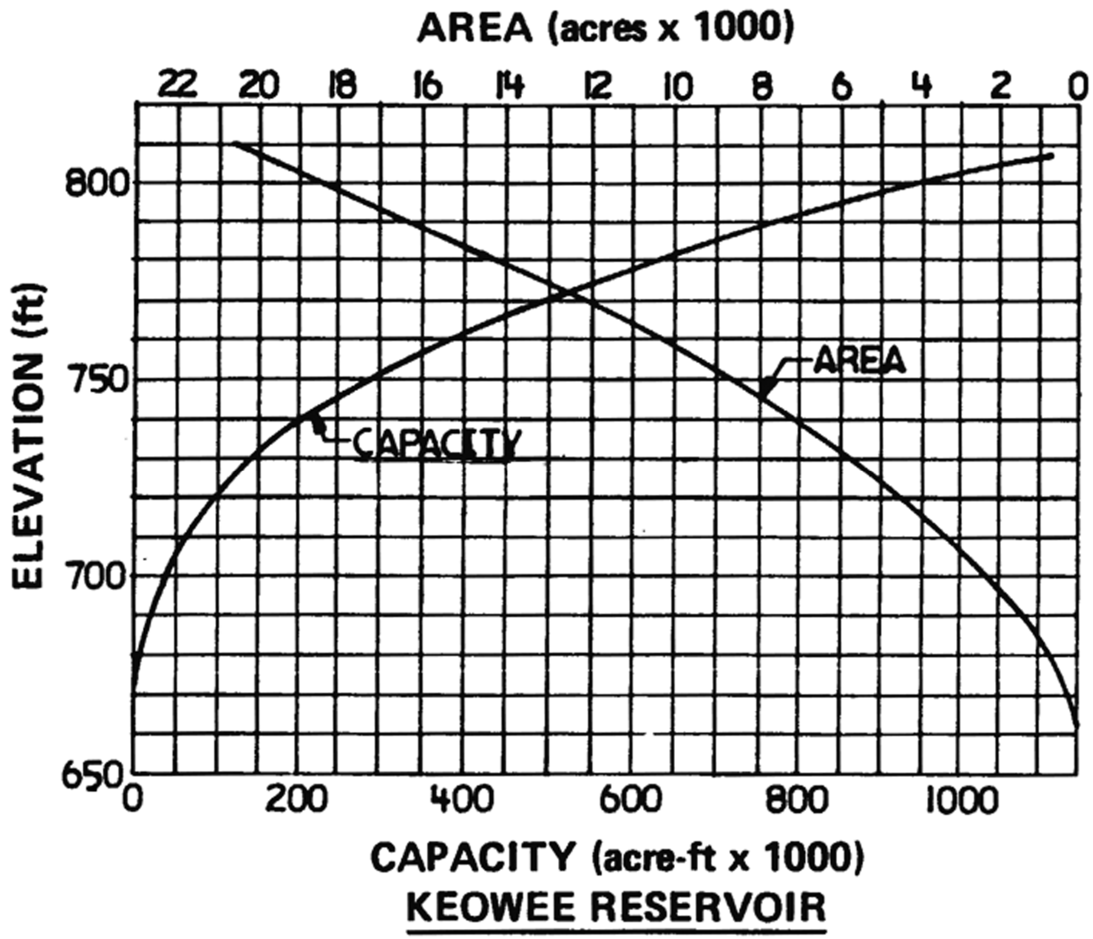
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

AREA AND CAPACITY CURVES FOR
MAJOR UPSTREAM DAMS

FIGURE 2.4.4–4 (SHEET 1 OF 5)



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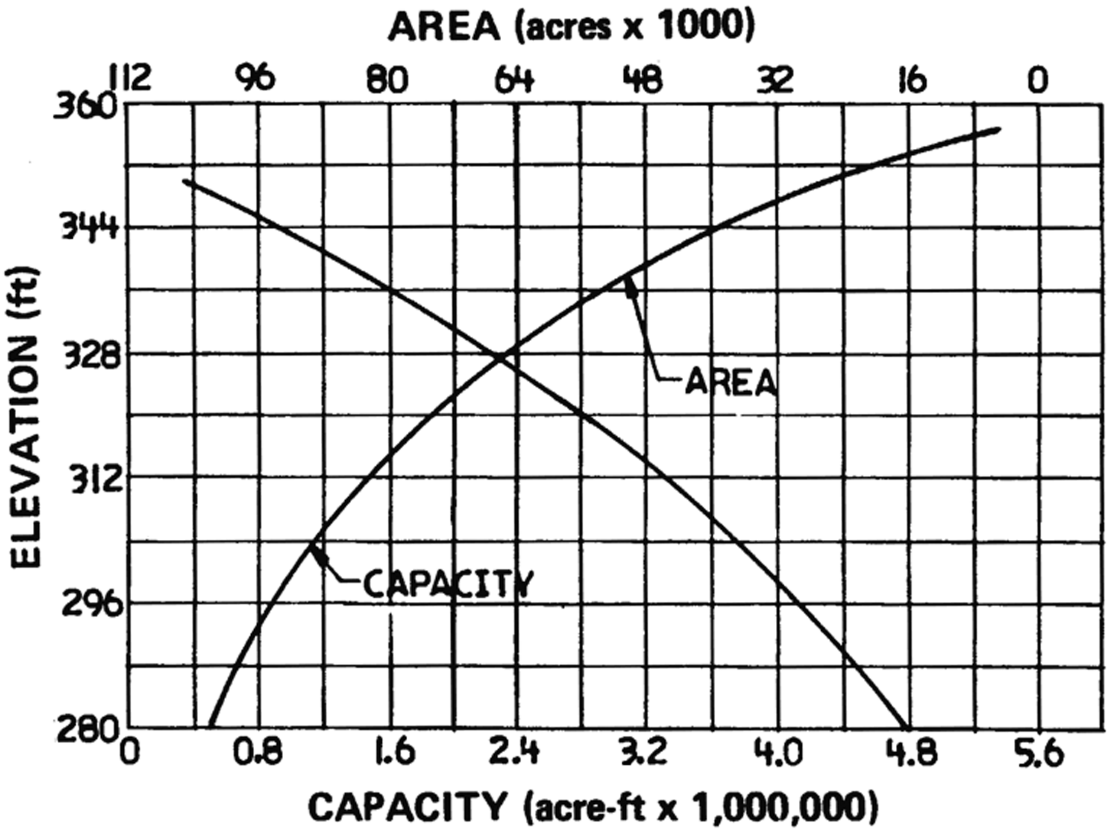
VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

AREA AND CAPACITY CURVES FOR
MAJOR UPSTREAM DAMS

FIGURE 2.4.4–4 (SHEET 2 OF 5)



CLARK HILL RESERVOIR

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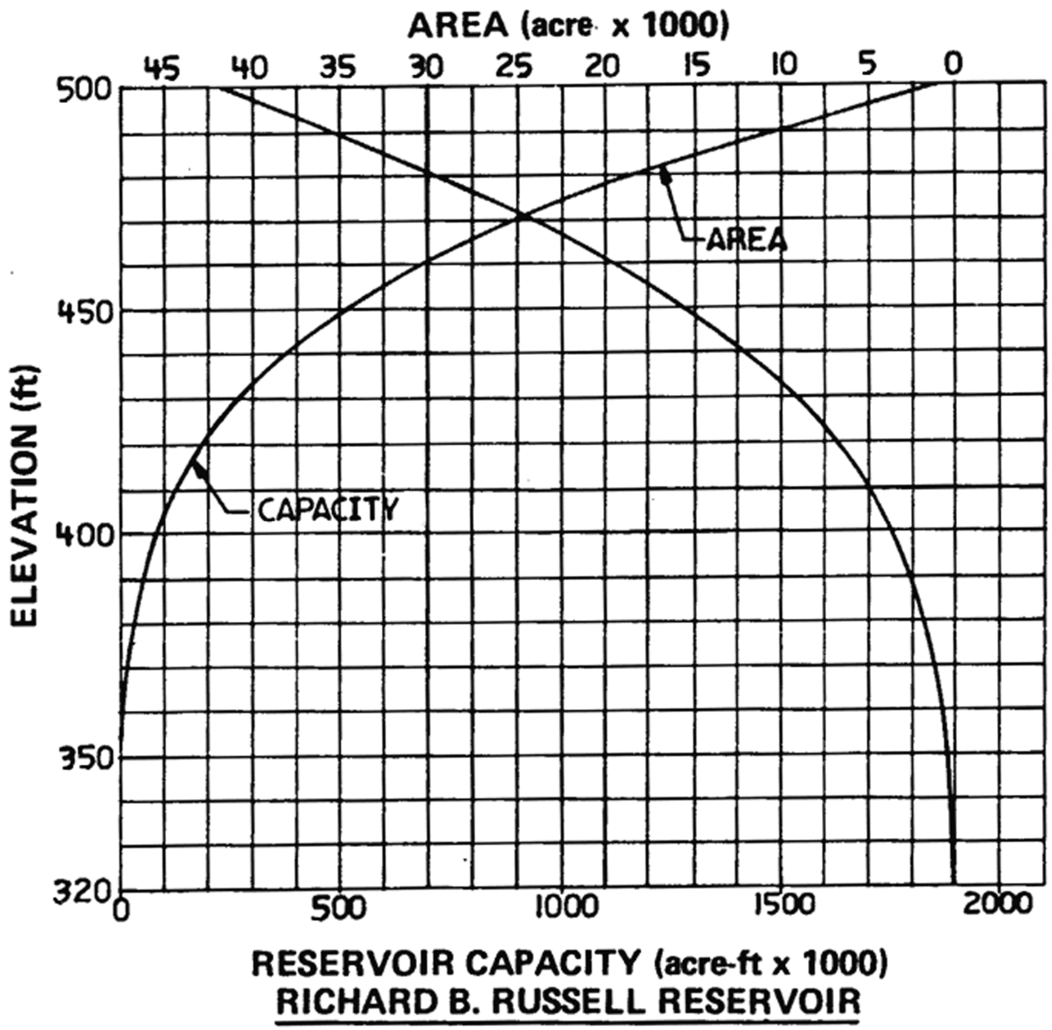
SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	AREA AND CAPACITY CURVES FOR MAJOR UPSTREAM DAMS
		FIGURE 2.4.4–4 (SHEET 3 OF 5)



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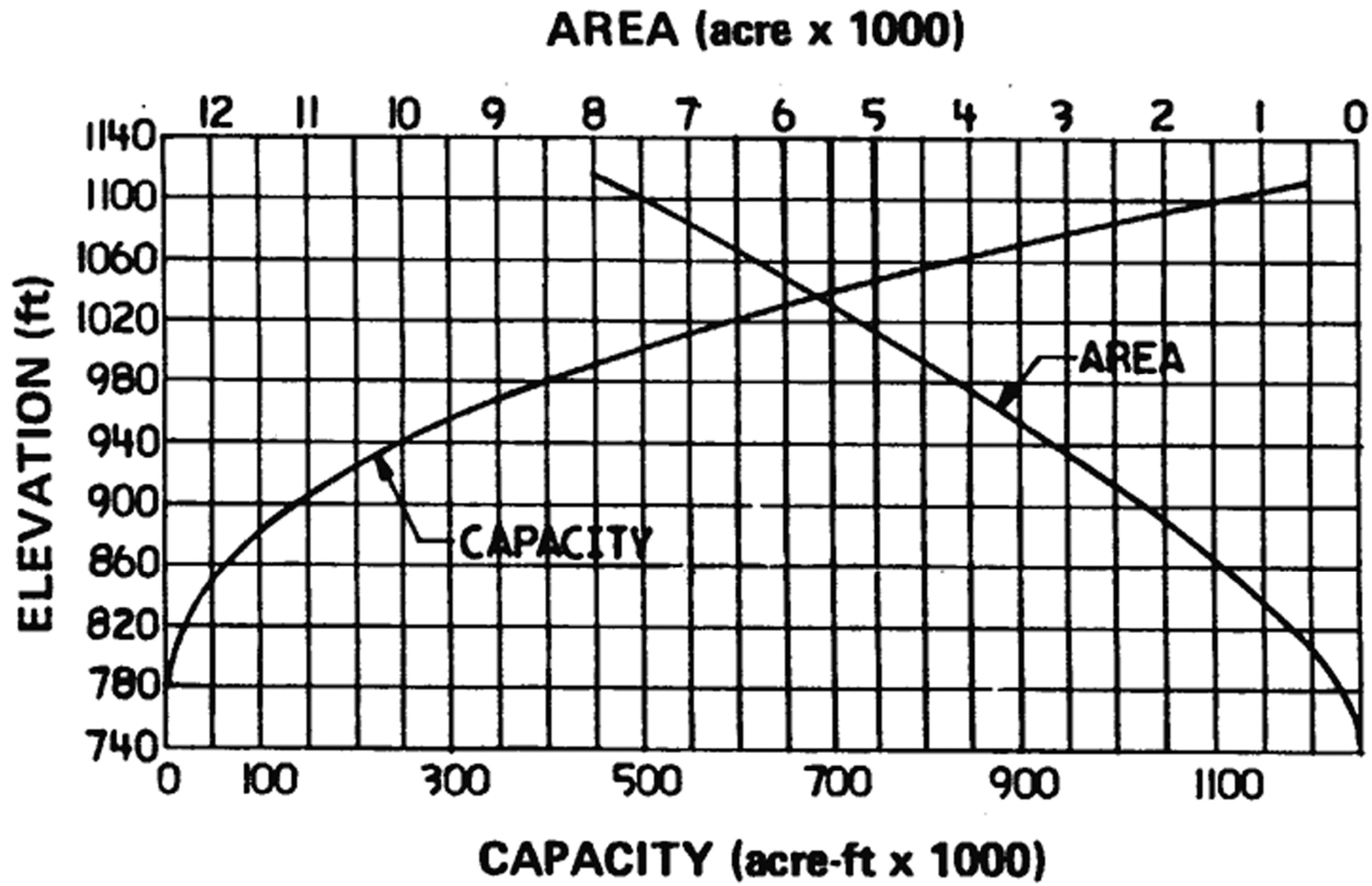
VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

AREA AND CAPACITY CURVES FOR
MAJOR UPSTREAM DAMS

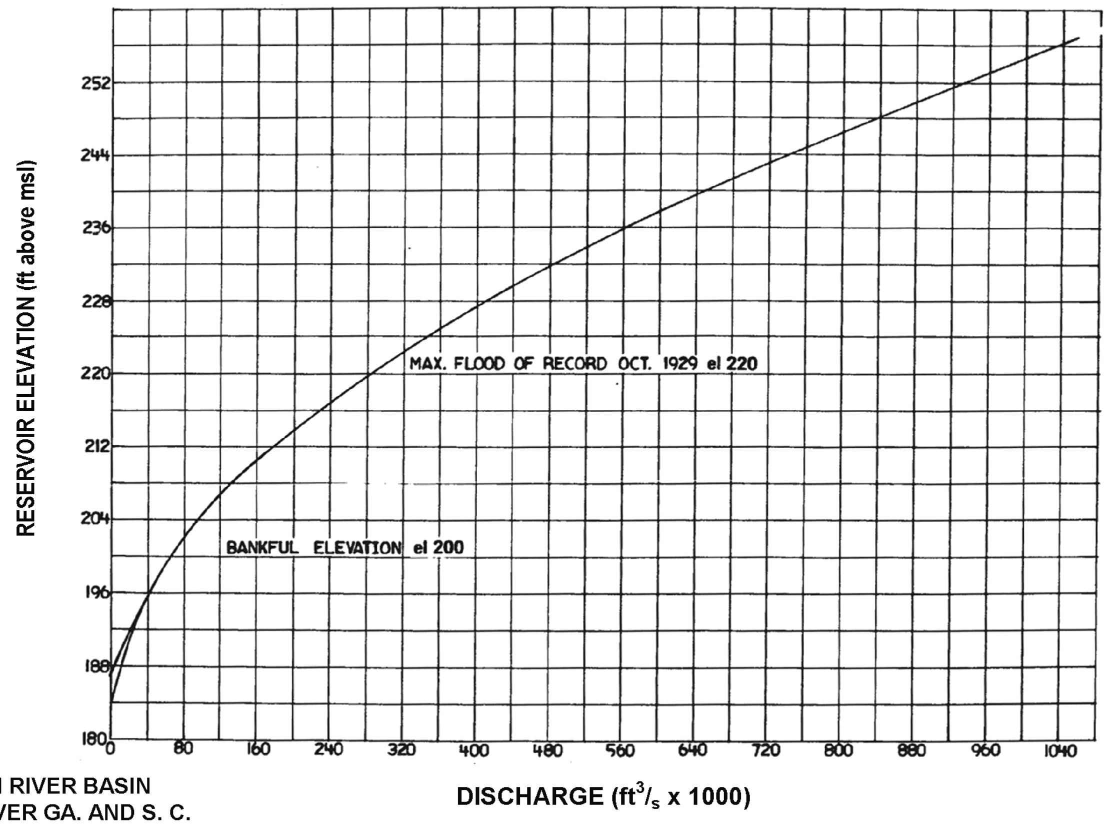
FIGURE 2.4.4–4 (SHEET 4 OF 5)



JOCASSEE RESERVOIR

REV 14 10/07

SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	AREA AND CAPACITY CURVES FOR MAJOR UPSTREAM DAMS
		FIGURE 2.4.4–4 (SHEET 5 OF 5)



SAVANNAH RIVER BASIN
SAVANNAH RIVER GA. AND S. C.
RESERVOIR REGULATION MANUAL
CLARK HILL LAKE
U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

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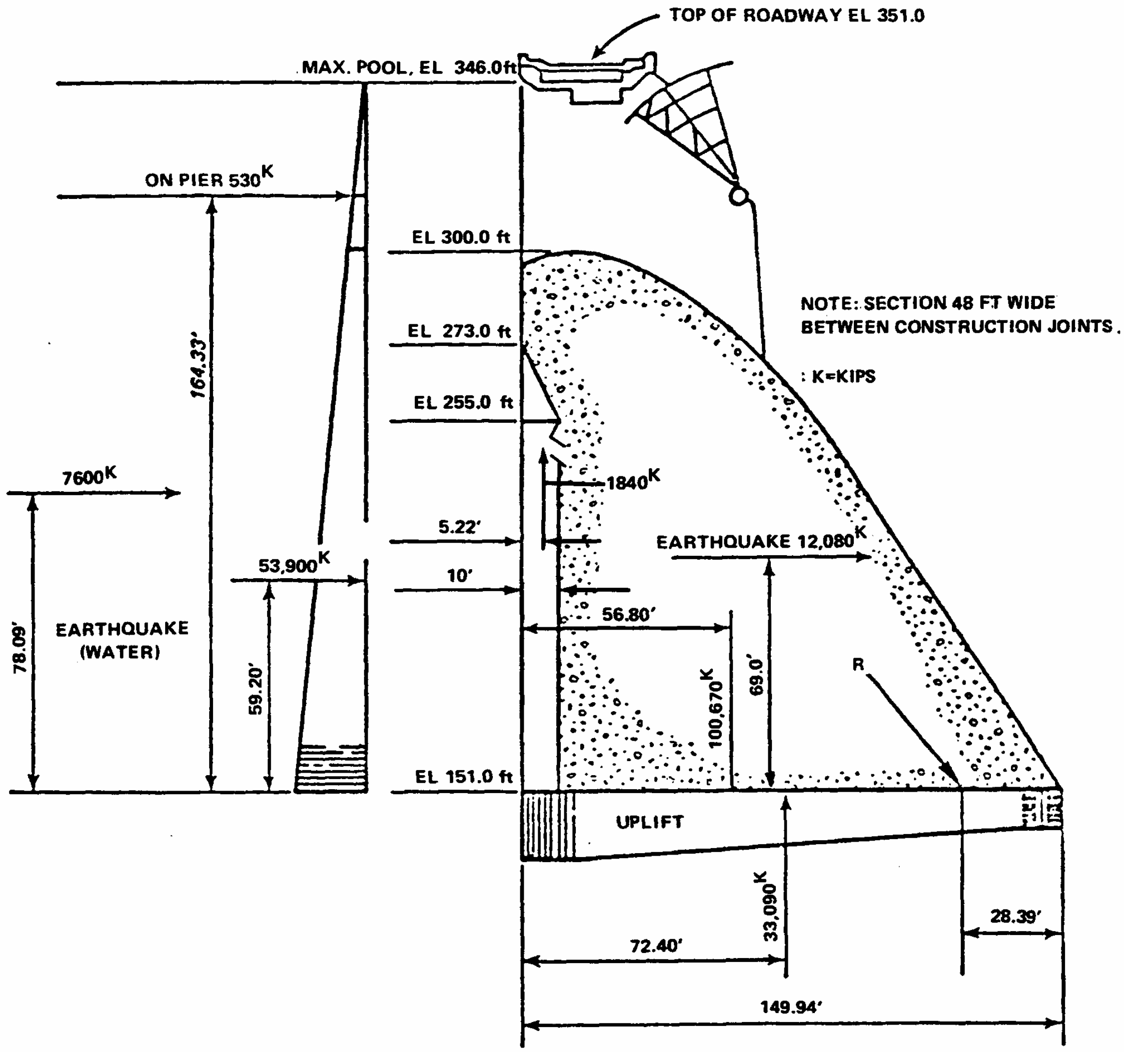
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

CLARK HILL TAILWATER
RATING CURVE

FIGURE 2.4.4–5



SPILLWAY SECTION

EARTHQUAKE LOADING
$\alpha = 0.12g$

ELEV.	ΣV KIPS	ΣH KIPS	$\frac{\Sigma H}{\Sigma V}$	SHEAR FRICTION FACTOR	RESULTANT OUTSIDE KERN	CONCRETE STRESSES (lbs/in.2) HEEL	TOE
151	65,740	74,112	1.1	6.3	21.59'	-13	207

STRESS TABULATION

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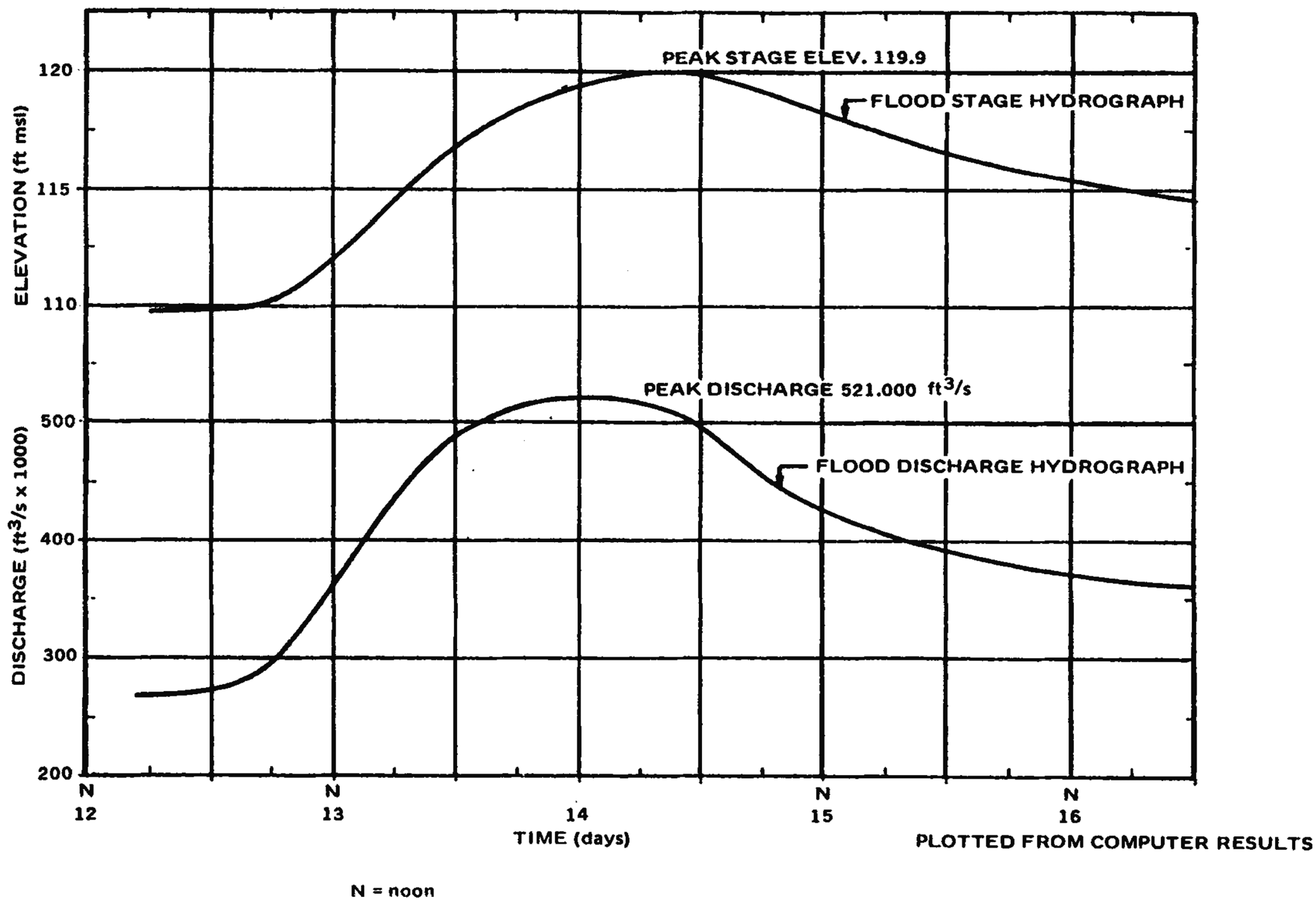
UNIT 1 AND UNIT 2

STABILITY OF CLARK HILL DAM SPILLWAY SECTION

FIGURE 2.4.4–6



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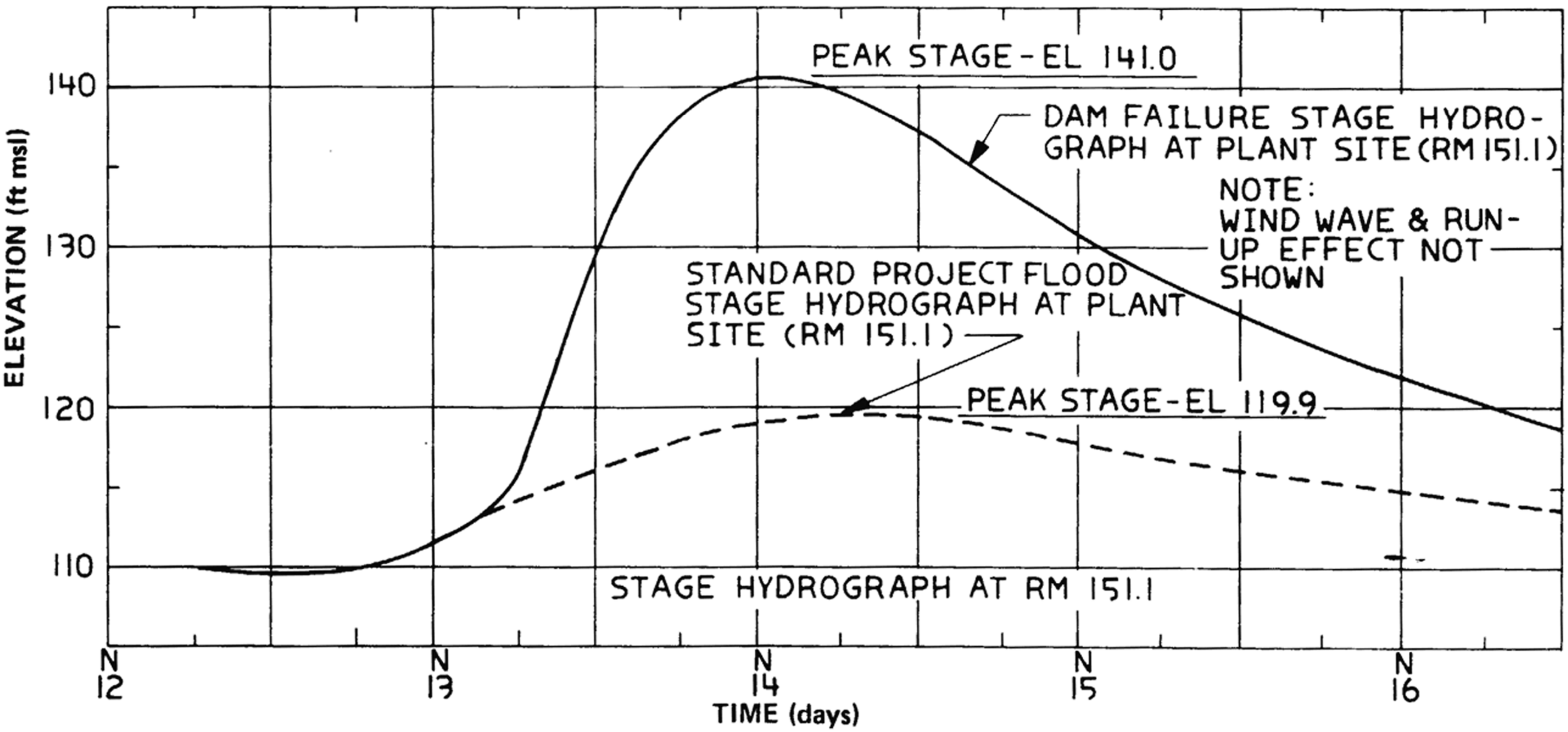
UNIT 1 AND UNIT 2

STANDARD PROJECT FLOOD
AT RIVER MILE 151.1

FIGURE 2.4.4–7



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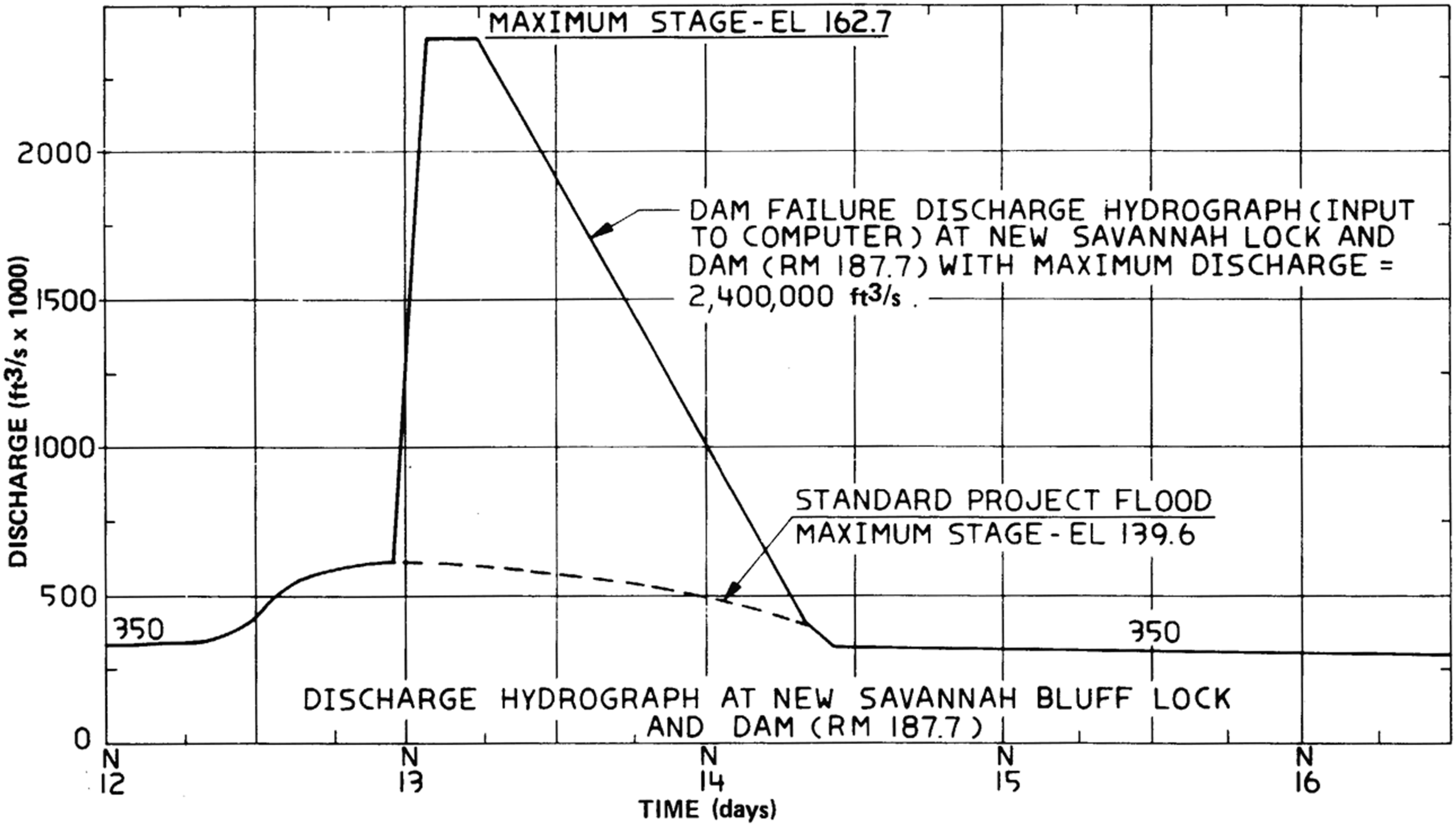
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

EFFECT OF DAM FAILURE
AT VEGP SITE

FIGURE 2.4.4–8 (SHEET 1 OF 2)



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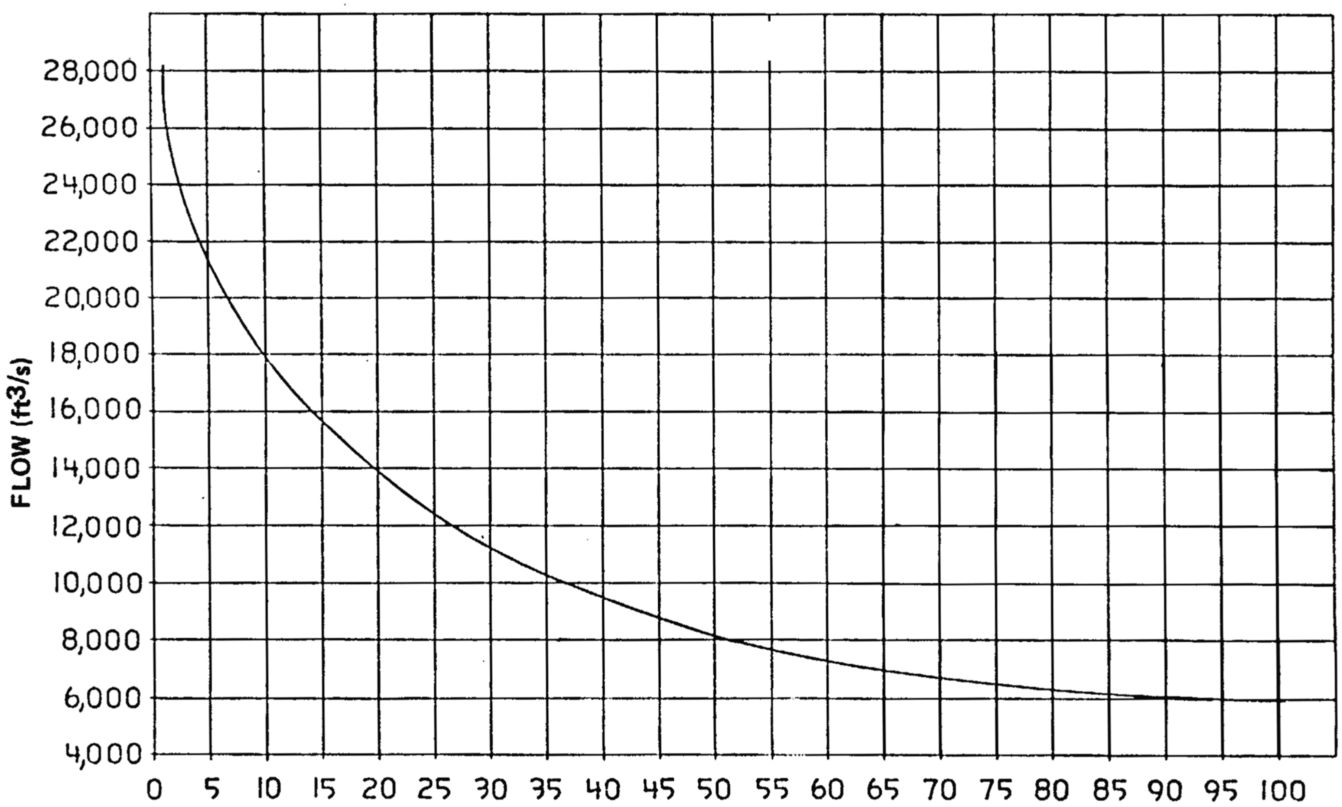
UNIT 1 AND UNIT 2

EFFECT OF DAM FAILURE
AT VEGP SITE

FIGURE 2.4.4–8 (SHEET 2 OF 2)

NOTES:
1. INFORMATION FURNISHED BY CORPS OF ENGINEERS, SAVANNAH DISTRICT.
2. FLOW IS CONTROLLED DUE TO CLARK HILL DAM UPSTREAM.



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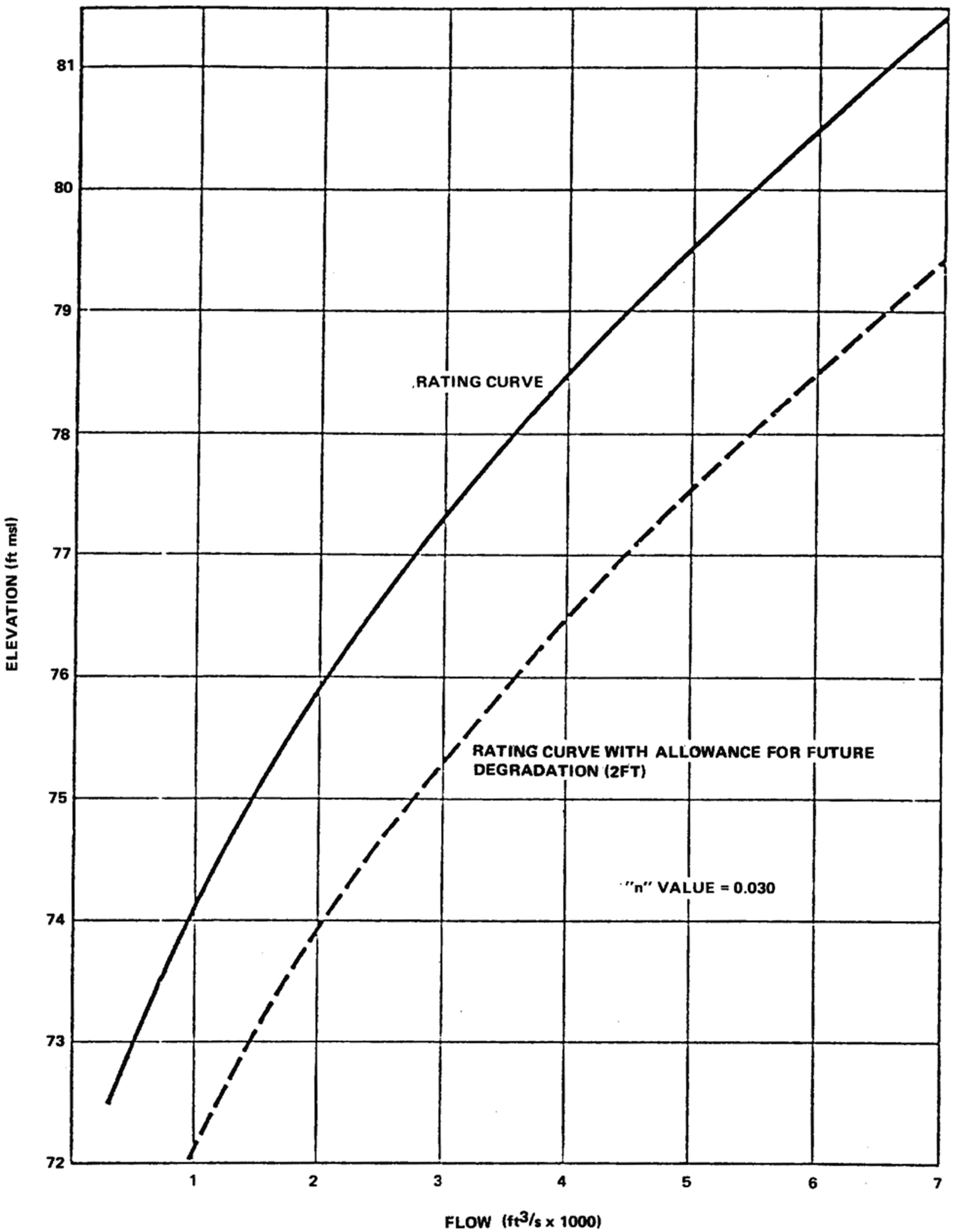
SOUTHERN COMPANY Energy to Serve Your World®	**VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2**	FLOW DURATION CURVE, SAVANNAH RIVER AT BUTLER CREEK–RIVER MILE 187.0 FIGURE 2.4.11–1



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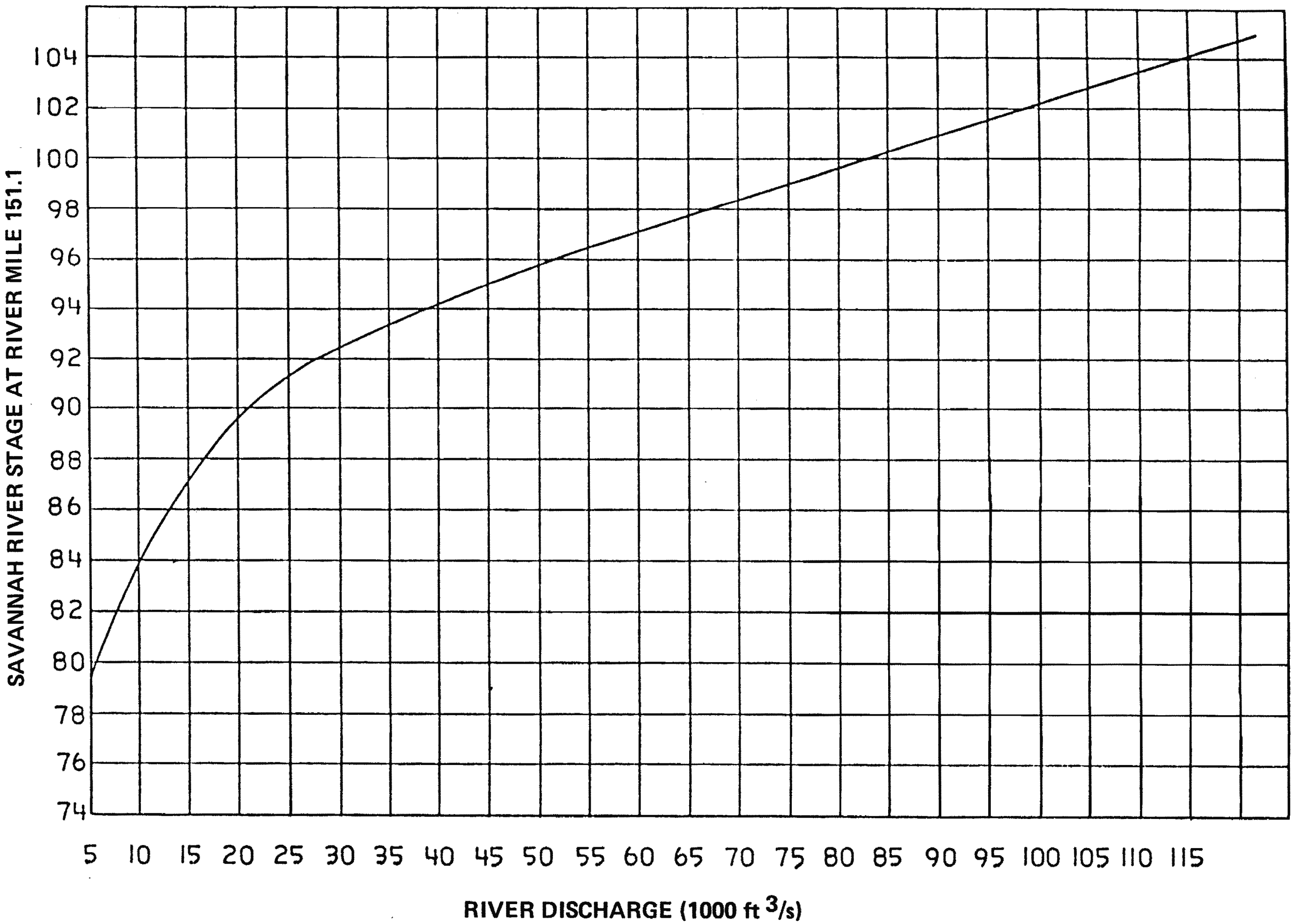
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

SAVANNAH RIVER LOW FLOW RATING CURVES AT INTAKE STRUCTURE RIVER MILE 151.1

FIGURE 2.4.11–2



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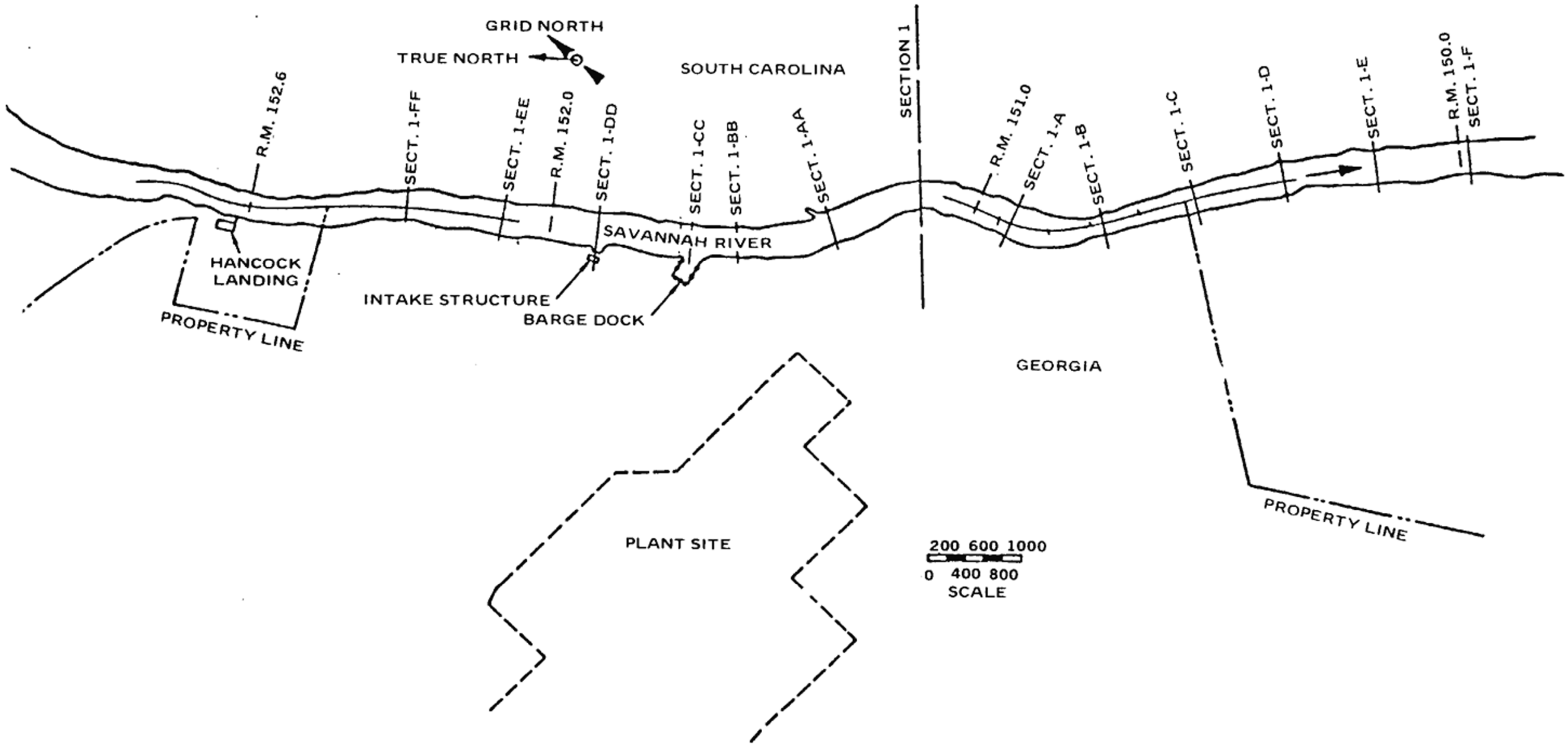
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

SAVANNAH RIVER INTERMEDIATE FLOW RATING CURVE AT INTAKE STRUCTURE RIVER MILE 151.1

FIGURE 2.4.11–3



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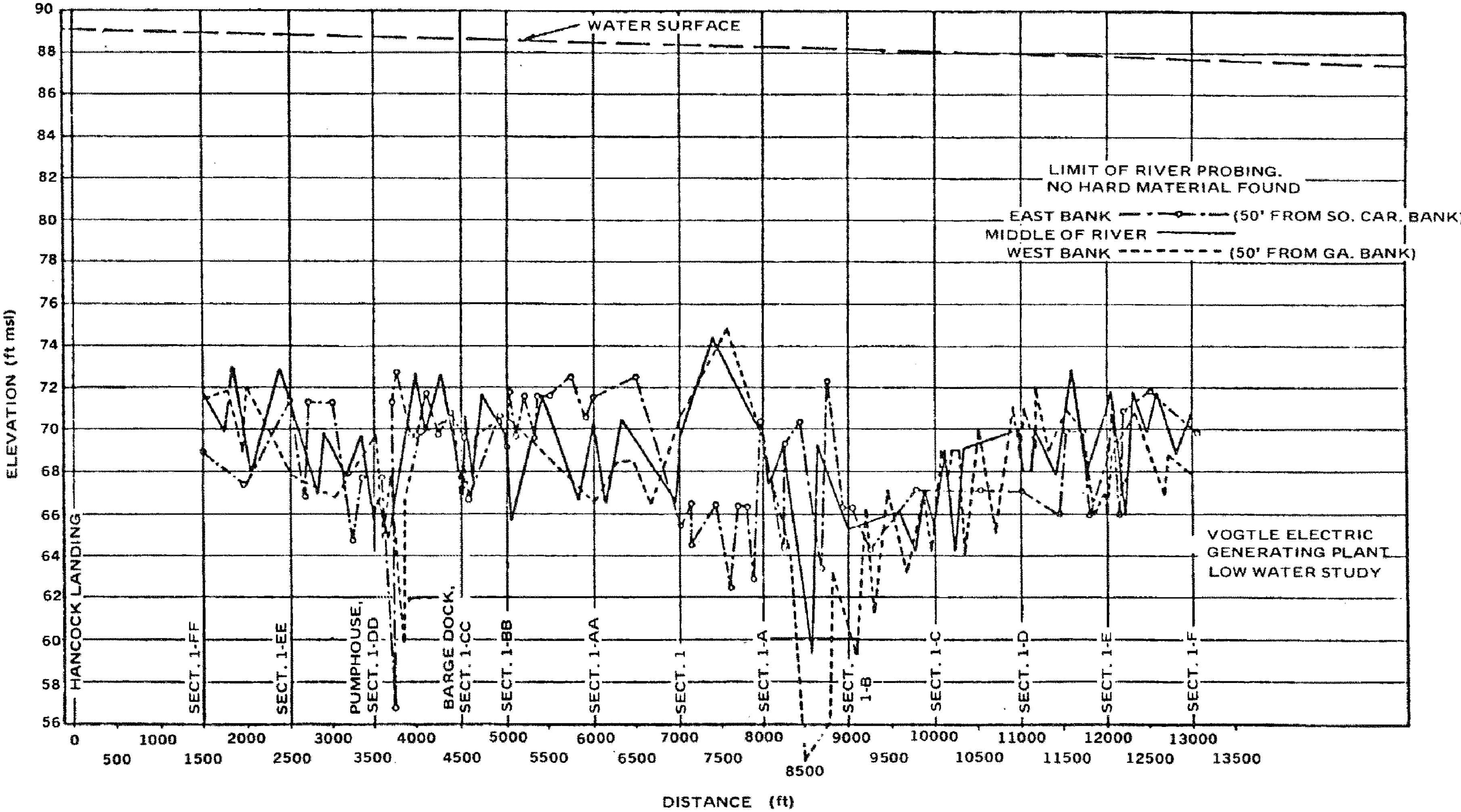
UNIT 1 AND UNIT 2

LOCATIONS OF SECTIONS
FOR LOW WATER STUDY

FIGURE 2.4.11–4



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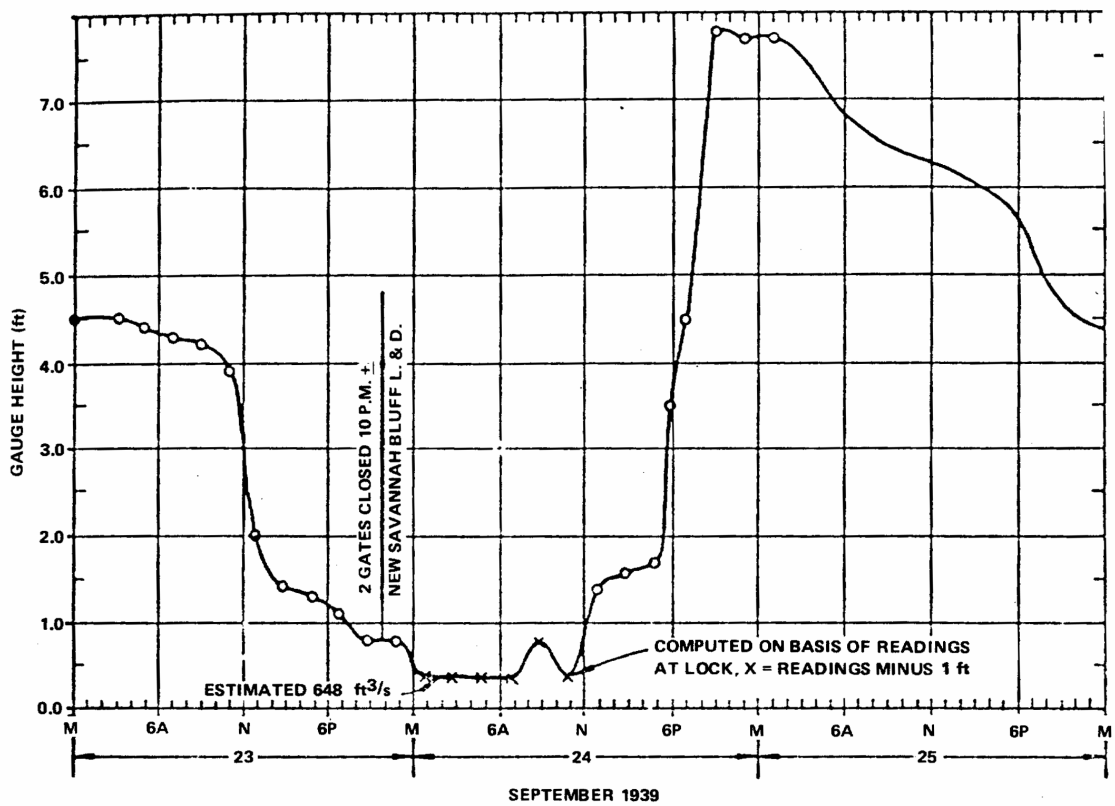
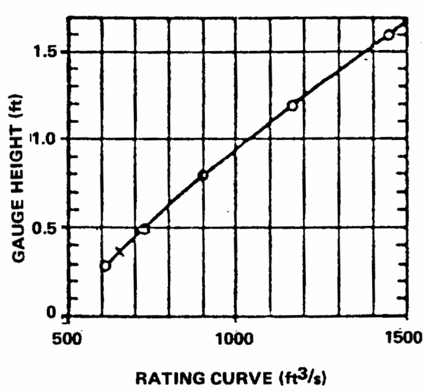
UNIT 1 AND UNIT 2

VEGP LOW WATER STUDY

FIGURE 2.4.11–5



TAKEN FROM COPY OF ORIGINAL RECORDER GRAPH AT BUTLER CREEK, 0.3 MILE DOWN-STREAM FROM NEW SAVANNAH BLUFF LOCK AND DAM. COPY FURNISHED BY U S G S



AVERAGE DAILY FLOW

DATE	FLOW ft^3/s
9-23	2230
9-24	2940
9-25	5350

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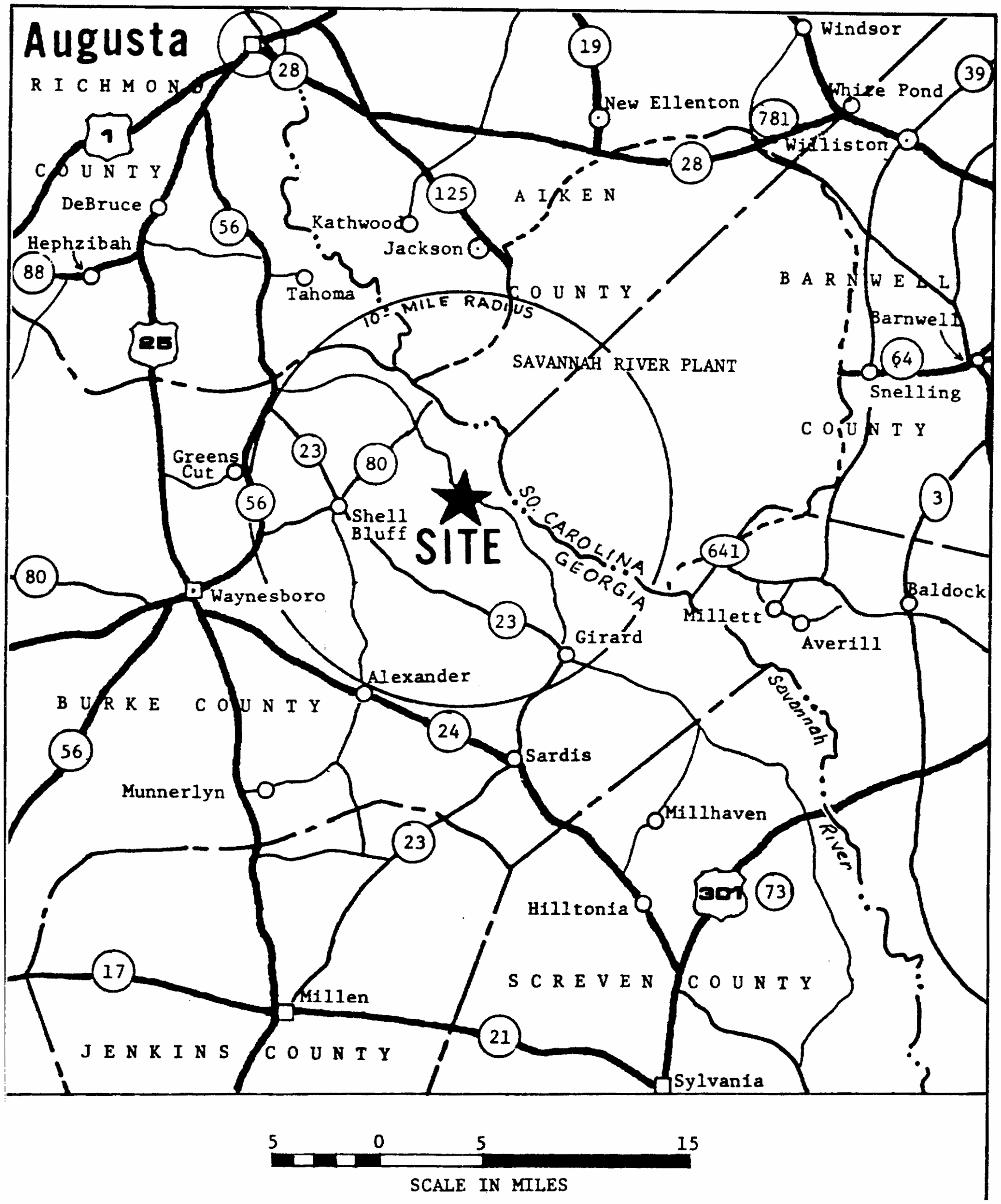
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

SAVANNAH RIVER AT AUGUSTA GAUGE READINGS FOR SEPTEMBER 23, 24 AND 25, 1939, MINIMUM FLOW OF RECORD

FIGURE 2.4.11–6



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SOUTHERN COMPANY
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VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2

SITE AREA

FIGURE 2.4.12–1

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

A summary of subsections 2.5.1 through 2.5.6 is provided below:

A. Summary of Investigation Program

The VEGP site is located approximately 26 miles south- southwest of Augusta, Georgia. The location of the site is shown on drawing AX6DD338. Comprehensive aerial geology and site-specific foundation investigations and examinations of the VEGP site were completed in accordance with the criteria outlined in Appendix A, Seismic and Geologic Siting Criteria for Nuclear Power Plants, of 10 CFR 100. The purpose of the investigation program and the subsequent evaluation of the geologic and soils data developed was to demonstrate the suitability of the site for a nuclear power generating facility.

Field investigations involved geologic mapping, drilling, geophysical survey, and ground water studies. During the Preliminary Safety Analysis Report (PSAR) phase of the investigations, 474 holes were drilled for a total of 60,000 ft of hole. A total of 111 holes were drilled subsequent to the PSAR investigations. The exploration program included electric logging, natural gamma, density, neutron, caliper, and three-dimensional velocity logs in selected drill holes. Water pressure tests and Menard pressure meter tests were performed to determine in situ properties of the marl stratum which provides bearing for plant structures and Seismic Category 1 backfill. Samples for fossil, mineral, or soluble carbonate analysis were taken in drill holes as required. The geophysical survey provided a total of 28,400 ft of shallow refraction seismic lines, 5000 ft of deep refraction lines, and cross-hole velocities in the upper 290 ft of materials.

Laboratory testing of selected soil samples consisted of the following tests:

1. Moisture content.
2. In situ unit weight.
3. Atterberg limits.
4. Static triaxial shear.
5. Direct shear.
6. Consolidation.
7. Relative density.
8. Unconfined compression.
9. Dynamic triaxial shear.
10. Resonant column.
11. Moisture density relationship.

Included in the investigation program was a thorough literature search, stereoscopic examination of color air photographs, evaluation of geologic conditions at and within 5 miles of the site, and geologic reconnaissance along 12 miles of the river bluff upstream and downstream of the site.

Drilling services were provided by Georgia Power Company, Girdler Drilling Company, and Law Engineering Testing Company. Geophysical seismic surveys were performed by Weston Engineers, Weston, Massachusetts. Down-hole geophysical surveys were performed by the Birdwell division of Seismograph Service, Incorporated. Laboratory testing services were provided by Law Engineering Testing Company, Atlanta, Georgia, and Geotechnical Engineers, Incorporated, Winchester, Massachusetts.

B. Summary of Geology

The site is located in the Atlantic Coastal Plain physiographic province in central Georgia. The portion of the Coastal Plain province in which the site occurs is known as the Tifton Upland which is characterized by rolling hills ranging in elevation from 80 to 280 ft in the site vicinity.

The geology within a 25-mile radius of the site consists of Precambrian and Paleozoic igneous and metamorphic basement rocks (gneisses and granites of the Kiokee Belt and phyllites and greenstones of the Belair Belt) overlain locally by Triassic basin sediments (Dunbarton Basin); these are, in turn, overlain by Cretaceous through Miocene Coastal Plain (shallow marine) sediments. Quaternary alluvial deposits occur along the Savannah River and its tributaries.

Virtually all tectonic activity occurred prior to the deposition of the Cretaceous sediments. The complex folding, faulting, and shear structures that developed in the basement rocks originated in the Precambrian and Paleozoic eras during orogenic episodes associated with the development of the southern Appalachians. Relatively undeformed Coastal Plain sediments indicate that this orogenic activity ceased prior to the Cretaceous period.

The geology within a 5-mile radius of the site reflects the geology of the region. The contact between the basement complex and Cretaceous sediments occurs more than 1000 ft below the surface. As a result of regional elevation fluctuations following the deposition of the basal Cretaceous sediments (Tuscaloosa formation), overlying Paleocene through Miocene sediments represent marine transgressive and regressive sequences. Strata include shallow marine sand, clay, gravel, limestone, and marl. Quaternary deposits of sand, gravel, silt, and clay occur as flood plain deposits in the Savannah River valley and the larger tributaries to the river. However, the Quaternary system is principally represented by erosion and weathering rather than deposition.

Cretaceous and post-Cretaceous formations underlying the site are essentially flat lying or gently dipping to the southeast, reflecting a regional dip of about 30 ft/mi. Localized solution occurs in a shallow formation stratigraphically above the marl. Therefore, the solution activity will have no effect on plant foundations.

No faults or lineaments have been found within 5 miles of the site, other than those associated with the Triassic Dunbarton Basin, and these structures do not extend into overlying Tertiary strata. There are no capable faults as defined by 10 CFR 100, Appendix A, anywhere within the site region. There are no other geologic hazards which could affect site safety or suitability for a nuclear power plant.

C. Summary of Seismology

The Atlantic Coastal Plain tectonic province, in which the site is located, is a large area with generally low seismic activity. The general seismicity of this province is expected to remain subdued, while high earthquake activity will be confined to the Charleston-Summerville seismic zone. In the adjacent southern Appalachian Mountains region, earthquakes are irregularly distributed, with concentrations in northeast Georgia, northwest South Carolina, eastern Tennessee, and Virginia, all at distances greater than 100 miles from the site.

The closest damaging earthquake to the site occurred on November 1, 1875. It was centered 60 miles to the northwest and had a maximum intensity of VI near the epicenter. This event may have been felt with low intensity at the site.

The New Madrid events of 1811-1812, which were centered 530 miles from the site, were probably felt at the site with an intensity less than VI. The Union County, South Carolina, earthquake of January 1, 1913, had an epicentral intensity of VII to VIII. However, the felt area does not include the VEGP site.

The source of seismicity most affecting the site, both in maximum historical intensity and number of earthquakes, is the Charleston-Summerville, South Carolina, area. The main shocks of August 31, 1886, probably produced an intensity of VI at the site. Recent studies have defined a linear north-northwest trending zone of epicenters in the Charleston- Summerville area. The closest approach of the Charleston-Summerville seismic zone to the site is 78 miles.

Detailed studies have revealed no seismological or geological evidence for capable faults within 200 miles of the site.

The maximum credible site intensity is VI-VII to VII. For conservatism a safe shutdown earthquake site intensity of VII-VIII is chosen. This intensity is associated with approximately 0.2 g peak horizontal acceleration.

Evidence indicates that the maximum historical intensity at the site was VI. For conservatism an operating basis earthquake of intensity VII is adopted. This intensity is associated with a peak horizontal ground surface acceleration of 0.12 g. A probabilistic analysis shows that the likelihood of this acceleration being exceeded during the 40-year[a] operating life of the plant is less than 8%.

---

[a] The operating licenses for both VEGP units have been renewed and the original licensed operating terms have been extended by 20 years. Seismic analyses are not related to aging and therefore are outside the scope of license renewal. NEI 98-03 guidance indicates that seismic data used to support original plant design bases are considered historical and do not need to be actively maintained. Therefore, this statement is not required to be updated as a result of license renewal.

D. Summary of Geotechnical Engineering

The surface soils at the VEGP site consist of three principal strata, i.e., the upper sand stratum, the marl stratum, and the lower sand stratum. The upper sand stratum is about 90 ft deep. The relative density of the upper sand stratum is very variable and ranges from very loose to dense. Based on the results of a liquefaction analysis, it was determined that the upper sand stratum would have a potential for liquefaction in the event of a seismic occurrence equivalent to the safe shutdown earthquake. The upper sand stratum was excavated down to the marl stratum and replaced with select sand and silty sand backfill compacted to an average of 97% of the maximum density determined by ASTM D 1557. This high degree of compaction will ensure an adequate factor of safety against liquefaction and reduce settlements to a tolerable level.

The auxiliary building, nuclear service cooling water towers, and instrumentation cavity of the containment are supported on the marl stratum. All the other power block structures are supported on compacted backfill. The marl stratum which directly or indirectly supports all the power block structures ranges in thickness from 60 to 100 ft. There is no evidence that the marl stratum has been subjected to or is potentially subject to subsidence, collapse, or uplift due to earthquake, solution processes, or other geological phenomena. The lower sand stratum underlying the marl stratum is estimated to be at least 750 ft thick and determined to be dense to very dense in relative density. The VEGP power block structures are therefore supported on competent bearing strata.

E. Conclusions

The studies described in subsections 2.5.1 through 2.5.6 have led to the following conclusions:

1. The site is suitable for design and construction of a multiple-unit nuclear power generating facility.

2. The geologic conditions exposed in the power block excavation confirm the conditions described in the PSAR.

3. No geologic features which could affect licensing aspects of the plant exist in the power block area.

4. The marl layer, which provides foundation for both Category 1 structures and Category 1 backfill, is as sound and as competent as anticipated and is free of solution cavities.

5. Results of laboratory testing and monitoring of the rebound of the marl confirm its competency.

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

The following paragraphs contain the results and conclusions of the regional and site geologic and seismic investigations.

Information on regional and local ground water conditions is included in subsection 2.4.12 and is only summarized in the following geology subsections. The characteristics of the foundation materials with respect to their ability to support the major plant structures are discussed in subsection 2.5.4 and cross-referenced in this section. Sources used in the preparation of text, figures, and tables are contained in the references cited at the end of this subsection.

2.5.1.1 Regional Geology

This paragraph on regional geology describes the area surrounding the site and relates it to the Atlantic Coastal Plain and Piedmont regions in terms of physiography, geomorphology, geologic history, lithology, stratigraphy, structure, and tectonic conditions.

2.5.1.1.1 Regional Physiography and Geomorphology

The VEGP site is located approximately 26 miles south-southwest of Augusta, Georgia, in the Atlantic Coastal Plain province (drawing AX6DD338). The Atlantic Coastal Plain province covers approximately 60% of the surface area of the State of Georgia. Along its inner margin at the boundary with the Piedmont province is the Fall Line. This line marks the contact between the crystalline basement and the onlapped Cretaceous and Cenozoic sediments. The plant site is located near the eastern margin of the Tifton Upland topographic area, a major topographic division of the Atlantic Coastal Plain province (drawing AX6DD338). The Atlantic Coastal Plain province in Georgia also includes parts of three other major topographic areas:(1)(2) from its western margin seaward, they are the Fall Line Hills, the Dougherty Plain, and the coastal terraces (drawing AX6DD338).

The Fall Line Hills, located northwest of the site, form a series of maturely dissected hills which adjoin the Fall Line to the northwest and the Tifton Upland to the southeast. The Fall Line Hills form a broad and continuous belt with the exception of some transverse divides. The Red Hills topographic subdivision, commonly considered part of the Fall Line Hills in Georgia, occupies the area immediately to the north of the Tipton Upland.(1) In Georgia, the Red Hills are equivalent to the tabular and incompletely dissected Louisville Plateau. The Fall Line Hills, including the Red Hills, form a zone between 20 and 40 miles wide.

The Dougherty Plain topographic division is widely exposed in west-central Georgia, situated between the Fall Line Hills on the north and the Tifton Upland on the south. The most noticeable features on its nearly flat plain are shallow, flat- bottomed or rounded depressions resulting from solution of the underlying rocks.

The plant site is located near the eastern margin of the Tifton Upland topographic division, a submaturely dissected area of the Coastal Plain just seaward of the Fall Line Hills in north-eastern Georgia. The dissection of the weak rocks has resulted in gently rolling, well-drained hills with broad, rounded summits and with relief of generally less than 50 and rarely more than 100 ft.(1)

In western South Carolina the trend of the Fall Line Hills is interrupted by the Aiken Plateau.(3)(4) The trend is continued in west-central South Carolina as the Congaree Sand Hills.(3)

The remainder of the Coastal Plain province in both Georgia and South Carolina is comprised of belts of coastal terraces of Pleistocene age. The older terraces adjacent to the Tifton Upland show moderate erosion, whereas the younger terraces are only slightly eroded.

Immediately to the northwest of the Atlantic Coastal Plain province is the Piedmont province. Farther to the northwest, within a 200-mile radius of the site are the Blue Ridge and the Valley and Ridge provinces. These provinces are shown on drawing AX6DD338.

The Piedmont province is a northeast trending belt that adjoins the Atlantic Coastal Plain province to the south and the Blue Ridge province and, in part, the Valley and Ridge province to the north. Extending from central Alabama to southern New York, the Piedmont province is an intricately dissected upland plain. A few linear ridges are present in west-central Georgia. Approximately 20% of the State of Georgia lies in the Piedmont province.

The Fall Line forms the border between the Piedmont and Atlantic Coastal Plain provinces. In the vicinity of the Fall Line the upland plain of the Piedmont slopes down to meet the Coastal Plains. In this area erosion has caused large streams of the Piedmont to cut gorges and create waterfalls and rapids as they flow over this steeper gradient into the weaker Coastal Plain sediments.

The Blue Ridge province is situated to the northwest of the Piedmont province and extends for 550 miles from northeastern Georgia to southern Pennsylvania. The Blue Ridge province is relatively narrow north of North Carolina and is dominated by a central ridge, whereas the area in Georgia is a broad, mountainous upland.

The Valley and Ridge province lies to the northwest of the Blue Ridge province and extends from central Alabama to the St. Lawrence Lowland. The province can be divided along its length into two major parts. The eastern part consists of a broad valley; the western part consists of a series of ridges and valleys resulting in a pronounced northeast-southwest regional grain.

The Savannah River cuts a deep, transverse valley through the Atlantic Coastal Plain province along the eastern margin of the site. The river valley is of old age, with a broad flood plain at an elevation of approximately 85 ft adjacent to the site. The bordering upland has a general elevation of 250 to 300 ft above mean sea level and is dissected by the tributaries of the Savannah River. Drainage patterns are well developed consisting of deeply incised streams which have broad, gently sloping valleys.

Occasional small, shallow, closed surface depressions are present above el 200 ft in the site area. These are formed where leaching of a soluble carbonate horizon in the upper deposits has occurred, causing surface subsidence over the affected area. Site geologic conditions are discussed in paragraph 2.5.1.2 and appendix 2B. Stability of subsurface material at the site is discussed in subsection 2.5.4.

2.5.1.1.2 Regional Geologic History

Large-scale compressional and extensional tectonics characterize the history and evolution of the Appalachian Mountains and eastern margin of North America. Tectonic models initially proposed by Hatcher(5)(6), Rodgers(7) and Rankin(8)(9) provide the general tectonic framework for evolution of the central and southern Appalachians. The models generally describe a period of major extensional rifting during the late Precambrian, followed by the development of both eastward- and westward-dipping subduction zones at different times during the Paleozoic, followed by renewed extensional rifting during the Triassic and Jurassic.

Sequential tectonic development of the Appalachians has been confirmed and elaborated with respect to plate-tectonic interactions by more recent geological mapping and deep-crustal geophysical studies in the central and southern Appalachians.[10-37] These studies indicate that the Appalachians consist of a number of lithologically and structurally distinct terranes ("suspect terranes") sutured to the continent during Paleozoic plate collisions. Strike-slip displacement may have occurred along many of these terrane boundaries concurrent with or subsequent to the suturing process as indicated by paleomagnetic data from the New England and maritime Canada area.[38] Subsequent major lateral displacements of some or all of these terranes occurred in the late Paleozoic mainly along low angle detachment surfaces or decollements.

The Appalachians are classically divided into a series of northeast-southwest-trending physiographic provinces. From northwest to southeast these include the Valley and Ridge, Blue Ridge, Piedmont, and Coastal Plain (drawing AX6DD338). In the southern Appalachians, the Piedmont Province includes a number of Lithotectonic belts: the Inner Piedmont belt, the Kings Mountain belt, the Pine Mountain belt, the Charlotte belt, and the Carolina Slate belt.[5][39] Smaller local lithotectonic belts are also defined. Near the site, north of Augusta, the Kiokee belt parallels the southeastern margin of the Carolina Slate belt, separated from it by the Modoc fault. At the edge of Coastal Plain onlap, the Augusta fault is interpreted to juxtapose yet another belt (the "Belair belt") against the southeastern edge of the Kiokee belt (figures 2 and 10C of Cook, <u>et al</u>.[24]) Both the Brevard and the Augusta faults would represent, within the crystalline Piedmont rocks, major, east dipping ramp faults that are listric into the basal decollement at depth (figures 10C, 11, and 12 of Cook, <u>et al</u>.[24]). The regional distribution of these provinces and belts is shown in figure 1 of Hatcher and Odom[12] and figure 1 of Hatcher and Zietz[32].

Williams and Hatcher [20][21] group many of these various lithotectonic belts of the Appalachian Orogen into "suspect" Appalachian terranes because their internal consistency in structure, lithology, geologic history and provenance implies that they were once spatially separate crustal entities brought into mutual proximity by subsequent plate accretion. In the southern Appalachians, for example, the Charlotte and Carolina slate belts and part of the Kings Mountain belt comprise the Avalon terrane and the eastern Blue Ridge, Inner Piedmont, Chuaga belt, and much of the Kings Mountain belt comprise the Piedmont terrane (Williams and Hatcher [20][21]). To the southeast, magnetic and gravity data provide strong evidence for additional exotic terranes beneath the Coastal Plain sediments of Georgia and the Carolinas (Higgins and Zietz[35]). These include the Suwannee and Brunswick terranes of Williams and Hatcher[21].

The geologic history presented below is a synthesis of the current technical literature on Appalachian tectonics described above. Age constraints on the timing of tectonic events are determined primarily by the depositional history of sediments in the Appalachian Orogen and from radiometric dating of metamorphic and igneous rocks. This age control is summarized by Hatcher,[5-6] Hatcher and Odom,[12] and Glover.[18] A concise summary of the geologic history of the southern Appalachians is presented by Cook, <u>et al</u>.[24] following the work of Hatcher[6] and Hatcher and Odom[12].

2.5.1.1.2.1 <u>Precambrian and Paleozoic Eras.</u> In the late Precambrian (approximately 820 million years ago) extensional tectonics rifted the North American craton from the ancestral craton of Africa, Europe, and South America.[8][9] The present eastern and southern portions of the North American craton became a passive continental margin on which shallow marine shelf deposits accumulated (Chilhowee-Knox), transitional to a basinal facies farther east (Ocoee). Various continental fragments were distributed in the expanding proto-Atlantic or "Iapetus" Ocean as the major continents separated.

During the ensuing plate convergence or contraction of the Iapetus Ocean early in the Paleozoic, several island arcs developed offshore on these continental fragments or on oceanic crust. The Carolina Slate and Charlotte belts, for example, are believed to be one or more island arcs subsequently accreted to the margin of North America.[(6)(18)(28)] In the southern Appalachians, closure of the oceanic basin between North American and previously rifted continental and island arc fragments, and subsequent collision of these land masses during the Cambrian and Ordovician, produced the Taconic orogeny (480 to 450 million years ago). Fragments composing the Inner Piedmont-Blue Ridge terrane were accreted to North America by overthrusting.[(6)(9)(23)] Extreme deformation, metamorphism, and plutonism in the Inner Piedmont accompanied the orogeny. Sediment loading occurred on the cratonic lithosphere in past-Taconic time from detritus eroded from mountainous terrain to the east, representing a radical change in provenance compared to pre-Taconic paleogeography.

Continued closure of the Iapetus Ocean by subduction resulted in the collision of the Carolina Slate Belt with the Inner Piedmont-Blue Ridge terrane about 300 to 400 million years ago, approximately concurrent with the Acadian orogeny (Late Silurian through Late Devonian), and was accompanied by additional metamorphism and deformation. The intervening Kings Mountain belt and Charlotte belt include rocks caught in the collision. Rocks of the Charlotte belt represent high-grade metamorphic equivalents of the Carolina Slate belt and are of island arc affinity. Rocks of the Kings Mountain belt are predominantly a mixture of Charlotte belt and Inner Piedmont rocks. During the collision the allochthonous Inner Piedmont-Blue Ridge fragment was thrust further westward over the old continental margin.

The last phase of paleozoic plate convergence along the eastern margin of North America culminated in the continental collision between proto-North America and proto-Africa during the Alleghenian orogeny, from Middle Pennsylvanian through Early Permian time (300 to 250 million years ago). Most of the prominent Appalachian structures and provinces assumed their present configuration during this orogeny. Extensive igneous plutonism in the eastern Piedmont accompanied the orogeny, and large scale overthrusting deformed the sedimentary rocks in the Valley and Ridge province. Thrusting of the Piedmont over the Blue Ridge occurred along the Brevard zone at this time.

Over the past several years deep seismic reflection studies have suggested that these various thrust faults conveying the allochthonous slices westward are splays of a deeper, regionally extensive detachment surface or decollement. These studies suggest that the decollement extends eastward beneath the Piedmont province and may continue beneath the Coastal Plain and Atlantic shelf.[(10)(11)(13)(16)(17)(22 24)(26)(27)(34)(36) (40-43)] The seismic profiles from Pennsylvania, Virginia, eastern Tennessee, South Carolina, and Georgia show laterally continuous subhorizontal reflectors ranging in depth from less than 6 km in the Valley and Ridge to more than 11 km beneath the Coastal Plain. Cook, et al.,[(22, 24)] and Harris and Bayer[(27)] interpret these subsurface reflectors to be elements of a regionally extensive decollement underlain by relatively undeformed, weakly unmetamorphosed, layered Paleozoic sedimentary rocks. The subhorizontal east dipping decollement presumably accommodated crustal foreshortening associated with continental collision in late Paleozoic time when metamorphic Precambrian and early Paleozoic rocks of the Blue Ridge and Piedmont were thrust to the west and northwest over relatively unmetamorphosed sedimentary rocks of the Valley and Ridge.

The southeastern extent of the decollement is still poorly known. The seismic reflection profiles confirm the presence of a decollement from the Valley and Ridge to at least as far as the inner Piedmont where the subsurface reflectors thicken, become discontinuous, and dip steeply to the southeast.[(13)(22- 24)] In one interpretation, the steeply dipping reflectors mark the root zone of the decollement beneath the Kings Mountain Belt of the outer Piedmont.[(12)(21)(32)(36)] In a second interpretation, Cook, et al., [(22)(24)] suggest that the decollement may be rooted offshore where a complex structural configuration is evident on the seismic profiles.

Most investigators agree that major movement on the decollement occurred during the Alleghenian orogeny, although it must also have been active in some form during the earlier Taconic and Acadian orgenies. Such a model provides a rational kinematic mechanism for the thin-skinned fold and thrust deformation of the Valley and Ridge Province and the emplacement of the crystalline Blue Ridge and piedmont rocks as relatively thin, allochthonous sheets. Latest thrust movement is constrained to a time preceding the onset of Mesozoic extensional tectonics. Ellwood, Whitney, and Wenner(44) describe a 350-million-year-old intrusive complex (Elberton Granite) which was emplaced across the decollement; the intrusive complex would therefore provide a minimum age for thrusting.

2.5.1.1.2.2 Mesozoic Era.

2.5.1.1.2.2.1 Triassic and Jurassic Periods. The tectonic model which best explains the stratigraphic distribution of lower Mesozoic rocks on the eastern coast of North America includes the following sequence: (1) Permian to Late Triassic uplift and crustal thinning along the axis of the future Atlantic Ocean, (2) Middle to Late Triassic strike-slip faulting and volcanism along east-trending fracture zones followed by the advance of the Tethys Sea, and (3) Late Triassic rifting along the axis of the proto-Atlantic Ocean and shearing along east-west fracture zones. This action had the combined effect of decoupling segments of the African and North American plates and causing deposition of clastic sediments in the Triassic basins that formed. Late Triassic to Early Jurassic crustal extension and extrusion of basaltic lavas were followed by collapse of the continental margins and simultaneous deposition of marine carbonates.(45)

On the eastern seaboard of the United States many of the Triassic basins are exposed at the surface in the Piedmont province (see, for example, plate 1 of Wentworth and Mergner-Keefer(46)). Most, however, are covered by Coastal Plain sediments and have been delineated on the basis of core holes and wells (Plate 1 of Chowns and Williams(15)), aeromagnetic and gravity anomalies,(25)(42)(47) and seismic reflection and refraction studies.(11)(24) These Triassic basins are shown on drawing AX6DD409. (For a general discussion of the development of Mesozoic rift basins in the southeastern United States, see Klitgord, Dillon, Popenoe(37) and Dillon, Klitgord, and Paull.(48))

The Triassic basins were filled with fanglomerates, landslide debris, and mudflow deposits originating along steep fault scarps. Streams draining the surrounding metamorphic highlands pumped most of the very coarse material close to the edge of the basin, but periodic increases in stream energy, due either to increased precipitation or to renewed uplift, carried coarser particles to the center of the basin.(49-51)

The Triassic sediments in the Dunbarton Basin beneath the plant site at one time may have reached a maximum thickness of 6000 to 8000 ft greater than at present. This estimate is based on the conversion of montmorillonite to illite, which varies with depth of burial.(49) Subsequent erosion reduced both mountains and valleys to a single peneplained surface.

2.5.1.1.2.2.2 Cretaceous Period. Both Cretaceous and Tertiary sediments of the Coastal Plain Province accumulated on the trailing eastern margin of the continent. The composition of these sediments and their gentle dip away from the Appalachian Mountains implies that the Appalachians have stood as an eroding structural high for over 200 million years.(52)

Following a period of uplift and erosion during the Late Jurassic and Early Cretaceous, there was a transgression of Late Cretaceous seas over part of the Coastal Plain.(53) The basal clastic formation in the vicinity of the plant site is the subaerial Tuscaloosa formation.

Deposition of this formation began sometime between 100 and 94 million years ago. A period of nondeposition occurred in the Upper Cretaceous series of South Carolina between 94 and 82 million years ago,[(54)(55)] which may be correlated in part with an erosional surface within the Tuscaloosa formation downdip of the plant site.[(56)(57)] Following this period of erosion the sea again transgressed onto the continent, and deposition of the Tuscaloosa continued in an estuarine environment near the plant site.

Rocks deposited during the close of the Cretaceous are not present in Georgia or South Carolina.[(56-58)] The Cretaceous-Tertiary boundary is marked by an erosional surface which would be due, in part, to a fall in sea level.[(53)]

During the Cretaceous Period and continuing into the Cenozoic Era structural deformations in the form of mild regional warping and faulting, or reactivation of older faults, occurred.

The Southeast Georgia Embayment of Toulmin[(59)] includes an area of downwarping and sediment thickening which formed during Cretaceous and Cenozoic time.[(56)(60)] This feature has also been called the Okefenokee Embayment[(61)] and the Atlantic Embayment of Georgia.[(62)] A second sedimentary basin, the Appalachicola Embayment, is an area of thickened Tertiary sediments extending into the southwest corner of Georgia. This feature has also been called the Southwest Georgia Basin (LeGrand, 1961;[(63)] Murray, 1961[(64)]). Between these two embayments is a positive feature called the Central Georgia Uplift,[(61)] which is defined as a southeast-northwest striking upwarped feature between the two flanking downwarped areas. The southern extension of the Central Georgia Uplift is the Peninsula Arch[(65)] which also forms the spine of Florida.

The Yamacraw Ridge is a basement feature trending parallel to the Georgia and South Carolina coastlines.[(60)] Maps by Herrick and Vorhis[(62)] show that this feature may have had some influence on Upper Cretaceous sedimentation.[(56)]

Several small undulations appear within the confines of the Appalachicola Embayment. Most have been recognized from subsurface data, although a few are expressed as surface features. The folding in southwestern Georgia appears to be of Tertiary age, and some folding may have occurred as late as Miocene.[(60)(66)]

Faults with minor displacement of Cretaceous and Cenozoic deposits are present in the southeastern United States.[(67)] Recent detailed work has indicated that northeast-trending faults with Later Cretaceous and Cenozoic displacements such as the Belair, Cooke, and Stafford fault zones exist in the Atlantic Coastal Plain and Piedmont.[(40)(68)(69)] Wentworth and Mergner-Keefer[(70)] propose that many of these faults may be reactivated Mesozoic and older high angle normal faults. Other isolated instances of Cretaceous and Cenozoic faulting in the coastal plain region have been listed by Prowell.[(71)]

Herrick and Vorhis [(62)] identified a feature they called the Gulf Trough within the Appalachicola Embayment from isopach and structure contour maps prepared from subsurface data. The structure is linear and more sharply defined than the surrounding folds, and for these reasons Cramer[(60)] concludes that the trough was formed by faulting. In an earlier work, Callahan[(72)] interpreted this feature as two parallel, down-to-the-southeast faults. Cramer and Arden[(56)] also suggested evidence for faulting within the trough. Various authors have suggested possible mechanisms for its formation.[(56)(62)(73)(74)] Patterson and Herrick[(74)] have reviewed the proposals which include: normal faulting producing a graben; down-warping forming a syncline; and a Tertiary marine strait or valley.

2.5.1.1.2.3 Cenozoic Era.

2.5.1.1.2.3.1 Tertiary Period.

2.5.1.1.2.3.1.1 Paleocene Epoch. Sediments deposited during the early Paleocene are thickest in the southwest, indicating that seas transgressed from that direction. Following this period of deposition, uplift of the region resulted in the erosion and removal of most of these rocks in Georgia.(56)(75)This uplift was accompanied by faulting in response to the tectonic forces resulting from the northwestward drift of a passive continental margin.(76)

A second transgression occurred in the late Paleocene.(56)(75) Although this transgression is thought to have been extensive, no upper Paleocene sediments are interpreted to exist in the plant site area.

2.5.1.1.2.3.1.2 Eocene Epoch. Following a period of erosion during the early Eocene, the sea again transgressed over the Georgia Coastal Plain during the middle Eocene. The bulk of the middle Eocene sediments are carbonates, with up to 10% chert and evaporite. Toward the Fall Line all of the carbonate rocks become coarser and grade into calcareous sands, indicating a higher energy environment. Onlap of the marine sediments onto the Coastal Plain is evident and paleontological data indicate that the transgression was very slow.(56) Following the transgression of the middle Eocene seas, regression again occurred and erosion of the middle Eocene deposits began.

Late Eocene deposition is represented by a relatively thin, uniform blanket of shelf limestones and calcareous sands, which unconformably overlie deposits of middle Eocene age. Northeastward along the Fall Line the fluctuating strandline of the middle Eocene sea is apparent in the intertonguing of carbonate and clastic formations. A period of regression is apparent, and deposits of late Eocene age are overlain by upper Oligocene deposits.

2.5.1.1.2.3.1.3 Oligocene Epoch. At least two transgression/regression cycles occurred during the Oligocene. Only the late Oligocene transgression deposited material in the site area. The full extent of this overlap (Suwannee) is not known, since an undetermined quantity of updip rocks have been removed by erosion. Facies patterns indicate that the overlap was probably extensive. The Suwannee rocks that remain are shelf deposits, with none of the updip clastic facies preserved.

2.5.1.1.2.3.1.4 Miocene Epoch. The deposits of Miocene age appear to be a sequence of predominantly clastic sediments deposited during and following the regression of the coastline. In some places (VEGP site included) erosion has continued from the Miocene to the present.

2.5.1.1.2.3.2 Quaternary Period. During Pleistocene time the sea transgressed over the eastern part of the Coastal Plain several times. Each transgression/regression cycle left a distinct terrace as evidence of its occurrence. Surface uplift and subsidence of the Coastal Plain of Georgia and surrounding states continued through the Pleistocene.(77) Sediments have accumulated and related geomorphic features such as erosional scarps, and terraces have continued to develop over the last 1.8 million years.

2.5.1.1.3 Stratigraphy and Lithology

The stratigraphic nomenclature used to describe the formations in the Coastal Plain of Georgia has recently undergone a certain amount of reinterpretation and some of the formation names and intraformational boundaries are currently being changed.

The names of the stratigraphic units may change, but the formations are distinct, both lithologically and geophysically, and can be correlated. The stratigraphic nomenclature adopted for this report accepts, in general, the current thought of geologists working in the area. The terminology used was selected because, although not yet completely formalized, it is thought to accurately represent the best knowledge of the stratigraphic framework of the study area. A correlation chart (drawing AX6DD339) and a lithologic chart (drawing AX6DD340) are provided for reference.

2.5.1.1.3.1 Precambrian and Paleozoic Rocks. The crystalline basement rock ranges in age from late Precambrian through Paleozoic. The basement rocks exposed northwest of the VEGP site include the gneisses and granites of the Kiokee Belt and the phyllites and greenstones of the Belair Belt.[78][79] The upper surface of the basement rock has been eroded, tilted to the southeast, and buried. The general plane of this surface strikes approximately N62°E and dips southeast at 36 ft/mi.[80]

2.5.1.1.3.2 Mesozoic Rocks.

2.5.1.1.3.2.1 Triassic System. Triassic basins occur along the eastern seaboard from Connecticut south to Florida (drawing AX6DD409). Basins north of South Carolina are exposed in Piedmont crystalline rocks, while those south of North Carolina are overlain by Cretaceous and Cenozoic sediments.

The sediments within these basins have been tentatively correlated with the Newark Supergroup of Late Triassic through Early Jurassic age.[45][80-83] It is difficult to obtain an accurate age for the sedimentary rocks within these basins due to their time-transgressive nature.[45]

As shown on drawing AX6DD409 the plant site is underlain by the buried Dunbarton Triassic Basin. The sediments within this basin have been identified as Triassic, based on stratigraphic position and lithology. No microfossils[49][50][84] or igneous rocks[51][85] indicating Jurassic age have been found in the Dunbarton Basin.

Marine and Siple[84] have presented a complete lithologic description of the Triassic rocks of the Dunbarton Basin based on drill cores. In the central northwest portion of the basin, sediments consist of red-brown breccias in a matrix of claystone and siltstone. The central part of the basin is composed of alternating layers of sandstone and mudstone. Rocks from what may be the southeastern part of the basin include siltstones, claystones, and fine-grained sandstones which contain calcareous nodules.

2.5.1.1.3.2.2 Cretaceous System. The Upper Cretaceous Tuscaloosa Formation consists of fluvial and estuarine deposits of cross-bedded arkosic sand and minor gravel intercalated with lenses of variegated white, pink, red, brown, and purple silt and clay.[56][57][80] Coarse and fine sediments are interbedded in an irregular sequence and grade laterally into one another or pinch out within short distances. Abundant kaolin is present along with other clay minerals.

2.5.1.1.3.3 Cenozoic Deposits.

2.5.1.1.3.3.1 Tertiary System.

2.5.1.1.3.3.1.1 Paleocene Series. The lower Paleocene series in the vicinity of the site consists of the Ellenton and the Huber Formations (drawing AX6DD339).

Ellenton Formation

The Ellenton Formation is a dark-gray to black sandy lignitic micaceous clay interbedded with medium- to coarse-grained quartz sand. Authigenic gypsum is commonly associated. The lower part of the Ellenton is sandy lignitic clay with the sand portion becoming very coarse and gravelly.

The Ellenton is unconformable with the underlying Tuscaloosa Formation. The contact is characterized by a change in the color of the clay and in the composition of the sand. The Ellenton grades into the overlying Huber Formation in the vicinity of the plant site.(86)

Siple(80) originally assigned the Ellenton to the Late Cretaceous, but recent workers have assigned it an age of early Paleocene.(87)(88)

Huber Formation

The Huber Formation lies between the top of the Ellenton Formation and base of the overlying sands and limestones of middle Eocene age. The lithology of the Huber Formation is diverse, ranging from beds of multicolored clays, high-purity and sandy kaolin, to thick cross-bedded members of coarse, pebbly sand and conglomerate composed of boulders of pisolitic kaolin.(89) In drill cores the uppermost part of the Huber Formation shows signs of weathering and chemical reduction.

2.5.1.1.3.3.1.2 Eocene Series. The Eocene series consists of the middle Eocene Lisbon Formation and the upper Eocene Barnwell Group.

Lisbon Formation

The Lisbon Formation occurs between the top of the Huber Formation and an unconformity at the base of the Barnwell Group. In east-central Georgia the Lisbon Formation is subdivided into three members: an unnamed basal sand and limestone member, the Blue Bluff Member, and the McBean Limestone Member.

The lowermost portion consists of a quartz sand which grades both up section and downdip into a calcareous sand. Overlying these sands is a limestone. The Blue Bluff Member is a greenish- to bluish-gray, moderately hard calcareous siltstone of marl. In core holes recently drilled near the plant site the marl is thinly interbedded to laminated with isolated limestone nodules and shell fragments.(86) Updip, the McBean Limestone Member is composed of soft, gray limestone and calcareous sand. Downdip, the Blue Bluff Member interfingers with an unnamed gray calcareous sand and fossiliferous limestone.

At the plant site, the Blue Bluff Member is a bluish-gray marl. This marl forms the foundation for critical plant structures and structural backfill. This is discussed further in paragraph 2.5.1.2, subsection 2.5.4, and appendix 2B.

Barnwell Group

In east-central Georgia the Barnwell Group consists of the Clinchfield Formation which contains the Utley Limestone Member; the Dry Branch Formation which contains the Irwinton Sand, Griffins Landing, and Twiggs Clay Members; and the Tobacco Road Sand. Downdip the Barnwell Group grades into the carbonate facies of the Ocmulgee, Crystal River, and Williston Formations of the Ocala Group.

Clinchfield Formation

The Utley Limestone Member of the Clinchfield Formation is typically a sandy, glauconitic slightly argillaceous, and locally cavernous limestone of varying degrees of induration.(90)

Dry Branch Formation

The Dry Branch Formation consists of three distinct, interfingering lithofacies: a montmorillonite clay (Twiggs Clay); a distinctly bedded sand (Irwinton Sand); and an indistinctly to massively bedded, calcareous, fossiliferous sand (Griffins Landing).

The Twiggs Clay is a pale-greenish, olive-green, bluish-gray, dark-gray, or locally almost black silty clay with hackly, blocky, subconchoidal to conchoidal fracture and is found as interbeds in both the Irwinton Sand and the Griffins Landing Members of the Dry Branch Formation. The Irwinton Sand consists of fine- to medium-grained, well-sorted, deeply weathered, almost pure quartz sand that shows well developed horizontal and local cross-bedding in outcrop. Downdip, the Irwinton Sand interfingers with the Griffins Landing Member, a fairly well-sorted, massive to indistinctly bedded calcareous sand. The unit often contains lenses of Twiggs Clay associated with oyster shell beds. Downdip, the Griffins Landing grades into the Williston Formation, a nonfossiliferous, sandy equigranular limestone.

Tobacco Road Sand

The uppermost formation within the Barnwell Group is the Tobacco Road Sand, which is predominately quartz sand. The sand in the Tobacco Road varies from fine-grained and well-sorted to very coarse-grained, granular, pebbly and poorly sorted. The Tobacco Road is characteristically massively bedded and bioturbated, although locally the formation may be thinly and distinctly bedded, even laminated.(90)(91) In east-central Georgia the Tobacco Road Sand grades downdip into the limestone facies of the Ocmulgee Formation. Further downdip, the Ocmulgee grades into the Crystal River Formation.(86)

2.5.1.1.3.3.1.3 Oligocene Series.

Suwannee Limestone

Downdip of the plant site the Suwannee Limestone rests unconformably upon the Ocala Group which is the downdip equivalent of the Barnwell Group. (See drawing AX6DD339.) The basal part of the Suwannee consists of a sandy limestone that contains few fossils. Above this is a layer of predominately cream-colored, relatively soft, somewhat chalky, fossiliferous limestone. The upper part is a light-gray to cream color, dense nodular, cherty, and somewhat sandy limestone.(56)

2.5.1.1.3.3.1.4 Miocene Series. The Hawthorne Formation is the youngest Tertiary Formation in the vicinity of the plant site. The formation has been assigned to earliest Miocene (25 to 23 million years before present) or Altamaha age.(88) Hawthorne sediments include poorly sorted clayey sands and gravels, containing cross-bedded stringers of liminite-geothite pebbles. The sediments are variegated, orange through violet, with mottled or alligator-skin appearance due to weathering.

Exposures of Hawthorne and Barnwell Formation sediments in the region commonly contain patterned weathering structures. The weathering has produced an upper zone, commonly 2 to 3 ft thick, of mottled blotches and horizontal planes of offwhite bleached zones within the deep red sediments. Below this zone a series of vertical weathered fractures is found. The vertical features normally taper downward and pinch out within 10 ft of the upper Tertiary sediment surface. These features have been described as clastic dikes by various authors.

The occurrence of clastic dikes in Coastal Plain sediments has resulted primarily from alteration along near vertical fractures during a paleosol development. The material within the dikes consists of the same material as the host sediments, with some dikes containing high proportions of clay. The origin of the fractures is difficult to determine. Large exposures of dikes show polygon development associated with desiccation. Local small faults associated with solution collapse structures have dike alteration along them. In several locations the near vertical dike faults are offset by low angle reverse faults. These thrust faults are interpreted to result from later settlement and collapse, although the timing and exact relationship is unknown.

The geographic distribution of clastic dikes is the result of the paleoenvironment which caused the desiccation and alteration. The grain size and conduit geometry of the liquefaction feature studied by Cox[92] is very different from the clastic dikes found in the site area. It is concluded that clastic dikes near the site cannot be attributed to tectonic activity.

2.5.1.1.3.3.2 Quaternary System. The Quaternary system is represented by alluvial deposits consisting of coarse gravel and poorly sorted sand which occur irregularly and discontinuously in the tributary and main channels of the Savannah River.

2.5.1.1.4 Regional Structural Geology

Major structural and tectonic features in Georgia and South Carolina are shown on drawing AX6DD409. The major structural trend affecting the region is the pre-Mesozoic southern Appalachian Mountain system, exposed west of the Fall Line. Virtually all tectonic activity occurred prior to the deposition of the Cretaceous sediments east of the Fall Line. The complex folding, faulting, and shear structures that developed in the Piedmont, Blue Ridge, and Valley and Ridge Fold belts originated in the Precambrian and Paleozoic eras during one or more of the orogenic episodes associated with the development of the southern Appalachians. Relatively undeformed Coastal Plain sediments show that this orogenic activity ceased prior to the Cretaceous. Additional evidence for the pre-Cretaceous termination of orogenic activity is the lack of offset of the numerous Triassic diabase dikes which cross the earlier structural features.

The crystalline basement underlying the Georgia Coastal Plain dips toward the southeast at approximately 36 ft/mi. This regional dip is interrupted by several local structures.

2.5.1.1.4.1 Tectonic Framework of the Georgia Coastal Plain.

2.5.1.1.4.1.1 Triassic Features. The Dunbarton Basin (drawing AX6DD409) is one of several elongated basins filled with Triassic (and in some other cases Jurassic) rocks found buried beneath the Cretaceous and Cenezoic age sediments of the Georgia Coastal Plain.

The most probable origin of the Dunbarton Basin is the formation of a graben by normal faulting. Evidence has been presented for a northwest border fault of unknown displacement, and faulting has been hypothesized for the southeastern margin.[49] Substantial evidence for a southeastern border fault is lacking, however, and the nature and extent of this margin of the Dunbarton Basin is derived from gravity and aeromagnetic surveys.[49][84] The basin is oriented northeast-southwest and is about 31 miles long and 6 miles wide (drawing AX6DD409) based on an aeromagnetic survey.

Recent geophysical studies have indicated the possibility of intrabasinal faulting, but an attempt to verify this by analyzing drill cores was inconclusive.(49)(84)

Because of stratigraphic thickness and the nature of the gravity and magnetic data, faulting is a likely explanation for the southeastern boundary of the Dunbarton Basin.

2.5.1.1.4.1.2 Cretaceous and Cenozoic Features. The dominant structural features of the Georgia Coastal Plain are two large sedimentary basins separated by structural drawing AX6DD409). The southeast Georgia Embayment(59) includes an area of downwarping and sediment thickening which formed during Cretaceous and Cenozoic time.(56)(60) A second sedimentary basin, the Appalachicola Embayment, is an area of thickened Tertiary sediments into the southwest corner of Georgia.(61) Between these two embayments is a positive feature called the Central Georgia Uplift,(61) which is defined as a southeast-northwest striking upwarped feature between the two flanking downwarped areas. The southern extension of the Central Georgia Uplift is the Peninsular Arch,(65) which also forms the spine of Florida. The Yamacraw Ridge is a basement feature trending parallel to the coastlines of Georgia and South Carolina which may have had some influence on Upper Cretaceous sedimentation.(60)

2.5.1.1.4.2 Faulting. Faults with minor displacement of Cretaceous and Cenozoic deposits are present in the southeastern United States.(67) The geology of the southeastern Atlantic Coastal Plain, however, is such that faulting is not easily recognized. Recent detailed work has demonstrated that northeast-trending faults with Late Cretaceous and Cenozoic reverse displacements do exist in the Atlantic Coastal Plain and Piedmont.(40)(68)(69)(93)

2.5.1.1.4.2.1 Belair Fault Zone. The Belair fault zone is a structural feature extending along the inner margin of the Atlantic Coastal Plain (drawing AX6DD409). This fault is located a few miles west of Augusta and extends for about 29 miles from Fort Gordon Military Reservation on the south to a quarry just west of the Savannah River on the north.(78)(69)(94)

The Belair fault zone has been shown to consist of at least eight en echelon reverse faults trending from N23°E to N50°E and dipping 50° to the southeast.(69) The fault zone juxtaposes crystalline phyllite of the Little River Series of late Precambrian or Cambrian age with Coastal Plain kaolinitic sands and gravels, which are formally correlated with the Upper Cretaceous Tuscaloosa Formation.(46)(93) Individual fault segments are from 1 to 3 miles in length, with gouge zones only a few feet wide at most. According to Prowell and others(94) the basal Tuscaloosa unconformity is vertically displaced from 15 to 100 ft. The most recent documentable movement along the Belair fault zone occurred about 40 million years ago.(46)(70)

2.5.1.1.4.2.2 Gulf Trough. Much controversy surrounds the structure and origin of the Gulf Trough of Georgia (drawing AX6DD409). The structure is linear and more sharply defined than the surrounding folds, and for these reasons Cramer(60) concludes that the trough was formed by faulting. If the Gulf Trough is due to faulting, the available data indicate that this movement would have occurred prior to the beginning of the Miocene.(62)(95) The evidence suggests that other geologic phenomena, such as erosion, local variations in regional tilt, local subsidence, or warping can also explain the trough.(95)

The origin of the Gulf Trough is not clear. Various authors have suggested possible mechanisms for its formation.(60)(73)(74) (96) Patterson and Herrick(74) have reviewed the proposals which include: (1) normal faulting producing a graben, (2) downwarping forming a syncline, and (3) a Tertiary marine strait or valley.

2.5.1.2 Site Geology

2.5.1.2.1 Site Physiography and Geomorphology

The site is located near the boundary between two topographic subdivisions of the Atlantic Coastal Plain province (drawing AX6DD338). These are the Tifton Upland to the southwest, upon which the site is located, and the older terraces to the northeast. The nearly flat topography of the older terraces is separated from the moderately hilly Tifton Upland by an abrupt 70- to 100-ft-high bluff cut by the Savannah River, which flows along its base.

2.5.1.2.1.1 Tifton Upland. The plant is located on rolling hills at about el 300 ft. Elevations in the area range from 80 ft at the Savannah River to 280 ft at the crest of a knoll near the plant. Surface drainage is primarily northeastward toward the river via a dendritic stream pattern which surrounds the property. Rainfall is relatively evenly distributed on a monthly basis, and, except during heavy storms, rain tends to soak in rather than run off. The solution and removal of carbonates from shallow underlying beds of calcareous sands and shells have resulted in the formation of local depressions, creating areas of internal drainage. Since these soluble zones occur within nearly horizontal strata resting upon an essentially impervious, hard, clay marl, springs generally have emerged at the top of exposures of the marl, causing sapping and headward erosion of the overlying sands and clays and the formation of amphitheaters and eventually ravines. Where shell deposits are thick, small-scale cavernous conditions occur along preferred percolation paths. The coalescing of the solution depressions or collapse of these small subterranean channels on the top of the clay marl results in ravines with apparently small drainage areas and with amphitheaters at the head.

2.5.1.2.1.2 Older Terraces. The older terraces subdivision of the Atlantic Coastal Plain province (drawing AX6DD338) is represented principally by the Savannah River alluvial plain, which in the site area is broad and flat and at an elevation of 80 to 90 ft. The river valley is broad and mature and includes low, dissected, old marine terraces as well as various river plain features, such as cutoff oxbows and natural levees.

2.5.1.2.1.3 Site Geologic History. The site area is located upon a seaward-thickening wedge of sediments 950 ft thick at the plant, deposited upon the truncated and peneplained roots of the ancestral southern Appalachian Mountain system (paragraph 2.5.1.1.2). Igneous and metamorphic rocks of Precambrian through Paleozoic age and early Mesozoic Triassic sediments comprise the basement rock at the site. These deformed and faulted basement rocks reflect the complex geologic history of the Appalachian Mountain system, which has been essentially quiescent since late Mesozoic peneplanation. This period of tectonic stability during and following deposition of the sediments is evidenced by their nearly flat-lying and relatively undeformed nature. The seaward thickening of the sedimentary mantle indicates a progressive downwarping of the continental margin. Regional uplift of the Coastal Plain is the latest and current stage of the geologic history of the site area.

The stratigraphic and structural relationships of the lithologic units at the site reflect the geologic history of the region. The site area was relatively stable following the deposition of the nonmarine Cretaceous Tuscaloosa Formation and the overlying Ellenton Formation of Early Paleocene age. Unconformably overlying the Ellenton Formation in the site area are the Lisbon Formation and Barnwell Group of Eocene age, which are in turn unconformably overlain by the Hawthorne Formation of Miocene age. The Hawthorne Formation is the youngest deposit of formation status exposed in the site area.

The Tertiary shallow marine deposits represent periods of marine transgressions and regressions from Eocene through Miocene times, most likely the result of periods of minor regional uplift and subsidence. For example, the Barnwell Group includes lithologic units varying from coarse sand to clay and marl, zones of weathering and variations in fossil abundances indicative of variable near-shore and tidal conditions.

The current stage of regional uplift is evidenced in the site area by exposures of the Miocene marine Hawthorne Formation at elevations above 250 ft. The mature geomorphic expression and deep weathering of the Hawthorne Formation and exposures of the underlying Barnwell Group indicate an extended period of orderly erosion on a stable surface of emergence.

2.5.1.2.2 Site Lithology and Stratigraphy

The site lithology has been determined from the following:

- Geologic and foundation exploration borings.
- Seismic refraction surveys.
- Correlations between holes using spontaneous potential, resistivity, and gamma logs.
- Geological mapping of the surface and foundation excavations for plant structures.
- Millett fault study of 1982.

Surface distribution of geologic materials is shown on the 5-mile-radius local geologic map (drawing AX6DD345) and the site geologic map (drawing AX6DD351). Subsurface geological conditions are shown on the geologic sections (drawing AX6DD352). The drill logs of borings are discussed in appendix 2B. The stratigraphic succession of the lithologic units at the site and as used in this report is shown on drawings AX6DD339 and AX6DD340.

The geologic formations which were encountered during site exploration are shown on drawing AX6DD352 and table 2.5.1-1 and include materials ranging in age from Cretaceous to Eocene. Cretaceous sediments are known to underlie the site area and crop out a little more than 5 miles northeasterly from the site near the old town site of Ellenton. However, no materials identified as Cretaceous crop out within a radius of 5 miles of the site.

2.5.1.2.2.1 Pre-Tertiary. The pre-Tertiary rocks which underlie the site are described in paragraph 2.5.1.1. To summarize, approximately 600 ft of Cretaceous sediments rest unconformably upon a truncated and peneplained lithologic complex of Triassic, Paleozoic, and Precambrian age composed of indurated sediments, intrusive and extrusive igneous rocks, and metamorphic rocks.

2.5.1.2.2.2 Tertiary System. In the site area, all geologic exposures are sediments of Eocene through Miocene age, except for local alluvial cover. Most exploratory drill hole intercepts include sediments of Eocene age and, where drilling started at higher ground surface elevations, sediments of Miocene age. Deep borings, such as TW-1, encountered Paleocene and Cretaceous sediments. The regional stratigraphy is discussed in paragraph 2.5.1.1 and shown on drawings AX6DD339 and AX6DD340. The generalized lithology of the site, which is based in part on data obtained from exploratory drilling at the vicinity of the plant site, is presented in table 2.5.1-1.

2.5.1.2.2.2.1 Eocene Series. The Eocene series in the site area consists of two lithologic units. The older is the Lisbon Formation, which includes the bearing unit for the plant structures; the younger is the Barnwell Group. The local lithologic characteristics and stratigraphy of these formations are summarized in table 2.5.1-1 and discussed in paragraphs 2.5.1.2.2.2.1.1 and 2.5.1.2.2.2.1.2.

2.5.1.2.2.2.1.1 Lisbon Formation. The Lisbon Formation of middle Eocene age is exposed only along the Georgia side of the Savannah River. In general, the exposed lithologic unit of this formation is an approximately 60-ft-thick, greenish-gray, fossiliferous clay marl with intercalated lenses of limestone. This clay marl unit, which is the bearing bed for the plant structures, is the Blue Bluff Member of the Lisbon Formation.

The lower portion of the Lisbon Formation, which is known in the site area only from exploration drilling, consists of an unnamed, approximately 100-ft-thick bed of fine-grained sand. The lower contact of the Lisbon Formation with the Paleocene Huber and Ellenton Formations is not exposed in the mapping area. Below the Lisbon Formation is the approximately 50-ft-thick lithologic unit comprised of interbedded clay, silty sand, and lignitic beds representing the Huber and Ellenton Formations. The upper contact of the Lisbon Formation with the Barnwell Group is well exposed in the power block excavation for the VEGP and along the Savannah River in the vicinity of the plant.

The best natural exposures of the Lisbon Formation within the mapping area are at Blue Bluff. They are described in detail in the report of investigation of the marl.[97] Excellent exposure of the Lisbon Formation in the auxiliary building excavation for the VEGP was mapped and described in the reports of the power block excavations.[98][99] Drawings AX6DD352, AX6DD360, AX6DD361, AX6DD362, AX6DD363, AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, and AX6DD370 show in detail a lithologic stratigraphic succession similar to that reported in the section at Blue Bluff.

Numerous black shark teeth were found in the interval immediately below the marl, and microfossil analysis of a sample taken from hole 152 just below the base of the Blue Bluff marl indicates an Eocene age for this material.

The Blue Bluff marl is a distinct unit that is relatively constant in thickness over many square miles, although variable in lithology. As may be seen from drawing AX6DD352, the marl has been eroded from much of the Savannah River flood plain and covered over in part by the higher river terraces. It is completely eroded from the section in hole 36. In hole 45, some 3 miles farther away, a facies change has occurred, with the marl becoming dense gray-green, silty sand and silty clay.

Parallel to the river, however, it is 50 ft thick at Shell Bluff, approximately 1.5 miles northwest of the site, and 65 ft thick at hole 156 on the Griffin Landing Road, nearly 5 miles to the southeast. Isopachs of the marl in the plant area are shown on drawing AX6DD371.

2.5.1.2.2.2.1.1.1 Lisbon Formation in the Power Block Excavation. The upper Eocene Lisbon Formation is represented in the site area by the Blue Bluff Member marl, which is the foundation for structures in the power block area. The marl has a total thickness of about 70 ft in the site area (on drawing AX6DD371). The upper approximately 25 ft of the marl were exposed in excavations and mapped in detail (drawings AX6DD360, AX6DD361, AX6DD362, AX6DD363, AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, AX6DD370, AX6DD372, AX6DD373, AX6DD374, and AX6DD375). A vertical section between el 108.6 ft (final excavated grade) and el 132 ft was exposed in the auxiliary building basement excavation. Ten subunits of the marl were recognized and mapped in this vertical section. The subunits, designated A through J are shown on drawings AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, and AX6DD370 and described in the following paragraphs.

Unit A, near the top of the excavation walls, is generally above el 128 ft to the upper contact of the marl with the Utley Limestone Member of the Barnwell Group. It consists of dark-gray, silty to clayey marl with very fine light-gray to white, fine, sandy laminations which are undulatory and discontinuous.

Scattered shell fragments and well-cemented lenses of sand up to 0.1 ft thick are present locally. The laminations are oriented parallel to the lower contact of the unit, and parting along the laminations is common. Unit A is dense and well consolidated. Surfaces exposed to the atmosphere tend to dessicate rapidly. Unit A interfingers with the underlying unit B. This was especially evident in the south wall of the vicinity of stations 0+70, 1+50, and 4+30 (drawings AX6DD365, AX6DD367, and AX6DD368). The contact with unit B is everywhere gradational.

Unit B, directly beneath unit A, was continuous around the auxiliary building basement excavation walls and varies from 1 to over 4 ft in thickness. It consists of massive to faintly laminated gray, sandy marl. It has a sugary texture and does not tend to dessicate as readily as does unit A. This property provides an easy means for differentiating the units after exposure to the atmosphere. Unit B is dense but poorly cemented and contains widely scattered shell fragments. A subunit of B, designated B_1, has been identified and is present locally within B. This subunit consists of laminated sandy marl which is locally fossiliferous. Subunit B_1 has been mapped at the base of B in the east wall and the easterly portions of the north and south walls. (For example, see drawing AX6DD366.) The contacts between B and B_1 are highly gradational.

Unit B is in turn underlain by a thin, relatively discontinuous but laterally extensive limestone, designated as unit C. This limestone is light gray and well indurated, and it exhibits conchoidal fracturing. It was continuous in the west end of the south wall but becomes discontinuous east of station 0+80. East of station 3+65, the limestone becomes a series of small, irregular, discontinuous pods at varying elevations (drawing AX6DD367). Where exposed in the north, east, and west walls, the limestone formed discontinuous lenses at a relatively consistent elevation. It averaged about 1 ft in thickness and dipped slightly to the east, being present at about el 127 to 128 ft at the west end of the auxiliary building and 125 ft at the east end.

During excavation of the auxiliary building basement, the irregularity of portions of unit C led to a special study to determine whether the irregularities could be related to fault offset. The concern was that lenses and pods of the limestone occurring at slightly different elevations might have been offset from one another. The study focused on an area of the south wall at station 2+80 and the north wall at station 1+70 (drawings AX6DD365 and AX6DD367). As both excavation and mapping of stratigraphically lower units progressed, it became very evident that the irregularities of unit C were due to processes other than faulting. The continuity of the lower units in the areas of interest precluded the possibility of fault offset. A report prepared by Bechtel[98] concluded that the only plausible explanation for the observed irregularities was a combination of erosional and depositional processes.

Underlying the limestone of unit C is medium-gray, highly fossiliferous, sandy to silty marl, designated as unit D. This zone, averaging 8 ft in thickness, was continuous around the walls of the auxiliary building excavation. The lithology of unit D is very uniform, and its upper and lower contacts are quite sharp. An abundance of pelecypods retaining both valves characterizes this unit. Near the base, a number of very hard, lime-cemented pods and lenses are present at roughly equivalent elevations and have highly gradational contacts with the surrounding marl. These pods and lenses are believed to represent accumulations of calcium carbonate cement leached from the surrounding fossiliferous marl. They are collectively considered to be a subunit of D, designated D_1.

Unit E underlies D and is thin, relatively continuous, impure limestone. It is light gray, very well indurated, and fossiliferous. It averages 1 ft in thickness and varies in elevation from 121 ft in the northwest corner of the auxiliary building to 116 ft in the southeast corner. Locally, unit E is difficult to distinguish from D_1. This was seen in the north wall between stations 1+40 and 1+70 (drawing AX6DD365), where E is discontinuous and D_1 is represented by some fairly continuous lenses. In these cases, unit E is arbitrarily selected as the unit displaying the sharpest contacts with surrounding units and the one stratigraphically between the overlying unit D and underlying unit F. The similarity between portions of E and D_1 suggests that both may be cemented deposits resulting from leaching and redeposition of calcium carbonate from the overlying fossiliferous deposits. The relative continuity of E indicates a basic permeability change occurring at the horizon in the geologic past. This is a basis for differentiating the overlying unit D and underlying unit F.

Unit F, like D, is a fossiliferous marl which was seen to be continuous around the basement excavation walls. It is medium gray and sandy to silty; it varies in thickness from 1 to 4 ft. It is dense and well consolidated but poorly cemented and tends to dessicate upon exposure to the atmosphere. Unit F includes some cemented limey pods similar to D_1. These have gradational contacts with surrounding material and appear to be secondary in origin.

Unit G is light-to-dark gray laminated marl, which is present locally as lenses interfingering with units F and H. It was relatively continuous in the westerly portion of the south wall but pinches out at station 1+50. It reappeared between stations 1+85 and 2+25 (drawings AX6DD367 and AX6DD368) but then disappeared from the remainder of the south wall. It was present in portions of the west and north walls and was absent in the east wall. The unit is characterized by very fine, sinuous, and discontinuous sandy laminations; scattered shell fragments; and small, lenticular clay pods. It contains scattered carbonaceous lenses and is well consolidated.

Unit H underlies G and consists of massive gray marl which was continuous around the excavation. It is dense, well consolidated, and poorly cemented. Shell fragments are sparse in the upper part of the unit but become increasingly abundant toward the base. Unit H varies in thickness from 1 to 6 ft.

Unit I underlies H and is very similar to unit E. It is a thin, relatively continuous, light-gray, impure limestone which is generally less than 1 ft thick. It was continuous around the excavation walls, with the exception of the east wall between station 0+79 and the south end of the wall, where it was absent.

Unit J, the deepest marl unit exposed in the auxiliary building excavation, consists of medium gray, massive, fossiliferous marl similar to the stratigraphically higher units D and F. It was continuous around the excavation walls, with the exception of the east end of the excavation, where the upper contact of the unit dipped beneath the base of the excavation.

From the preceding descriptions, it is seen that the portion of the marl section exposed in the auxiliary building excavation represents cycles of fossil abundance and absence, interspersed with periods of formation of secondary limestone pods and lenses as a result of leaching of calcium carbonate from fossiliferous zones. Erosional and depositional processes have combined to create some of the interfingering of units as well as the irregularity of some of the limestone layers.

The upper contact of the Lisbon Formation was exposed around the perimeter of the power block excavation, because it exists at an elevation higher than the top of the more localized auxiliary building excavation. The top of the Lisbon Formation corresponds with the top of the Blue Bluff marl. This upper contact was examined in detail and surveyed. It varies in elevation from a high of 138.6 ft on the north side of the excavation to a low of 132.0 ft on the south side. The contact is erosional with very minor relief present. The uppermost few feet of the marl are locally weathered to a greenish color, and bioturbations (disturbance of the sediment due to the activity of organisms) were noted locally.

2.5.1.2.2.2.1.2 Barnwell Group. Late Eocene beds known as the Barnwell Group (paragraph 2.5.1.1.3.3.1.2) are present over much of the area within 5 miles of the site (drawing AX6DD345).

The formation is primarily comprised of tan, yellow, red, and white sands and clayey sands, although exposures of claystone, shelly limestone, and reef deposits are common. The Barnwell Group is comprised of four basic lithologic units, shown on drawing AX6DD352, which are listed below, from oldest to youngest:

- Utley Limestone.
- Twiggs Clay Member.
- Irwinton Sand Member.
- Tobacco Road Sand (Upper Sand).
- Upper Sand (Tobacco Road Sand).

The Barnwell Group rests unconformably upon an erosion surface at the top of the Lisbon Formation. The Barnwell Group is shown as a single lithologic unit on the 5-mile-radius geologic map (drawing AX6DD345).

The best exposures of the Barnwell Group within the mapping area include the power block excavation for the VEGP, a road cut just west of the intersection of River Road and Little Beaver Dam Creek, the intersection of River Road and Newberry Creek, and the intersection of the road just south of the railroad and Newberry Creek. Excellent exposures outside the mapping area which should be noted include a road cut just south of Brier Creek on Thomas Bridge Road and just south of McBean on State Highway 56. The Barnwell Formation was also encountered in numerous exploratory drill holes, which add to the data collected from surface exposure.

The oldest unit of the Barnwell Group is the Utley Limestone Member of the Clinchfield Formation. The Utley Limestone is a white to light-gray fossiliferous limestone, which has been referred to as the shell zone. The limestone was well exposed in the power block excavation for the VEGP and is locally exposed along the Georgia side of the Savannah River. This limestone layer, which is also thought to be of middle Eocene age, exhibits the effects of leaching. Surface topography, losses of drilling fluids during the exploratory drilling, and direct visual observation in the excavation and natural exposures all indicate the presence of solution cavities. The thickness of this unit varies from 0 to 100 ft.

Locally overlying the Utley Limestone of the Barnwell Group is the Late Eocene, Twiggs Clay Member. The Twiggs Clay was exposed only in the power block excavation where it was a medium-gray, moderately hard, sandy claystone. The upper 2 to 5 ft are weathered to greenish-gray, reflecting the unconformable relationship with the overlying sand units.

Unconformably overlying the Twiggs Clay is the Irwinton Sand Member. The Irwinton Sand is present through much of the mapped area. Although the Irwinton Sand was well exposed in the power block excavation, it apparently pinches out to the west.

The Irwinton Sand is typically represented by unconsolidated, tan, and white, medium-grained sand and clayey sand. The sands are typically massive, although some cross-bedding is present. Tan clay seams and clayey zones along with scattered shell fragments and carbonaceous zones are present. The upper few feet of this unit in the power block excavation are comprised of shell fragments in a matrix of clay with manganese straining, providing a relatively sharp contact with the overlying sands.

Overlying the Irwinton Sand is the Tobacco Road Sand unit of the Barnwell Group. Tobacco Road Sand is typically red, although yellow, brown, tan, and mottled units are present. The sand is typically medium grained and locally cross-bedded. This sand unit is present throughout much of the mapping area and was particularly well exposed in the power block excavation and near the intersection of River Road and Little Beaver Dam Creek.

The upper portion of the Tobacco Road Sand locally contains lenses of limestone or relic features of limestone which have been leached. This limestone is well exposed near the intersection of Brier Creek and Thomas Bridge Road and near the intersection of the railroad and Newberry Creek. It should be noted that sands of the upper Barnwell Group are affected by surface weathering, forming mottled clayey sands and in many road cuts.

2.5.1.2.2.2.1.2.1 Barnwell Group in the Power Block Excavation. All of the sediments that were exposed in the sidewalls of the power block excavation are of Eocene age. Above the Blue Bluff marl of the Lisbon Formation, the exposures were comprised entirely of sedimentary beds of the Barnwell Group and include the units referred to in paragraph 2.5.1.2.2.2.1.1. The stratigraphic column shown on drawing AX6DD352 gives a summary of the lithologic characteristics of these sediments, as encountered in the power block excavation. Drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363, detailed geologic maps of the geological investigation of the excavation, provides comprehensive lithologic descriptions of the Barnwell Group sediments found in the excavation.

Although examined and described in detail, the deposits between the top of the Blue Bluff marl and approximately el 170 ft could not be mapped in detail. Consequently, the geologic map of the power block excavation (drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363) shows only the detailed lithology of the Tobacco Road Sand and the upper portion of the Irwinton Sand. This was due to extensive slumping of the slopes when excavation and dewatering were suspended during the period between September 1974 and June 1976. Extensive regrading obscured the contacts between units in this zone. Several portions of the slopes were covered with riprap in order to control seepage and improve stability, further obscuring contacts. Since seepage from the slopes was creating local stability problems, it was decided not to excavate back into the slopes to expose contacts and risk large slope stability problems. Detailed mapping of the units above and below this zone demonstrated the continuity of the strata and the absence of faulting.

The lowermost exposed unit within the Barnwell Group in the power block excavation is the Utley Limestone. The lower part of the limestone is grayish yellow, well indurated, and fossiliferous, grading locally into coquina. It was continuous around the power block excavation and varies in thickness from 0.5 to 3 ft. The upper part of the limestone is white to light gray and varies from 0 to 12 ft in thickness, present only in the north and northwest portions of the power block excavations. Although well indurated, this thicker limestone has been subjected to extensive leaching, producing a honeycomb network of cavities. Some individual cavities had mean diameters of several feet before being removed by excavation or filled in place. The location and extent of the cavities exposed in the slopes are shown on drawing AX6DD363. The filling of cavities in the limestone intersected by the excavation slopes is described in paragraph 2.5.4.5. Within the cavities, the limestone typically displayed a weathered and soft zone immediately adjacent to the cavity walls, which graded within a few inches to hard, unweathered limestone. Locally, extensive leaching of the limestone had left a residue of silt and clay impurities forming a soft mottled blackish material. Included in the Utley Limestone is a highly fossiliferous clay deposit which varies in color from tan to dark gray. The difference in colors appears to be due primarily to weathering effects. Prior to its removal, this clay was present mainly in the northwest portion of the power block excavation. It contains abundant specimens of the oyster Crassostrea gigantissima, a key Eocene near-shore pelecypod. Lesser quantities of other pelecypods, gastropods, arthropod arts, and shark teeth have been identified in this clay.[78]

Unconformably overlying the Utley Limestone is the Twiggs Clay. This consists primarily of medium-gray, moderately indurated, laminated sandy claystone, which is quite similar to the underlying Blue Bluff marl of the Lisbon Formation. The Twiggs Clay was exposed only in the southeast portion of the power block excavation and varies in thickness from 0 to 13 ft. The upper 2 to 5 ft are weathered to a distinctive greenish-yellow color. The Twiggs Clay has alternating thin and thick beds (from less than 1 in. to greater than 1 ft), with gradational contacts between beds. No joints, fractures, or discontinuities were observed in the clay.

The Irwinton Sand of the Barnwell Group unconformably overlies the Twiggs Clay in the southeast portion of the power block and overlies the Utley Limestone elsewhere. The Irwinton Sand consists of an approximately 50-ft-thick vertical sequence of sands, clays, and reef deposits. At the base of the sequence is a massive, white, quartz-rich sand deposit. The presence of fossil shrimp burrows identifies this as an intertidal deposit. The upper surface of the sand is highly irregular, with reef-type accumulations of Crassostrea gigantissima present on the highs. These shell accumulations are well cemented and highly calcareous. This sand is fine to medium grained and very well sorted, and it exhibits extensive cross-bedding. It is extremely friable and tends to rapidly slump and ravel, assuming its angle of repose soon after excavation.

Above the white sand and reef deposits is a sequence of tan sand and clay. The sand is generally fine to medium and moderately sorted, and it contains thin seams of tan clay having high plasticity. Two continuous marker horizons are present within this sequence. The first, a zone of manganese-straining and shell debris, occurs generally between el 170 and 180 ft and was somewhat higher than this on the west side of the excavation. This zone, called the shell hash horizon, varies in thickness from less than 1 in. to almost 6 ft and could be traced continuously around the excavation slopes (drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363). A second shell hash horizon is locally present beneath the first one but is discontinuous. The second marker horizon is a zone of abundant tan clay seams, which varies from approximately 1 to almost 6 ft in thickness, and was found between el 180 and 200 ft. This clay zone marks the top of the Irwinton sand.

Both of the marker horizons undulate along the strike, with flexures in the bedding reflecting underlying reef highs as well as lows due to collapse of cavities in the stratigraphically lower Utley Limestone. These flexures are discussed further in paragraph 2.5.1.2.3.4.

Above the Irwinton Sand is the Tobacco Road Sand of the Barnwell Group. This sand extended up to the top of the excavation slopes and consisted of a thick (up to 40 ft) zone of predominately red sand with zones of lavender, purple, mustard yellow, and orange sand. The color changes are due to weathering effects and are not related to structure or lithology. The sand consists of fine- to medium-quartz grains which are moderately to well sorted and angular to subrounded. Colors are imparted by clay coatings on the individual grains. Differential weathering has produced mottled zones of bright colors which form an alligator-skin effect near the top of the unit. The sand is dense, well consolidated, and completely uncemented.

At the top of the excavation slopes, recent deposits of buff-colored, alluvial and windblown sand were present locally. These deposits form a thin veneer of fine- to medium-grained, angular to subangular, well-sorted quartz sand which is highly gradational with the underlying sand of the Barnwell Group.

2.5.1.2.2.2.2 Miocene Series. The Hawthorne Formation of Miocene age caps the ridge and hills above el 200 ft around the site area and lies unconformably upon the eroded surface of the upper sand member of the Eocene Barnwell Group. The Hawthorne Formation is typically red to brown mottled sandy clay and clayey sand. Lateral facies changes, however, result in significant lithologic variations including massive and cross-bedded lavender, purple, red, and brown medium- to coarse-grained sand. Channel deposits and localized lithologic changes are well illustrated in the railroad cut just east of Daniel Grove Baptist Church.

The contact between the Hawthorne Formation and the Barnwell Group is difficult to distinguish in the field. In gross terms, all lithologies present in the upper Barnwell Group occur in the Hawthorne Formation. The only unique distinguishing property of the Hawthorne Formation is the presence of siliceous gravel near the base of the formation. This gravel is present in road cuts near the intersection of Brier Creek and Thomas Bridge Road in the road cut across from DeLaigle Trailer Park, just west of the VEGP site entrance. In areas where the gravel is not present, it is difficult to differentiate the two formations.

The clastic dikes associated with the Hawthorne Formation are discussed in paragraph 2.5.1.1.3. No exposures of these clastic dikes have been found in the vicinity of the site.

2.5.1.2.2.3 Quaternary Deposits. The Quaternary sediments in the site area consist of sands, gravels, silts, and clays of Holocene and Pleistocene ages. The Quaternary is largely represented by flood plain deposits in the Savannah River Valley and alluvial trains along the courses of larger streams tributary to the Savannah River. At the plant site elevation, the Quaternary is principally represented by erosion and weathering rather than depositional processes, although deposits of buff-colored, windblown sand are seen on higher ground.

2.5.1.2.3 Site Structural Geology

The formations underlying the site area are essentially flat lying or gently dipping to the southeast, reflecting the regional dip. The site area structure is illustrated on drawings AX6DD377 and AX6DD378, which show subsurface contours on the top and bottom of the marl. The dip in the plant site area is about 30 ft/mi in a southeasterly direction. This gentle homoclinal structure is unbroken in the area except for a gentle dip reversal which is of depositional and differential compaction origin. This feature is discussed in paragraph 2.5.1.2.3.2.

Solution depressions are apparent on the geologic map of the area (drawing AX6DD351). These features, which have been investigated and found to be confined and related to lithologic units stratigraphically above the Blue Bluff marl, are discussed in paragraphs 2.5.1.2.3.3 and 2.5.1.2.2.

2.5.1.2.3.1 Faults and Lineaments. No faults or lineaments have been found within 5 miles of the site, other than those associated with the Triassic Basin, discussed in paragraph 2.5.1.2.3.2. These structures do not extend into the overlying Tertiary deposits. Examination of sediments exposed in the walls of the power block excavation has shown no evidence of faulting (paragraph 2.5.1.2.3.4).

In early 1982, U.S. Geological Survey Open-File Report 82-156 postulated the presence of two faults in the vicinity of VEGP.[100] The Millett fault was described as a northeast trending fault extending from northern Jenkins County, Georgia, to central Barnwell County, South Carolina. Its postulated trace passed close to the small community of Girard, Georgia, about 7 miles south of VEGP. The Statesboro fault was described as a northeast trending fault extending from central Bulloch County, Georgia, to eastern Allendale County, South Carolina. Its postulated trace was about 32 miles south of VEGP.

In response to the open-file report, a detailed and comprehensive investigation was conducted to evaluate whether or not the postulated faults were capable, i.e., had moved once in the past 35,000 years or more than once in the past 500,000 years. Based on information in the open-file report, the postulated Millett fault was of primary interest and the postulated Statesboro fault was of secondary interest to the study. A review of the data and evaluations used in the open-file report indicated that additional data and evaluations would be needed to adequately determine the capability of the postulated faults. If a fault or faults were found to be present, the capability of such faults was to be determined.

The investigations encompassed several scientific fields which address the question of faulting. These include surface geology, subsurface geologic and geophysical characteristics, ground water aquifer characteristics, surface water hydrology, and the nature and distribution of historic seismicity in the area.

To provide guidance and review of the studies, a number of eminent consultants were retained. These were chosen because their fields of expertise were related to the planned studies: Dr. Bruce Bolt, director of the seismographic station at the University of California, Berkeley; Dr. R. D. Hatcher of the University of South Carolina; Dr. V. J. Henry of the University of Georgia; Dr. P. E. LaMoreaux, president of P. E. LaMoreaux and Associates; Mr. H. LeGrand, and independent consultant in geohydrology; Dr. R. Lyon of Stanford University; Dr. S. Papadopulos, president of S. Papadopulos and Associates; Mr. Carl Savit, senior vice president of Western Geophysical; Dr. Carl Stepp, affiliated with Woodward-Clyde Consultants, and Mr. L. Wood, ground water geology specialist with S. Papadopulos and Associates.

The results of the studies conclusively demonstrate the absence of a capable fault in the vicinity of the postulated Millet fault and strongly suggest that no capable fault exists near the location of the postulated Statesboro fault.

These conclusions are based on the following:

A. Core drilling and geophysical logging clearly demonstrate subsurface continuity of beds 40 to 80 million years before present across the trace of the postulated Millett fault.

B. Acoustic reflection surveys performed in the Savannah River demonstrate continuity of subsurface strata deposited across the strike of both the postulated Millett and Statesboro faults during the last 80 million years.

C. Geologic mapping and remote sensing studies reveal no surface expression of faulting.

D. Examination of recorded and reported seismic events indicate that there is no historic seismicity which can be associated with either of the postulated faults.

E. Surface and ground water hydrology studies do not support the presence of faults.

These studies are described in detail in Bechtel Power Corporation (1982).[86]

It is concluded that no capable faults exist in the vicinity of the postulated Millett and Statesboro faults; therefore, they can have no impact on the existing accepted seismic design bases of the VEGP.

2.5.1.2.3.2 Flexures and Folds. The foundation materials for the plant structures are comprised of essentially flat-lying Tertiary sediments, which include the Lisbon Formation of Eocene age. The bearing unit for the major plant structures is the Blue Bluff marl, the upper member of the Lisbon Formation. Contours of the upper and lower surfaces as well as an isopach map of the marl in the vicinity of the plant are shown on drawings AX6DD352, AX6DD371, and AX6DD372. The contours on these map figures were derived from outcrop and drill hole information and reflect the southeasterly regional dip of approximately 30 ft/mi throughout the plant siting area. This regional dip is interrupted approximately 1000 ft northwest of the plant siting area by a gentle dip reversal of a maximum of 3° (5%) northwesterly, along a northeast-southwest trend. The apparent dip reversal is shown with two-to-one, vertical-to-horizontal exaggeration on geologic section B-B' (drawing AX6DD352), where it resembles a monoclinal flexure.

This local anomaly in the regional dip has been the subject of detailed investigation and discussion. The hypotheses which have been considered concerning the origin of the anomaly include:

- Structural monocline.
- Solution collapse.
- Erosional-depositional feature.
- Stratigraphic facies change.

Numerous holes were drilled to determine the characteristics of the dip reversal, and water pressure tests were made to determine whether it affects the watertightness of the marl. No indication that it is a fault-controlled feature was found during the extensive investigations. It does not appear to be simply an erosional feature on the top of the unit, as it is reflected in both the top and bottom of the marl to an approximately equal extent. It dips in the wrong direction to reflect possible near-surface expression of the underlying Triassic Basin boundaries. No relationship to the assumed boundary fault contact at the northern edge of the Triassic basin could be found other than coincidence of location. As the assumed northern Triassic basin boundary fault would have to be down thrown towards the sea, the fact that the flexure in the bearing horizon slopes in the opposite direction (i.e., to the northwest) seems to negate any structural relationship.

The dip reversal seems to have been formed prior to and in part during the deposition of the Lisbon Formation. A local, well-developed, striated bedding plane was found in one hole (No. 246) at the base of the Blue Bluff marl. A few short, discontinuous fractures with slick surfaces have been noted. These appear to be similar to fractures found in clay during some compaction processes. These occur near the southwestern or lower side of the anomaly and well away from the plant area. Water losses related to jointing in the upper 15 ft of the marl were noted during the exploratory drilling investigation of this feature. These phenomena were not observed elsewhere throughout the plant site investigation. It is believed that the reversal represents deposition on an erosional irregularity on the underlying sands, with the possibility of some local differential compaction during or shortly after deposition of the overlying stratum.

A special study was made of the available information on the Triassic basin because of the proximity of the dip reversal in the bearing horizon to the best approximation of the basin's northern border. No evidence has been found that suggests a reflection of the subsurface basin boundaries in the overlying pre-Cretaceous and Cenozoic strata. The most recent and comprehensive subsurface investigations of this basin have been undertaken in the Savannah

River plant, some 5 to 11 miles east-northeast of the site. They have been reported by Marine;[84] some of the more pertinent portions are quoted below:

> Well P5R, drilled in 1962, was the discovery well for the buried Triassic sedimentary basin; it penetrated only about 95 feet of these sedimentary beds, consisting of maroon claystone and fine-grained sandstone. Well DRB 9 was drilled in 1969 at a location selected, on the basis of the short reflection seismic survey, to penetrate the edge of the Triassic basin and then pass into the crystalline metamorphic basement. At this location, the Triassic sedimentary rocks are 1593 feet thick and consist of maroon fanglomerate made up of clasts of pink weathered gneiss set in a maroon siltstone matrix. The Triassic is underlain by augen gneiss, with pink feldspar augen about 0.5 inches in diameter set in a matrix of fine-grained green hornblende. The well is about 0.4 mile from the edge of the basin; the contact of the Triassic with the augen gneiss dips about 35 degrees to the southeast, as do a few faint bedding planes in the fanglomerate. Well DRB 10 was drilled to a depth of 4206 feet and penetrated about 3035 feet of red siltstone and sandstone. Crystalline metamorphic rock was not expected in this well, and none was encountered. No well in the basin has penetrated any igneous rock or coal, even though they occur in other East Coast Triassic basins.
>
> Geophysical work, consisting of reflection seismic and gravity-magnetic surveys, was done or analyzed after the information from these three wells was available. The reflection seismic surveys showed a sharp northwest boundary but did not indicate termination of the Triassic rocks where the southeast border was inferred from the aeromagnetic map. The contact of the coastal-plain beds and the Triassic rocks could be followed as a reflection but with scattered apparent discontinuities (labeled inferred faults in figure 4). The contact of the basement with the Triassic rocks could not be detected at all.
>
> Gravity and magnetic surveys were made on the ground along many of the same traverse lines as the seismic survey in an effort to estimate the thickness of the Triassic rocks and to develop more information on the possible faults indicated by the reflection seismic surveys. One of the faults interpreted from the seismic data was selected for more intensive investigation by drilling. Two wells (DRB 11 and P12R) were drilled on either side of the fault. The elevation of the top of the Triassic was the same in both wells, showing that the last movement on the fault, if it exists, was before the development of the erosional surface. The age of the erosional surface is pre-Late Cretaceous or at least 100 million years. Thus, there has been no movement on the fault in the last 100 million years. This conclusion is substantiated by correlation of distinctive peaks on the gamma ray and electrical logs of the coastal-plain beds.
>
> Well DRB 11 was designed to deviate 15 degrees from vertical in order to intercept the fault that had been indicated by the seismic and gravity magnetic surveys. The well was drilled to a total depth of 3320 feet, which represented a true vertical depth of 3278 feet and a horizontal migration of 387 feet to the northwest. The well was cored completely, but the fault indicated by the geophysical surveys was not penetrated.

Wendell Marine noted in personal communication that, based on the investigations on the Savannah River plant site, he would conclude that all known faulting antedates the erosion surface between the coastal plain sediments and the Triassic sediments in the basin.

Early exploration drilling and water-pressure testing in the Blue Bluff marl resulted in water-loss data which seemed to show the marl to be permeable, possibly due to fracturing related to folding or faulting. Later, holes drilled within 15 feet of the early holes to further investigate the apparent water losses proved the marl to be quite tight. The previously indicated losses within the bearing stratum seem to have been caused by mechanical problems. The overlying shell zone (basal limestone) varies from moderately to extremely pervious. If the drill casing was not tightly seated to form a seal where it penetrated the marl, the vibrations caused by drilling may have loosened it, allowing the drilling fluid to escape beneath the base of the casing and into the pervious shell zone. This was reported by the driller as a water loss in the marl.

A similar situation exists with the pressure tests. The Blue Bluff marl, being nodular, causes drill hole diameter fluctuations, locally quite irregular (see caliper logs, drawings AX6DD379, AX6DD380, AX6DD381, AX6DD382, AX6DD383, and AX6DD384). This condition made difficult the proper leakage-free placement of packers during the pressure tests. Consequently, an apparent water loss into the marl would be indicated in the earlier pressure tests if either packer leakage occurred or its casing was not tightly seated. The holes showing possible water loss in the marl were checked out with the 500-series holes in which the casing was grouted to the surface of the marl before coring of the marl commenced. Packer settings were adjusted until a good seal was obtained before the interval was tested. The test results of the 500-series holes show that the marl is tight, and therefore the hypothesis of fold- and fault-related fracturing is unsupported.

Solution collapse, or apparent folding caused by subsidence of the marl into an underlying zone of solution, and removal of carbonates is a reasonable hypothesis. Slump and foldlike features have been ascribed to the leaching process in the Lisbon Formation in South Carolina. However, investigative drilling has found none of the associated features, which include tension fracturing, clastic dikes, and low-density, carbonate-impoverished materials or cavities within or beneath the marl.

Erosional and depositional processes, together with differential intraformational compaction of units below or within the marl, could produce apparent warping or folding in cohesive sediments. Such features have been described in the power block excavation. However, the magnitude of differential settlement, a net elevation change of 50 ft in 1000 ft of horizontal distance, seems to be large in consideration of the sandy nature and thickness of underlying strata.

Detailed mapping of the Lisbon Formation in excavations at the plant site has demonstrated rapid lithologic changes laterally and rapid changes in the thickness of mappable units within the formation. The upper boundary of the bearing stratum, the Blue Bluff marl, is basically established by the contact between it and an overlying shell bed, the Utley Limestone. The base is generally established by the presence of an underlying sand bed. In the excavation and in investigative borings, similar lithologic sequences are repeated vertically. The principal difference is one of scale. It can be shown within the excavation that thickness changes of 12 to 15 ft in a horizontal distance of less than 200 ft can occur in the Utley Limestone and in the Twiggs Clay, which is practically indistinguishable from the Blue Bluff marl. It seems that over a distance of 1000 ft, the magnitude of change of thickness, the pinching out of key units, and the appearance of similar key units at lower elevations could create the appearance of a flexure.

In conclusion, the origin of the dip reversal, or flexure, is probably due to erosional-depositional features possibly exaggerated by stratigraphic facies changes. There is no evidence to support either a structural monocline or solution collapse hypothesis.

2.5.1.2.3.3 Solution Depressions. Solution depressions are readily apparent on the geologic map (drawing AX6DD351) of the site and were the subject of considerable investigation using surface and down-hole geophysics, coring, and pressure-testing techniques.

The depressions at the site area originate from the solution and removal of calcareous shell material present in the sand above the marl horizon. Calcareous shell material is subjected to leaching by percolating meteoric water on its migration downward from the porous sandy soil to the water table, the base of which is formed by the top of the Blue Bluff marl. Subsequent settling of the overlying material results in the formation of the shallow depressions. To indicate the relationship of the surface depressions with the Utley Limestone and bearing horizon, the depressions have been accentuated by hachuring on drawings AX6DD371, AX6DD376, AX6DD377, and AX6DD378.

The calcareous shell material of the Utley Limestone above the Blue Bluff marl is of limited extent and varies considerably in thickness, as may be seen on drawing AX6DD376. The surface depressions are restricted to areas where the shell material is, or clearly has been, present. Where thick shell accumulations are present, solution features are common, including the depressions, cirque-like ravine heads, and, locally, a stream flowing from a small solution cave. The numerous drill holes which extend through the marl have intercepted no solution-related features in the marl or beneath it. The solution features are confined to the Utley Limestone zone above the marl.

The following data are submitted in support of the shallow origin of these depressions:

A. The depressions are broad and shallow rather than deep and steep sided, as would be the case in a near-surface cavern collapse. Solution and removal of calcium carbonate from a sand-shell mixture, however, produce the type of depression found here. That the shell materials above the marl are being dissolved is indicated by the nature of the core recovered and the voids and fluid losses commonly experienced while drilling the shell zone. Conversely, the impermeable nature of the marl determined from core and pressure tests data indicates that it acts as an aquiclude, essentially preventing the further vertical migration of the surface waters.

B. In the plant vicinity the depressions are confined to areas underlain by the Utley Limestone zone. (See drawing AX6DD376.)

C. There are no disappearing streams in which the flow reappears within or below the marl of the bearing horizon, as would be expected were the depressions caused by solution and removal of calcium carbonate from beneath the Utley Limestone. On the contrary, it appears that the marl forms the base of the streams over much of their courses until they start cutting down to the Savannah River base level. To the northwest, the stream drainage into Mathes Pond is via a small solution cave in the Utley Limestone in which the Blue Bluff marl forms the cave bottom.

D. Drawings AX6DD377, AX6DD378, and AX6DD371 present contour elevations on the top of the marl, the bottom of the marl, and the thickness of the marl, respectively. If the origin of the surface depressions were related to conditions beneath the shell zone, it would be reflected in these contours. A number of the drill logs in which the datum points for these contours were obtained were from holes in or adjacent to these surface depressions. This provides good contour control in these areas for these maps. Their data clearly show no correlation between the surface depressions and contours of the isopach maps of the top of the bearing horizon.

Thus, the depressions are caused by the solution and removal of calcareous material above the Blue Bluff marl. Since the foundation excavation for the plant has removed all materials above the marl (drawings AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, and AX6DD370) from the overlying Utley Limestone, solution and removal of carbonate do not pose a geologic hazard for the plant.

2.5.1.2.3.4 Power Block Excavation. Two separate marker horizons in the Irwinton Sand have been mapped around the side slopes of the power block (drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363). Both horizons are continuous and unbroken, demonstrating the absence of faulting in these materials. In addition, the upper contact of the stratigraphically lower Lisbon Formation marl has been mapped with survey accuracy and has also been found to be uninterrupted by offsets (drawings AX6DD372 and AX6DD373). Subunits within the marl have been mapped around the walls of the auxiliary building basement excavation. These zones were likewise found to be undisturbed by faulting. Minor stratigraphic irregularities noted were shown to be related to erosional and depositional processes in the report of marl investigation.[97]

Only one joint was recognized in the entire power block excavation, and this was of limited extent. The joint was exposed in the southeast corner of the power block excavation, where it extended from the upper surface of the marl down to el 127 ft where it terminated at a depth of about 6 ft. The joint trended N81°E; it was approximately vertical and was tightly closed. Some secondary dark-green mineralization was noted as a fine coating on the joint faces. No other joints or fractures were identified in any other lithologic units.

The strata of the Irwinton Sand exhibited flexures in the power block excavation which are related to the following phenomena:

- Differential compaction of underlying sediments.
- Subsidence due to leaching of underlying calcareous materials and collapse of solution cavities.
- Deposition on uneven surfaces.

These flexures are particularly evident in the two marker horizons of the Irwinton sand.

Evidence for differential compaction includes the typical association of highs in the marker horizons with the occurrence of underlying reef deposits of shells. These cemented shell deposits form hard spots in comparison with the relatively more compressible sediments between them. As the load of the overburden increased during deposition, the sediments between the reefs were compressed relatively more than those immediately above them. This created downward flexures in the overlying marker horizons.

Evidence for subsidence due to leaching and collapse of cavities includes the presence of solution cavities in the limestone underlying zones of downwarping of the relatively incompetent sands and clays. Recent surface depressions in the area give added evidence for subsidence. Solution activity and cavity formation are confined to the limestone above the Blue Bluff marl. No cavities have been found in the marl, and the upper surface of the marl exposed in the power block excavation contains no depressions.

2.5.1.2.4 Site Geologic History

The geologic history of the site area reflects the geologic history of the region (paragraph 2.5.1.1.2), which was an active part of the ancestral Appalachian Mountain system. The complex geologic history of the Appalachian Mountain system is reflected by the igneous and metamorphic rocks of Precambrian through Paleozoic age and Triassic sediments underlying the site, which has been essentially quiescent since the deposition of the Cretaceous sediments in the coastal plains. Although rocks comprising the basement complex were not encountered in drill holes at the site, seismic refraction surveys at the site indicate that the basement complex Cretaceous contact occurs at a depth of over 950 ft.

Due to regional elevation fluctuations following deposition of the Cretaceous Tuscaloosa Formation and the overlying Paleocene Huber and Ellenton Formations, the time interval from Eocene through Miocene was one of marine transgressions and regressions. For example, the Lisbon Formation of Eocene age, overlying the Huber/Ellenton Formations in the site area, is representative of near-shore or tidal deposits. The presence of alternating, typically shallow water sediments, along with variations in fossil abundances in the Lisbon Formation and the Barnwell Group, suggests changing near-shore environments.

Variegated clays and sands, lithologically similar to the Miocene Hawthorne Formation, were encountered in the upper portion of some of the holes drilled at the site and are also the youngest Tertiary sediments underlying the soil horizon of the higher elevations in the site area.

2.5.1.2.5 Site Geologic Maps

The site geologic map is shown on drawing AX6DD361. The 5-mile-radius geologic map of the local area is shown on drawing AX6DD345. The 5-mile-radius map shows the location of the known and inferred contacts between surface materials at the site. A detailed geologic map of the power block area, drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363, was prepared from data obtained during the geologic mapping program conducted concurrent with excavation and grading. The geologic map of the power block area and geologic data concerning the area are discussed in paragraphs 2.5.1.2.2 and 2.5.1.2.3. Geologic sections through the power block excavations are shown on drawings AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, AX6DD370, AX6DD372, and AX6DD373. The regional geologic map is shown on drawings AX6DD341 and AX6DD342.

2.5.1.2.6 Plot Plan

Information concerning the locations of major structures of the plant, including all Seismic Category 1 structures, and exploration and test borings made at the site area is presented on drawings AX6DD343, AX6DD344, and AX6DD351. These figures are discussed in subsection 2.5.4. The geologic logs of the borings are discussed in appendix 2B.

2.5.1.2.7 Subsurface Profiles and Plant Foundations

Subsurface profiles showing lithologic correlations from drill hole data are given on drawing AX6DD352. Geologic profiles and geologic sections prepared from data obtained during the geologic mapping of the foundation excavations are presented on drawings AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, AX6DD372, and AX6DD373. They are discussed in paragraphs 2.5.1.2, 2.5.1.3, and 2.5.4.5. The site ground water conditions are discussed in detail in subsection 2.4.12 and summarized in paragraph 2.5.1.2.8.7. The significant engineering characteristics of the subsurface materials are discussed in subsection 2.5.4. All Seismic Category 1 structures are founded on the Blue Bluff marl or upon compacted structural backfill placed upon the marl. The Blue Bluff marl is a gray-green, hard, sandy to silty, calcareous clay marl with thin limestone lenses, small calcareous nodules, and occasional macrofossils.

2.5.1.2.8 Engineering Geology Evaluation

2.5.1.2.8.1 Engineering Properties of Foundation Materials. The strength of foundation materials, static and dynamic properties, bearing capacities and settlement, rebound and heave, and foundation design criteria are discussed in subsection 2.5.4.

2.5.1.2.8.2 Prior Earthquake Effects. There is no evidence to suggest that surficial or subsurface materials have been affected by prior earthquake activity. No evidence of texture faults were found from any of the site exploration borings or in the power block excavations.

2.5.1.2.8.3 Deformational Zones. Examination of outcrops, excavation exposures, and subsurface samples have revealed that there are no deformational zones within the Blue Bluff marl, which is the foundation material for the major plant structures. Approximately 1000 ft northwest of the major structures, there is, however, a dip reversal of about 3° to the northwest. This gentle dip reversal in the otherwise very gently southeasterly dipping (approximately 30 ft/mi southeasterly) homocline of Tertiary sediments is of depositional origin and does not represent a structural (tectonic) deformation. Paragraph 2.5.1.2.3 contains a discussion of this anomaly.

During the construction phase at VEGP, a comprehensive inspection program was carried out to continuously monitor and assess the condition and character of all excavated marl throughout the power block area. A total of four joints was found in the uppermost strata of the marl. Two were found during routine inspection of the exposed marl surface prior to backfilling, and two were found during inspection of the radwaste solidification building caisson foundation. Each joint was independently investigated and found to be of limited depth and aerial extent and of nontectonic origin. Evidence produced by the investigations suggests that the joints were formed either during or immediately following late-stage diagenesis of the marl. Depositional loading from overlying sediments may have been a contributing factor. There is no evidence to suggest that these features are related to any processes that have occurred within recent geologic time.

With the exception of the joints described above, no other fractures, partings, or anomalous features were found in the marl.

2.5.1.2.8.4 Zones of Alternation or Weakness. The Blue Bluff marl is basically unweathered and unaltered, except for the uppermost section, which is up to 5 ft thick and slightly discolored by weathering from gray-green to green. This weathered zone was completely removed in foundation preparation.

2.5.1.2.8.5 Bedrock Stress. Over 1000 ft of unlithified to poorly lithified sediments overlie the pre-Cretaceous basement rock at the site. Rebound of the Blue Bluff marl, the bearing stratum, has been monitored by in situ instruments in the power block excavation. A total of nine heave points were installed between el 104 and 126 ft at the locations shown in figure 2.5.4-2. From 1974 to 1977, rebound ranged from less than 1 to 1.6 in., substantially less than predicted. A discussion of heave is contained in paragraph 2.5.4.10.

2.5.1.2.8.6 Effects of Man's Activity. There are no mining or underground mineral extraction activities occurring on or near the site. Ground water extraction is nominal in this area of low population. Therefore, there are no human activities which will affect site geologic conditions.

2.5.1.2.8.7 Site Ground Water. There are two confined aquifers beneath the site. The Cretaceous aquifer is the lowermost, and it consists primarily of the sands and gravels of the Tuscaloosa formation. It is often referred to as the Tuscaloosa aquifer. The overlying Tertiary aquifer is represented beneath the site by the "unnamed sands" member of the Lisbon Formation. These Tertiary sands are the local, minor equivalent of the regional principal artesian aquifer which consists primarily of permeable sands and limestones of several Tertiary formations extending throughout the Atlantic Coastal Plain. The Cretaceous aquifer and lesser Tertiary aquifer are believed to be hydraulically connected beneath the plant site. The beds that normally separate the Tertiary aquifer from the underlying Cretaceous aquifer are somewhat more permeable than they are elsewhere.

Overlying these aquifers is the Blue Bluff marl, the upper member of the Lisbon Formation. The marl layer, approximately 70 ft thick, is a near-impermeable layer that effectively confines the Tertiary and Cretaceous aquifers.

Ground water also exists in an unconfined water table aquifer in the Barnwell sands and limestone which overlie the marl. The water table aquifer a the site is on an interfluvial ridge, or a topographically high area in which the ground water in the water-table discharges along streams which nearly surround the high. The streams discharge to the Savannah River. A detailed discussion of site ground water conditions is contained in subsection 2.4.12.

2.5.1.3 References

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2.5.2 VIBRATORY GROUND MOTION

2.5.2.1 Seismicity

All significant historically reported earthquakes, that is, all earthquakes that could have reasonably affected the site region, are considered in this section. The site region as used here is the area within approximately 200 miles of the site. Within this site region all earthquakes of Modified Mercalli intensity IV or greater or magnitude 3.0 or greater are considered. Earthquakes that occurred outside the region but that were probably felt at the site are also treated.

All significant site region earthquakes are listed in table 2.5.2-1. To avoid crowding, only those Charleston Summerville area events of magnitude greater than or equal to magnitude 4.75 or intensity VI are shown on drawing AX6DD385. Elsewhere, all events in table 2.5.2-1 are shown on drawing AX6DD385. Several sources were used to compile the data given in table 2.5.2-1:

A. The Earthquake Data File of the National Geophysical and Solar-Terrestrial Data Center[1][2] is a collection of many published individual sources, including Earthquake History of the United States[3] (data source EQH), U.S. Earthquakes[4] (data source USE), and the U.S. Geological Survey (data source GS). Citations for these sources can be found in the Earthquake Data File records documentation.[1]

B. Data source CSC is from the earthquake catalog for the South Carolina seismic network of the University of South Carolina and is included in the Earthquake Data File.

C. Data source STR is a compilation of state earthquake catalogs by Stover and others[5][6] and Reagor and others.[7][8]

D. The Bollinger study[9] (data source BOL) is a catalog of southeastern United States from 1754 through 1974.

E. The Nuttli catalog (data source NUT) is an update of the Nuttli catalog of reference 10 and covers the central United States from 1811 through 1979.[11]

F. Data source EUS is attributed to Tarr of the U.S. Geological Survey. This catalog is a composite of many of the above catalogs, covering east and central United States. A number of events, usually of low intensity, listed in the EUS catalog are unique to that catalog and their existence could not be confirmed.[11] These events are generally small.

G. The Dewey and Gordon catalog[12] (data source D&G) covers the eastern continental United States and adjacent Canada, an area ranging between 52° to 104°W, 25° to 49°N. This is a compilation of previously cataloged events from 1925 through 1976, relocated with the benefit of newly developed location methods and improved regional travel-time curves, developed from data obtained from several dense seismic networks installed in the 1970s.

H. The Virginia Polytechnic Institute catalog (data source VPI) is from the Bulletin of the Southeastern United States Seismic Network, compiled at the Virginia Polytechnic Institute and State University (various publication dates since 1978). This catalog covers events primarily in the states of Alabama, Georgia, South Carolina, North Carolina, Tennessee, Kentucky, Virginia, and West Virginia, between July 1977 and June 1983. A recent effort by the author to remove blast- and reservoir-induced events from this catalog has been incorporated into this study's data set.

Before the early 1970s, most of what is known about site region seismicity was based on intensity data. These data are not derived from measurements made by instruments but are only chronicles of the sensible effects of earthquakes on people, structures, and landforms. The Modified Mercalli Scale of 1931[13] (an abridged version of this scale appears in table 2.5.2-2) is used throughout this subsection for the descriptive ranking of earthquake effects. The epicenters of earlier earthquakes were set at or near the center of the maximum intensity of shaking effects (I_o), which at times, especially in less populated areas, were sparsely reported. In the later years of this period, instrumental recordings from a few regional seismographic stations brought improvements to epicentral locations. The locations of site region earthquakes are given to an accuracy of 1/10° in table 2.5.2-1. This choice implies an uncertainty in epicentral position on the order of 5 to 10 miles or so. This is estimated to be a fair representation of the uncertainty in earthquake locations for site region events averaged over the chronological range of their occurrence. Roundoff at the 1/10° level is used in several of the sources,[3][9] from which the information in table 2.5.2-1 and drawing AX6DD385 is compiled, so that it is convenient and logical to follow this convention. However, it must be remembered that the 5- to 10-mile uncertainty is only an average estimate. For some of these events the actual uncertainties in location are realistically measured in several times this average.

Earthquake detection and location greatly improved in the 1970s with the installation of dense seismic networks in various areas in the eastern United States. A permanent seismic network was installed in 1974 in South Carolina between Charleston on the Atlantic Coast and Columbia in the central part of the state.

Other stations have been installed permanently or temporarily at sites of particular interest, especially at nearby reservoirs and at the Savannah River Plant just across the Savannah River from the project site. Recent events which have been recorded instrumentally with greater precision are given in table 2.5.2-1 with locations to an accuracy of 1/100°, implying an uncertainty in epicentral position on the order of 3 miles. This precision has also been given for events before the early 1970s that were relocated by Dewey and Gordon.

Few focal depths have been determined for events before the early 1970s. In their relocations of earthquakes Dewey and Gordon note that few of their calculated focal depths are estimated to be precise to within about 6 miles, although the data do indicate that small and moderate earthquakes in eastern North America typically have sources in the upper part of the earth's crust (about 20 miles). Site region earthquakes recorded since the mid-1970s frequently have focal depths, when determined, of improved precision to as fine as 1 mile. Asterisks (*) indicate that the depth has been constrained to enable a stable earthquake location solution.

The magnitude of an earthquake depends for its definition upon the amplitude of motion on a standard instrument normalized to take into account the separation of the earthquake location from the instrumental recording site. In the site region several magnitude scales are commonly employed which are not exactly equivalent. For the earthquakes of table 2.5.2-1 and drawing AX6DD385 the magnitudes are equivalent or roughly equivalent to body wave magnitudes, except where noted. The uncertainty in instrumentally determined magnitudes is roughly ±0.3 magnitude units. Magnitudes estimated from various forms of intensity information, including intensity fall-off with distance and extent of felt area as well as by empirical conversions from site intensity, have uncertainties on the order of ±0.3 to ±0.6 magnitude units.

Distances between each site region epicenter and the site are tabulated in table 2.5.2-1. In view of the uncertainties in earthquake location noted above, the distances are given only to the nearest 5 miles for those events with locations given to an accuracy of 1/10° and to the nearest 1 mile for those events with locations given to an accuracy of 1/100°.

These estimates are based on available felt report data from several standard sources [3][4] supplemented by a number of specialized references.[14-20] The highest intensity at the site is VI, which is associated with the August 31, 1886, Charleston, South Carolina, earthquake. The earthquakes closest to the site are the October 28, 1974, intensity IV event about 45 miles north of the site and the July 26, 1945, intensity V event about 48 miles to the northwest. Although no specific felt reports are listed at the site for these events, U.S. Earthquakes indicates a site intensity of I-III in Augusta for the 1945 event.

The earthquake of epicentral intensity greater than or equal to VII occurring nearest the site, exclusive of those earthquakes in the Charleston-Summerville, South Carolina, area, was the Union County, South Carolina, event of January 1, 1913. A recent evaluation of the felt reports from this earthquake[16] indicates an epicentral intensity of VII-VIII and a felt area that does not include the site. Therefore, for this study it is concluded that this earthquake, occurring about 110 miles north, was not felt at the site.

Several other site region earthquakes, including some aftershocks of the 1886 Charleston earthquake, are estimated to have been felt with intensity less than or equal to IV at the site.

In addition to the earthquakes within the site region, several large events at greater distances are significant. For example, the New Madrid, Missouri, earthquake sequence of 1811 and 1812 occurred about 530 miles west-northwest but was probably felt at the VEGP site. The epicentral intensities of the largest earthquakes of this sequence have been variously reported as X-XI[21] or XII.[3] Only limited felt information in the site region is available. Fuller[22] cites evidence that may be interpreted as indicating a maximum intensity of VI anywhere in Georgia. This is in close agreement with a recent reevaluation of felt data from the New Madrid sequence by Nuttli.[21] A generalized isoseismal map for the December 16, 1811, earthquake appearing in this reevaluation shows the site to lie in an area between the V and VI isoseismal lines. Therefore, the site intensity from this sequence is conservatively estimated to have been less than VI.

The Giles County, Virginia, earthquake of May 31, 1897, occurred about 280 miles north of the site. The epicentral intensity of this earthquake has been estimated to be as high as VIII.[16] A recent evaluation of the felt reports associated with this earthquake indicates that the southernmost extent of intensity III effects may have just included the site.[16] Therefore, it is possible that this earthquake may have been felt with low intensity at the site.

From the brief summary above, table 2.5.2-1, and drawing AX6DD385, it is clear that the Charleston-Summerville, South Carolina, area is the source of the seismicity most affecting the VEGP site. This is true both in terms of the maximum historical site intensity and the number of earthquakes felt at the site with lesser intensity.

Historic data indicate that this small area has been the location of earthquakes within South Carolina for as long as records have been kept. For example, a recent archival study[23] has indicated the occurrence of 18 probable earthquakes in South Carolina between 1698 and August 1886. Of these 18 events, 13 are believed to have originated near Charleston. This localization is in all likelihood due at least in part to demographic distribution. None of these 13 probable earthquakes had an intensity of greater than V. The seismicity of South Carolina as a whole was not anomalously high prior to the August 31, 1886, earthquake relative to that of the neighboring states of Tennessee, Georgia, and North Carolina.[23]

The Charleston earthquake of 1886 was associated with two main shocks 8 min apart and hundreds of aftershocks over the next several years. The center of the area of maximum intensity was located near Middleton Place, about 14 miles northwest of Charleston and 10 miles south-southeast of Summerville. There were two discrete zones of maximum damage: one near Middleton Place and Summerville and the other about 10 miles southwest near Ravenel and Rantowles. Both of these areas are included in the meizoseismal area of intensity X shown on drawing AX6DD385. It has been thought that this bimodal pattern resulted from each of the two main shocks having a different epicenter. It is possible, however, that local ground conditions and demography may have been responsible for the details of the high intensity pattern. The earthquakes were felt over an area of more than 2,000,000 mi^2; they were felt as far away as Boston, Milwaukee, New York, Cuba, and Bermuda. Minor damage was reported at Savannah, Georgia; Augusta, Georgia; and Columbia, South Carolina.

In the meizoseismal area more than 62 miles of railroad tracks were damaged. The damage included lateral and vertical displacement, formation of S-shaped curves, and the longitudinal movement of hundreds of meters of track.[15] This type of damage is first given at the intensity X level.[24] The formation of small sand craters and the ejection of sand were widespread effects. This is characteristic of intensity X if occurrence is on a large scale.[24]

Structures in the city of Charleston suffered extensive damage, but most masonry and frame buildings were not destroyed. Detailed descriptions of structural damage in Charleston may be found in Dutton.[14] This and the absence of rail damage and extensive ground effects indicate an intensity of IX in Charleston.[15]

A detailed isoseismal map throughout the State of South Carolina for the main shocks of the 1886 sequence is shown on drawing AX6DD385. This figure shows that the VEGP site was probably within an area of intensity VI, although pockets of higher intensity are present both closer to and farther from the epicenter.

Since the main earthquakes on August 31, 1886, both aftershocks and more recent seismic activity of epicentral intensity up to VII have occurred in the Charleston-Summerville zone. None of these earthquakes has been felt at the site with an intensity above IV.

2.5.2.2 Geologic Structures and Tectonic Activity

The site is located in the Atlantic Coastal Plain province at a point about 25 miles southeast of the boundary between this and the Piedmont province. Within 200 miles of the site are also the Blue Ridge province to the northwest of the Piedmont province and a small section of the Valley and Ridge province northwest of the Blue Ridge province. These physiographic provinces and the site location are shown on drawing AX6DD338.

Physiographic and geologic descriptions of these provinces may be found in paragraph 2.5.1.1.1. Regional geologic history is discussed in paragraph 2.5.1.1.2. This history consists of important episodes of Paleozoic orogeny, more modest tensional tectonism in the early Mesozoic resulting in Triassic basins and subsequent diabasic intrusions, and essential quiescence since the deposition of the Cretaceous sediments in the coastal plain. Some regional epeirogenic upwarping and arching during the late Eocene ended before Miocene deposition about 25,000,000 years ago.

The Pleistocene and recent history of the site region is largely represented by erosion of the southern Appalachian provinces and flood plain deposition and valley fill associated predominantly with the rivers and larger streams in the Atlantic Coastal Plain province.

In addition to the larger scale regional physiographic provinces and small scale Triassic basins, the Belair fault has been noted as a geologic structure of possible interest for evaluations of vibratory ground motion. This fault zone is discussed in paragraph 2.5.1.1.4.1. The most recent investigation of this fault zone[25] concludes that the last episode of movement occurred sometime within the last 60,000,000 years but prior to 23,000 to 2000 years ago. No intermediate age strata have been found that would provide a more definitive date of the last movement of the fault.

2.5.2.3 Correlation of Earthquake Activity with Geologic Structures or Tectonic Provinces

With the exception of the Charleston-Summerville area, seismicity of the site region is generally diffuse. There have been no definite correlations between earthquake epicenters and geologic structures. The evidence on the Belair fault zone is inconclusive as discussed in paragraphs 2.5.1.1.4.1 and 2.5.2.2. However, although lack of movement in the last 35,000 years has not been absolutely demonstrated, there is no correlation of any macroseismicity with this fault, and the general tectonic quiescence of the region argues against its likely significance. In a recent consideration,[26] it was concluded that the Belair fault zone is not a capable fault within the meaning of Appendix A to 10 CFR 100, section 3(g). This convention is followed in this study.

During the months of March through August 1982, further studies were made to determine the existence and capability of the postulated Millett Fault introduced in an open-file United States Geological Survey report.[27] No evidence was found in support of the existence of any fault in the region designated by the report. Details of the study can be found in a report entitled Studies of Postulated Millett Fault.[28]

Thus, for the purpose of vibratory ground motion at the VEGP site, historic earthquake activity is most logically correlated with tectonic provinces. Within 200 miles of the site, four distinct tectonic provinces are traditionally recognized. These are the Valley and Ridge, Blue Ridge, and Piedmont provinces of the Southern Appalachian Mountains and the Atlantic Coastal Plain province. The boundaries of these provinces in the site region are shown on drawing AX6DD341.

In the last 10 years, several working hypotheses have been proposed in an attempt to associate geologic structure and tectonic activity in the Charleston region with the source of the Charleston earthquake. Seeber and Armbruster[29] suggest backslip on the Appalachian detachment (decollement) as a possible mechanism for the 1886 Charleston earthquake and cite potential normal displacement in the area of the Brevard fault zone and a number of coseismic phenomena in 1886 as supporting evidence for this hypothesis. Behrendt[30] proposes reverse slip on steeply dipping faults of small offset northwest of Charleston and associated listrically with the decollement. Wentworth and Mergner-Keefer[31][32] have proposed reactivation of faults bounding Triassic basins throughout the east coast as a model for recurrence of a Charleston type fault.

These working hypotheses both assume and require that the principal horizontal stress orientation in the Charleston region is essentially northwest-southwest as indicated by Zoback and Zoback.[33] Latest information[34] indicates that a principal compression almost at right angles to this direction is apparently now acting in this region. This, and the failure of the proposed types of structure to be correlated with well-located instrumental hypocenters, recent earthquake focal mechanisms, or any evidence of geologically young faulting, argue against the adoption of these models at this time.

In this report, the southern Appalachian Mountains provinces are treated as a single region. This usage is for convenience only and is not intended to imply that the distinct Valley and Ridge, Blue Ridge, and Piedmont tectonic provinces, in general, should be considered as a single unit. However, such a usage is adequate within the narrow context of the assumptions employed to determine design vibratory ground motion at VEGP. This point is discussed in paragraph 2.5.2.4. In this report, these three tectonic provinces are called, collectively, the Southern Appalachian Mountains Region.

The Southern Appalachian Mountains Region so defined is bounded on the east, southeast, and south by the fall line and the Atlantic Coastal Plain tectonic province. On the northwest, it is bounded by the Cumberland and Allegheny Plateaus. On the northeast, along structural trend, geologic and seismic discriminates are more tenuous. Here this boundary is chosen to include northern Pennsylvania, northern New Jersey, and southernmost New York State and to exclude the Appalachians north of this area. The outline of the Southern Appalachian Mountains Region in the site region is shown on drawing AX6DD385. This area is a region of consistent northeast-southwest structural trends. As may be seen on drawing AX6DD385, epicenters in this area are irregularly distributed, with concentrations in northeast Georgia, northwest South Carolina, and eastern Tennessee in the site region and in Virginia farther to the north. These concentrations are all at distances greater than 100 miles from the site.

The Atlantic Coastal Plain tectonic province extends from the fall line on the northwest to the edge of the Continental Shelf to the east and southeast. To the northeast the boundary is near the northeastward extent of the Southern Appalachian Mountains Region as defined above. To the southwest the Atlantic Coastal Plain is conveniently separated from the Gulf Coastal Plain by the Pickens-Gilbertown fault zone and a line extending down from the northwestern border of the Southern Appalachian Mountains Region. The Atlantic Coastal Plain tectonic province thus defined is a large area with generally low seismic activity. With the exception of the Charleston-Summerville area within the Atlantic Coastal Plain, seismicity is more infrequent and of smaller size than in the Southern Appalachian Mountains Region. As used in this study, the Charleston-Summerville seismic zone area includes the 1886 meizoseismal zone and the epicenters of the cluster of earthquakes in this immediate vicinity that have occurred subsequently. This zone is sketched on drawing AX6DD385.

As discussed in paragraph 2.5.2.1, the intensity X earthquake of 1886 occurred in an area of previously low seismicity. It is the largest earthquake ever reported in the Southeastern United States. The seismic activity of the meizoseismal zone of this earthquake has remained anomalously high from that time. The occurrence of the 1886 earthquakes and the continued high activity of this area have prompted many recent studies aimed at better defining and understanding the source of this localized activity. Efforts to monitor local microearthquake activity [35-38] have revealed that a number of previously unrecordable small earthquakes do occur within the Charleston-Summerville seismic zone as shown on drawing AX6DD385. These epicenters near Middleton Place define a linear north-northwest trending zone about 15 miles long and 2 miles wide. Focal depths of earthquakes in this zone range from 1.25 to 7.5 miles, and hypocenter plots indicate that this zone is nearly vertical. The focal mechanism solution for the November 22, 1974, event (drawing AX6DD385) of intensity VI within this zone is consistent with dip-slip faulting on a nearly vertical fault striking northwest-southeast.[38] Studies of epicenters in this area are continuing.

The localized zone of seismicity is spatially associated with the northeast edge of a positive Bouguer gravity anomaly and the northeast lobe of a three-lobed positive magnetic anomaly. These anomalies have been interpreted to reflect the presence of mafic intrusives in the basement at depths of 1.5 to 2.8 miles.[39] This is within the range of depths established for the nearly vertical zone of seismicity. In turn, the presence of mafic intrusives with elastic rigidities differing from those of the surrounding shallow crustal rocks has been used to hypothesize several stress amplification mechanisms[40-43] that could help explain the localization of earthquakes in the Charleston-Summerville area. Scenarios of this type have not provided definitive conclusions to date. However, because of the clustering of historical earthquakes in the Charleston-Summerville area, because of the continuing microearthquakes there at levels higher than normal background seismicity for the Southeastern United States, and because of the geologic features peculiar to that area that may ultimately provide a mechanism for earthquake localization, it is accepted for the purposes of this analysis that near-future earthquakes in the Atlantic Coastal Plain tectonic province will continue to exhibit the features distinctive of the near past. That is, general seismicity will remain subdued, while high activity earthquake clustering in the site region will remain confined to the Charleston-Summerville seismic zone.

2.5.2.4 Maximum Earthquake Potential

The largest historical earthquakes within the Charleston-Summerville seismic zone are the two main events of August 31, 1886. As discussed in paragraph 2.5.2.1, recent reinterpretation of the damage reports associated with these earthquakes[15] indicates that the maximum intensity was X on the Modified Mercalli Scale.[13] The closest approach of the Charleston-Summerville seismic zone as drawn on drawing AX6DD385 and discussed in paragraph 2.5.2.3 is about 78 miles.

Exclusive of the Charleston-Summerville seismic zone, the maximum historical earthquake in the Atlantic Coastal Plain within the site region is of intensity VI. Several intensity VII earthquakes have been reported on or near the fall line between the Piedmont and Atlantic Coastal Plain tectonic provinces north of the site region; for example, the February 21, 1774, earthquake near Petersburg, Virginia; the November 11, 1840, and October 7, 1871, earthquakes near Philadelphia, Pennsylvania; and the December 23, 1875, event near Richmond, Virginia. As is discussed below, such events do not control the vibratory ground motion design even if it is assumed that they or similar earthquakes should be considered, for conservatism, at the site.

The maximum intensity site region earthquake to occur in the Southern Appalachian Mountains Region is the VII-VIII[9] event of 1913 in Union County, South Carolina. The greatest historic earthquake in this region, as it has been defined in paragraph 2.5.2.3, is the intensity VIII Giles County, Virginia, earthquake of 1897. In both cases the currently accepted maximum intensities of these events have resulted from conservative reinterpretations of the previously accepted lower maximum intensities of VI-VIII and VII, respectively.[3] In addition, the Giles County earthquake occurred in the Valley and Ridge tectonic province that is typically separated from the Blue Ridge and Piedmont provinces for purposes of seismotectonic province characterization. However, as is discussed below, even if an intensity VIII earthquake is considered capable of occurring within the Southern Appalachian Mountain Region at its 26-mile closest approach to the site, it is not the controlling event for seismic design. This is the principal reason that the Southern Appalachian Mountains Region was defined as it was in paragraph 2.5.2.3.

In order to evaluate the effects at the site of the three maximum earthquakes discussed above, several intensity attenuation formulas were considered. Two of these[15][44] have been derived from studies of the intensity distribution for the 1886 Charleston earthquake, while the third[45] is derived from isoseismals from a number of more recent events in the Eastern United States.

McGuire[44] derives an intensity attenuation function from 783 intensity reports from the 1886 Charleston, South Carolina, earthquake as these reports were reevaluated in a recent study.[46] Bollinger[15] follows the same procedure but fits the data to a formula of slightly different form. These two attenuation relations are, respectively:

$$I_s = I_o + 3.08 - 1.34 \; \ell n \; R \text{ and}$$

$$I_s = I_o + 2.87 - 0.00052 \; R - 2.88 \log R$$

Where ℓn is the logarithm to the base e (natural logarithm) and log is the base 10 (common) logarithm, I_s is near site intensity, I_o is epicentral intensity, and R is epicentral distance in kilometers. Substituting X for I_o and 78 miles x 1.6 km/mi = 125 km for R, the intensity at the site from a recurrence of the 1886 earthquake is 6.61 and 6.77, respectively. This is, as expected, entirely consistent with drawing AX6DD385, which shows the site at intensity VI to be in an area of lower than average intensity for its distance from the epicenter. Thus, best data indicates a recurrence of the 1886 Charleston earthquake would result in an intensity VI-VII to VII at the site.

To calculate the intensity at the site from the maximum earthquake in the Southern Appalachian Mountains Region, a recent formula developed by Anderson[45] is used. This formula is derived from the statistical consideration of isoseismal maps of 66 earthquakes in the Central and Eastern United States, including the 1897 Giles County earthquake. In this case the mean site intensity is given as:

$$I_s = I_o + 3.2 - 2.7 \; (0.00039 \; R + \log R)$$

For I_o = VII-VIII, R = 26 miles x 1.6 km/mi = 41.6 km, I_s = 6.28. For the more conservative I_o = VIII, I_s = 6.78. Thus, the effect at the site from the maximum, credible Southern Appalachian Mountains Region earthquake is very similar to the Charleston-Summerville event; that is, I_s = VI-VII to VII.

Finally, the site intensity from an earthquake in the Atlantic Coastal Plain province, exclusive of the Charleston-Summerville seismic zone, will be the same as the maximum historical event in this province. As discussed above, this is intensity VI.

Even if this event were assumed to be VII, it would not significantly supersede the site intensities from the other two pertinent earthquake sources in the site region.

Thus, the maximum potential earthquake intensity at the site is conservatively estimated to be VI-VII to VII.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

The site is underlain by Atlantic Coastal Plain sediments consisting of 800 to 1000 ft of predominantly clays, sands, limestone, and marl, ranging in age from Cretaceous to recent. Material properties for the sediments under the site and the methods used to determine these properties are discussed in paragraph 2.5.4.2.

No analysis to specifically take account of the site soil column is undertaken to translate the maximum potential site intensity into the safe shutdown earthquake (SSE) ground motion. This additional analysis is neglected because of the uncertainties involved in its performance and because the intensities of paragraph 2.5.2.4 are converted into peak accelerations in a conservative manner, using formulas relating peak acceleration to intensity on a wide variety of foundation conditions, including those similar to the foundation conditions at the site.

2.5.2.6 Safe Shutdown Earthquake

As discussed in paragraph 2.5.2.4, the maximum credible site intensity is VI-VII to VII. For additional conservatism a SSE site intensity of VII-VIII is chosen. According to the best available data relating peak horizontal ground surface acceleration to intensity on a variety of foundations,[47][48] intensity VII-VIII is associated with approximately 0.2 g peak horizontal acceleration. This relationship is appropriate for sites near the zone of energy release or for sites where I_s is not much less than I_o. This is the case for the Atlantic Coastal Plain maximum credible event and approximately the case for the Southern Appalachian Mountains maximum credible event. This condition is not well realized for the Charleston-Summerville seismic zone maximum credible event. Since acceleration attenuates more rapidly than intensity, the use of the empirical relationships noted above is probably very conservative in this case. However, the larger Charleston earthquake is likely to be richer in low frequency ground motion at the site than the other design earthquakes. Thus, for conservatism a single design response spectrum is proposed to define site SSE ground motion. This ground motion is defined in terms of Regulatory Guide 1.60 horizontal and vertical design response spectra[49] normalized to a peak ground acceleration of 0.20 g. These spectra are shown in section 3.7.

2.5.2.7 Operating Basis Earthquake (OBE)

As discussed in paragraph 2.5.2.1, the most recent best evidence indicates that the maximum historical intensity at the site was VI, associated with the Charleston earthquake of 1886. A less finely detailed study of attenuation of intensity from that earthquake would indicate a somewhat higher intensity should be expected at the site. For additional conservatism an OBE intensity of VII is adopted. Using the intensity/acceleration relationships noted in paragraph 2.5.2.6, this intensity is associated with a peak horizontal ground surface acceleration on average foundation condition and near the zone of energy release of approximately 0.12 g. This acceleration, and spectra of identical form as those characterized in paragraph 2.5.2.6, are used to define the OBE ground motion at the site and are shown in section 3.7.

A probabilistic estimate of the occurrence of the OBE acceleration at the site during its 40-year[a] operating life may be made based on the work of Algermissen and Perkins.(50) This study shows that the site acceleration with a 90% chance of nonexceedence in a 50-year interval is about 0.10 to 0.11 g.

Assuming, as is implicit in this characterization, that earthquakes occur as a Poisson point process, this is equivalent to estimating an 8% chance of occurrence of a site acceleration exceeding 0.10 to 0.11 g during the 40-year operating life of the plant. The chance of exceeding the 0.12 g OBE is, therefore, somewhat less than this.

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[a] The operating licenses for both VEGP units have been renewed and the original licensed operating terms have been extended by 20 years. Seismic analyses are not related to aging and therefore are outside the scope of license renewal. NEI 98-03 guidance indicates that seismic data used to support original plant design bases are considered historical and do not need to be actively maintained. Therefore, this statement is not required to be updated as a result of license renewal.

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47. Neumann, F., Earthquake Intensity and Related Ground Motion, University of Washington Press, Seattle, Washington, p 93, 1954.

48. Trifunac, M. D., and Brady, A. G., "On the Correlation of Seismic Intensity Scales With the Peaks of Recorded Strong Ground Motion," Bulletin of the Seismiological Society of America, Vol 65, pp 139-162, 1975.

49. U.S. Nuclear Regulatory Commission, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Regulatory Guide 1.60, Revision 1, December 1973.

50. Algermissen, S. T., and Perkins, D. M., "A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States," U.S. Geological Survey Open-File Report 76-416, p 45, 1976.

2.5.3 SURFACE FAULTING

No evidence of surface faulting has been uncovered in the site area. Detailed stratigraphic study and mapping of the excavations for Category 1 structures are discussed in paragraph 2.5.1.2.2. Mappable lithologic units may be traced unbroken around the perimeter of the excavations, demonstrating the absence of faulting. The 5-mile-radius site investigation showed no evidence of surface displacement that might localize earthquakes in the immediate vicinity of the site. This region has been relatively stable for a considerable length of time, and known faults in the Piedmont province to the west and the Triassic basin underlying the site are inactive. The geology of this area is discussed fully in paragraph 2.5.1.2.

2.5.3.1 Geologic Conditions of the Site

The lithologic, stratigraphic, and structural geologic conditions of the site are presented in paragraphs 2.5.1.2.2 and 2.5.1.2.3. Regional, local, site, and site excavation geologic maps are shown on drawings AX6DD341, AX6DD345, AX6DD351, AX6DD360, AX6DD361, AX6DD362, and AX6DD363, respectively. The regional geology and geologic history are discussed in paragraph 2.5.1.1.

2.5.3.2 Evidence of Fault Offset

The area within 5 miles of the site is not associated with known or suspected faulting. Geologic sections throughout the site and power block area, which are based on data obtained from field mapping, exploration borings, and mapping of the foundation excavation, reveal no evidence for the existence of any fault offset at the site (drawings AX6DD345, AX6DD351, AX6DD360, AX6DD361, AX6DD362, and AX6DD363).

2.5.3.3 Earthquakes Associated with Capable Faults

There are no known capable faults within 5 miles of the site.

2.5.3.4 Investigations of Capable Faults

Field investigations for this project, including exploratory drilling, field mapping, studies of aerial photography, and geophysical studies, show that no capable faults exist within 5 miles of the site. Reversal of the regional dip northwest of the site has been investigated and shown to be related to depositional and erosional processes, as discussed in paragraph 2.5.1.2.3. Stratigraphic irregularities discovered during excavation for power block foundation have been studied and shown to be related to depositional and erosional processes, as discussed in paragraph 2.5.1.2.2.2.

2.5.3.5 Correlation of Epicenters with Capable Faults

There are no known capable faults within 5 miles of the site. There is no correlation of epicenters with known or suspect faults within 5 miles of the site.

2.5.3.6 Description of Capable Faults

No capable faults are known to occur within 5 miles of the site.

2.5.3.7 Zone Requiring Detailed Faulting Investigation

In 1982, the U.S. Geological Survey released Open-File Report 82-156, which postulated the existence of two potentially capable faults within 32 miles of VEGP. According to the report, the Millett fault was located approximately 7 miles south of VEGP, while the Statesboro fault was located approximately 32 miles south of VEGP. The report did not assert that either of the faults were capable, but due to their proximity to the site, especially the Millett fault, a full-scale investigation was undertaken to determine exact location and date of last movement.

2.5.3.8 Results of Faulting Investigation

The investigation of the postulated Millett and Statesboro faults was completed with the conclusion that these faults did not exist within the depths to which the investigation extended and that, if they exist at some depth greater than the investigated depth, then they are not capable faults by virtue of the age of undisturbed overlying sediments. The investigative program is described completely in a separate report entitled, Studies of Postulated Millett Fault, prepared by Bechtel Power Corporation, dated October 1982.

2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

2.5.4.1 Geologic Features

There is no evidence that the Blue Bluff marl, which is the bearing stratum, has been subjected to or is potentially subject to subsidence, collapse or uplift due to earthquake, solution processes, or other geological phenomena (paragraph 2.5.1.2). Surface materials, comprised of strata which overlie the marl, have been subjected to and are potentially subject to subsidence due to solution processes (paragraph 2.5.1.2). These materials have been completely removed in the power block, and all Category 1 structures in the plant area are founded directly or indirectly on the marl (paragraphs 2.5.1.2.2 and 2.5.1.2.3.3).

The geologic history (paragraph 2.5.1.2.4) indicates that the plant site is located upon an area of regional uplift and has been subjected to subaerial erosion during Quaternary times. The stratigraphic sequence and investigative work (paragraph 2.5.1.2) indicate approximately 950 ft of unlithified to poorly lithified sediments resting upon pre-Cretaceous basement rock. Rebound in the marl, the bearing stratum has been monitored and is discussed in paragraph 2.5.4.10.

The surface of the shallow (unconfined) ground water table historically has been approximately el 160 ft. The marl, which is the bearing stratum, is essentially impermeable and is an effective aquiclude comprising the base of the ground water table and the cap of a confined aquifer. The hydrostatic surface elevation of the confined aquifer is approximately 115 ft.

There are no deformational zones, irregular weathering, jointing or fracturing systems, crushed zones, or other indications of structural weakness in the marl which is the bearing stratum (paragraphs 2.5.1.2.3 and 2.5.1.2.8).

There are no materials at the site that are hazardous or may become hazardous due to lack of induration or consolidation, variability, high water content, solubility, or undesirable response to natural or induced conditions.

2.5.4.2 Properties of Subsurface Materials

The subsurface conditions in the plant site may be subdivided into three principal strata. The top stratum consists of sands, silty sands, and clayey sands with occasional clay seams. This stratum, referred to hereinafter as the upper sand stratum (Barnwell Group), is about 90 ft thick. At the base of the upper sand stratum is a shelly limestone (Utley Limestone) which is about 5 ft thick on an average. Below the upper sand stratum is a stratum consisting of a very hard calcareous clay marl (Blue Bluff marl), ranging in thickness from 60 to 100 ft. This stratum is referred to as the marl bearing stratum. The stratum beneath the marl bearing stratum consists principally of dense, coarse to fine sand with minor interbedded silty clay and clayey silt. This unit (Ellenton Formation) is called the lower sand stratum. The thickness of this stratum is estimated to be at least 750 ft.

Based on the results of the site exploration, it was determined that the upper sand stratum would have a potential for liquefaction in the event of a seismic occurrence equivalent to the safe shutdown earthquake (SSE).(1) It was also determined that the shelly limestone layer is characterized by solution channels, cracks, and discontinuities within it. Consequently, it was concluded that the upper sand stratum materials and the shelly limestone layer should be excavated down to the marl bearing stratum and replaced with select sand and silty sand backfill compacted to a sufficient degree to preclude the possibility of liquefaction and to reduce settlement to a tolerable level. With the exception of the auxiliary building, nuclear service cooling water towers, and instrumentation cavity of the containment which are founded on the marl bearing stratum, all the power block structures including the containment basemat and the non-Category 1 turbine building are supported on Category 1 backfill. The location of these structures is shown in drawings CX2D45V003 and AX1D45A01. Compacted fill and marl foundations are indicated on drawing AX6DD386.

The static and dynamic engineering properties of the three principal soil strata and for compacted Category 1 backfill were determined by field investigation and laboratory testing. The results of all the field and laboratory work and data evaluation are covered in five separate reports.(1-5) A discussion and summary of the static and dynamic soil properties of the upper sand, marl, and lower sand strata are presented in paragraphs 2.5.4.2.1, 2.5.4.2.2, and 2.5.4.2.3, respectively. The static and dynamic soil properties of compacted Category 1 backfill are summarized and discussed separately in paragraph 2.5.4.5.2.

2.5.4.2.1 Properties of Upper Sand Stratum (Barnwell Group)

The static engineering properties of the upper sand stratum are summarized in table 2.5.4-1. A range of values is given for most properties. The standard penetration test data indicate that the relative density of the upper sand stratum is extremely variable and ranges from very loose to dense. The consistency of the clay lenses in this stratum ranges from soft to medium.

Unconsolidated undrained triaxial test results from samples in this stratum indicate that the Mohr strength envelope of total stresses may be defined by parameters ranging from about $c=2100$ lb/ft^2, $\phi=6^\circ$ to $c=440$ lb/ft^2, $\phi=32^\circ$ depending upon the predominance of clay or sand.

Similarly, consolidated undrained triaxial test results ranged from $c=1650$ lb/ft^2, $\phi=17^\circ$ to $c=4000$ lb/ft^2, $\phi=25^\circ$ for the Mohr strength envelope of total stresses and from $\phi=33^\circ$ to $\phi=34.5^\circ$ for the Mohr strength envelope of effective stresses. The design properties shown in table 2.5.4-2 were developed from the static engineering properties summarized in table 2.5.4-1.

The test data and the procedures used to obtain these data are included in reference 1 and its appendices.

A summary of the design dynamic shear modulus at strain levels of 10^{-4}% or lower for the upper sand stratum is given in table 2.5.4-3. The basic properties of the upper sand stratum to be used in dynamic analyses are summarized in table 2.5.4-4.

Values of the dynamic shear modulus are computed from in situ shear wave velocity measurements as follows:

$$G = \frac{\gamma}{g}(V_s)^2$$

where:

G = shear modulus (lb/ft^2).

γ = unit weight (lb/ft^3).

g = acceleration due to gravity (ft/s^2).

V_s = shear wave velocity (ft/s).

2.5.4.2.2 Properties of Marl Bearing Stratum (Blue Bluff Marl)

The marl bearing stratum is a zone of hard, slightly sandy, cemented, calcareous clay. It is the uppermost stratum capable of supporting heavy structural loads. Consistency of the marl varies from hard to very hard, moderately brittle material resembling a calcareous siltstone or claystone. Seismic explorations indicate a velocity interface about 15 ft below the top of the stratum. The material below that level has a compressional wave velocity approaching 7000 ft/s as compared to about 5000 ft/s for the upper portion of the stratum.

This is probably due to some degree of weathering of the upper 15 ft. The static engineering properties of the clay marl bearing stratum are summarized in table 2.5.4-1. Ranges of value are given for the most important properties.

The standard penetration test values range from 10 blows/ft in the weathered marl at the contact with the shell zone to well in excess of 100 blows/ft. The unconsolidated undrained shear strength based on one-point tests ranged from c=260 lb/ft^2 to c=500,000 lb/ft^2, with 10,000 lb/ft^2 being the value adopted for design. Samples that yield undrained strengths less than 10,000 lb/ft^2 exhibit large strains to failure which normally indicate sample disturbance in brittle materials of this type.

Laboratory tests indicate that the marl bearing stratum is highly preconsolidated. Atterberg limit tests indicate that the plasticity index is between 2 and 70%. Using an average of 25% this would yield a S_u/p ratio of about 0.2 based on work by Skempton,[6] where S_u is the undrained shear strength and p is the effective consolidation pressure at sample depth. This indicates that the preconsolidation pressure, would be 80 k/ft^2 for the average undrained shear strength of 16.0 k/ft^2. The average undrained strength is taken to be the average of all samples which failed at strengths less than 50 k/ft^2. With such a high preconsolidation pressure, it would be expected that settlements under structure loads would be small and would occur rapidly as load is applied. The design properties shown in table 2.5.4-2 were developed from the data summarized in table 2.5.4-1.

The basic test data and procedures used to obtain these data are contained in reference 1 and its appendixes.

The undrained shear strength of the marl bearing stratum was verified after completion of the power block excavations by testing representative cores. The results of these tests are included in reference 2. These test results verified that the recommended design strength parameter of c=10,000 lb/ft^2, ϕ=0° is appropriately conservative. Actually, the average undrained shear strength of all core samples that failed was approximately 20 k/ft^2, and the lowest measured value was 11.7 k/ft^2. Therefore, all samples tested exceeded the design strength of 10 k/ft^2. During the excavation, the heave of the marl stratum was observed and recorded. Heave values were measured at nine different locations within the power block area. These data are included in reference 2. An average heave of approximately 1.25 in. was measured in the power block after the excavation was complete. Based on the heave data, the undrained Young's modulus of the clay marl stratum was computed to be 10,000 k/ft^2. This value was consistent with the range of values for the Young's modulus obtained from Menard pressuremeter and seismic velocity measurements but was significantly higher than the 4000 k/ft^2 used to evaluate elastic settlements in the Preliminary Safety Analysis Report (PSAR).

A summary of the design dynamic shear modulus at strain levels of 10^{-4}% or lower for the clay marl stratum is given in table 2.5.4-3. Reduction factors to be used for determining the shear modulus at strains larger than 10^{-4}% are given in figure 3.7.B.2-6. The properties used in dynamic analyses are summarized in table 2.5.4-4. Values of the dynamic shear modulus were computed from in situ shear wave velocity measurements as previously described in paragraph 2.5.4.2.1.

2.5.4.2.3 Properties of Lower Sand Stratum (Ellenton Formation)

Pertinent static engineering properties of the lower sand stratum are summarized in table 2.5.4-1. Design values for dry unit weight, in situ moisture content, and standard penetration test are summarized in table 2.5.4-2.

The standard penetration test values varied from 70 blows/ft to more than 100 blows/ft with the majority of values exceeding 100 blows/ft. Such high blow counts indicate the very dense state of the lower sand stratum. Based on seismic investigations, the lower sand stratum extends to basement rock which is located approximately 900 to 1000 ft below natural ground. Design dynamic properties of the lower sand stratum are summarized in tables 2.5.4-3 and 2.5.4-4.

2.5.4.3 Exploration

Geologic and soils investigation of the VEGP site has been completed and the results are described in subsection 2.5.1. This section summarizes the exploratory work performed.

The geologic and soils data and their evaluation were obtained in separate and individual investigations conducted by Law Engineering Testing Company (LETCO) and Bechtel Power Corporation. Included were a thorough search of the literature, stereoscopic examination of color air photographs, evaluation of geologic conditions at and within 5 miles of the site, and geologic reconnaissance along 12 miles of the river bluff upstream and downstream of the site.

2.5.4.3.1 Borings and Samplings

Field investigations started in January 1971 and involved drilling, geophysical survey, and ground water studies. Drilling was also carried out during construction excavation to verify and obtain further details concerning subsurface conditions in the power block area. In all, 474 holes have been drilled for a total of 60,000 ft of hole. The drilling logs of holes used for the primary geologic investigation and foundation investigation for the plant facilities for the PSAR are discussed in appendix 2B, and details of these borings are tabulated in table 2B-1. An inventory of cores retained and stored is contained in appendix 2B, table 2B-2. Drilling statistics of 111 holes drilled subsequent to PSAR investigations are as follows: 41 were drilled in the river facilities area to define soil conditions and lateral extent of the marl and to obtain permeability data; 38 were drilled in the power block area to obtain samples of the marl for laboratory testing for a record of the properties of the marl; and 32 were drilled for natural draft cooling tower foundation information.

The locations of the borings are on drawings AX6DD343 and AX6DD344 with the exception of the 31 "CS" series holes drilled in the power block, which are shown on drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363.

The exploration program included electric logging, natural gamma, density, neutron, caliper, and "3D" (Birdwell) velocity logs in selected drill holes. Water pressure tests and Menard pressuremeter tests were conducted to determine in situ properties of the bearing horizon, the marl. Samples for fossil, mineral, or soluble carbonate analysis were taken in those drill holes where conditions were suitable. The geophysical survey conducted to supplement the drilling program is described in paragraph 2.5.4.4. It provided a total of 28,400 ft of shallow refraction seismic lines, 5000 ft of deep refraction lines, and cross-hole velocities in the upper 290 ft of materials. Seismic survey lines and profiles are shown on drawings AX6DD387, AX6DD388, AX6DD389, AX6DD390, AX6DD391, AX6DD392, and AX6DD393. Down-hole geophysical logs are discussed in appendix 2B.

Drilling services were provided by Georgia Power Company (GPC), Girdler Drilling Company, and LETCO. Geophysical seismic surveys were performed by Weston Engineers, Weston, Massachusetts. Down-hole geophysical surveys were performed by the Birdwell Division of Seismograph Service, Incorporated.

2.5.4.3.2 Backfill

For a summary of backfill exploration refer to paragraph 2.5.4.5.2.2.

2.5.4.4 Geophysical Surveys

Geophysical seismic refraction and cross-hole surveys were conducted at the site to evaluate the occurrence and characteristics of subsurface materials. The seismic refraction survey was used to determine depths to seismic discontinuities, based on measured compressional wave velocities. Shallow and deep refraction profiles were obtained throughout the site area, totaling 28,400 and 5000 linear ft, respectively. The cross-hole seismic survey was conducted in the power block area to determine in situ velocity data for both compression and shear waves to a depth of 290 ft (82 ft below sea level) in bore holes 136, 146G, 148, 149, 151, and 154. In this procedure, three-dimensional detectors were lowered into four of the bore holes to equal elevation levels. Energy was generated in a fifth bore hole, at the same elevation level, to determine cross-hole velocities.

The locations of the seismic survey lines, the borings used for cross-hole velocity measurements, and the seismic profiles are shown on drawings AX6DD387, AX6DD388, AX6DD389, AX6DD390, AX6DD391, AX6DD392,and AX6DD393. Table 2.5.4-5 is the compilation of the results of the cross-hole measurements. The seismic velocity zones are summarized and related to other data in table 2.5.4-6. These data were used in determining the elastic moduli, compiled in table 2.5.4-7.

2.5.4.5 Excavation and Backfill

2.5.4.5.1 Excavation

The natural ground surface in the plant area varied between el 200 and 230 ft. The power block area was excavated and graded to an elevation of approximately 130 to 135 ft near the top of the marl bearing stratum which is the clayey marl of the Blue Bluff Member of the Lisbon Formation. In the following and previous discussions, this is called the marl or the clay marl bearing stratum. The excavation for the power block structures at the VEGP site is roughly square in shape; there are three access ramps, one each in the northwest, southeast, and southwest corners of the excavation. It measures approximately 1400 ft on an edge at the top and 1000 ft on an edge at the toe. The side slopes were cut at a gradient of two horizontal to one vertical. The total excavated volume in the power block was approximately 5,000,000 yd^3 including the access ramp. Drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363 geologic maps of the excavation as of 1977, before access roads were completed.

Within the excavation, a deeper localized excavation was made for the auxiliary building basement (drawings AX6DD374 and AX6DD375). This consisted of a rectangular area measuring approximately 120 ft by 440 ft. The base of this excavation was at approximately el 108 ft, and the walls were cut vertically, with a horizontal bench at el 118 ft. The four nuclear service cooling water towers are founded directly on the marl just south of the auxiliary building. The other major power block structures are founded on structural backfill at elevations above the floor of the excavation.

Excavation work was started in May 1974 and postponed on September 12, 1974. The bottom elevation of the excavation averaged approximately 145 ft at this time and close to 900,000 yd^3 of excavation remained. The excavation work was resumed in February 1977 and the auxiliary building excavation was bottomed out in October 1977.

As excavation progressed, the exposed materials were geologically mapped (drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363), including the deeper localized excavation for the auxiliary building (drawings AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, AX6DD370, and AX6DD371). A discussion of the mapping is presented in paragraph 2.5.4.5.1.2.

2.5.4.5.1.1 Excavation Procedures. Excavation work started and progressed very rapidly using scrapers and bulldozers in the upper sands (above the water table) which are at about el 160 ft. Very little, if any, ripping was required because of the sandy nature of the deposits; a maximum rate of 120,000 yd^3/day was attained at the peak of activity. Upon reaching the water table, construction dewatering was begun. The site ground water conditions are discussed in detail in subsection 2.4.12. The procedures utilized during excavation for construction dewatering are discussed in paragraph 2.4.12.1.3.1.

When the excavation reached the zones of hard shell-rich limestone described earlier (Utley Limestone), limited blasting of the rock was utilized to facilitate its removal. Since the shell-rich limestone was immediately above the marl, it was necessary to control any required blasting in such a manner as to protect the underlying marl (marl bearing stratum) from damage. The major portion of the rock was removed by first breaking it with a hydraulic ram mounted on a backhoe, then loading it out with conventional equipment.

Excavation of the marl was accomplished by ripping, followed by conventional earth moving. The auxiliary building basement excavation was cut with bulldozers and front-end loaders. Trimming of the walls was accomplished with a backhoe. Some of the hard, indurated limestone layers within the marl were first broken with the backhoe-mounted hydraulic ram, then removed by front-end loader. Fine grading of the floor of the power block was accomplished with motor graders in areas underlying future structural backfill and with Gradalls in the nuclear service cooling water tower foundation areas. In the foundation areas, shovels and air hoses were used for cleanup of loose material.

2.5.4.5.1.2 Geologic Mapping Procedures. The geologic mapping and recording of features exposed during excavation are described in the Bechtel Report of Geology and Foundation Conditions (appendix 2B.3). The mapping entailed these phases:

A. Detailed mapping of deposits above the marl; May 1974 to October 1974.

B. Detailed mapping of features within the marl and surveying of the upper contact of the marl; February 1977 to October 1977.

C. Detailed inspection and recording of areas in the marl approved for placement of concrete or backfill; June 1977 to January 1979.

The first phase of mapping was performed in conjunction with the excavation of the sediments above the marl. Features were located in the side slopes of the excavation as the bottom elevation was progressively lowered. The side slopes were cut at a gradient of two horizontal to one vertical, and survey stakes were installed on a grid pattern on the slopes. Locations of geologic features were measured by tape and hand-level methods using the slope stakes as reference points. Accuracy of these measurements is estimated to be within 0.5 ft. Mapping of the 2:1 slopes was recorded in plan on a base map compiled from the project excavation drawings and is shown on drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363.

The second phase of mapping was accomplished as the marl was exposed and prepared for placement of concrete and backfill. To demonstrate the absence of faulting in the marl, the contact between the marl and the overlying sediments was mapped and recorded with survey accuracy around the perimeter of the excavation. Five hundred seventy-five survey points were established by the geologists along this contact and these points were located instrumentally. The nature of the contact between the points was examined closely for continuity and absence of breaks. The contact is shown in plan view on drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363, and the details of the survey results are shown in both plan and section on drawings AX6DD372 and AX6DD373.

In addition to examining the upper contact, features within the marl were examined and recorded. The deep excavation for the auxiliary building basement, within the larger power block excavation, provided an excellent opportunity for this. The sides of the excavation exposed a vertical section of approximately 22 ft in height in the marl. A system of reference points was established on the walls of the excavation and stations were established for the purpose of describing locations of features. The stationing system adopted is shown on drawings AX6DD374 and AX6DD375. The mapping was recorded in the vertical plane and is presented as geologic sections on drawings AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, and AX6DD370. An explanation of geologic units used for mapping purposes is shown on drawing AX6DD370. By referring to drawings AX6DD374 and AX6DD375, the location of any section can be easily ascertained. Measurements were made by tape and hand-level methods on the excavation walls. Accuracy is generally within 0.1 ft.

The third phase of geologic mapping consisted of detailed inspection and photography of foundation areas rather than mapping in a strict sense. This effort was initiated in June 1977 when the first portion of the auxiliary building basemat excavation (vertical surface) was cleaned off at final grade and prepared for application of a protective seal. Inspection and approval of final grade in the marl was documented and transmitted from the inspecting Bechtel geologists to GPC. This documentation has been transferred to Southern Nuclear Operating Company (SNC), as the exclusive operating licensee.

2.5.4.5.1.3 Construction Dewatering. A discussion of construction dewatering is contained in paragraph 2.4.12.1.3.3.1.

2.5.4.5.1.4 Slope Protection. During the early stages of excavation, intense rainfall caused erosion of the 2:1 side slopes of the power block excavation. The uncemented sands above the marl were eroded, resulting in deeply incised gullies in some areas. These gullies were backfilled with the native soil material, and local areas of the slope were regraded. One such area is seen on the geologic map (drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363) in the upper part of the east slope between stations N83+00 and N84+00. Another larger area exists in the south slope of the access ramp east of station E100+00. After regrading the eroded areas, berms were constructed around the tops of the slopes to control runoff. The surfaces of the slopes were sprayed with the chemical stabilizing agent Petroset, a colorless liquid which sets up and tends to bond the sand grains together. These measures proved to be generally successful in controlling further erosion.

After resumption of excavation work in 1977, erosion problems further down the slopes were encountered due to seepage of the perched ground water out of the slopes. Since stabilizing agents were expected to be ineffective under these conditions, the lower portions of the slopes were blanketed with a transition zone and covered with riprap to improve stability.

At the base of the upper sand stratum where the 2:1 slopes intersected a limestone shell bed (Utley Limestone on drawing AX6DD352), several cavities of varying size were exposed in the slopes. The location and extent of the cavities exposed in the slopes is shown on drawing AX6DD363. The largest of these existed in the northwest corner of the power block and had an opening measuring 10 ft by 10 ft. This cavity extended back into the slope some 30 ft before narrowing down to a small size. Other small cavities were encountered at varying intervals all along the north side of the power block excavation. It was necessary to fill these cavities so that an effective buttress would be formed against which the future structural backfill could be placed and compacted. The cavities were first cleaned of loose debris, then backfilled with crushed rock (Georgia State Standard No. 467). The crushed rock was packed into the cavities by means of a 20-ft-long ram attached to the blade of a bulldozer. The large cavity in the northwest corner was effectively filled in this manner to at least a distance of 25 ft back of the entrance.

To retard erosion of temporary slopes in Category 1 backfill placed in the power block excavation, these slopes were sprayed with a commercial compound known by the trade name Glassroot. It consists of a glass fiber material which is sprayed onto the slope, then coated with a film of asphalt emulsion. Other measures which also proved to be effective in controlling erosion of the compacted sandy backfill included the use of gunite, plastic sheeting, and sand bags.

2.5.4.5.1.5 Foundation Cleanup and Protection. As mentioned previously, the Blue Bluff marl (marl bearing stratum) at final grade in foundation areas was exposed using either a motor grader or Gradall. Loose material was then removed by shovel, broom, and air hose. On the vertical walls of the auxiliary building excavation, final trim to neat line was accomplished with a backhoe followed by pick and shovel and air hose techniques.

In all cases where final grade was exposed and cleaned off, the marl surface had to be covered in a manner approved by the geologist within 24 h of exposure. On horizontal surfaces the marl was covered either by structural backfill, a gunite protective layer, or a lean concrete mudmat depending on whether the particular area exposed was in a foundation or backfill area. The vertical walls of the auxiliary building basement excavation were coated with a 4-in.-thick layer of gunite reinforced with welded wire mesh.

In some cases, temporary covers such as loose soil or plastic sheeting were employed when the permanent cover material could not be applied within the 24-h limit. In all cases the temporary cover procedure was approved by either the geologist or the GPC inspector. Before placing the permanent cover material in any foundation area, the marl was inspected and approved by the geologist or soils engineer in accordance with prescribed procedures.

2.5.4.5.1.6 Foundation Inspection and Approval Procedures. All areas of marl (Blue Bluff marl) exposed and cleaned off in preparation for placement of concrete or backfill were examined closely for any evidence of loose or soft zones, geologic discontinuities, or unusual geologic features. After confirming the absence of such features, the inspecting geologist photographed and approved the excavated foundation area and documented the approval on a special form. The photographs and approval documents are part of the permanent project records.

2.5.4.5.1.7 Foundation Testing. During the general geologic mapping of the marl and other inspecting functions, a program of coring and testing samples of the marl was conducted to confirm the material properties used for design. The coring and sampling operation was performed under the direction of a Bechtel geologist and a GPC inspector, and the test assignments were made by the Bechtel foundation engineer.

A total of 38 core holes and offset replacement core holes were drilled by the rotary method in the floor of the power block excavation at 29 locations selected by the geologist. The hole locations are shown on drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363. The marl was cored to depths between 4 and 11 ft beneath the final excavated grade. Selected samples of 4-in.-diameter core were labeled and placed in wooden boxes for permanent storage at the site. Samples selected for laboratory testing were wrapped in cellophane, sealed with wax, and placed in special boxes for transportation to the laboratories of LETCO in Atlanta.

The results of the testing program are discussed in paragraph 2.5.4.2.2.

2.5.4.5.2 Backfill

Compacted backfill is placed in the power block area from the top of the marl stratum at approximately el 130 ft to the design elevation for each structure. The plant grade elevation is at 219 ft 6 in. or below. The auxiliary building and nuclear service cooling water towers, containment instrumentation cavity, and radwaste solidification building are supported directly on the marl stratum. The other safety-related power block structures are supported on compacted backfill. The foundation elevations of these structures are given in table 3.7.B.1-2. The radwaste solidification building foundation consists of large diameter drilled caissons extending into the marl stratum.

With the exception of an area north of the turbine building, an area over the in-situ slopes forming the west side of the power block excavation, and localized areas around nonsafety-related buried piping above the water table, all backfill in the power block area is compacted to an average of 97% of the maximum density determined by American Society of Testing

Materials (ASTM) D 1557, with no tests below 93% and not more than 10% of the tests between 95 and 93%. A procedure to achieve the required degree of compaction was developed in a test fill program. The results of the test fill program are discussed in paragraph 2.5.4.5.2.7 and presented in detail in reference 7.

The area north of the turbine building was compacted to an average of 95% of the maximum density determined by ASTM D 1557 with not more than 10% of tests between 93 and 95% and no test below 93%. The static stability and liquefaction analyses (paragraphs 2.5.4.8 and 2.5.4.10) were performed for the case where the power block backfill was assumed compacted to 97% relative compaction. A 95% relative compaction for the area north of the turbine building between el 185.5 to 219.5 ft has no effect on safety-related structures, since no Category 1 structures rely on this material for a load bearing foundation. The integrity of the turbine building design is not affected because the area does not project below the bottom of the building and does not provide foundation support for the turbine building. Since the area north of the turbine building is away from Category 1 structures and represents less than 10% of the total power block backfill, the factor of safety against liquefaction is not affected.

The area over the in-situ slopes forming the west side of the power block excavation was compacted with Category 2 material to an average of 95% of the maximum density determined by ASTM D 1557 with not more than 10% of tests between 93 and 95% and no test below 93%. The area is above elevation 206 and bounded by the coordinates N73+49, to N86+20 and E89+79 to E91+38. The use of Category 2 material compacted to 95% in this area has no effect on safety-related structures, since neither Category 1 structures nor the turbine building rely on this material for a load bearing foundation. The material is directly above the in-situ soils which are themselves not used to support Category 1 structures. Because the material is also over 40 ft above the water table and away from Category 1 structures the factor of safety against liquefaction is not affected.

Approximately 2.5 ft of undocumented fill was placed beneath a nonsafety-related concrete slab located above the in-situ excavation slopes on the south side of the power block. The slab is bounded by the coordinates N73+55 to N74+40 and E93+98 to E95+28, and is located at grade. The undocumented soils do not support, nor are they adjacent to, safety-related structures or equipment. They are located directly above in-situ soils which are themselves not used to support safety-related structures or equipment. Because the material is over 45 ft above the water table and located away from Category 1 structures, the factor of safety against liquefaction is not affected.

An approximate 6 inch lift of undocumented backfill was placed at grade level in a trench used for routing two separate 4-inch diameter PVC drain lines in the low voltage switchyard area. These additional pipe drains were installed to remove rainfall runoff that was accumulating in the bottom of the station service trenches and in two pull boxes. The undocumented soils do not support, nor are they adjacent to, safety-related structures or equipment. Because this material is at least 45 feet above the water table and located away from Category 1 structures, the safety factor against liquifaction was not affected.

Up to 24 in. of graded aggregate material was substituted for sand and silty sand backfill as the power block fill was brought to finished grade. This material was compacted to 97% of the maximum density determined in accordance with ASTM D 1557 and served to protect the sand and silty sand Category 1 backfill from erosion and vehicular traffic.

The localized area around nonsafety-related buried piping and similar conduits is compacted with concrete sand or other sands with similar properties to an average of 95% of maximum density determined by ASTM D 1557, with no tests below 93% and not more than 10% of the tests between 93 and 95%, unless compacted to an average of 97% using Category 1 backfill as defined below for safety-related piping. Typically, this localized area consists of backfill 3 ft above, 1 ft below, and a maximum of 5 ft on either side of nonsafety-related buried piping or similar conduits. Only a few% of the total power block backfill utilizes this compaction criteria. All such areas are located above the water table so that the factor of safety against liquefaction is not affected. Sand compacted to an average of 95% in the limited areas around piping and similar conduits will not affect the structural integrity of any Category 1 structures. Sand compacted to an average of 95% will have static and dynamic properties consistent with those properties assumed for design of power block structures and piping. A static cone penetrometer reading of 200 is used to decide on the adequacy of concrete sand or other sands with similar properties between and below nonsafety-related piping in areas where constrained access prevents the use of the sand cone test.

Trenches containing safety-related piping or similar conduits are backfilled by placing lean concrete to the bottom of the pipe to provide continuous support and backfilling with Category 1 backfill, using wooden tampers, hand-held power tampers, or hand-held vibratory compactors as required. Use of these methods produces an average compaction of at least 97% of the maximum dry density determined in accordance with ASTM D 1557, with no tests below 93% and not more than 10% of the tests between 93 and 95%. Category 1 backfill material compacted between and immediately around pipes has a fines content below 10%. Static cone penetrometer readings developed from correlation with sand cone tests are used to decide on the adequacy of the compaction in areas where constrained access prevents the use of the sand cone test.

Post construction (after 1989) compaction may also be measured by a nuclear density test which has been correlated and calibrated with sand cone test per ASTM D2922-81 and FSAR paragraph 2.5.4.5.2.7.1D.

Lean concrete is used to backfill localized areas where placement of backfill material is impractical.

Undocumented backfill may be installed in areas designated for Category 1, Noncategory 1, and Category 2 backfill, on a case by case basis, and approved only when it is confirmed that there will be no effects on soil liquefaction, settlement, and load carrying capacities. The locations of the installment of undocumented backfill in a Category 1 area will be recorded on engineering drawings AX2D46T025 or AX2D46T001 to provide for proper consideration in regard to future installation or construction of structures, railroads, pipelines, utilities, etc.

2.5.4.5.2.1 Sources and Quantities of Backfill Material. An estimate of the total quantity of Category 1 backfill required in the power block was made by first determining the volume of the excavated area. The quantity obtained was increased by 10% to allow for compaction. The volume of all the structures below the grade level was then deducted from the total volume to obtain quantity of Category 1 backfill required. The quantity of Category 1 backfill required beneath and around the power block structures was estimated to be about 4.2 million yd^3.

Seven borrow sources within the plant site were investigated. These sources were designated as borrow areas 1, 2, 3, 4, 5, 1-A, and 1-B. In addition, two stockpiles, namely stockpiles A and B, were also investigated. These stockpiles contained material that had been obtained from excavations in the power block area. The approximate limits of all borrow areas and stockpiles are shown on drawing AX6DD394.

Detailed field exploration and laboratory testing programs were performed to identify suitable material in the borrow areas and stockpiles. The results of these investigations are presented in references 2, 3, and 4. Based on an evaluation of the field and laboratory data, it was determined that materials selected for Category 1 backfill should be sand and silty sand with not more than 25 weight% passing the U.S. No. 200 sieve size. This criterion was used to judge the suitability of materials for backfill. It was concluded that borrow areas 1, 2, 3, 4, 5, 1-A, 1-B, and stockpile A were good sources of Category 1 backfill and that substantial quantities of sand and silty sand could be obtained by selective excavation. It was also concluded that stockpile B consisted of mainly clayey soils. From considerations of anticipated excessive structure settlement associated with clayey backfill and the difficulty of selective excavation to remove clayey soils, it was decided to exclude stockpile B as a suitable source for Category 1 backfill.

Estimated quantities of material suitable for Category 1 backfill were as follows:

Borrow Area	Estimated Quantity (yd^3)
1	1,503,000
2	251,000
3	1,007,000
4	1,197,000
5	190,000
1-A	1,700,000
1-B	1,700,000

In addition, the total quantity of material available in stockpile A was estimated to be approximately 600,000 yd^3. Thus, a total quantity of 8,148,000 yd^3 of Category 1 backfill was identified from the aforementioned sources, which was considered more than sufficient for backfill requirements in the power block.

An additional borrow area (3-A) was investigated through field exploration and laboratory testing during October and November 1992 to identify Category 1 backfill for future general use around the plant site. It was concluded that borrow area 3-A is a good source of Category 1 backfill, consisting of sand and silty sand meeting the above stated criteria, with the use of selective excavation.

The estimated quantity of Category 1 backfill, resulting from the 1992 investigation, which may be used for future plant requirements is as follows:

Borrow Area	Estimated Quantity (yd^3)
3-A	1,456,000

A portion of borrow area 3-A is a subset of old borrow area 3. This portion of borrow area 3 was never excavated. Future landfill trenches are planned due west of the existing landfill trench as shown on drawing AX6DD394. Therefore, it is likely that soil from borrow area 3-A will be excavated. Soil from borrow area 3-A which meets the criteria for Category 1 backfill (reference 28) may be stockpiled at the time of landfill excavation.

2.5.4.5.2.2 Exploration. Field exploration for borrow areas 1, 2, 3, 4, and 5 and stockpiles A and B was accomplished in early 1977. Subsurface exploration involved test pits excavated to a maximum depth of 25 ft by means of a backhoe. A total of 26, 8, 40, 12, and 3 test pits was excavated and logged in borrow areas 1, 2, 3, 4, and 5, respectively. Thirty-four test pits were excavated and logged in the two stockpiles. An appropriate number of jar and bulk samples were taken for laboratory testing.

Borrow area 1-A was investigated in the summer of 1978. Eighteen borings evenly spaced in a grid pattern covering the area were drilled and logged using a hollow stem auger. The borings extended to depths ranging from 13.5 to 66 ft below the existing grade and were terminated at depths below the water table ranging from 0 to 14 ft. Representative soil samples were obtained at 5-ft intervals and whenever a change in soil type occurred.

Investigations in borrow area 1-B were performed in June and July 1979. Sixty borings were drilled and logged during this investigation. Holes were advanced using both rotary drilling and auger drilling techniques. The depth of borings ranged from 40.5 to 81.5 ft below existing grade and were terminated upon reaching the water table. In most of the borings representative split-spoon soil samples were obtained at 5-ft intervals. In some borings sampling was done at 2 1/2-ft intervals. In addition, bulk samples were obtained.

A field exploration of borrow area 3-A was conducted during October 1992. Fourteen soil borings were drilled using a 4-1/4 inch hollow stem auger and continuously sampled with a 3-1/2 inch I.D., 5-foot long continuous sampler. Samples were logged during the drilling operations. Three bulk samples were also obtained. The boring depths ranged from 23.8 feet to a maximum depth of 45 feet. Most of the borings were terminated upon encountering clay deposits. No borings were drilled beyond a depth of 45 feet due to the impracticality of excavating backfill to greater depths.

Logs of all test pits and borings drilled in the borrow areas and stockpiles are contained in references 2, 3, 4, and 28. Locations of all test pits and borings are shown on drawing AX6DD394.

2.5.4.5.2.3 Laboratory Testing. In order to classify the soils in borrow areas 1, 2, 3, 4, 5, 1-A, and 1-B; and stockpiles A and B and obtain the static and dynamic engineering properties of compacted backfill, many laboratory tests were performed on samples obtained from the field explorations.

These tests are listed below:

- Laboratory classification of soils.
- Grain size distribution.
- Atterberg limits.
- Moisture content of soil.
- Specific gravity.
- Moisture-density relation.
- Relative density.
- Static consolidated drained triaxial compression.
- Static consolidated undrained triaxial compression.
- Consolidation.
- Stress-controlled consolidated undrained cyclic triaxial.
- Strain-controlled consolidated undrained cyclic triaxial compression.

- Resonant column.
- Triaxial tests to determine volume changes due to cyclic loading.
  - Cyclically loaded without permitting drainage.
  - Cyclically loaded while permitting drainage.

In order to determine the suitability of soils in borrow area 3-A for use as Category 1 backfill, the following laboratory tests were performed:

- Grain size analysis, including hydrometer analysis on selected samples.
- Atterberg limits.
- Moisture content.
- Moisture-density relation.

All tests were performed in accordance with applicable ASTM test methods or recognized procedures where no ASTM was available.

Details of the test results and test procedures are included in references 2, 3, 4, 5, and 28.

2.5.4.5.2.4 Criteria for Category 1 Backfill Suitability. Soil classification test data obtained in accordance with ASTM D 2487, D 2488, D 1140, D 422, D 423, and D 424 were used to identify materials suitable for use as Category 1 backfill in the borrow areas and stockpiles. Cross-sections were developed based on the classification test data to facilitate selective excavation of acceptable material in the borrow sources. Summaries of classification test data and cross-sections for each borrow source are contained in references 2, 3, 4, and 28.

Select sand and silty sand materials are used for Category 1 backfill. In selecting soils for use as Category 1 backfill, only those soils with 25 weight% or less passing the U.S. No. 200 sieve size were considered suitable. Periodic tests in accordance with ASTM D 422 and D 1140 were used to ensure that the above criterion was being satisfied during backfilling operations.

2.5.4.5.2.5 Design Static Properties. For sand and silty sand backfill compacted to an average of 97% of the maximum dry density determined by ASTM D 1557, the static properties shown in table 2.5.4-8 were used for design. These properties were based on data obtained from the tests referred to in paragraph 2.5.4.5.2.3.

2.5.4.5.2.6 Design Dynamic Properties. For compacted sand and silty sand, the dynamic properties shown in table 2.5.4-9 are used for design.

For computing the dynamic shear modulus at higher strain levels, the attenuation factors shown in figure 3.7.B.2-5 were used for design.

The variation of damping ratio with shear strain (figure 3.7.B.1-8) was also used for design.

2.5.4.5.2.7 Test Fill Studies.

2.5.4.5.2.7.1 Test Fill for Heavy Equipment Compaction. A test fill program was performed to evaluate the performance of three different pieces of compactors: Ingersoll-Rand SPF 60, Ingersoll-Rand SP 60, and Raygo 600A. The purpose of the program was to determine the appropriate lift thickness and number of passes required to achieve an average of 97% of the maximum density according to ASTM D 1557, with no tests below 93% and not more than 10% of tests between 95 and 93%. The material used for the test fill program consisted of sand and silty sand. Seven test fills were constructed.

The results of the test fill program are discussed in detail in reference 7.

A brief summary of the conclusions of the test fill program is given below:

A. The moisture content of the sand and silty sand material should be within $\pm$2% of the optimum moisture content determined by ASTM D 1557. Upon initiation of the backfill program, the moisture content range was $\pm$2%. This was later modified by program specifications to -3% to +2%. This modification was based on a review of the original test fill data and additional field tests performed during the backfill operation. Material too wet or too dry should not be compacted until brought within the required limits.

B. The Raygo 600A roller travelling at 1.5 mph can compact a 6-in. uncompacted lift, moisture conditioned within the range specified above, in four passes. Also, a combination of the Raygo 600A and Ingersoll-Rand SPF 60 rollers travelling at 1.5 mph can compact a 6-in. uncompacted lift with a total of four passes (two each). No test fills were made with the Ingersoll-Rand SPF 60 and SP 60 compactors for a 6-in. uncompacted lift thickness. These two rollers are capable of delivering a compaction effort comparable to or greater than the Raygo 600A compactor. Therefore the Ingersoll-Rand SPF 60 and SP 60 can be used for compacting a 6-in. uncompacted lift with four passes each separately.

C. The roller speed must be maintained at 1.5 mph.

D. The sand cone method (ASTM D 1556), modified to increase the minimum hole volume to 0.2 ft^3, provided consistent test results as compared to other test procedures. Therefore, it was used for all quality control testing during construction.

However, the nuclear density testing device may be used post construction (after 1989) in lieu of the sand cone method provided: 1) an acceptable laboratory calibration and field (soil specific) correlation between sand cone and nuclear test results can be achieved per the provisions of ASTM D2922-81, Calibration; 2) at least 10 test comparisons are made and the correlation is checked in an ongoing manner for every subsequent 10 nuclear density determinations; 3) the minimum 0.2 ft^3 sand cone test volume is employed; and 4) the correlation is established as per paragraphs 4.2.1, 4.2.2, and 4.2.3 of ASTM D2922-81 in order that minimal deviation is obtained.

Table 2.5.4-10 summarizes the results of the heavy equipment test fill program.

2.5.4.5.2.7.2 Test Fill for Hand Compaction Equipment. A test fill program was also performed to determine satisfactory compaction procedures involving three different types of hand compaction equipment. The test fill program was performed using the Wacker WS-74 Dual Drum, Wacker 100 (Jumping Jack), and Ingersoll-Rand SP 24 vibratory hand compactors. Based on the data obtained during the test fill operation, it was determined that the procedures outlined below will meet the Compaction requirements specified in paragraph 2.5.4.5.2.

Type of Equipment	Thickness of Lift (in.)	No. of Passes	Speed (ft/min)	Vibrations (per min)
Wacker WS-74 Dual Drum	6	4	60	3000
Wacker 100	6	2	20	630
Ingersoll-Rand SP 24	6	4	60	4000

Table 2.5.4-11 summarizes the results of the hand compaction equipment test fill program.

2.5.4.5.2.8 Nonsafety-Related Pipe Trench Backfill in Power Block Area. Trench backfill for nonsafety-related piping in Category 1 fill areas is compacted to an average of 95% relative compaction as defined in paragraph 2.5.4.5.2. The backfill material used is concrete sand with 2% or less fines. The sand is saturated and compacted by internal vibration using concrete vibrators.

A test fill program was implemented to determine whether the required degree of compaction could be achieved by the vibrated sand method. The resulting data demonstrate that the compaction above, between, and below the pipes meets the required compaction criteria. Results of the test fill program are summarized in reference 17.

2.5.4.5.2.9 Soil-Cement-Flyash Backfill. Plastic backfill consisting of cement, flyash, sand, and water is used as bedding material for Category 2 circulating water lines located in the Category 1 backfill zone north of the turbine building. Plastic backfill is being used in lieu of compacted sand and silty sand backfill because of the difficulty in obtaining the required compaction around the pipes.

Static and dynamic tests were performed on specimens consisting of different proportions of cement, flyash, sand, and water. The tests demonstrated that specimens of plastic backfill tested possess static and dynamic properties comparable to Category 1 backfill. The properties summarized below are of the plastic backfill that is used. The properties correspond to a plastic backfill mix of 65 lb of cement, 385 lb of flyash, 2586 lb of sand, and 469 lb of water per cubic yard of backfill.

Plastic unit weight	129.2 lb/ft^3
Slump	5 in.
Air content	3.5%
Unconfined compressive strength	
Average at 7 days	20.7 psi
Average at 28 days	30.5 psi
Average at 91 days	61.4 psi
Dry unit weight	
Average at 7 days	114.4 lb/ft^3
Average at 28 days	114.5 lb/ft^3
Average at 91 days	114.5 lb/ft^3
Moisture content	
Average at 7 days	14.0%

Average at 28 days	14.2%
Average at 91 days	14.6%
Cohesion	
Range at approximately 100 days	2100-5000 lb/ft^2
Angle of friction	
Range at approximately 100 days	36-48.5°
Range of shear modulus at approximately 100 days, under a confining pressure of 2 ksf for strain level of 10^{-4}%	4200-4400 lb/ft^2
Range of damping at approximately 100 days for strain level of 10^{-4}%	2.4-2.6%

2.5.4.6 Site Ground Water Conditions

The occurrence and movement of ground water beneath the site are described in detail in section 2.4.12. The first ground water body encountered beneath the VEGP site is a water table (unconfined) aquifer in the Barnwell sands and Utley limestone. It overlies the Blue Bluff marl. The site is on an interfluvial ridge that is nearly surrounded by streams that have cut down through the Barnwell sands and Utley limestone to the marl. This has isolated the water table aquifer beneath the site from adjacent areas. Ground water discharges from the water table aquifer to the surrounding streams. The streams discharge to the Savannah River.

Underlying the water table aquifer is the Blue Bluff marl, the upper member of the Lisbon Formation. The marl layer, approximately 70 ft thick, is a near-impermeable layer that effectively confines the underlying Tertiary and Cretaceous aquifers.

The Tertiary aquifer is represented beneath the site by the "unnamed sands" member of the Lisbon Formation. These sands are the local, minor equivalent of the regional Tertiary aquifer that is referred to as the principal artesian aquifer. The Cretaceous aquifer, the lowermost confined aquifer consists primarily of the sands and gravels of the Tuscaloosa Formation. It is often referred to as the Tuscaloosa aquifer. The Cretaceous aquifer and the lesser Tertiary aquifer are believed to be hydraulically connected beneath the plant site. Excavations for structures at the site do not extend through the marl; the marl remains as a hydrologic barrier beneath the site. Therefore, the confined aquifers will have no direct effect on structures.

Replenishment of the water table aquifer is by infiltration of precipitation, and after percolation to the water table, it moves laterally to the bordering interceptor streams. Contours of the water table for November 1971 and December 1984 are shown on drawings AX6DD329 and AX6DD330. The water table is, in general, subdued reflection of the ground surface, and movement is from the central portions of the interfluve toward the bordering interceptor streams.

Foundation design for the power-block facilities required excavation of the materials comprising the water table aquifer overlying the Blue Bluff marl. To construct and maintain the excavation the materials were dewatered by a series of ditches oriented in an east-west direction. They were connected by a north-south ditch, which drained to a sump in the southwest corner of the excavation. The sump was equipped with four pumps with a capacity of 500 gal/min each to remove inflows from ground water. Additional capacity was provided for the removal of inflows of storm water into the excavation. Dewatering for construction was terminated, in March 1983, and the water levels and flow pattern of the water table aquifer have returned near the preconstruction pattern. Dewatering is discussed in more detail in paragraph 2.4.12.1.3.3.1, and in Appendix 2B.

Upon completion of construction, recharge is expected to be less in the plant area than prior to construction because of the structures, pavements, and surface drainage systems. Future recharge conditions will thus be such that the water table is not expected to rise as high as under preconstruction conditions. Power block structures are designed to accommodate ground water levels of el 165 ft; hence, no permanent dewatering system is required.

At the VEGP site the piezometric surface of the Tertiary aquifer, determined from observation wells set in the unnamed sands below the confining (marl) layer, slopes to the northeast toward the Savannah River. The river has cut through the marl in the vicinity of VEGP, and it is in hydraulic contact with the underlying Tertiary aquifer. This allows the aquifer to discharge to the river in this area. This is a relatively local condition, as downstream of the VEGP site, the confining layer is intact below the river, and the direction of ground water movement in the confined aquifers is to the southeast, the regional direction of migration of the aquifer.

Permeabilities of the aquifers and the confining layer were measured by field and laboratory methods. Details of the tests and the results are described in paragraph 2.4.12.2.4.

Permeability of Barnwell sands and clayey sands (water table aquifer) was measured in situ at two exploratory holes at the plant site and in the laboratory on three undisturbed samples. The results ranged from 10 to 302 ft/year. One disturbed sample of Barnwell sands (considered for use as backfill) and two "grab" samples of backfill material were measured at different densities. The results ranged from 430 to 20,000 ft/year. Two test wells, each with an array of 4 observation wells, were used to conduct field tests in the Utley limestone, which is at the base of water table aquifer. Data from the tests indicated that the permeability of the Utley limestone varies considerably from place to place. Calculated permeabilities range from 96 to 125,400 ft/year. The results of permeability tests of water table aquifer materials are summarized in table 2.4.12-12 and 2.4.12-13.
In-situ permeability tests in the Blue Bluff marl (the confining layer) were conducted in 95 intervals at different depths in 28 exploratory holes. In 90% of the intervals tested, no measurable water inflow occurred. In only three holes was any inflow confirmed: two of these were in near-surface, weathered marl. The range of laboratory permeability measurements is from 5.2×10^{-3} ft/year to 8.8 ft/year. Results of permeability tests in the confining layer are summarized in table 2.4.12-10.

Large quantities of ground water are stored in the confined aquifers underlying the region of the VEGP site, and relatively small withdrawals have occurred to date. Although many small communities derive water from wells, the draft on the aquifers is low because of the low population density, limited industrial development, abundant surface waters, and abundant rainfall (agricultural crops of the area do not require significant quantities of applied water). Future use of ground water for industrial and domestic use is expected to increase to some degree, but withdrawals from the confined aquifers are estimated to be small. This assessment takes into account the planned requirements of the VEGP project, which will draw from the Cretaceous aquifer for makeup water (paragraph 2.4.12.1.3.).

A comprehensive ground water monitoring program has been implemented at the VEGP. This program has been designed to monitor piezometric levels in the water table aquifer, the confined aquifers (Tertiary and Cretaceous), and hydrostatic pore pressure in the confining layer (marl). The program consists of various wells monitoring the unconfined aquifer, the Tertiary aquifer, the Cretaceous aquifer, and the confining layer. The ground water monitoring program is discussed in detail in paragraph 2.4.12.2.3.1.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

This subject is addressed in subsections 3.7.1 and 3.7.2.

2.5.4.8 Liquefaction Potential

The liquefaction potential of the upper sand stratum was evaluated using the standard penetration test blow counts obtained during the investigation and the simplified procedure of Seed and Idriss.[8] This evaluation is described in detail in reference 1 and indicates that the upper sand below the ground water level is susceptible to liquefaction when subjected to the maximum SSE acceleration of 0.2 g. Based on this evaluation the upper sand stratum was removed to an approximate elevation of 130 to 135 ft in the power block area. Select sand and silty sand compacted to 97% of the maximum density determined by ASTM D 1557 is placed from the top of the marl stratum to the design elevation of the various power block structures with the exception of an area north of the turbine building as noted in paragraph 2.5.4.5.2. The liquefaction potential of compacted backfill in the power block area was evaluated for the PSAR and is discussed in detail in reference 1. The analysis indicated a factor of safety against liquefaction on the order of 1.9 to 2.0. The analysis was done utilizing cyclic strength data obtained from tests on specimens of compacted backfill.

During the investigations for borrow sources, additional dynamic data were obtained to supplement the cyclic strength data obtained previously and reported in reference 1. Cyclic triaxial tests were performed on compacted specimens of sands obtained from stockpile A and borrow area 1. The cyclic stress ratios versus the number of cycles to 2.5% total strain (initial liquefaction) are shown on drawings AX6DD395 and AX6DD396. The results show that the stress ratios for the cleaner sands are substantially lower than for silty sands. In the liquefaction analysis done previously (1) stress ratios for the cleaner sands were used to obtain the safety factor against liquefaction. Therefore, the cyclic stress ratios for the cleaner sands obtained during investigations for borrow material were compared with values obtained during the PSAR investigations. A comparison of the two test data is shown on drawings AX6DD397 and AX6DD398. The comparison indicates that the PSAR data represent a lower bound of test values. If the liquefaction analysis were performed using the upper bound values obtained during the borrow investigation, a factor of safety higher than 1.9 to 2.0 would have been obtained for the design SSE conditions.

From the discussion presented above, it is concluded that there exists an adequate factor of safety against liquefaction for backfill compacted to 97% of the maximum density obtained by ASTM D 1557.

2.5.4.9 Earthquake Design Basis

The design bases for the SSE and operating basis earthquake are addressed in paragraphs 2.5.2.6 and 2.5.2.7.

2.5.4.10 Static Stability

2.5.4.10.1 Bearing Capacity of Compacted Backfill and Marl Bearing Stratum Supporting Mat Foundations

The ultimate bearing capacity of the backfill is evaluated for the backfill consisting of sand and silty sand compacted to 97% of the maximum dry density (ASTM D 1557).

The ultimate bearing capacity of a soil is defined as the load at which shear failure will occur. A factor of safety of at least three is considered acceptable for the allowable bearing capacity for static loads. For dynamic loads, a minimum safety factor of two is required. The net ultimate bearing capacity of sand backfill supporting a rectangular foundation above the water table is given by the expression:(9)

$$q_{ult} = \gamma D n_q \left(1 + 0.2\frac{B}{L}\right) + 1/2\,\gamma B \left(1 - 0.3\frac{B}{L}\right) N\gamma - \gamma D$$

where:

q_{ult}	=	the net ultimate bearing capacity (k/ft^2).
γ	=	the total unit weight of the backfill (k/ft^3).
D	=	depth of embedment of the footing (ft).
B	=	width of the footing (ft).
L	=	length of the footing (ft).
N_q,$N\gamma$	=	dimensionless bearing capacity factors.

For a circular foundation the expression is:

$$q_{ult} = 1/2\,\gamma B N\gamma\,(0.6) + \gamma D\,(N_q - 1)$$

If the water table is located at the bottom of a foundation supported by cohesionless material, the values obtained from the above expressions are approximately halved.

For a rectangular foundation supported entirely on the marl bearing stratum, the net ultimate bearing capacity is given by the expression:(10)

$$q_{ult} = c N_c \left(1 + 0.2\frac{D}{B}\right)\left(1 + 0.2\frac{B}{L}\right)$$

c	=	undrained shear strength of the marl bearing stratum (k/ft^2).
N_c	=	dimensionless bearing capacity factor.

For a circular foundation:

$$q_{ult} = 1.2\, cN_c.$$

For sand and silty sand backfill compacted to 97% relative compaction (ASTM D 1557), strength parameters of c=0 and ϕ=34° derived from triaxial test data were used (paragraph 2.5.4.5.2). For the marl bearing stratum (Blue Bluff marl), strength parameters of c=10 k/ft^2 and ϕ=0 were used (paragraph 2.5.4.2.2).

A summary of power block structure loads and allowable bearing capacity is presented in table 2.5.4-12. The bearing capacity of compacted backfill was determined to be very high for the large structures under consideration. Consequently, the strength of the marl bearing stratum will govern the allowable bearing capacity of the plant structures. Since the net allowable bearing pressures in all cases far exceed the net static loads, bearing capacity of the supporting soils is not a problem. Settlement of structures will therefore govern the allowable bearing pressures.

2.5.4.10.2 Settlement of Power Block Structures on Mat Foundations

When a load of limited size is applied to a sand stratum, it will undergo shear deformation beneath the loaded area. The vertical component of this deformation is called the "initial" or elastic settlement which will occur immediately upon application of the load. Sand and silty sand drain relatively fast upon loading, and therefore, long-term volume changes with dissipation of pore water pressure do not occur in these soils.

Therefore, while estimating settlements in these soils, only elastic settlements based upon the Young's modulus of elasticity were considered.

When a load is applied to a column of saturated clay soil, the clay will deform and pore water pressures will be induced in it. Immediately after the application of the load, little, if any, pore water will be squeezed out and the clay will deform at constant volume. The vertical component of movement is called the initial or elastic settlement. In the course of time, pore water will be squeezed out of the clay and its volume will decrease. The vertical component of this volume decrease is known as "consolidation" settlement. Therefore, for estimating settlements in the marl bearing stratum, both elastic and consolidation settlements were taken into consideration.

Soil stresses and settlements were computed using the Settlement Problem Oriented Language (SEPOL) computer program developed at Massachusetts Institute of Technology.[11] The sand backfill, the marl bearing stratum, and the lower sand stratum were treated as layered systems and divided into layers of different thicknesses. The SEPOL program computes the stress and strain at the midpoint of each layer based on the theory of elasticity. The elastic settlement is computed by multiplying the calculated strain in the layer by the layer thickness. Consolidation settlement is obtained using the calculated vertical stress and the rebound portion of the laboratory consolidation test curve. The settlement of each layer is determined by taking the sum of elastic and consolidation settlement for the layer, and the total settlement is calculated as the sum of the settlements contributed by each layer.

The following soil parameters for sand and silty sand backfill, the marl bearing stratum, and the lower sand stratum are used in the calculation of elastic settlements:

Soil Parameter	Sand, Silty Sand Backfill	Marl Bearing Stratum	Lower Sand Stratum
Moist unit weight (lb/ft^3)	120	-	-
Saturated unit weight (lb/ft^3)	130	115	115
Submerged unit weight (lb/ft^3)	68	53	53
Poisson's ratio	0.4	0.5	0.4
Young's modulus (k/ft^2)	1500	10,000	See figure 2.5.4-3

For compacted backfill, the Young's modulus of 1500 k/ft^2 was obtained from static triaxial tests. For the marl bearing stratum, the value of Young's modulus of 10,000 k/ft^2 was derived from the heave data, in situ pressure meter, and seismic velocity test data. The values of Young's modulus for the lower sand stratum (figure 2.5.4-3) are based on shear wave velocity data and empirical correlations between shear wave (low-strain) Young's modulus and higher-strain static

Young's modulus. Using the available shear wave velocity data for the upper 140 ft of the lower sand stratum from table 2.5.4-5, a plot of low strain Young's modulus versus depth is obtained as shown on figure 2.5.4-3. Values of the low-strain Young's modulus below a depth of 140 ft in the lower sand stratum were obtained by extrapolation using the Seed-Idriss relationship: (18)

$$G_d = 1000K_2\,(\sigma_m^{'})^{1/2}$$

where

K_2 = a parameter that depends on the void ratio and the strain amplitude of motions.

$\sigma_m^{'}$ = mean effective stress in lb/ft^3

G_d = the low strain shear modulus in lb/ft^2.

The low strain Young's modulus (E_d) is obtained from the equation:

$$E_d = 2(1+\upsilon)G_d$$

Where υ is the Poisson's ratio of the lower sand stratum.

Based on studies made by Swiger(19) the static Young's modulus for the lower sand stratum may be obtained by taking it equal to about 1/3 the low strain value (E_d). Therefore, from figure 2.5.4-3, the static Young's moduli of the various layers in the lower sand stratum are as follows:

Depth Below Grade (ft)	E(k/ft^2)
160-260	10,800
260-460	13,500
460-760	17,500
760-1160	22,000

The lower sand stratum is approximately 1000-ft thick based on seismic investigations (paragraph 2.5.4.2.3). For the settlement analyses, the lower sand stratum was divided into four layers of thickness; 100, 200, 300, and 400 ft, respectively. Each layer was assigned an appropriate value of Young's modulus as shown above. Total elastic settlements were obtained by adding the elastic settlements obtained in the backfill, marl, and the various layers of the lower sand stratum. Elastic settlement of each layer was calculated by multiplying the strain (Σ_z) in the layer by the height of the layer. The strain (Σ_z) is given by the following expression: (9)

$$\Sigma_z = \frac{1}{E}\left\{\sigma - \upsilon\,(\sigma_x + \sigma_y)\right\}$$

where:

$\sigma_z^{'}\sigma_x^{'}\sigma_y$ = the principal stresses induced at center of the layer by the loads.

E = Young's modulus.

υ = Poisson's ratio.

The stresses σ_z, σ_x and σ_y were obtained at the midpoint of each layer from the SEPOL computer program. In calculating stresses and settlements, both the weight of the backfill and structure loadings were taken into consideration.

The consolidation settlement was computed using the following formula:[14, 15]

$$\rho_c = \frac{C_c h}{1 + e_o} \log \frac{(\sigma_\upsilon^{'} + \Delta\sigma_\upsilon)}{\sigma_\upsilon}$$

where:

ρ_C	=	consolidation settlement.
C_C	=	compression index.
h	=	layer thickness.
e_0	=	initial void ratio.
$\sigma_\upsilon^{'}$	=	in situ effective vertical stress at midpoint of the marl stratum.
$\Delta\sigma_\upsilon$	=	effective additional vertical stress at middepth of the marl stratum due to the surface load.

The thickness of the marl stratum was taken as 70 ft for purposes of computing consolidation settlement. Considering the highly preconsolidated nature of the marl stratum, the compression index (C_c) used in the above formula was taken equal to the rebound index (C_r) obtained from laboratory consolidated tests. [1] The value of $C_c/1 + e_0$ was determined for each consolidation test, and an average of 0.0046 was used for settlement calculations.

The results of the settlement analysis of the power block structures are presented in figure 2.5.4-1. The estimated total settlement at the center and corner of each structure are shown. Total settlements include both elastic and consolidation settlements of the marl stratum and elastic settlements of the sand backfill and the lower sand stratum. Due to the highly preconsolidated nature of the marl stratum, the consolidation settlements are small, compared to the elastic settlements. The total settlements do not include settlements in the marl and lower sand strata as a result of fill under foundations, since these settlements will occur prior to placement of building loads.

2.5.4.10.3 Caisson Foundation in Radwaste Solidification Building

2.5.4.10.3.1 Caisson Ultimate Downward Capacity. The ultimate downward capacity of a caisson supporting the radwaste solidification building is given by:

$$V_u = q_{nu} \quad A_b + f_S \quad A_S$$

where:

V_u	=	ultimate downward capacity of caisson (k).
q_{nu}	=	net ultimate bearing capacity of caisson (k/ft^2).

f_S = frictional resistance of caisson shaft (k/ft^2).

A_b = area of caisson base (ft^2).

A_S = peripheral area of caisson shaft (ft^2).

The net ultimate bearing capacity (q_{nu}) of a caisson foundation bearing on marl is given by:

q_{nu} = 9 S_u, where S_u = undrained shear strength of marl.

= 90 k/ft^2 for S_u = 10 k/ft^2.

For computing the frictional resistance along the caisson shaft embedded in marl, the adhesion between the marl and the concrete is taken as 1.5 k/ft^2. Shaft friction in the upper sand stratum is conservatively ignored.

The equation for the ultimate downward capacity of a caisson bearing on marl can therefore be rewritten as follows:

$$V_u = 90\ A_b + 1.5\ pH_m$$

where:

V_u = ultimate downward capacity of caisson (k).

A_b = area of caisson base (ft^2).

p = perimeter of caisson shaft (ft).

H_m = depth of caisson penetration in marl (ft)

The estimated total dead plus sustained live load transmitted to the radwaste building foundation is approximately 116,000 k. The foundation system consists of 54, 8-ft diameter caissons spaced 18 ft center to center. The required design capacity per caisson is approximately 2150 k. An 8-ft diameter caisson with 20 ft of penetration into the marl would develop an ultimate downward capacity of approximately 5280 k and is satisfactory for radwaste solidification building foundation support.

2.5.4.10.3.2 Caisson Settlement. A load caisson would undergo settlement as a result of three causes:

A. Elastic compression of the shaft of the caisson.

B. Elastic compression of the marl on which the caisson is supported.

C. Consolidation settlement of the marl due to dissipation of pore water pressure occurring over a period of time.

The total settlement that would result at the top of the caisson would be the sum of A, B, and C above.

The applied pressure at the top of an 8-ft diameter caisson is estimated to be approximately 40 k/ft^2. The total settlement of the caisson with an applied stress of 40 k/ft^2 at the top is not expected to exceed 1 in.

2.5.4.10.4 Foundation Heave

Prior to excavations, the soil conditions in the power block consisted of an upper sand stratum, followed by a 70-ft layer of the clay marl bearing stratum and a lower stratum of dense sand with clay to a 750-ft depth. All of the upper sand stratum was removed in the power block area. Mass excavations were carried out from the existing grade elevation of 210 ft to the top of the clay bearing stratum which is at an approximate elevation of 130 ft. The excavation commenced in May 1974 and continued through September 1974. Because of project suspension, no excavation was done from September 1974 to February 1977. Upon restart of the project in February 1977, further excavation was resumed and it was completed in August 1977. During this period of time the heave of the clay marl stratum resulting from the removal of the overburden was frequently observed and recorded by GPC. Heave values were measured at different locations within the power block area.

An average heave of approximately 1.25 in. was measured in the power block area. Field records indicate that practically the entire heave as a result of the excavations has probably occurred over the period of time of the excavations.

2.5.4.10.5 Lateral Earth Pressures

The lateral earth pressure on subterranean walls of the power block structures was computed for sand and silty sand backfill having the properties discussed in paragraph 2.5.4.5. The coefficient of earth pressure "at rest" is used and a value of 0.7 (appropriate for 97% relative compaction) are considered in the computations. Additionally, the walls are designed for surcharge loadings and dynamic soil pressures where appropriate.

2.5.4.10.6 Hydrostatic Ground Water Pressures

The maximum predicted ground water elevation is 35 ft above the top of the clay marl bearing stratum. The effect of hydrostatic ground water pressures was considered in evaluating bearing capacity and settlement of soils supporting the power block structures, as discussed in paragraph 2.5.4.10.1.

Based on information obtained from piezometers extending into the lower sand stratum, the piezometric surface of the water contained in the lower sand stratum is approximately 20 to 30 ft below the top of the clay marl bearing stratum. This pressure has no effect on safety-related structures supported on compacted backfill and the marl stratum.

2.5.4.11 <u>Design Criteria</u>

As discussed in paragraph 2.5.4.2, the upper sand stratum would have a potential for liquefaction from an occurrence of the SSE event at the site. Further, the shelly limestone layer is characterized by voids and discontinuities and is unsuitable for foundation support from the standpoint of bearing capacity and settlement. Therefore, all Category 1 structures are supported on either the clay marl bearing stratum or on sand and silty sand backfill compacted to an average of 97% of the maximum density determined by ASTM D 1557. The allowable bearing pressure under both static and dynamic conditions satisfies these requirements:

A. A minimum factor of safety of three against shear failure under sustained dead load plus live load.

B. A minimum factor of safety of two against shear failure under sustained dead load plus maximum live load.

C. Structure settlements within tolerable limits for sustained dead load plus live load.

Evaluation of the liquefaction potential of Category 1 backfill is based on a minimum factor of safety of 1.5 against liquefaction. Computed factors of safety against liquefaction and bearing capacity failure are identified in paragraphs 2.5.4.8 and 2.5.4.10, respectively. Methods of analyses, including assumptions made in the analyses, are also discussed in the referenced paragraphs.

2.5.4.12 Techniques to Improve Subsurface Conditions

2.5.4.12.1 Foundations in the Clay Marl Stratum

No special treatment was required to improve foundation conditions beneath Category 1 structures supported on the marl stratum.

2.5.4.12.2 Foundations in Soil

Category 1 foundations in soil are supported on sand and silty sand backfill compacted to an average of at least 97% of the maximum determined by ASTM D 1557. This subject has been addressed in paragraph 2.5.4.5.

2.5.4.13 Subsurface Instrumentation

2.5.4.13.1 Heave Instrumentation

The heave of the clay marl bearing stratum was monitored during the period of excavations in the power block area from 1974 to 1977.

The general procedure for measuring heave is outlined below:

A. Two permanent reference benchmarks were established far outside the excavation and away from all related construction activity.

B. Nine heave point tips with polyvinyl chloride protective sleeves were installed at selected locations approximately 5 ft below the eventual bottom of the excavation.

C. Invar steel reading rods were lowered through each protective sleeve to mate with the heave point tip. The rods were tensioned to alleviate any "snaking" and to rigidly clamp them into position.

D. The heave point tip elevation was determined by conducting a first order level survey from a benchmark to the top of the reading rods.

The locations of the heave points are shown in figure 2.5.4-2. Three of the heave points were damaged after installation; therefore, data from only six heave points are available. A report of heave point measurements is presented in reference 16 and summarized in table 2.5.4-13. The data show that the measured heave of the marl stratum ranged from 0.6 to 1.7 in., with an average of 1.25 in. Results are also plotted on drawings AX6DD399 and AX6DD400.

2.5.4.13.2 Settlement Monitoring

2.5.4.13.2.1 Program Description. The foundation design parameters for all power-block structures were based on measured soil parameters obtained by field exploration and laboratory testing. The structures and the interconnecting piping are designed for building settlement. A settlement monitoring program was initiated to record settlements at various locations in the structures.

This monitoring program consists of two permanent benchmarks installed as reference points for measurements and monitoring points (i.e., settlement markers) as shown on drawing AX2D55V001. The settlement monitoring program is a separate engineering program implemented by plant procedures. Changes to this program will be made through the engineering design process.

2.5.4.13.2.2 Marker Reading Frequency. Markers were originally read at approximately 60-day intervals. Subsequent to October 1987, the 58 markers which have been demonstrated to be essentially stable have been read at maximum 6-month intervals and readings on two redundant markers (423-1 and 423-1B) have been discontinued (references 21, 22, and 24). Subsequent to September 1991, an additional 63 markers, which have been demonstrated to be essentially stable, have been read at maximum 6-month intervals, and readings on five markers associated with the radwaste solidification building and radwaste transfer tunnel (155, 158, 159-R, 160-R, and 161-R) have been discontinued. Subsequent to June 1, 1994, all markers except four (249, 252, 260, and 292) have been demonstrated to be essentially stable and were read annually. Subsequent to July 1, 1995, all markers have been demonstrated to be essentially stable and were read annually. Subsequent to July 1, 2002, the following markers have been demonstrated to be essentially stable and were read approximately every 24 months: 5, 14, 20, 26, 32, 40, 48, 54, 63, 72, 81, 87, 100-R, 101-R, 102-R, 103-R, 104-R, 105-R, 106, 119, 120, 121-R, 122, 123-R, 126, 129-R, 130-R, 131-R, 133-R, 135, 138, 139, 143, 146-R, 147, 149-R, 150-R, 157, 162, 163, 164-R, 165-R, 166-R, 167-R, 168-R, 169-R, 170, 171, 172-R, 173, 174, 175, 179, 184, 191, 197, 198, 199-R, 200-R, 201-R, 202-R, 203-R, 204-R, 205-R, 206, 219, 221, 222-R, 224, 225-R, 228, 229, 230, 234, 236, 237-R, 238-R, 239, 242, 247-R, 248-R, 250-R, 251-R, 254-R, 261, 262, 263-R, 264-R, 265-R, 266-R, 267-R, 268, 269, 270-R, 271, 272, 273, 276, 279, 280, 281, 282, 283-R, 284-R, 286, 287, 288, 290, 291, 293, 294, 295-R, 298, 420-2, 423-1A, 426, 427, 503, 506, 509, 512, 603, 606, 609, 612, 1000-R, 1002, 1003-R, 1004-R, 1005, 1006, 2000-R, 2001, and 2002. Subsequent to July 1, 2004, markers that are not required for differential settlement, Category 1 buried pipe, base mat, and building tilt evaluations and have been demonstrated to be essentially stable will be inactivated and no longer read. Most of the remaining active markers have been demonstrated to be essentially stable and will be read approximately every 5 years. However, some markers have indicated a higher percentage of allowable differential settlement or increasing differential settlement and these markers will be read annually. Active markers read annually or approximately every 5 years are identified in the plant procedures. A review of settlement monitoring data was performed in 2020 under RER SNC1117235 and determined that settlement is essentially stable and regular surveys are no longer required except under certain conditions. Either of the two following conditions will result in the immediate monitoring of the settlement markers.

Settlement monitoring will be performed immediately after any earthquake event equal to or exceeding a free-field acceleration of 1/2 OBE (0.06g). A settlement survey will also be performed if the groundwater level in the power block area drops more than 10 ft. below the reference groundwater level of 160 msl in more than one observation well monitoring the backfill.

2.5.4.13.2.3 Long Term Monitoring Commitments. Settlement monitoring for both units will be continued for important markers, as described in paragraph 2.5.4.13.2.4, through the first year following issuance of the operating license for Unit 2. At the end of this period, a brief technical report was provided to the NRC with supporting settlement data and graphical plots, and an evaluation of the settlement effects that justified a reduction in the frequency and number of markers monitored.

2.5.4.13.2.4 Marker Relocations. Throughout the life of the plant, important settlement markers that are destroyed or become inaccessible will be replaced with markers as near as possible to their original locations so that continuous readings can be provided and the total settlement at the original locations can be determined. Important settlement markers are defined as those markers located in safety-related structures whose total settlements have exceeded 1 in. and/or which are needed to continue the determination of differential settlements across a structure, or to establish differential settlements at piping penetrations.

In addition to markers which become inaccessible or are accidentally destroyed, a maximum of 96 markers are being moved from their original locations to new locations which provide more convenient access (references 21 and 22). The original markers will remain in place and will be available should any future correlations be required. Markers relocated to the outside of structures or to higher elevations for more convenient access, will remain as close as practicable to their original plan view locations (reference 23).

2.5.4.13.2.5 Total Structure Settlement. Actual total structure settlements have been compared with the predicted totals in FSAR figure 2.5.4-1 and were provided in a report to the NRC staff 3 months prior to fuel load of Unit 1 (reference 20). If total predicted settlements are exceeded, the settlement analysis will be reevaluated.

For additional discussion, see reference 25.

2.5.4.13.2.6 Differential Settlement Effect on Piping. Actual differential settlements between structures will be compared to those used in design. Piping has been installed as late in the construction schedule as practicable, and supports on either side of building interfaces typically have not been installed until structure construction is essentially complete. This has resulted in most structure settlements having taken place prior to permanent installation of piping. Subsequent to permanent installation of piping, differential settlement is monitored at building interfaces.

If future evaluations of the actual differential settlement indicate that 75% of the amount used in the design of piping has been reached, the situation will be reviewed. If it appears that the design differential settlement may be exceeded, the piping will be reanalyzed for an increased differential settlement and/or the supports will be adjusted or modified to satisfy the design requirements.

For additional discussion, see references 26 and 27.

2.5.4.13.2.7 Differential Settlement within Structures. The effect of differential settlements within structures on the structures themselves is addressed by reviewing the maximum net slope of the deflection curve (δ/ℓ) relative to structure tilt. This review is performed for large power block structures, auxiliary building, control building, and fuel handling building. (Refer to reference 26.) Unless differential settlement within structures approach a value of δ/ℓ equal to 1/670, it is deemed unnecessary to evaluate such effects on the design of structures. Even at this net slope, no adverse effects are likely.

An update of differential settlements was provided in a report to NRC staff 3 months prior to fuel load of Unit 1 (reference 20). The report included an assessment of the net slope of the deflection curve (δ/ℓ) relative to structural tilt. The net slope was demonstrated to be well within the 1/670 limit. The review was performed for the control building, the auxiliary building and the fuel handling building. These large power block structures contain the number of markers required and the accessibility needed to perform such a review. The report, however, included the total and differential settlement for all other safety-related structures to demonstrate that the settlements which occurred were reasonable and within predicted limits.

The slope of the deflection curve relative to structure tilt for the above structures will be reviewed when settlement data warrants evaluation.

For additional discussion, see reference 26.

2.5.4.14 Construction Notes

There have been no significant construction problems, apart from the erosion of Category 1 backfill, that occurred as a result of heavy rainfall in early November 1979. Areas within the power block subjected to erosion are described in detail in a report submitted to the Nuclear Regulatory Commission.[14] The report outlined steps that had been initiated subsequent to the erosion to repair the affected and adjacent areas and to facilitate resumption of backfilling operations in the power block area. Also included in the report were recommended methods of repair and a description of future erosion and ground water control measures to prevent a recurrence of the problem.

All erosion in the power block backfill was satisfactorily repaired according to recommended procedures, with the exception of minor deviations that were necessitated by practical considerations.

Extensive field and laboratory tests were performed to verify the extent of disturbed material in the eroded areas. These tests were used to verify the competency of the backfill adjacent to the foundations of various Category 1 structures. The evaluation of the effect of erosion on Category 1 structure foundations was based on data developed during testing and visual observations made during the entire period of repair. The data and evaluation are contained in reference 15. The field testing and evaluations described in reference 15 provided adequate data which defined the disturbed zones in Category 1 backfill. All erosion was successfully repaired. This evaluation has established that there is no detrimental effect on the existing structures as a result of the heavy rainfall of early November 1979.

2.5.4.15 Standard Review Plan Evaluation

The Standard Review Plan calls for probabilistic as well as deterministic analyses of liquefaction potential at the site.

The liquefaction analyses performed for VEGP were of the deterministic type only.

The foundation properties for materials underlying Seismic Category 1 structures are known with much greater accuracy at VEGP than at most nuclear power plant sites. This is because all potentially liquefiable foundation materials have been removed and replaced with homogeneous, well-compacted structural backfill. All Seismic Category 1 structures are founded either on this backfill or on the underlying very competent marl.

The deterministic evaluation of the liquefaction potential described in subsection 2.5.4 involved use of extensive laboratory test data that covered the upper and lower bound cyclic shear strengths of compacted Category 1 backfill. The deterministic analyses have demonstrated (paragraph 2.5.4.8) that an adequate factor of safety exists against liquefaction.

The backfill supporting Category 1 structure foundations has been placed under extremely well-controlled conditions and exceeds the minimum design compaction requirements (97% of the maximum density determined by ASTM D 1557). The auxiliary building and nuclear service cooling water towers are supported on the marl stratum and surrounded by compacted backfill. The remaining Category 1 structures, including containment, control, fuel, and diesel generating buildings, are supported on compacted sand and silty sand backfill. The inplace density of the sand backfill is so high that, when sheared, the backfill increases in volume and relieves all excess pore water pressures. Liquefaction will therefore not occur in compacted backfill under SSE conditions. The properties of the marl stratum have been well established based on extensive field and laboratory test data and show that it will adequately support Category 1 structures and that it is nonliquefiable. The backfill surrounding Category 1 structures is of sufficient width and length (paragraph 2.5.4.5) such that in the event of an SSE, none of the Category 1 structures will be affected by the potential liquefaction of the in situ upper sand stratum in the areas outside the backfill.

Because the foundation conditions are so well controlled and defined as described above and because liquefaction is shown to be clearly not possible under the design SSE, a probabilistic evaluation of liquefaction potential is not considered necessary for the VEGP site.

2.5.4.16 References

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17. Letter from D. O. Foster (Georgia Power Company) to E. G. Adensam (Nuclear Regulatory Commission), dated October 27, 1983.

18. Seed, H. B. and Idriss, I. M., "Soil Moduli and Damping Factors for Dynamic Response Analyses," EERC, University of California, Berkely, California, December 1970.

19. Swiger, F., "Evaluation of Soil Moduli for Soil Structure Interaction Analysis," presented at the Conference on Analysis and Design in Geotechnical Engineering, Austin, Texas, 1974.

20. Bechtel Power Corporation, VEGP Report on Settlement, August 1986.

21. Letter GN-1368, dated May 22, 1987.

22. Letter SL-3002, dated August 3, 1987.

23. Letter GN-1398, dated September 22, 1987.

24. Letter from Nuclear Regulatory Commission to Georgia Power Company, dated October 20, 1987.

25. Letter GN-350, dated May 9, 1984.

26. Letter GN-350, dated May 9, 1984.

27. Letter GN-350, dated May 9, 1984.

28. Category 1 Backfill Investigation, dated November 30, 1992 (Response to REA-VG-0672).

2.5.5 STABILITY OF SLOPES

2.5.5.1 Slope Characteristics

Category 1 slopes consisted of excavation cut slopes and temporary backfill slopes. The excavation slopes were cut in the upper sand stratum and shell zone at two horizontal to one vertical. The lower 5 ft of the cut was in the clay bearing stratum. Parameters for design of the excavation slopes were based on data developed for the upper sand and clay bearing strata (paragraph 2.5.4.2). A total stress design shear strength of c=0, ϕ=34° was used for the upper sand stratum and c=10,000 lb/ft^2, ϕ=0° for the clay bearing stratum (table 2.5.4-2).

Temporary fill slopes were constructed at a minimum of 1.5 horizontal to one vertical where the slope height exceeded 3 ft except for a few deviations which are addressed in references 1 and 2. Slopes or portions of slopes of heights less than 3 ft were placed with stable side slopes. Except for north of the turbine building, fill slopes consisted of sand and silty sand material compacted to an average of 97% of the maximum density by American Society of Testing Materials (ASTM) D 1557. Fill slopes north of the turbine building consisted of sand and silty sand backfill compacted to an average of 95% of the maximum density by ASTM D 1557. Parameters for design of temporary fill slopes were based on data developed for compacted Category 1 backfill (paragraph 2.5.4.5). Design effective stress parameters of C'=0, ϕ=34° were used in analyzing temporary fill slopes.

2.5.5.2 Design Criteria and Analysis

The stability of the excavation cut slopes in in situ soil was determined using a computer program based on a modification of the Swedish Slip Circle method of slices analysis.(3) The slopes were analyzed for stability by assuming the material below the water table to be dewatered. A peripheral dewatering system is being used to control ground water and will be continued until backfilling is completed above the ground water table. In a dewatered condition, the factor of safety against sliding for a slope of two horizontal to one vertical was determined to be 1.3. This was considered satisfactory for a temporary construction slope. Earthquake forces were not considered in the design of these slopes since they are temporary during the construction period only.

For temporary fill slopes (1.5 horizontal to one vertical), slope stability analysis was performed using the Integrated Civil Engineering Systems LEASE computer program.(4)

The analysis revealed that a deep seated sliding failure will not occur, and any instability in the fill will be manifested in the form of minor raveling of the fill surface if it is steeper than the effective angle of skin friction. Infinite slope analysis based on the design friction angle of 34° indicated that temporary fill slopes will have a minimum factor of safety against raveling of 1.01. This was considered satisfactory for temporary fill slopes in a dewatered condition.

Surcharge loadings, such as buildings, on the top of a slope will affect the slope stability. To prevent loss of bearing capacity for the structure foundation and to ensure slope stability, buildings were located a sufficient distance away from the top of the slope. When situations arose during construction that required a building to be placed near a temporary fill slope, each case was analyzed to determine the minimum setback distance.

2.5.5.3 Log of Borings

Log of borings is listed in references 5, 6, and 7.

2.5.5.4 Compacted Backfill

This subject is discussed in paragraph 2.5.4.5.2.

2.5.5.5 References

1. Letter, with attachments, from D. E. Dutton of GPC to J. P. O'Reilly of the NRC, dated January 8, 1980.

2. Bechtel Power Corporation, Final Report on Dewatering and Repair of Erosion in Category 1 Backfill in Power Block Area, August 1980.

3. U.S. Corps of Engineers, "The Method of Slices," Civil Works Engineering Manual, SM 1110-2-1902.

4. Berkley, W. A., and Christian, J. T., "ICES LEASE-1: A Problem Oriented Language for Slope Stability Analysis," User's Manual, Soil Mechanics Publication No. 235, Massachusetts Institute of Technology, April 1969.

5. Bechtel Power Corporation, Report on Backfill Material Investigations, Vogtle Electric Generating Plant, January 1978.

6. Bechtel Power Corporation, Report on Backfill Material Investigations, Addendum No. 1, Vogtle Electric Generating Plant, October 1978.

7. Bechtel Power Corporation, Report on Backfill Material Investigations, Addendum No. 2, Vogtle Electric Generating Plant, November 1979.

2.5.6 EMBANKMENTS AND DAMS

There are no earth, rock, or earth and rock fill embankments required for plant flood protection or for impounding cooling water required for the operation of the plant.

TABLE 2.5.1-1

STRATIGRAPHIC UNITS IN THE VICINITY OF VEGP

System		Series	Formation	Description
Quaternary	Recent to Pleistocene		Alluvium	Alluvial fill and terrace deposits in stream valleys, consisting of tan to gray sand, clay, slit, and gravel.
Tertiary	Miocene		Hawthorne Formation	Tan, red, and purple sandy clay, interbedded lenses of gravel, and numerous clastic dikes.
Tertiary	Eocene	Jackson Age	Barnwell Group	Red, brown, yellow, and buff, fine to course, massive to crossbedded sand and sandy clay.
		Claiborne Age	Lisbon Formation	Yellow-brown to green, fine to coarse, glauconitic quartz sand, interbedded with green, red, yellow, and tan clay, sandy marl or limestone, and lenses of siliceous limestone.
Tertiary	Paleocene		Huber/Ellenton Formation	Dark-gray to black, lignitic, micaceous clay containing disseminated crystals of gypsum. Medium- to dark-gray coarse sand and white kaolin.
Cretaceous	Upper		Tuscaloosa Formation	Tan, buff, red, and white cross- bedded micaceous quartzite and arkosic sand and gravel, interbedded with red, brown, and purple clay and white kaolin.
Triassic	Upper		Newark Group	Gray, dark-brown, and brick-red sandstone, siltstone, gray-wacke, and claystone with included sections of fanglomerate or conglomerate.
Paleozoic and Precambrian			Basement Rock of the Carolina Slate Belt and Charlotte Belt	Granite, gneiss, chlorite- hornblende, and chlorite- tremolite schist, slate, and volcanic rocks.

TABLE 2.5.2-1 (SHEET 1 OF 9)

SIGNIFICANT EARTHQUAKES WITHIN 200 MILES OF THE SITE

(Intensity ≥ 4 or Magnitude ≥ 3)

DATE GMT YR	MO	DY	HR	TIME MN	SEC	LOCATION	DEG N	DEPTH DEG W	INTEN. MI	MAG. MMI	SITE MI	DATA DIST. SOURCE
1776	11	5				35.3	83.2	-	IV	-	170	BOL
1799	4	4				32.9	80.0	-	V	-	105	STR
1799	4	11	8	20		32.9	80.0	-	V	-	105	STR
1799	4	11	14	55		34.3	80.6	-	V	-	105	BOL
1817	1	8	4			32.8	79.8	-	V	-	115	BOL
1820	9	3	8	30		33.4	79.3	-	IV	-	145	STR
1851	8	11	1	55		35.6	82.6	-	V	-	175	STR
1853	5	20				34.0	81.2	-	VI	-	70	STR
1857	12	19	9	4		32.8	79.8	-	V	-	115	BOL
1860	1	16	18			32.8	79.8	-	V	-	115	BOL
1869	0	0				32.9	80.0	-	IV	-	105	STR
1872	6	17	20			33.1	83.3	-	V	-	90	EQH
1874	2	10				35.7	82.1	-	VI	-	175	BOL
1874	2	22				35.7	82.1	-	IV	-	175	EUS
1874	3	17				35.7	82.1	-	IV	-	175	EUS
1874	3	26				35.7	82.1	-	IV	-	175	EUS
1874	4	14				35.7	82.1	-	IV	-	175	EUS
1874	4	17				35.7	82.1	-	IV	-	175	EUS
1875	11	2	2	55		33.8	82.5	-	VI	-	60	EUS
1876	12	12				32.9	80.0	-	IV	-	105	STR
1879	12	13				35.0	80.9	-	V	-	140	EUS
1885	10	17	17	30		33.0	83.0	-	IV	-	70	BOL
1886	8	27	8	30		33.0	80.2	-	V	-	90	BOL
1886	8	28	8	45		32.9	80.0	-	VI	-	105	STR

TABLE 2.5.2-1 (SHEET 2 OF 9)

DATE				TIME		LOCATION	DEPTH	INTEN.	MAG.	SITE	DATA	
	GMT					DEG N	DEG W	MI	MMI		DIST.	SOURCE
YR	MO	DY	HR	MN	SEC					MI		
1886	8	28	9	40	32.9	80.0	-	IV	-	105	STR	
1886	8	28	18	20	32.9	80.0	-	IV	-	105	STR	
1886	9	1	2	51	32.9	80.0	-	X	-	105	EQH	
1886	9	1	6	5	32.9	80.0	-	VI	-	105	STR	
1886	9	2	4	55	32.9	80.0	-	V	-	105	STR	
1886	9	3	21		30.4	81.7	-	IV	-	190	STR	
1886	9	4	4	1	32.9	80.0	-	VI	-	105	STR	
1886	9	4	9		30.4	81.7	-	IV	-	190	STR	
1886	9	5			30.4	81.7	-	IV	-	190	STR	
1886	9	6	4	6	32.9	80.0	-	VI	-	105	STR	
1886	9	6	16	35	32.9	80.0	-	IV	-	105	STR	
1886	9	8			30.4	81.7	-	IV	-	105	STR	
1886	9	9	18	47	30.4	81.7	-	IV	-	190	STR	
1886	9	17	6	29	32.9	80.0	-	VI	-	105	STR	
1886	9	21	10	15	32.9	80.0	-	VI	-	105	STR	
1886	9	21	10	30	32.9	80.0	-	V	-	190	STR	
1886	9	27	19	2	32.9	80.0	-	VI	-	105	STR	
1886	9	27	22	2	32.9	80.0	-	V	-	105	STR	
1886	10	9	3	40	32.9	80.0	-	IV	-	105	STR	
1886	10	9	5	40	32.9	80.0	-	IV	-	105	STR	
1886	10	9	6	48	32.9	80.0	-	VI	-	105	STR	
1886	10	22	10	20	32.9	80.0	-	VI	-	105	EQH	
1886	10	22	19	45	32.9	80.0	-	VII	-	105	EQH	
1886	10	23	1	7	32.9	80.0	-	IV	-	105	STR	
1886	11	5	17	20	32.9	80.0	-	VI	-	105	EQH	
1886	11	28	20	13	32.9	80.0	-	IV	-	105	STR	
1887	1	4	11	44	32.9	80.0	-	VI	-	105	STR	
1887	3	4	7		32.9	80.0	-	IV	-	105	STR	

TABLE 2.5.2-1 (SHEET 3 OF 9)

DATE GMT YR	MO	DY	TIME HR	MN	SEC	LOCATION DEG N	DEG W	DEPTH	INTEN. MI	MAG. MMI	SITE DIST. MI	DATA SOURCE
1887	3	17	14	9		32.9	80.0	-	V	-	105	STR
1887	3	18	23	10		32.9	80.0	-	IV	-	105	STR
1887	3	19				32.9	80.0	-	IV	-	105	STR
1887	3	24				32.9	80.0	-	IV	-	105	STR
1887	3	24	4	5		32.9	80.0	-	IV	-	105	STR
1887	3	28				32.9	80.0	-	IV	-	105	STR
1887	4	7	4			32.9	80.0	-	IV	-	105	STR
1887	4	8	9			32.9	80.0	-	IV	-	105	STR
1887	4	10	11	30		32.9	80.0	-	IV	-	105	STR
1887	4	14	7	25		32.9	80.0	-	IV	-	105	STR
1887	4	26	10			32.9	80.0	-	IV	-	105	STR
1887	4	28	8			32.9	80.0	-	V	-	105	STR
1887	5	6				32.9	80.0	-	IV	-	105	STR
1887	6	3	12			32.9	80.0	-	IV	-	105	STR
1887	7	10	18			32.9	80.0	-	IV	-	105	STR
1887	8	27	4	30		32.9	80.0	-	V	-	105	STR
1887	8	27	9	20		32.9	80.0	-	IV	-	105	STR
1888	1	12	15	54		32.9	80.0	-	VI	-	105	STR
1888	1	16	17	52		32.9	80.0	-	IV	-	105	STR
1888	2	29	11			32.9	80.0	-	V	-	105	STR
1888	3	3				32.9	80.0	-	IV	-	105	STR
1888	3	3	4	30		32.9	80.0	-	IV	-	105	STR
1888	3	4				32.9	80.0	-	IV	-	105	STR
1888	3	14	5			32.9	80.0	-	V	-	105	STR
1888	3	20	5			32.9	80.0	-	IV	-	105	STR
1888	3	25				32.9	80.0	-	IV	-	105	STR
1888	4	16				32.9	80.0	-	IV	-	105	STR
1888	4	16				32.9	80.0	-	IV	-	105	STR

TABLE 2.5.2-1 (SHEET 4 OF 9)

DATE GMT YR	MO	DY	HR	TIME MN	SEC	LOCATION	DEG N	DEPTH DEG W	INTEN. MI	MAG. MMI	SITE MI	DATA DIST.	SOURCE
1888	4	16				32.9	80.0	-	IV	-	105	STR	
1888	5	2				32.9	80.0	-	IV	-	105	STR	
1889	2	10		31		32.9	80.0	-	V	-	105	STR	
1889	7	12	2	54		32.9	80.0	-	IV	-	105	STR	
1891	10	13	5	55		32.9	80.0	-	IV	-	105	STR	
1893	6	21	4	5		30.4	81.7	-	IV	-	190	STR	
1893	6	21	7	7		30.4	81.7	-	IV	-	105	STR	
1893	7	5	8	10		32.9	80.0	-	IV	-	105	STR	
1893	7	6	9	5		32.9	80.0	-	IV	-	105	STR	
1893	7	8	7	48		32.9	80.0	-	IV	-	105	STR	
1893	7	8	15	25		32.9	80.0	-	IV	-	105	STR	
1893	9	19	7	5		32.9	80.0	-	IV	-	105	STR	
1893	9	19	7	40		32.9	80.0	-	IV	-	105	STR	
1893	9	19	8	55		32.9	80.0	-	IV	-	105	STR	
1893	11	8	4	40		32.9	80.0	-	IV	-	105	STR	
1893	11	8	6	5		32.9	80.0	-	IV	-	105	STR	
1893	12	27	6	51		32.9	80.0	-	IV	-	105	STR	
1893	12	27	7	17		32.9	80.0	-	IV	-	105	STR	
1893	12	27	9	9		32.9	80.0	-	IV	-	105	STR	
1893	12	27	9	56		32.9	80.0	-	IV	-	105	STR	
1893	12	28	2	20		32.9	80.0	-	IV	-	105	STR	
1894	1	10	8	5		32.9	80.0	-	IV	-	105	STR	
1894	1	10	8	49		32.9	80.0	-	IV	-	105	STR	
1894	1	10	9	15		32.9	80.0	-	IV	-	105	STR	
1894	1	30	4	5		32.9	80.0	-	IV	-	105	STR	
1894	2	1	5	21		32.9	80.0	-	IV	-	105	STR	
1894	6	16	2	16		32.9	80.0	-	IV	-	105	STR	
1894	12	11	5	27		32.9	80.0	-	IV	-	105	STR	

TABLE 2.5.2-1 (SHEET 5 OF 9)

DATE GMT YR	MO	DY	TIME HR	MN	SEC	LOCATION DEG N	DEG W	DEPTH	INTEN. MI	MAG. MMI	SITE DIST. MI	DATA SOURCE
1895	1	8	5	40		32.9	80.0	-	IV	-	105	STR
1895	1	8	5	58		32.9	80.0	-	IV	-	105	STR
1895	1	8	7	29		32.9	80.0	-	IV	-	105	STR
1895	4	27	7	40		32.9	80.0	-	IV	-	105	STR
1895	7	25	4	1		32.9	80.0	-	IV	-	105	STR
1895	10	6	6	25		32.9	80.0	-	IV	-	105	STR
1895	10	20	17	8		32.9	80.0	-	IV	-	105	STR
1895	11	12	23	33		32.9	80.0	-	IV	-	105	STR
1896	3	19	8	22		32.9	80.0	-	IV	-	105	STR
1896	8	11	5	58		32.9	80.0	-	IV	-	105	STR
1896	8	11	6	14		32.9	80.0	-	IV	-	105	STR
1896	8	11	8	15		32.9	80.0	-	IV	-	105	STR
1896	8	11	9	24		32.9	80.0	-	IV	-	105	STR
1896	8	12	7	42		32.9	80.0	-	IV	-	105	STR
1896	8	14	5	43		32.9	80.0	-	IV	-	105	STR
1896	8	30	3	24		32.9	80.0	-	IV	-	105	STR
1896	9	8	18	16		32.9	80.0	-	IV	-	105	STR
1896	11	14	8	15		32.9	80.0	-	IV	-	105	STR
1899	3	10	5	45		32.9	80.0	-	IV	-	105	STR
1899	12	4	12	48		32.9	80.0	-	IV	-	105	STR
1900	10	31	16	15		30.4	81.7	-	V	-	190	EQH
1901	12	2	0	26		32.9	80.0	-	IV	-	105	STR
1903	1	24	1			32.9	80.0	-	IV	-	105	STR
1903	1	24	1	15		32.1	81.1	-	VI	-	80	EQH

TABLE 2.5.2-1 (SHEET 6 OF 9)

DATE YR	GMT MO	DY	HR	TIME MN	SEC	LOCATION	DEG N	DEPTH DEG W	INTEN. MI	MAG. MMI	SITE MI	DATA DIST.	SOURCE
1903	1	31	10	54		32.9	80.0	-	IV	-	105	STR	
1903	2	3	10	6		32.9	80.0	-	IV	-	105	STR	
1907	4	19	8	30		32.9	80.0	-	V	-	105	EUS	
1911	4	20	22			35.2	82.7	-	V	-	150	BOL	
1911	4	21	3			35.2	82.7	-	V	-	150	EUS	
1911	6	12	10	30		32.9	80.0	-	VII	-	105	EUS	
1912	6	20				32.0	81.0	-	V	-	90	EUS	
1912	9	29	8	6		32.9	80.0	-	IV	-	105	STR	
1912	10	23	1	15		32.7	83.5	-	IV	-	105	STR	
1912	12	7	19	10		34.7	81.7	-	IV	-	105	STR	
1913	1	1	18	28		34.7	81.7	-	VII	-	105	EQH	
1914	3	5	20	5		33.5	83.5	-	VI	-	100	EQH	
1914	3	7	1	20		34.2	79.8	-	IV	-	135	STR	
1914	7	13	20	53		33.0	80.2	-	IV	-	90	BOL	
1914	9	22	2	4		33.0	80.2	-	V	-	90	BOL	
1915	10	29	6			35.8	82.7	-	V	-	190	EQH	
1916	2	21	17	39		35.5	82.5	-	VII	-	170	BOL	
1916	3	2	5	2		34.5	82.7	-	IV	-	105	EUS	
1923	12	31	20	6		34.8	82.5	-	IV	-	120	BOL	
1924	10	20	8	30		35.0	82.6	-	V	-	135	EQH	
1926	7	8	9	50		35.9	82.1	-	VI	-	190	EQH	
1928	11	20	3	45		35.8	82.3	-	IV	-	185	STR	
1928	12	23	2	30		35.3	80.3	-	IV	-	170	STR	
1929	1	3	12	5		33.9	80.3	-	IV	-	100	STR	
1929	10	28	2	15		34.3	82.4	-	IV	-	85	STR	
1930	12	10	0	2		34.3	82.4	-	IV	-	85	STR	
1930	12	26	3			34.5	80.3	-	IV	-	125	STR	
1931	5	6	12	18		34.3	82.4	-	IV	-	85	STR	

TABLE 2.5.2-1 (SHEET 7 OF 9)

DATE GMT YR	MO	DY	TIME HR	MN	SEC	LOCATION DEG N	DEG W	DEPTH	INTEN. MI	MAG. MMI	SITE DIST. MI	DATA SOURCE
1933	6	9	11	30		33.3	83.5	-	IV	-	100	STR
1933	12	19	14	12		33.0	80.2	-	IV	-	90	EUS
1933	12	23	9	40		32.9	80.0	-	V	-	105	STR
1933	12	23	9	55		32.9	80.0	-	IV	-	105	STR
1934	12	9	5			33.0	80.2	-	IV	-	90	BOL
1935	1	1	3	15		35.1	83.6	-	V	-	170	BOL
1938	3	31	10	10		35.6	83.5	-	IV	3.8	195	NUT
1940	12	25				35.9	82.9	-	V	-	200	USE
1941	5	10	11	12		35.6	82.6	-	IV	-	175	STR
1943	12	28	10	25		33.0	80.2	-	IV	-	90	BOL
1944	1	28	17	30		32.9	80.0	-	IV	-	105	STR
1945	1	30	20	20		32.9	80.0	-	IV	-	105	STR
1945	7	26	10	32	16.4	33.75	81.38	3	V	4.4	48	D&G
1947	11	2	4	30		32.9	80.0	-	IV	-	105	STR
1949	2	2	10	52		32.9	80.0	-	IV	-	105	STR
1949	6	27	6	53		32.9	80.0	-	IV	-	105	STR
1951	3	4	2	55		32.9	80.0	-	IV	-	105	STR
1951	12	30	7	55		32.9	80.0	-	IV	-	105	STR
1952	11	19				32.8	80.0	-	V	-	105	BOL
1956	1	5	3			34.3	82.4	-	IV	-	85	BOL
1956	5	19	14			34.3	82.4	-	IV	-	85	BOL
1956	5	27	18	25		34.3	82.4	-	IV	-	85	BOL
1957	5	13	14	24	51.1	35.80	82.14	3*	VI	4.1	185	D&G
1957	7	2	9	33	1.0	35.6	82.6	-	VI	4.6	175	EUS
1957	11	24	20	6	17.0	35.0	83.5	-	VI	-	160	BOL
1958	5	16	22	30		35.6	82.6	-	IV	-	175	EUS
1958	10	20	1	16		34.5	82.8	-	V	-	110	BOL
1959	8	3	6	8	36.8	33.05	80.13	1*	VI	4.4	96	D&G

TABLE 2.5.2-1 (SHEET 8 OF 9)

DATE GMT YR	MO	DY	HR	TIME MN	SEC	LOCATION DEG N	DEG W	DEPTH [1]	INTEN. MI	MAG. [2] MMI	SITE DIST. MI	DATA SOURCE
1959	8	8	6	8	30.0	33.0	79.5	-	VI	-	135	BOL
1959	10	27	2	7	28.0	34.5	80.2	-	VI	-	130	EUS
1960	1	3	7	30		35.9	82.1	-	IV	-	190	STR
1960	3	12	12	47	44.0	33.07	80.12	6	V	4.0	96	D&G
1960	7	23				33.0	80.0	-	V	-	105	USE
1960	7	28	3	37	30.0	32.8	82.7	-	V	-	60	EUS
1963	4	11	17	45		34.9	82.4	-	IV	-	125	EUS
1963	5	4	21	1	50.3	32.97	80.19	3*	IV	3.3	93	D&G
1964	1	20	13	37	52.0	35.9	82.3	-	IV	-	190	EUS
1964	3	7	18	2	58.6	33.72	82.39	3	-	3.3	53	D&G
1964	3	13	1	20	17.5	33.19	83.31	1*	V	3.9	89	D&G
1964	4	20	19	4	44.1	33.84	81.10	2	V	3.5	62	D&G
1965	9	9	14	42	20.0	34.7	81.2	-	-	3.9	110	STR
1967	10	23	9	4	2.5	32.80	80.22	12	V	3.4	94	D&G
1968	7	12	1	12		32.8	79.7	-	IV	-	125	STR
1968	9	22	21	41	18.2	34.11	81.48	1*	IV	3.5	69	D&G
1969	5	18				34.0	82.6	-	-	3.5 M1	75	STR
1969	12	13	10	19	29.7	35.04	82.85	4	V	3.7	145	D&G
1971	5	19	12	54	3.6	33.36	80.65	1*	IV	3.7	67	D&G
1971	7	13	6	42	26.0	34.8	83.0	-	IV	3.8	135	BOL
1971	7	31	20	16	55.0	33.34	80.63	2	III	3.8	68	D&G
1971	8	11	3	50		33.4	80.7	-	-	3.5	65	BOL
1972	2	3	23	11	9.7	33.31	80.58	1	V	4.5	71	D&G
1973	12	19	10	16	8.7	32.97	80.27	4	-	3.0	88	D&G
1974	8	2	8	52	11.1	33.91	82.53	2	VI	4.1	68	D&G
1974	10	28	11	33		33.79	81.92	-	IV	3.0	45	CSC
1974	11	5	3			33.73	82.22	-	III	3.7	48	CSC
1974	11	22	5	25	56.2	32.92	80.14	5	VI	4.3	96	D&G/BOL

TABLE 2.5.2-1 (SHEET 9 OF 9)

DATE YR	GMT MO	DY	HR	TIME MN	SEC	LOCATION DEG N	DEG W	DEPTH [1]	INTEN. MI	MAG. [2] MMI	SITE DIST. MI	DATA SOURCE
1974	12	3	8	25		33.95	82.50	-	III	3.6	69	CSC
1975	4	1	21	9		33.20	83.20	-	-	3.9 M1	82	STR
1975	4	28	5	46	52.6	33.00	80.22	6	IV	3.0 M1	91	STR
1975	10	18	4	31		34.90	83.00	-	IV	-	140	STR
1975	11	25	15	17	34.8	34.93	82.93	1	IV	3.2	140	D&G/GS
1976	12	27	6	57	15.2	32.06	82.50	9	V	3.7	86	D&G/GS
1977	1	18	18	29	14.2	33.04	80.21	7	VI	3.0 M1	91	STR
1977	3	30	8	27	47.8	32.95	80.18	5	V	2.9 Ms	94	STR
1977	8	25	4	20	7.0	33.39	80.69	-	V	3.1	65	GS
1977	12	15	19	16	43.1	32.92	80.22	-	V	3.0	92	GS
1979	9	6	20	38	16.3	35.30	83.24	6	-	3.2 M1	171	VPI
1980	6	10	23	47	32.1	35.46	82.81	1*	-	3.0	170	VPI
1981	4	9	7	10	31.2	35.51	82.05	-	-	3.0	164	VPI
1981	5	5	21	21	56.7	35.33	82.42	6	-	3.5	155	VPI
1982	10	31	3	12	12.2	32.64	84.89	0*	-	3.1	184	VPI
1982	12	11	0	25	6.7	32.71	83.47	0	-	3.0 M1	103	VPI

1 Asterisk (*) indicates that the depth has been constrained to enable stable earthquake location solution.

2 Magnitude is given as body wave magnitude (mb), unless otherwise specified as local magnitude (M1) or surface wave magnitude (Ms).

TABLE 2.5.2-2 (SHEET 1 OF 2)

MODIFIED MERCALLI INTENSITY SCALE OF 1931 (ABRIDGED)

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

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

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

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

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

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

VII. Everybody runs outdoors. Damage negligible in buildings of good structures; considerable in poorly built or badly designed structures; some chimneys are broken. Noticed by persons driving motor cars (VIII Rossi-Forel Scale).

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

TABLE 2.5.2-2 (SHEET 2 OF 2)

IX.	Damage considerable in specially designed structures; well designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken (IX+ Rossi-Forel Scale).
X.	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations, ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks (X Rossi-Forel Scale).
XI.	Few, if any, masonry structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII.	Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

TABLE 2.5.4-1

ENGINEERING PROPERTIES OF SITE SOILS

Static Properties	Upper Sand Stratum (Barnwell Group)	Marl Bearing Stratum (Blue Bluff Marl)	Lower Sand Stratum (Ellenton Formation)
In situ dry density (lb/ft^3)	41-120	51-155	69-118
In situ moisture content (percent)	5-86	3.2-60.6	15-45
Degree of saturation (percent)	18-100	100	-
ASTM D 1557 maximum dry density (lb/ft^3)	101-125	-	-
Optimum moisture content (percent)	7.5-17.4	-	-
Unconsolidated undrained shear strength c (lb/ft^2) and 0°	440-2100, 6°-32°	260-500,000, 0°	-
Consolidated undrained shear strength c (lb/ft^2) and 0°	1650-4000, 17°-25°	-	-
Consolidated drained shear strength c (lb/ft^2) and 0°	0, 33°-34.5°	-	-
Standard penetration test (blows/ft) range/average	2-60 / 30	10-100+ / 100+	70-100+ / 100+
Liquid limit (percent)	NP[a]	19-111	NP
Plastic limit (percent)	NP	15-55	NP
Plasticity index (percent)	NP	2-70	NP
Poisson's ratio	0.4-0.46	0.5	-
Porosity	-	0.403-0.619	-
Permeability (ft/year)	200-350	-	-
Specific gravity	2.67-2.74	2.37-2.84	
Modulus of elasticity (k/ft^2)	-	86-25,000	-

a. Majority of material nonplastic (NP): clay layers had liquid limit greater than 100.

TABLE 2.5.4-2

ENGINEERING PROPERTIES FOR DESIGN

Static Properties	Upper Sand Stratum (Barnwell Group)	Marl Bearing Stratum (Blue Bluff Marl)	Lower Sand Stratum (Ellenton Formation)
In situ dry density (lb/ft^3)	94	88	94
In situ moisture content (percent)	25	35	24
Degree of saturation (percent)	88	100	-
ASTM D 1557 maximum dry density (lb/ft^3)	115.2	-	-
Optimum moisture content (percent)	12.4	-	-
Unconsolidated undrained shear strength c (lb/ft^2) and 0°	2300, 6°	10,000, 0°	-
Consolidated undrained shear strength c (lb/ft^2) and 0°	1000, 18°	-	-
Consolidated drained shear strength c (lb/ft^2) and 0°	0, 34°	-	-
Standard penetration test (blows/ft)	30	100+	100+
Porosity	-	0.497	-
Poisson's ratio	0.4	0.5	-
Permeability upper sand stratum (ft/year)	350	-	-
Specific gravity	2.70	2.72	-
Modulus of elasticity (k/ft^2)	-	4000-10,000	-

TABLE 2.5.4-3

DESIGN VALUES OF SHEAR MODULUS[(a)]

In Situ Soils:

Elevation (ft)	Shear Modulus (lb/ft^2)
210 to 180	2.3×10^6
180 to -770	11.6×10^6

a. The values refer to shear modulus at strains of approximately 10^{-4} percent or lower.

TABLE 2.5.4-4

IN SITU SOILS - BASIC SOIL PROPERTIES FOR DYNAMIC DESIGN[a]

Stratum Designation	Elevation (ft)	Moist	Unit Weight Saturated (lb/ft)	Submerged	Poisson's Ratio
Upper sand stratum	225 to 135	115	115	52.6	0.4 to 0.46
Marl bearing stratum	135 to 70	-	115	52.6	0.5
Lower sand stratum	70 to -770	-	115	52.6	0.4 to 0.46

a. In sandy soils the specific value of Poisson's ratio within the given range will be the most conservative value for the particular dynamic analysis being carried out.

Figures describing the variation of shear modulus and damping ratio with shear strain for clay marl bearing stratum and lower sand stratum are provided in subsections 3.7.B.1 and 3.7.B.2.

TABLE 2.5.4-5

COMPILATION OF SHEAR WAVE DATA[a]

Depth (ft)	Elevation[b] (ft)	Compression Wave Velocity (ft/s)	Shear Wave Velocity (ft/s)
0-15	208-193	1400	600[c]
20	188	2500	1000
30	178	2800	1000
40	168	2500	900
50	158	4600	1000
60	148	5200	1200
70	138	5100	1400
80	128	6600	1600
90	118	6700	1700
100	108	6900	1800
110	98	6600	1700
120	88	6400	1700
130	78	6600	1800
140	68	6500	1700
150	58	6800	1600
160	48	6600	1600
170	38	6800	1800
180	28	6600	--
190	18	6500	1800
200	8	6600	1800
210	-2	6600	1700
220	-12	6600	1800
230	-22	6700	1800
240	-32	6400	1700
250	-42	6500	1800
260	-52	6700	1800
270	-62	6800	1800
280	-72	6700	1800
290	-82	6700	1700

a. From cross-hole measurements or as noted.

b. Ground surface elevations average 208 ft above sea level in this area.

c. From surface data.

TABLE 2.5.4-6

SUMMARY OF DATA COMPILATION BASED ON CROSS-HOLE DATA[(a)]

Bore Holes	Depth (ft) From	Depth (ft) To	Elevation (ft) From	Elevation (ft) To	Compression Wave Velocity (ft/s)	Shear Wave Velocity (ft/s)	Generalized Material Correlations
B-136	0	15	208	193	1400	600 (from surface data)	Sands
B-146G	15	40+	193	168-	2500	1000	Sands
B-148	40+	50	168-	158	4600	1000	Saturated materials
B-149	50	80-	158	128+	5000	1300	Saturated materials
B-151	80-	90	128+	118	6650	1650	Compact materials, high blow-count values
B-154	90	290	118	-82	6800	1600-1800 (mostly 1800)	Compact materials, high blow-count values

a. Ground surface elevation averages 208 ft above sea level in this area.

TABLE 2.5.4-7

ELASTIC MODULI RESULTS

Depth (ft) From	Depth (ft) To	Elevation (ft) From	Elevation (ft) To	P-Wave (ft/s)	S-Wave (ft/s)	Density (lb/ft^3)	Poisson's Ratio	Young's Modulus ($lb/in.^2$)	Shear Modulus ($lb/in.^2$)
0	15	208	193	1400	600	115	0.39	0.2×10^5	0.8×10^4
15	40+	193	168-	2500	1000	115	0.40	0.7×10^5	2.4×10^4
40+	50	168-	158	4600	1000	115	0.48	0.7×10^5	2.4×10^4
50	80-	158	128+	5000	1300	115	0.46	1.3×10^5	4.1×10^4
80-	90	128+	118	6650	1650	115	0.47	2.0×10^5	6.8×10^4
90	290	118	-82	6800	1800	115	0.46	2.3×10^5	8.0×10^4

TABLE 2.5.4-8

DESIGN STATIC PROPERTIES FOR BACKFILL COMPACTED
TO 97-PERCENT RELATIVE COMPACTION (ASTM D 1557)

Soil Properties	Sand, Silty Sand
Unit weights (lb/ft^3)	
Moist	126
Saturated	132
Submerged	69.6
Effective shear strength parameters	
Cohesion or c (k/ft^2)	0
Angle of internal friction or Σ (degrees)	34
Undrained modulus of elasticity or E (k/ft^2)	1500
Poisson's ratio (υ)	0.4
Compression index	-

TABLE 2.5.4-9

DESIGN DYNAMIC PROPERTIES FOR BACKFILL COMPACTED TO
97-PERCENT RELATIVE COMPACTION (ASTM D 1557)

Soil Properties	Sand, Silty Sand
Unit weights (lb/ft^3)	
Moist	126
Saturated	132
Submerged	69.6
Poisson's ratio	0.33
Damping ratio	See figure 3.7.B.1-8.

Shear modulus at strain of 10^{-4} percent[(a)]

Elevation (ft)	Depth (ft)	Shear Modulus (lb/ft^2)
210	10	2.3×10^6
195	25	3.6×10^6
165	55	5.3×10^6
150	70	5.7×10^6
130	90	6.2×10^6

a. Shear modulus $G = 1000\ k_2(\sigma'_m)^{1/2}\ lb/ft^2$, where $k_2 \approx 79$.

Variation of shear modulus with strain is given in figure 3.7.B.2-5.

TABLE 2.5.4-10
SUMMARY OF TEST FILL RESULTS FOR HEAVY EQUIPMENT COMPACTION

Test Fill No.	Material From Stockpile[a]	Roller	Roller Speed (mph)	Number of Passes	Lift Thickness (in.)	Field Density Method	Depth of Test (in.)	Number of Field Density Tests[b]	Percent of Tests: Less Than 97% Compaction	Percent of Tests: 93 to 95% Compaction	Percent of Tests: Less than 93% Compaction	Remarks
I	C	SPF 60 and Raygo 600A	1.5	2 each	6	Sand cone	12	24	0	0	0	Acceptable
							18	24	0	0	0	
						Nuclear	12	24	8	0	0	
							18	24	8	0	4	
II	C	Raygo 600A	1.5	4	6	Sand cone	12	24	0	0	0	Acceptable
							18	-	-	-	-	
						Nuclear	12	24	0	0	0	
							18	-	-	-	-	
III	A	SPF 60 and Raygo 600A	1.5	2 each	6	Sand cone	12	24	33	4	0	Acceptable
							18	-	-	-	-	
						Nuclear	12	24	100	46	29	
							18	-	-	-	-	
IV	A	Raygo 600A	1.5	4	6	Sand cone	12	24	4	0	0	Acceptable
							18	-	-	-	-	
						Nuclear	12	24	96	42	46	
							18	-	-	-	-	
V	A	SP 60	3	3	12	Sand cone	12	24	42	8	0	Not acceptable
							18	6	83	17	0	
						Nuclear	12	24	100	4	96	
							18	6	100	50	50	
VI	C	SP 60	2, 1.5	3	12	Sand cone	12	24	0	0	0	Acceptable
							18	6	0	0	0	
						Nuclear	12	24	33	8	8	
							18	6	67	17	17	
VII	B	Raygo 600A	1.5	4	6	Sand cone	12	10	30	20	0	Inconclusive
							18	-	-	-	-	
						Nuclear	12	10	100	10	90	
							18	-	-	-	-	

a. Letters refer to designations on drawing AX6DD394.
b. A laboratory compaction test was performed for each field density test.

TABLE 2.5.4-11

SUMMARY OF TEST FILL RESULTS FOR HAND COMPACTION EQUIPMENT

									Percent of Tests			
Test Fill No.	Material From Stockpile[a]	Roller	Roller Speed (mph)	Number of Passes	Lift Thickness (in.)	Field Density Method	Depth of Test (in.)	Number of Field Density Tests[b]	97% Compaction or Greater	93 to 95% Compaction	93% Compaction or Lower	Remarks
1	C	Wacker 74 Dual Drum	0.68	4	6	Sand cone	12	8	8	0	0	Acceptable
2	C	Wacker 100 Jumping Jack	0.23	2	6	Sand cone	12	8	8	0	0	Acceptable
3	C	Ingersoll - Rand- SP 24	0.68	4	6	Sand cone	12	16	15	1	0	Acceptable

a. Letters refer to designations on drawing AX6DD394.

b. A laboratory compaction test was performed for each field density test.

TABLE 2.5.4-12 (SHEET 1 OF 2)

SUMMARY OF RESULTS OF BEARING CAPACITY ANALYSIS

Structure	Supporting Stratum	El of Top of Foundation/El of Bottom of Foundation (ft)	Approx. Mat Size (ft)	Loading Pressure Gross Static (k/ft^2)	Loading Pressure NetLAM Static (k/ft^2)	Gross Dynamic (k/ft^2)	Net Dynamic (k/ft^2)	Ultimate[b] Net Bearing Capacity (k/ft^2)	Allowable[c] Net Bearing Capacity Static (k/ft^2)	Allowable[c] Net Bearing Capacity Dynamic (k/ft^2)	Factor of Safety = Ultimate Net Bearing Capacity ÷ Net Static or Dynamic Loading Pressure: Computed Static Factor of Safety	Factor of Safety = Ultimate Net Bearing Capacity ÷ Net Static or Dynamic Loading Pressure: Computed Dynamic Factor of Safety
Diesel generator building	Backfill	220/211	115x93	3.8	2.7	13.6	12.5	60.9	20.3	30.5	22.6	4.9
Turbine building	Backfill	195/186	184x604	3.6	-0.5	8.7	4.6	56.5	18.8	28.3	Very[d] high	12.3
Control building	Backfill	180/173	525x169	4.3	-1.3	13.4	7.8	57.8	19.3	28.9	Very[d] high	7.4
Containment building	Backfill	169/158.5	D=154.5	8.4	1.0	20.9	13.5	61.7	20.6	30.9	61.7	4.6
Fuel building	Backfill	160/154	198x75	8.1	0.1	23.4	15.4	64.0	21.3	32.0	640	4.2
Nuclear service cooling water tower	Marl	137/128	D=100	8.8	-2.4	34.2	23.1	61.7	20.6	30.9	Very[d] high	2.7
Auxiliary building	Marl	119.25/109.25	440x130	10.2	-3.3	28.7	15.2	63.7	21.2	31.9	Very[d] high	4.2
Nuclear service cooling water valve house	Backfill	204.5/198.5	110x33	3.2	0.6	12.6	10.0	133.5	44.5	66.8	222.5	13.4
Auxiliary feedwater pumphouse	Backfill	215/212	86x40	1.6	0.6	3.2	2.2	92.5	30.8	46.3	154.3	42.0

TABLE 2.5.4-12 (SHEET 2 OF 2)

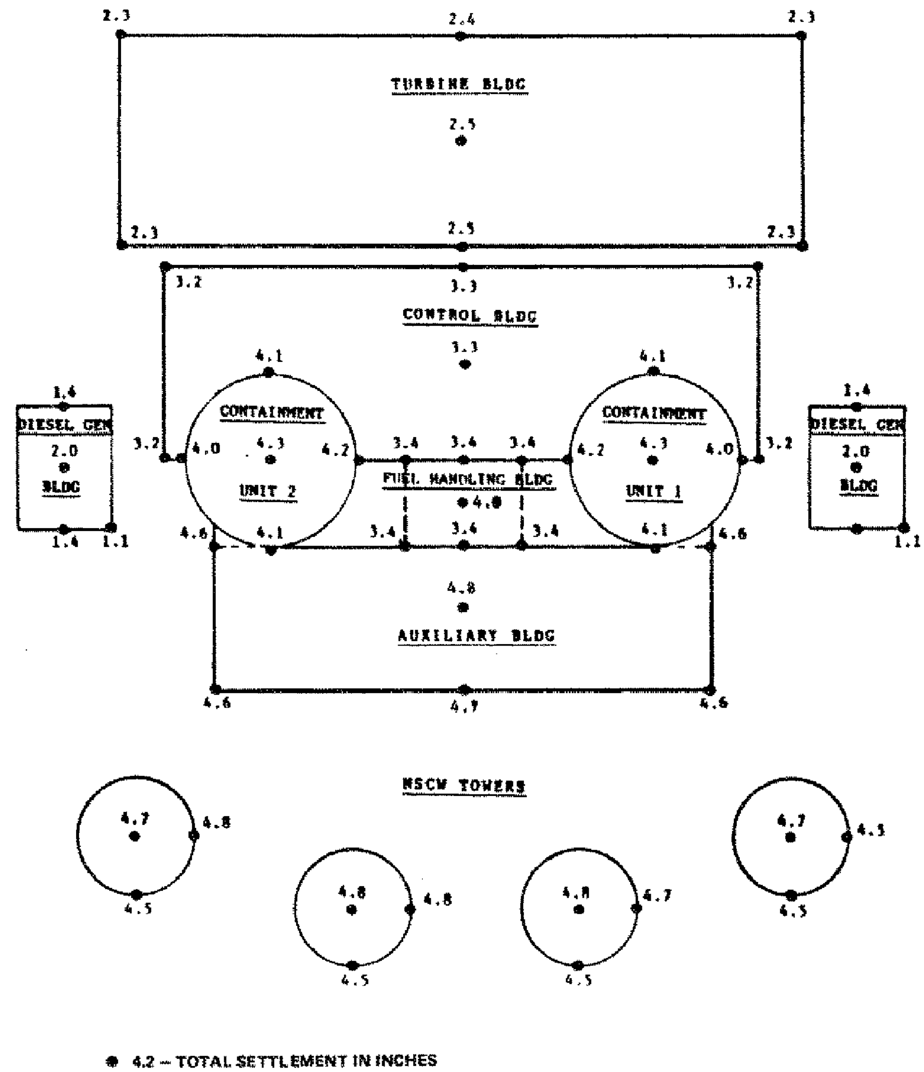
Structure	Supporting Stratum	El of Top of Foundation/El of Bottom of Foundation (ft)	Approx. Mat Size (ft)	Loading Pressure Gross Static (k/ft^2)	NetLAM Static (k/ft^2)	Gross Dynamic (k/ft^2)	Net Dynamic (k/ft^2)	Ultimate[b] Net Bearing Capacity (k/ft^2)	Allowable[c] Net Bearing Capacity Static (k/ft^2)	Dynamic (k/ft^2)	Factor of Safety = Ultimate Net Bearing Capacity ÷ Net Static or Dynamic Loading Pressure Computed Static Factor of Safety	Computed Dynamic Factor of Safety
Condensate storage tank	Backfill	220/212	115x63	3.1	2.1	5.3	4.3	115.3	38.4	57.7	54.9	26.8
Diesel fuel oil Storage tank Pumphouse	Backfill	211.5/209.5	114x26	1.7	0.4	2.5	1.2	81.9	27.3	41.0	204.8	68.3
Reactor makeup water storage tank	Backfill	220/212	51x51	2.3	1.3	5.3	4.3	95.7	31.9	47.9	73.6	22.3
Refueling water storage tank	Backfill	220/216	62x62	3.7	3.2	11.8	11.3	88.9	29.6	44.5	27.8	7.9
Radwaste transfer building	Backfill	220/216.5	93x45	3.6	3.2	7.7	7.3	86.5	28.8	43.2	27.0	12.0

a. The net static load is the load in excess of the overburden pressure at the base of the structure.
b. The ultimate net bearing capacity is the load in excess of the overburden pressure at the foundation level at which shear failure will occur in the foundation stratum.
c. The allowable net static and dynamic bearing capacities are obtained by dividing the net ultimate bearing capacity by factors of 3 and 2 respectively.
d. The net static bearing pressure is negative.

TABLE 2.5.4-13

SUMMARY OF MEASURED HEAVE IN POWER BLOCK

Heave Point No.	Period From	To	Measured Heave (in.)
1	06/22/74	08/07/77	1.1
2	06/16/74	06/22/76	1.4
3	06/16/74	10/02/74	0.6
5	06/16/74	02/26/77	1.7
7	06/16/74	06/05/77	1.2
9	06/30/74	08/07/77	1.5



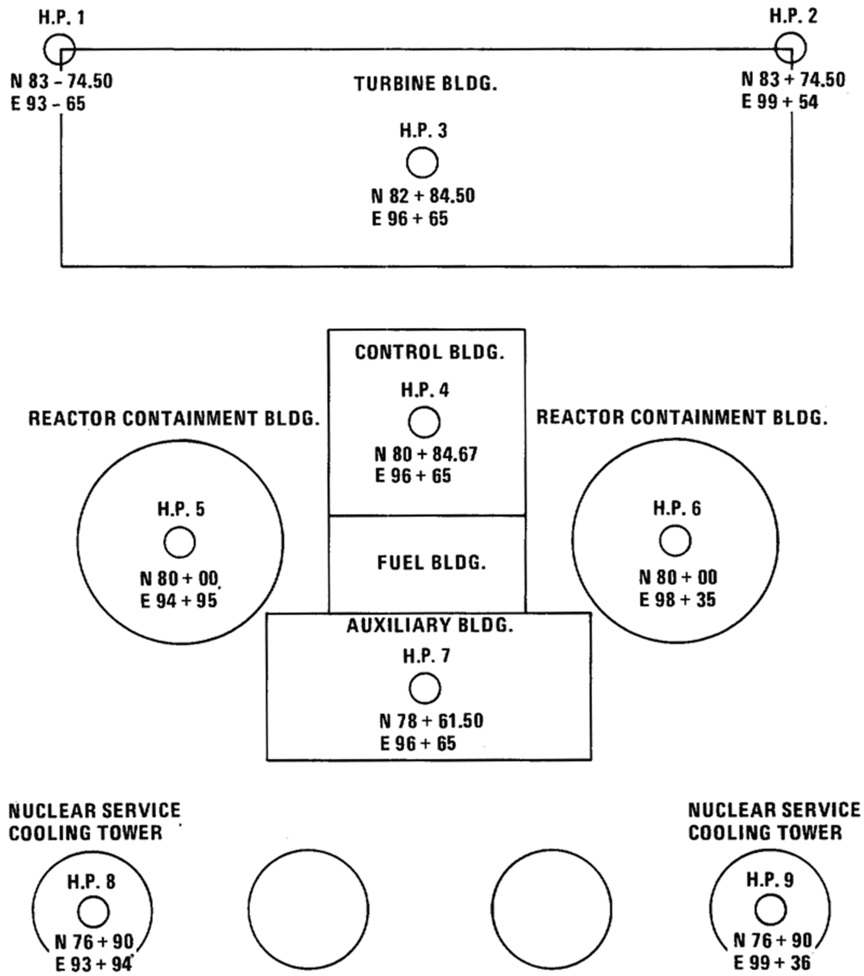
REV 13 4/06

SOUTHERN COMPANY
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

SETTLEMENT OF UNITS 1 AND 2
POWER BLOCK STRUCTURE

FIGURE 2.5.4–1

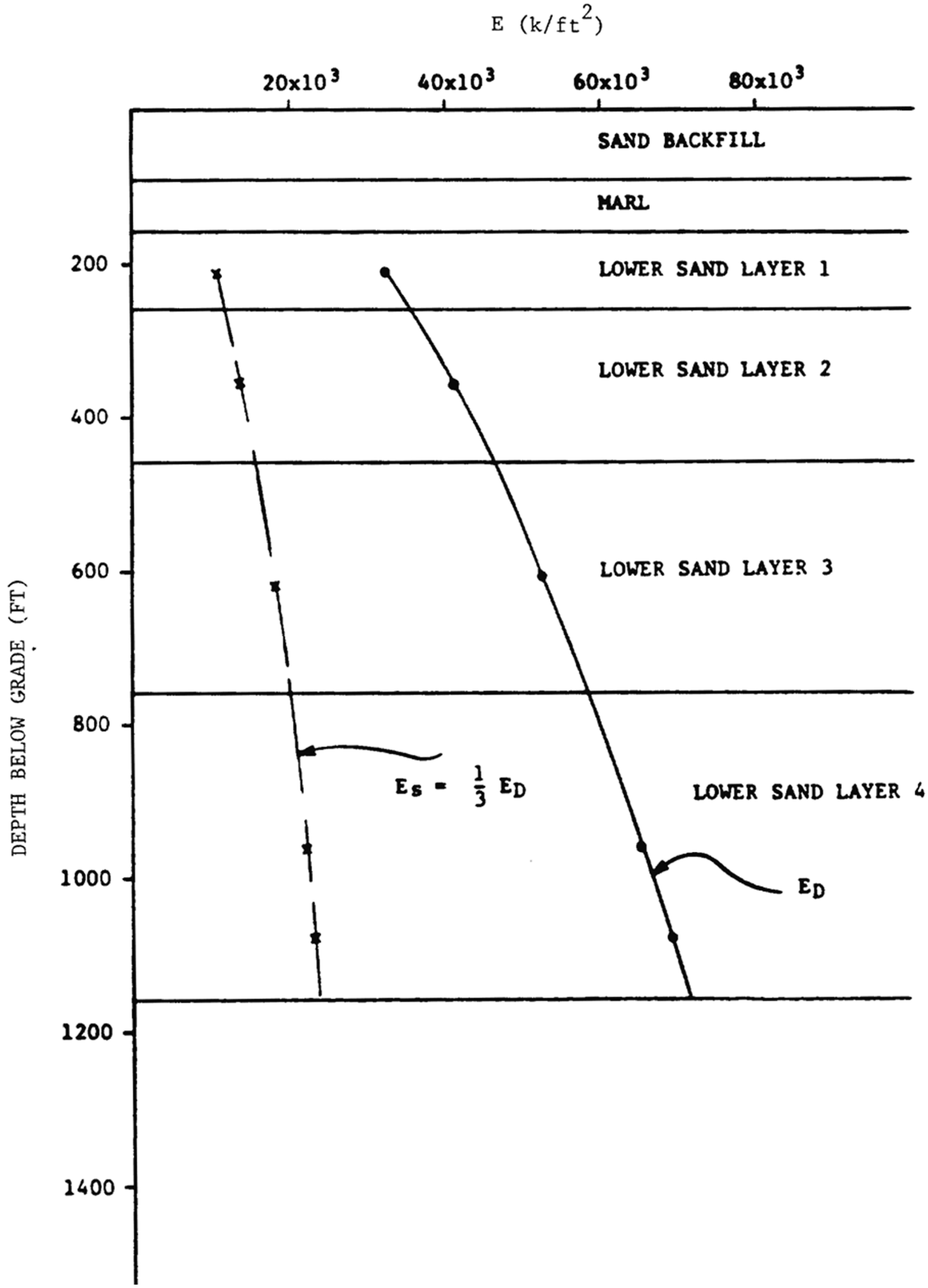


BENCHMARK NO. 1: N 80 + 00 E 139 + 00; TIP ELEV. = 105.0

BENCHMARK NO. 2: N 80 + 00 E 51 + 00; TIP ELEV. = 69.0

REV 13 4/06

SOUTHERN COMPANY *Energy to Serve Your World®* VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	HEAVE POINTS LOCATION PLAN
	FIGURE 2.5.4–2



REV 13 4/06

SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	YOUNG'S MODULUS vs DEPTH FIGURE 2.5.4–3

APPENDIX 2A (HISTORICAL)

POPULATION DISTRIBUTION METHODOLOGY

2A.1 INTRODUCTION (HISTORICAL)

This appendix documents the procedures followed in the preparation of paragraphs 2.1.3.1 and 2.1.3.2 of the Final Safety Analysis Report (0 to 50 miles) and paragraphs 2.1.2.1 and 2.1.2.2 of the Operating License Stage Environmental Report (50 to 500 miles). Included with a step-by-step documented review of the procedures is a presentation of the methodology used, a table of definitions, a review of the materials used, a discussion of the assumptions made, and a section addressing the procedures to follow should the submitted figures need updating.

2A.2 DEFINITIONS (HISTORICAL)

OPB: OPB is an acronym for the Office of Planning and Budget, State of Georgia.

SDC: SDC is an acronym for the State Data Center, State of South Carolina.

Sector: A sector is one of the 16 compass divisions comprising the area within any given circle centered on VEGP.

Segment: A segment is an area bounded by two sector divisions and two arcs.

USGS Quadrangle Maps: United States Geological Survey quadrangle maps are topographical maps bounded by parallels of latitude and meridians of longitude. Quadrangles covering 7 1/2 min of latitude and longitude are published at the scale of 1:24,000 (1 in. = 2000 ft.) Quadrangles covering 15 min of latitude and longitude are published at the scale of 1:62,500 (1 in. mile).

2A.3 ASSUMPTIONS (HISTORICAL)

The major assumptions taken to determine population distribution were as follows:

A. *The population density within the Savannah River Plant is zero.*

B. *The percentage of each county's population in a segment for 1980 will not change over the forecasted time span.*

C. *The curvature of the earth will not have a significant effect on the construction of rings and sectors for the 500-mile radius.*

D. *Due to possible human error in construction, rings are estimated to be accurate within ±1/2 mile.*

E. Population changes at Fort Gordon over the forecasted period are addressed in the OPB projections.

F. Population projections made for the area within a 500-mile radius of VEGP were based on 1980 census data and county population projections obtained from the OPB and the SDC.

G. The extrapolation method of population projections for county and subcounty areas is generally more accurate than the differential and share methods.

2A.4 MATERIALS (HISTORICAL)

The materials used to determine population distribution were:

A. The USGS quadrangle maps of the affected area.

B. The 1980 state maps depicting the affected counties.

C. U.S. Census Bureau figures indicating the population and number of housing units of the counties and cities in the affected area for 1980.

D. The OPB and the SDC county and city projections for the years 1990, 2000, 2010, and 2020.

E. The 1980 population figures for military installations in the area.

F. A house to house survey of the area within 5 miles of the VEGP site conducted in 1980.

The USGS quadrangle maps were used as the base maps for construction of rings and sectors and for the transcription of county and city boundaries; they were chosen as base maps because of their detailed representation of housing patterns. The OPB and SDC projections were chosen for future population estimates because of the high accuracy of their projections as demonstrated by a narrow margin of error between 1980 projected and actual population figures.

2A.5 METHODOLOGY (HISTORICAL)

Described in the steps below is the methodology used to determine population distribution:

A. Locate the center of VEGP on the USGS quadrangle maps. Construct concentric circles on the quadrangle maps at distances of 1 through 10, 20, 30, 40, 50, 60, 70, 85, 100, 150, 200, 350, and 500 miles.

B. Divide the constructed circles into sectors of 22 1/2° with each sector centered on one of the 16 compass points, e.g., true north, north-northeast.

C. Transcribe the city and county boundaries located within a 500-mile radius of VEGP to USGS quadrangle maps.

D. Estimate from the quadrangle maps the percentage of each city's and county's population that lies in each affected section.

E. Calculate from 1980 census statistics the percentage of each county's population that lies in each affected section.

F. Assume that the percentage of each county's population that resides in each affected segment remains the same over the forecasted time span.

G. Prepare population projections for the first year of plant operation (1987 for Unit 1) and beyond by first determining the margin of error for previous projections completed by the OPB and the SDC (the difference between 1980 population forecasts and 1980 census figures). Using the margin of error, adjust projections for the census years, the midpoint in the plant's operating life, and the endpoint in the plant's operating life.

H. Estimate a segment's population for any year by multiplying each affected county's adjusted population projection for that year by the percentage of that county's population which is in the segment.

To estimate a county's population for the anticipated first year of plant operation, the following methodology was used:

A. Subtract the county's previous census decade's count from the next projected census decade's estimate, divide by 10, and multiply this number times the number of years into the decade the first year of plant operation occurs.

B. Add the figure obtained in item A to the previous decade's estimate to obtain a county's estimated population for the anticipated initial year of plant operation. For example, assuming a starting date of 1987, the estimated population of county X for section A lying entirely within that county is determined as follows:

$$\frac{M - N}{10} \times 7 = 0.7M - 0.7N$$

$$\begin{aligned} 0.7M - 0.7N + N &= 0.7\ (M - N) + N \\ &= \textit{estimated population for } 1987 \end{aligned}$$

where:

M = 1990 estimated population for county X.

N = 1980 population count for county X.

For segments within 50 to 500 miles, statewide projected growth rates were used to determine individual county forecasts. Population estimates within 5 miles of VEGP were based on a 1980 house to house survey of the area. Multifamily housing units related to construction worker demand were included in the population estimates for 1987. It was assumed that these housing units will continue to be in use through 1989, estimated completion year of Unit 2. However, projections for 1990 and beyond do not include construction-related housing.

2A.6 PROCEDURES (HISTORICAL)

Population distribution by sector for the area within a 500-mile radius of VEGP was determined in the following manner:

A. *Base maps showing county boundaries within a 500-mile radius were overlain with annular rings and sectors.*

B. *Each segment's county composition was visually estimated. For example, sector 16-30 is composed of 4 percent Aiken County and 14 percent Richmond County.*

C. *All counties lying within the 500-mile radius were listed with their 1980 population. Cities over 25,000 were subtracted from county figures if they lay in more than one segment, or if the county lay in more than one segment. For example:*

	1980 Population
Etowah County	*103,057*
Gadsden City	*-47,255*
Remaining population	*55,802*

Each city was listed with the segment or segments in which it lay.

D. *Each segment's 1980 population was determined by multiplying the percentage of each county represented by its remaining 1980 population (U.S. Census Bureau).*

APPENDIX 2B

GEOLOGY

2B.1 INTRODUCTION

Comprehensive aerial geology and site specific foundation investigations and examinations of the VEGP site have been completed. The results and conclusions are described and contained in section 2.5 and subsection 2.4.12. The geologic logs and geophysical logs are submitted under a separate cover.

A table of drilling statistics is presented in table 2B-1. A description of foundation conditions encountered during construction is contained in subsection 2B.3.1.

2B.2 FIELD INVESTIGATIONS

A total of 370 borings was drilled for the primary geologic and site specific foundation investigations for the plant facilities. The drill logs of 354 of these borings are submitted as described in section 2B.1. The remaining 16 borings were done for the revised locations of the cooling towers and the drill logs for these borings are included in the report Foundations Investigations for Natural Draft Hyperbolic Cooling Towers, Addendum, prepared by Bechtel Power Corporation, December 1978. Selected marl core samples from principal borings have been placed in protective storage; table 2B-2 provides an inventory of these core samples. An additional 12 borings were made for the studies of the postulated Millett Fault conducted in 1982. These borings are offsite and are described in detail in the report Studies of Postulated Millett Fault, dated October 1982. Logs of these borings are included in that report.

2B.3 REPORT OF GEOLOGY AND FOUNDATION CONDITIONS

2B.3.1 INTRODUCTION

This report presents the results of geologic work performed in conjunction with the excavation of the power block areas at the VEGP site. The purpose of the work was to identify, locate, and record details of the geologic structure, stratigraphy, and lithology of the soil and rock strata encountered in the excavation. In addition, samples of foundation rock were obtained for testing of physical properties.

The geologic work was performed by Bechtel geologists during the period May 1974 through October 1977 with certain tasks continuing on an as-needed basis. This time period included the initial startup of the construction work, the interim postponement of work between September 1974 and July 1976, and the subsequent restart of construction.

The work performed included detailed geologic mapping of the soil and rock strata exposed in the power block excavation, and coring and testing of the Blue Bluff marl, which forms the foundation for power block structures and structural backfill. This marl has sometimes been referred to as the "clay bearing stratum." Geologic mapping was accomplished by recording the details of stratigraphy, structure, and lithology of the various soil and rock deposits on a base map prepared from excavation drawings. Mapping of the vertical surfaces of the auxiliary building excavation walls was recorded on geologic sections coinciding with the surfaces of the walls. All geologic mapping was performed using hand surveying techniques with the exception of the recording of the upper contact of the marl layer. This was recorded by instrumental survey of 575 points established by the geologists. Photography was employed as an aid to mapping and to provide a record of foundation geologic features.

As areas of the marl were cleaned off at final grade in the excavation, they were inspected and signed off by a qualified geologist or soil engineer. The documentation for the approved foundation areas was submitted to Georgia Power Company for permanent retention. This documentation has been transferred to Southern Nuclear Operating Company (SNC), as the exclusive operating licensee.

Subsection 2B.3.2 of this report presents a brief summary of conclusions from the studies performed. Subsection 2B.3.3 presents the details of the geologic structure, stratigraphy, and lithology of the various geologic materials encountered in the excavation. Ground water conditions encountered during the reference period are described. Geologic mapping procedures are discussed in detail. Subsection 2B.3.4 describes the excavation geometry along with the procedures utilized for advancing the excavation down to final grade. Temporary dewatering methods are described as are measures taken for protection of the side slopes from erosion. Foundation cleanup and protection procedures are discussed, and inspection and approval procedures are outlined. The marl testing program carried out to confirm the design physical properties of this material is discussed, and reference is made to the backfill report(1) in which the test results are compiled. The monitoring of rebound of the marl due to unloading by excavation of the overlying deposits is described. Subsection 2B.3.5 presents detailed conclusions drawn from the work described in the preceding sections.

The features described in the report are illustrated in maps, geologic sections, and miscellaneous figures which accompany the report.

This report is confined to geologic conditions exposed in the power block excavation at the VEGP site. For discussion of regional geology as well as other more general features of site geology, refer to section 2.5. The documents referenced at the end of this section present further details of topics mentioned briefly in this report.

The following organizations and individuals participated in the work covered by this report:

- Georgia Power Company--plant owner, construction manager, logistical support.
- Bechtel Power Corporation--geology, soil engineering.
- Law Engineering Testing Company--obtainment and laboratory testing of core samples.
- Goldberg, Zoino, Dunnicliff Associates--foundation instrumentation.
- R. Y. Bush--dewatering consultant.

2B.3.2 SUMMARY OF CONCLUSIONS

The studies described in this report have led to the following conclusions:

- The site is suitable for design and construction of a multiple-unit nuclear generating facility.
- The geologic conditions exposed in the power block excavation confirm the conditions described in the Preliminary Safety Analysis Report(PSAR).(2)
- No geologic features exist in the power block area, which could affect licensing aspects of the plant.
- The marl layer, which provides foundation for both Category 1 structures and Category 1 backfill, is as sound and as competent as anticipated and is free of solution cavities.
- Results of laboratory testing and monitoring of rebound of the marl confirm its competency.

2B.3.3 GEOLOGIC CONDITIONS

2B.3.3.1 General

The VEGP site is situated on the right (south) bank of the Savannah River in Burke County, Georgia, approximately 26 miles southeast of the city of Augusta. The area is a part of the Atlantic Coastal Plain physiographic province and is characterized by mature river valley topography. The elevation of the site varies from approximately 90 to about 255 ft above msl. The original ground surface elevation in the power block area varied from about 200 to 230 ft. The site topography is characterized by rolling hills and ridges dissected by tributaries of the Savannah River. Steep cliffs have formed along the south side of the Savannah River as a result of the steady southward migration of the main river channel. In contrast, the north bank is an extensive area of swampy lowland. This low area is outside the plant boundary.

The portion of the coastal plain in which the site lies consists of a thick sequence of Cretaceous and Tertiary shallow-water massive sediments overlying Triassic basement rocks. The sediments generally dip at a very slight angle towards the east and southeast. At the VEGP site, approximately 950 ft of these sediments cover the older Triassic rocks. The general stratigraphic sequence consists of a thin veneer of windblown sand on hills and ridges capping a sequence of shallow marine sands, clays, and isolated coquina and limestone deposits, which extend down to approximate el 135 ft. Below this, a layer of moderately to locally well-cemented marl having a thickness of about 70 ft overlies dense sands and clayey sands, which extend down to the Triassic basement.

The marl layer acts as an aquiclude separating the regional aquifer, confined beneath the marl, from the shallow water table existing in the deposits above the marl.

The excavation for the power block facilities cut through the upper sequence of sand, clay, limestone, and coquina and bottomed in the marl. Details of these deposits are discussed in the following paragraphs.

2B.3.3.2 Geologic Mapping Procedures

Geologic mapping and recording of the features exposed in the excavation was accomplished in three phases:

- Detailed mapping of deposits above the Blue Bluff marl, May 1974 to October 1974.
- Detailed mapping of features within the marl and surveying of the upper contact of the marl, February 1977 to October 1977.
- Detailed inspection and recording of areas in the marl approved for placement of concrete or backfill, June 1977 to January 1979.

The first phase of mapping was performed in conjunction with the excavation of the sediments above the marl. Features were located in the side slopes of the excavation as the bottom elevation was progressively lowered. The side slopes were cut at a gradient of two horizontal to one vertical (2:1), and survey stakes were installed on a grid pattern on the slopes. Locations of geologic features were measured by tape and hand-level methods using the slope stakes as reference points. Accuracy of these measurements is estimated to be within 0.5 ft. Mapping of the 2:1 slopes was recorded in plan view on a base map compiled from the project excavation drawings and is shown on drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363.

The second phase of mapping was accomplished as the marl layer was exposed and prepared for placement of concrete and backfill. Since it was desired to demonstrate the absence of faulting in the marl, the upper contact was measured and recorded with survey accuracy around the perimeter of the excavation. Five hundred and seventy-five survey points were established by the geologists along this contact, and these points were located instrumentally. The nature of the contact between the points was examined closely for continuity and absence of breaks. The contact is shown in plan view on drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363 and the details of the survey results are shown in both plan and section on drawing AX6DD372.

In addition to examining the upper contact, features within the marl were examined and recorded. The deep excavation for the auxiliary building basement, within the larger power block excavation, provided an excellent opportunity for this. The sides of the excavation exposed a vertical section approximately 22 ft in height in the marl. A system of reference points was established on the walls of the excavation, and stations were established for the purpose of describing locations of features. The stationing system adopted is shown on drawings AX6DD374 and AX6DD375. The mapping was recorded in the vertical plane and is presented as geologic sections on drawings AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, and AX6DD370. An explanation of geologic units used for mapping purposes is shown on drawings AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, and AX6DD370. By referring to drawings AX6DD374 and AX6DD375, the location of any section can be easily ascertained. Measurements were made by tape and hand-level methods on the excavation walls. Accuracy is estimated to be generally within 0.1 ft.

The third phase of geologic mapping consists of detailed inspection and photography of foundation areas rather than mapping in the strict sense. This effort was initiated in June 1977 when the first foundation area was cleaned off at final grade and prepared for the gunite protective seal. Inspection and approval of final grade in the marl continued on an intermittent basis as foundation areas were readied. Documentation of the geologic inspection and approval of foundation areas have been transmitted from the inspecting Bechtel geologists to Georgia Power Company site personnel for permanent retention. The documentation described in this section has been transferred to SNC as the exclusive operating licensee.

Details of the methods used for protection of final grade areas in the marl are discussed in a later section.

2B.3.3.3 Stratigraphy

2B.3.3.3.1 General

The stratigraphy of the VEGP site has been described in the PSAR.[(2)] The subsequent detailed excavation mapping, as well as recent regional work by Georgia State Geological Survey, has resulted in a much clearer and more detailed understanding of the site stratigraphy.

The sequence of geologic strata exposed in the power block excavation is shown in the stratigraphic column presented in figure 2.5.1-5. Five distinct stratigraphic units are recognized in the excavation. These are, in sequence from the oldest to the youngest:

- The Lisbon Formation of middle Eocene age.
- The Utley Limestone Member of the Barnwell Group of late Eocene age.
- The Twiggs Clay Member of the Barnwell Group.
- The Irwinton Sand Member of the Barnwell Group.
- The Tobacco Road Sand unit of the Barnwell Group.

The stratigraphic nomenclature adopted in this report was chosen to conform with the latest regional stratigraphic interpretation adopted by the Georgia State Geological Survey.[(3)] Stratigraphic nomenclature in the Georgia-South Carolina area has long been disputed and has not yet been fully resolved.

The formational boundary between the Lisbon Formation and Barnwell Group coincides with the upper contact of the marl.

2B.3.3.3.2 Lisbon Formation

The middle Eocene Lisbon Formation is represented in the site area by the Blue Bluff marl which forms the foundation for structures and backfill in the power block area. The marl has a total thickness of about 70 ft in the site area. The upper approximate 25 ft of the marl were exposed in excavations and mapped in detail. A vertical section between el 108.6 ft (final excavated grade) and 132 ft was exposed in the auxiliary building basement excavation. Ten subunits were recognized and mapped in this vertical section. The subunits, designated A through J, are shown on drawings AX6DD364, AX6DD365, AX6DD366, AX6DD367, AX6DD368, AX6DD369, and AX6DD370.

Unit A, near the top of the excavation walls, is generally above el 128 ft and includes the marl from this point up to the upper contact of the marl with the Utley Limestone Member of the Barnwell Group. It consists of dark gray silty to clayey marl with very fine light gray to white fine sandy laminations, which are undulatory and discontinuous. Scattered shell fragments and well-cemented lenses of sand up to 0.1 ft thick are present locally. The laminations are oriented parallel to the lower contact of the unit, and parting along the laminations is common. Unit A is dense and well consolidated. Surfaces exposed to the atmosphere tend to dessicate rapidly. Unit A interfingers with the underlying unit B. This is especially evident in the south wall in the vicinity of stations 0 + 70, 1 + 50, and 4 + 30. (See drawings AX6DD365, AX6DD367, and AX6DD368.) The contact with unit B is everywhere gradational.

Unit B, directly beneath unit A, is continuous around the auxiliary building basement walls and varies from 1 to over 4 ft in thickness. It consists of massive to faintly laminated gray sandy marl. It has a sugary texture and does not tend to dessicate as readily as unit A. This property provides an easy means for differentiating the units after exposure to the atmosphere. Unit B is dense but poorly cemented and contains widely scattered shell fragments.

A subunit of B, designated B_1, has been identified and is present locally within B. This subunit consists of laminated sandy marl, which is locally fossiliferous. Subunit B_1 has been mapped at the base of B in the easterly portions of the north and south wall and the east wall. (For example, see drawing AX6DD366.) The contacts between B and B_1 are highly gradational.

Unit B is in turn underlain by a thin, relatively discontinuous but laterally extensive limestone, designated unit C. This limestone is light gray, well indurated, and exhibits conchoidal fracture. It is continuous in the west end of the south wall but becomes discontinuous east of station 0 + 80. East of station 3 + 65, the limestone becomes a series of small, irregular discontinuous pods at varying elevations. (See drawing AX6DD367.) Where exposed in the north, east, and west walls, the limestone forms discontinuous lenses at a relatively consistent elevation. It averages about 1 ft in thickness and dips slightly to the east, being present at about el 127 to 128 ft at the west end of the auxiliary building and 125 ft at the east end.

During excavation of the auxiliary building basement, the irregularity of portions of unit C led to a special study to determine whether the irregularities could be related to fault offset. The concern was that lenses and pods of the limestone occurring at slightly different elevations might have been offset from one another. The study focused on an area of the south walls at station 2 + 80 and the north wall at station 1 + 70. (See drawings AX6DD365 and AX6DD367.) As both excavation and mapping of stratigraphically lower units progressed, it became very evident that the irregularities of unit C were due to processes other than faulting. The continuity of the lower units in the areas of interest precluded the possibility of fault offset. A report was prepared(4) which concluded that the only plausible explanation for the observed irregularities was a combination of erosional and depositional processes.

Underlying the limestone of unit C is medium gray, highly fossiliferous, sandy to silty marl, designated unit D. This zone, averaging 8 ft in thickness, is continuous around the walls of the auxiliary building excavation. The lithology of unit D is very uniform and its upper and lower contacts are quite sharp. An abundance of pelecypods retaining both valves characterizes this unit. Near the base, a number of very hard, lime-cemented pods and lenses are present at roughly equivalent elevations and have highly gradational contacts with the surrounding marl. These pods and lenses are believed to represent accumulations of calcium carbonate cement leached from the surrounding fossiliferous marl. They are collectively considered to be a subunit of D, designated D_1.

Unit E underlies D and is a thin, relatively continuous impure limestone. It is light gray, very well indurated, and fossiliferous. It averages 1 ft in thickness and varies in elevation from 121 ft in the northwest corner of the auxiliary building to 116 ft in the southeast corner. Locally, unit E is difficult to distinguish from D_1. This is seen in the north wall between stations 1 + 40 and 1 + 70 (drawing AX6DD365) where E is discontinuous and D_1 is represented by some fairly continuous lenses. In these cases unit E is arbitrarily selected as the unit displaying the sharpest contacts with surrounding units, and the one stratigraphically in between the overlying unit D and underlying unit F. The similarity between portions of E and D_1 suggests that both may be cemented deposits resulting from leaching and redeposition of calcium carbonate from the overlying fossiliferous deposits. The relative continuity of E indicates a basic permeability change occurring at that horizon in the geologic past. This is a basis for differentiating the overlying unit D from the underlying unit F.

Unit F, like D, is a fossiliferous marl, which is continuous around the basement excavation walls. It is medium gray, sandy to silty, and varies in thickness from 1 to 4 ft. It is dense and well consolidated but poorly cemented and tends to dessicate upon exposure to the atmosphere. Unit F includes some cemented limy pods similar to D_1. These have gradational contacts with the surrounding material and appear to be secondary in origin.

Unit G is light to dark gray laminated marl, which is present locally as lenses interfingering with units F and H. It is relatively continuous in the westernly portion of the south wall but pinches out at station 1 + 50. It reappears between stations 1 + 85 and 2 + 25 (drawings AX6DD367 and AX6DD368) but then disappears for the remainder of the south wall. It is present in portions of the west and north walls and is absent in the east wall. The unit is characterized by very fine sinuous and discontinuous sandy laminations, scattered shell fragments, and small lenticular clay pods. It contains scattered carbonaceous lenses and is well consolidated.

Unit H underlies G and consists of massive gray marl, which is continuous around the excavation. It is dense, well consolidated, and poorly cemented. Shell fragments are sparse in the upper part of the unit but become increasingly abundant towards the base. Unit H varies in thickness from 1 to 6 ft.

Unit I underlies H and is similar to unit E. It is a thin, relatively continuous light gray impure limestone, which is generally less than 1 ft thick. It is continuous around the excavation walls with the exception of the east wall between station 0 + 79 and the south end of the wall where it is absent.

Unit J, the deepest marl unit exposed in the auxiliary building excavation, consists of medium gray, massive, fossiliferous marl similar to the stratigraphically higher units D and F. It is continuous around the excavation walls with the exception of the east end of the excavation where the upper contact of the unit dips beneath the base of the excavation.

From the preceding descriptions it is seen that the portion of the marl section exposed in the auxiliary building excavation represents cycles of fossil abundance and absence, interspersed with the formation of secondary limestone pods and lenses as a result of leaching of calcium carbonate from fossiliferous zones. Erosional and depositional processes have combined to create some of the interfingering of units as well as irregularity of some of the limestone layers.[4]

The upper contact of the Lisbon Formation was exposed around the perimeter of the power block excavation because it exists at an elevation higher than the top of the more localized auxiliary building excavation. The top of the Lisbon Formation corresponds with the top of the Blue Bluff marl. This upper contact was examined in detail and surveyed. It varies from a high elevation of 138.6 ft on the north side of the excavation to a low of 132.0 ft on the south side. The contact is erosional with very minor relief present. The uppermost few feet of the marl is locally weathered to a greenish color, and bioturbations (disturbance of the sediment due to the activity of organisms) were noted locally.

2B.3.3.3.3 Barnwell Group

Deposits of the upper Eocene Barnwell Group overlie the Blue Bluff marl of the Lisbon Formation and include all of the sediments exposed in the side slopes of the power block excavation. The contact between Barnwell Group and Lisbon Formation deposits is a disconformity, representing a hiatus in the depositional history of the site.

As mentioned previously, four distinct units within the Barnwell deposits have been recognized and are described in this section. These units include, from oldest to youngest: The Utley Limestone Member, the Twiggs Clay Member, the Irwinton Sand Member, and the Tobacco Road Sand Member. These units are illustrated in the stratigraphic column in figure 2.5.1-5.

Although examined and described in detail, the deposits between the top of the Blue Bluff marl and approximate el 170 ft could not be mapped in detail. (See drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363.) Consequently, the geologic map of the power block excavation (drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363) shows only the detailed lithology of the Tobacco Road Sand and the upper portion of the Irwinton Sand. This was due to extensive slumping of the slopes when excavation and dewatering were suspended during the period between September 1974 and June 1976. Extensive regrading obscured the contacts between units in this zone. Several portions of the slopes were covered with riprap in order to control seepage and improve stability, thereby further obscuring contacts. Since seepage from the slopes was creating local stability problems, it was decided not to excavate back into the slopes to expose contacts and risk large slope stability problems. Detailed mapping of the units above and below this zone demonstrated the continuity of the strata and the absence of faulting.

The lowermost exposed unit within the Barnwell Group in the power block excavation is the Utley Limestone. The lower part of the limestone is grayish-yellow, well indurated, and fossiliferous, grading locally into coquina. It is continuous around the power block excavation and varies in thickness from 0.5 to 3 ft. The upper part of the limestone is white to light gray, varies from 0 to 12 ft in thickness, and is present only in the north and northwest portions of the power block excavations. Although well indurated, this thicker limestone has been subjected to extensive leaching, producing a honeycomb network of cavities. Some individual cavities had mean diameters of several feet before being removed by excavation or filled in place. The filling of cavities in the limestone intersected by the excavation slopes is described in paragraph 2.5.4.5. Within the cavities, the limestone typically displayed a weathered and soft zone immediately adjacent to the cavity walls, which graded within a few inches to hard, unweathered limestone. Locally, extensive leaching of the limestone left a residue of silt and clay impurities, forming a soft mottled blackish material. Included in the Utley Limestone is a highly fossiliferous clay deposit which varies in color from tan to dark gray. The difference in colors appears to be due primarily to weathering effects. Prior to its removal, this clay was present mainly in the northwest portion of the power block excavation. It contains abundant specimens of the oyster <u>Crassostrea gigantissima</u>, a key Eocene near-shore pelecypod. Lesser quantities of other pelecypods, gastropods, arthropod arts, and shark teeth have been identified in this clay.

Unconformably overlying the Utley Limestone is the Twiggs Clay. This consists primarily of medium gray, moderately indurated, laminated sandy claystone, which is quite similar to the underlying Blue Bluff marl of the Lisbon Formation. The Twiggs Clay is only present in the southeast portion of the power block excavation and varies in thickness from 0 to 13 ft. The upper 2 to 5 ft are weathered to a distinctive greenish-yellow color. The Twiggs Clay has alternating thin and thick beds (from less than 1 in. to greater than 1 ft), with gradational contacts between beds. No joints, fractures, or discontinuities were observed in the clay.

The Irwinton Sand of the Barnwell Group unconformably overlies the Twiggs Clay in the southeast portion of the power block and overlies the Utley Limestone elsewhere. The Irwinton Sand consists of an approximately 50 ft thick vertical sequence of sands, clays, and reef deposits. At the base of the sequence is a massive, white, quartz-rich sand deposit. The presence of fossil shrimp burrows identifies this as an intertidal deposit. The upper surface of the sand is highly irregular, with reef- type accumulations of Crassostrea gigantissima present on the highs. These shell accumulations are well cemented and highly calcareous. This sand is fine- to medium-grained and very well sorted, and it exhibits extensive crossbedding. It is extremely friable and tends to rapidly slump and ravel, assuming its angle of repose soon after excavation.

Above the white sand and reef deposits is a sequence of tan sand and clay. The sand is generally fine to medium and moderately sorted, and it contains thin seams of tan clay having high plasticity. Two continuous marker horizons are present within this sequence. The first, a zone of manganese-staining and shell debris, occurs generally between el 170 ft and 180 ft and is somewhat higher than this on the west side of the excavation. This zone, called the shell hash horizon, varies in thickness from less than 1 in. to almost 6 ft and can be traced continuously around the excavation slopes (drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363). A second shell hash horizon is locally present beneath the first one but is discontinuous. The second marker horizon is a zone of abundant tan clay seams, which varies from approximately 1 ft to almost 6 ft in thickness, and is found between el 180 ft and 200 ft. This clay zone marks the top of the Irwinton Sand.

Both of the marker horizons undulate along the strike, with flexures in the bedding reflecting underlying reef highs as well as lows due to collapse of cavities in the stratigraphically lower Utley Limestone. These flexures are discussed further in paragraph 2.5.1.2.3.4.

Above the Irwinton Sand is the Tobacco Road Sand of the Barnwell Group. This sand extends up to the top of the excavation slopes and consists of a thick (up to 40 ft) zone of predominately red sand with zones of lavender, purple, mustard yellow, and orange sand. The color changes are due to weathering effects and are not related to structure of lithology. The sand consists of fine to medium quartz grains which are moderately to well sorted and angular to subrounded. Colors are imparted by clay coatings on the individual grains. Differential weathering has produced mottled zones of bright colors which form an alligator-skin effect near the top of the unit. The sand is dense, well consolidated, and completely uncemented.

At the top of the excavation slopes, recent deposits of buff-colored, alluvial, and windblown sand are present locally. These deposits form a thin veneer of fine- to medium-grained, angular to subangular, well-sorted quartz sand which is highly gradational with the underlying sand of the Barnwell Group.

2B.3.3.4 Structure

The sedimentary sequence exposed in the power block excavation is flat lying, although many of the units within the Barnwell Group are discontinuous and highly variable in thickness. The regional dip is to the southeast at about 30 ft to the mile.

A complete absence of faulting of the sediments is demonstrated by the continuity of the upper surface of the Blue Bluff marl (Lisbon Formation), as well as of the two marker horizons in the Irwinton Sand discussed in the preceding section. Continuity of subunits of the marl mapped on the walls of the auxiliary building basement excavation give added proof of absence of faulting.

Only one joint was recognized in the entire power block excavation and this was only of limited extent. The joint was exposed in the southeast corner of the power block excavation where it extended from the upper surface of the marl down to el 127 ft, where it terminated at a depth of about 6 ft. The joint trended N 81° E, was approximately vertical, and was tightly closed. Some secondary dark green mineralization was noted as a fine coating on the joint faces. No other joints or fractures were identified in any other lithologic units.

The strata of the Irwinton Sand exhibit flexures that are related to the following phenomena:

- Differential compaction of the underlying sediments.
- Subsidence due to leaching of underlying calcerous materials and collapse of solution cavities.
- Deposition of an uneven surface.

These flexures are particularly evident in the two marker horizons of the Irwinton Sand.

Evidence for differential compaction includes the typical association of highs in the marker horizons, with the occurrence of underlying reef deposits of shells. These cemented shell deposits form hard spots compared to the relatively more compressible sediments between them. As the load of the overburden increased during deposition, the sediments between the reefs were compressed relatively more than those immediately above them. This created downward flexures in the overlying marker horizons between the shell deposits.

Evidence for subsidence due to leaching and collapse of cavities includes the presence of solution cavities in the limestone underlying zones of downwarping of the relatively incompetent sands and clays above the cavities. Recent surface depressions in the area give added evidence for subsidence. Leaching and cavity formation are confined to the limestone above the marl. No cavities have been found to exist in the marl, and the upper surface of the marl contains no depressions where exposed in the power block excavation.

2B.3.3.5 Ground Water Conditions

Ground water conditions at the VEGP site are discussed in subsection 2.4.12. The Blue Bluff marl of the Lisbon Formation forms an aquiclude between the shallow water table aquifer and the deeper confined regional aquifer. Since the excavation for the power block bottomed in the marl, only the shallow water table aquifer was encountered in the excavation.

The original water table elevation in the power block area prior to excavation was approximately 160 ft. When excavation had progressed to this level, seepage was encountered and temporary construction dewatering initiated. Dewatering methods employed are discussed in paragraph 2B.3.4.3. As excavation progressed downwards, seepage continued from the side slopes. The seepage was from water perched on clay seams in the Irwinton Sand. This water continued to seep out of the slopes even though the water surface was drawn down elsewhere in the excavation. Locations of some of the more prominent ground water seeps are shown on drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363.

In September 1974 the VEGP project was postponed indefinitely and the dewatering effort was terminated. Seepage continued into the excavation, which had bottomed generally above el 140 ft, and the water level rose in the excavation to about 155 ft where it remained for the next 2 years. In June 1976 the project was restarted and the excavation dewatered once more. As excavation progressed through the cavernous Utley Limestone above the marl, significant flows of ground water were encountered from cavities requiring special localized dewatering procedures. Slight artesian conditions were encountered locally within the limestone.

When excavation had extended into the marl and perimeter drainage had been provided, work progressed under dry conditions. The marl contains no free ground water, and no springs were observed which might indicate a hydraulic connection with the deeper artesian aquifer below the marl.

Seepage from the side slopes of the power block excavation continued during this period with gradual decline in the elevation of the top of the seepage zone. The zone of seepage was effectively obscured when the side slopes were lined with a blanket of riprap up to el 160 ft. Temporary construction dewatering was continued throughout a significant portion of the construction period.

2B.3.4 EXCAVATION AND FOUNDATION CONSIDERATIONS

2B.3.4.1 <u>General</u>

The excavation for the power block structures for Units 1 and 2 at the VEGP site is roughly square in shape, with two access ramps exiting from the southeast and southwest corners of the excavation. It measures approximately 1400 ft on an edge at the top and 1000 ft on an edge at the toe. The side slopes were cut a gradient of 2:1. The total excavated volume in the power block was approximately 5 million yd^3 including the access ramps.

The original ground surface in the power block area varied from an elevation of about 200 ft to slightly over 230 ft. The major portion of the excavation bottomed in the marl layer at approximately el 130 ft.

Within this larger excavation, a deeper localized excavation was made for the auxiliary building basemat. This consisted of a rectangular area measuring approximately 120 ft by 440 ft. The base of this excavation was at el 108.6 ft, and the walls were cut vertically with a horizontal bench at el 118 ft. The other major power block structures are founded primarily on structural backfill at elevations above the floor of the excavation.

The excavation is shown in plan view on drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363. These drawings show only the access road at the southeast corner since the one in the southwest corner was graded after geologic mapping had been performed on the slopes as shown in the drawings.

Excavation work was started in May 1974 and postponed on September 12, 1974. The bottom elevation of the excavation averaged approximately 145 ft at this time and close to 900,000 yd^3 of excavation remained. The excavation work resumed by February 1977, and the auxiliary building excavation was bottomed out in October 1977.

2B.3.4.2 Excavation Procedures

Excavation work started and progressed very rapidly in the upper sands above the water table at el 160 ft. A large fleet of bulldozers and scrapers was assembled for the job. Very little, if any, ripping was required because of the sandy nature of the deposits; and progress was extremely fast, attaining a maximum rate of 120,000 yd^3/day at the peak of activity. Upon reaching the water table, excavation progress was significantly slowed because of the tendency of the equipment to mire in the saturated sands. At this point construction dewatering was begun. The procedures utilized for dewatering are discussed in the following section, but the general approach consisted of trenching a system of parallel ditches to permit drainage from the area between the ditches. Once dry, these areas would be excavated by bulldozers and scrapers while the ditches were progressively deepened to maintain dry conditions between the ditches. Excavation below water in the ditches was accomplished by means of two draglines.

When the excavation reached the zones of hard Utley Limestone described earlier, limited blasting of the rock was utilized to facilitate its removal. Since the limestone to be removed directly overlaid the marl, which was to form the foundation for structural backfill, it was necessary to control the blasting in such a manner as to protect the underlying marl from damage.

First, the stipulation was made that the contractor not use explosives if conventional methods could be used, even if some difficulty resulted. Further, the use of explosives would be discontinued in any case, if, in the opinion of the engineer, the marl might be damaged as a result of blasting. Blast holes were not permitted to penetrate lower than el 135 ft, and a minimum stem of 18 in. was recommended below the charge in each hole. It was recommended that no blast holes exceed 3 in. in diameter and that the maximum charge weight should not exceed 30 lb per delay. The maximum allowable powder factor was set at 1 lb/yd^3.

Because of the concern for protecting the marl, only very limited blasting of the limestone was performed. The major portion of the rock was removed by first breaking it with a hydraulic ram mounted on a backhoe, then loading it out with conventional equipment.

Excavation of the marl was accomplished by ripping, followed by conventional earth moving. The auxiliary building basement excavation was cut with bulldozers and front-end loaders.

Trimming of the walls was accomplished with a backhoe. Some of the hard, indurated limestone layers within the marl described in paragraph 2B.3.3.3.2 were first broken with the backhoe-mounted hydraulic ram, then removed by front-end loader. Fine grading of the floor of the power block was accomplished with motor graders in areas underlying future structural backfill and with Gradalls in the nuclear service cooling water tower foundation areas. In the foundation areas, shovels and air hoses were used for cleanup of loose material.

2B.3.4.3 Construction Dewatering

The construction dewatering system utilized in the power block excavation consisted of a system of east-west dewatering ditches connected by a north-south ditch leading to a sump and pumping plant in the southwest corner of the excavation. Because of the low permeability of the deposits, the dewatering consultant, Mr. R. Y. Bush, decided that a conventional well-point system would be ineffective, hence the ditch and sump approach.(5) This scheme proved to be successful when the invert elevation of the ditches was maintained 15 to 20 ft below the adjacent grade. This permitted conventional procedures in reasonably dry materials.

Upon reaching the marl, the system of ditches and sump was replaced by a perimeter drainage system as shown on drawing AX6DD324. This consisted of a buried porous concrete pipe around the perimeter of the power block excavation feeding into three small sumps at the toe of the south slope. Water pumped from the sumps was discharged to debris basin No. 1 southeast of the power block. The buried porous concrete pipe was encased in a granular filter material which was carried up the surface of the adjacent 2:1 slope to about el 160 ft. This filter blanket was placed so that there was a minimum of 4 ft of filter material measured horizontally from the face of the slope out to the face of the filter blanket. (See drawing AX6DD324.)

This dewatering scheme proved to be entirely successful and construction in the marl layer was able to proceed under totally dry conditions.

2B.3.4.4 Slope Protection

During the early stages of excavation, intense rainfall of short duration caused severe erosion of the 2:1 side slopes of the power block excavation. The uncemented sands rapidly washed out forming deeply incised gullies in some areas. These gullies were backfilled with the native soil material and local areas of the slope regraded. One such area is seen on the geologic map (drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363) in the upper part of the east slope between stations N83 + 00 and N84 + 00. Another larger area exists in the south slope of the access ramp each of station E100 + 00. After regrading the eroded areas, berms were constructed around the tops of the slopes to control runoff. The surfaces of the slopes were sprayed with the chemical stabilizing agent Petroset, a colorless liquid that sets up and tends to bond the sand grains together. These measures proved to be reasonably successful in controlling further erosion.

After the resumption of excavation work in 1977, erosion problems further down the slopes were encountered due to seepage of the perched ground water out of the slopes. Since stabilizing agents were expected to be ineffective under these conditions, the lower portions of the slopes were blanketed with riprap to improve stability. The riprap was subsequently covered with a finer grained filter transition material.

Where the 2:1 slopes intersected the cavernous limestone deposit, several cavities of varying sizes were exposed in the slopes. The largest of these existed in the northwest corner of the power block and had an opening measuring 10 ft by 10 ft. This cavity extended back into the slope some 30 ft before narrowing down to a small size. Other small cavities were encountered at varying intervals all along the north side of the power block excavation. It was necessary to fill these cavities so that an effective buttress would be formed against which the future structural backfill could be placed and compacted. This consideration did not require complete filling of the cavities since a prism of fill material placed in the entrance and extending some distance into the cavity would provide an unyielding mass against which the structural backfill could be placed. The cavities were first cleaned of loose debris, then backfilled with crushed rock (Georgia State Standard No. 467). The crushed rock was packed into the cavities by means of a 20-ft-long ram attached to the blade of a bulldozer. This method proved to be very successful and actually resulted in the crushed rock being forced into small crevices, effecting an essentially complete filling of some of the cavities. The large cavity in the northwest corner was effectively filled in this manner. From the volume of crushed rock forced into the cavity, it was estimated that the cavity was completely filled to at least a distance of 25 ft back of the entrance.

To retard erosion of temporary slopes in Category 1 backfill placed in the power block excavation, these slopes were sprayed with a commercial compound known by the trade name Glassroot. It consists of a glass fiber material which was sprayed onto the slope and then coated with a film of asphalt emulsion. This proved to be effective in controlling erosion of the compacted sandy backfill but only for a limited period of time. By late 1979, the glassroot/asphalt coating began to show signs of excessive deterioration. Consequently, the slopes were stripped clean and recoated with gunite, which proved to be more durable and easier to maintain.

2B.3.4.5 Foundation Cleanup and Protection

As mentioned previously, the marl at final grade in foundation areas was exposed using either a motor grader or Gradall. Loose material was then removed by shovel, broom, and airhose. On the vertical walls of the auxiliary building excavation, final trim to neat line was accomplished with a backhoe followed by pick and shovel and airhose techniques.

In all cases where final grade was exposed and cleaned off, the marl surface had to be covered in a manner approved by the geologist within 24 h of exposure. On horizontal surfaces the marl was covered either by structural backfill or by mudmat concrete depending upon whether the particular area exposed was in a foundation or backfill area. The vertical walls of the auxiliary building basement excavation were coated with a 4-in.-thick layer of gunite reinforced with welded wire mesh.

In some cases temporary covers such as loose soil or plastic sheeting were employed when the permanent cover material could not be applied within the 24-h limit. In all cases the temporary cover procedure was approved by either the geologist or the Georgia Power Company inspector. Before placing the permanent cover material in any foundation area, the marl was inspected and approved by the geologist or soil engineer in accordance with the procedures described in the following section.

2B.3.4.6 Foundation Inspection and Approval Procedures

All areas of marl exposed and cleaned off in preparation for placement of concrete or backfill were examined closely for any evidence of loose or soft zones or geologic discontinuities. After confirming the absence of such features, the inspecting geologist documented the approval of the area on field foundation approval forms. These field approval forms were transmitted to the Georgia Power Company site personnel for permanent retention. At intervals, the forms were countersigned by the supervising geologist and soil engineer for the project.

In addition, photographs of the foundation areas were taken. These were logged and transmitted to Georgia Power Company for permanent retention in the field office. The documentation described in this section has been transferred to SNC as the exclusive operating licensee.

2B.3.4.7 Foundation Testing

As a part of the general marl geologic mapping and inspecting functions, it was decided to carry out a program of coring and testing samples of the marl to confirm the material properties used for design. It was desired to obtain samples for record purposes. The coring and sampling operation was performed under the direction of the geologist and inspector, and the test assignments were made by the soils engineer.

A total of 38 core holes was drilled by rotary methods in the floor of the power block excavation at locations selected by the geologist. The hole locations are shown on drawings AX6DD360, AX6DD361, AX6DD362, and AX6DD363. The marl was cored to depths between 4 and 11 ft beneath the ground surface. Four-in.-diameter core samples were obtained, labeled, and placed in wooden boxes for permanent storage at the site. Samples for testing were selected by the geologist. These were then wrapped in cellophane, sealed with wax, and placed in special boxes for transportation to the laboratories of Law Engineering Testing Company in Atlanta.

A total of 31 core samples was tested for moisture content, bulk unit weight, unconfined compressive strength, and shear strength from one-point unconsolidated-undrained triaxial shear tests. The average wet unit was found to be 105.6 lb/ft^3, while the average moisture content was 36.2 percent. The average deviator stress at failure in the strength tests was 39.14 k/ft^2 (272 psi). A complete summary of test results is found in appendix 7 of reference 1. The results obtained are in the range anticipated and show that the marl is a competent foundation material.

2B.3.4.8 Foundation Rebound Monitoring

In order to monitor the rebound occurring in the Blue Bluff marl layer as a result of removal of approximately 100 ft of the overlying materials, the specialist firm of Goldberg, Zoino, Dunnicliff Associates was commissioned to provide in situ instrumentation. A total of nine heave points was installed at the bottom of drill holes made for this purpose. The heave points were installed at the locations shown in figure 2B-1, between approximate el 104 and 126 ft.

Throughout the excavation period, elevation changes of the heave points were surveyed. The measured heave is summarized in the table below:

	Period		
Measured Heave Point No.	From	To	(in.)
1	6/22/74	8/07/77	1.1
2	6/16/74	6/22/76	1.4
3	6/16/74	10/02/74	0.6

5	6/16/74	2/26/77	1.7
7	6/16/74	6/05/77	1.2
9	6/30/77	8/07/77	1.5

More complete data is presented in appendix 6 of reference 1.

The measured heave was substantially less than that predicted.

2B.3.5 CONCLUSIONS

The detailed geologic mapping of the strata exposed in the power block excavation at the VEGP site has better defined the structure and stratigraphy of this area. A much more comprehensive and detailed picture of the site geology has emerged as a result of this effort. The general conclusions of the PSAR(2) have been confirmed.

Two separate marker horizons in the Irwinton Sand have been mapped around the side slopes of the power block. Both horizons are continuous and unbroken, demonstrating the absence of faulting in these materials. The upper contact of the stratigraphically lower Blue Bluff marl of the Lisbon Formation has been mapped with survey accuracy and has also been found to be uninterrupted by offsets. Subunits within the marl have been mapped around the walls of the auxiliary building basement excavation. These zones were likewise found to be undisturbed by faulting. Minor stratigraphic irregularities noted were shown to be related to erosional and depositional processes.(4)

Surface depressions and subsidence features mapped in the upper sands were found to be related to collapse of solution cavities in the underlying limestone. Detailed examination of the exposed marl and surveying of its upper surface configuration has shown that the marl is free of solution cavities such as those present in the overlying limestone. The marl was found to contain no freely draining water and its function as an aquiclude was confirmed. Where the side slopes of the power block intersected solution cavities in the limestone layer, these cavities were backfilled with crushed rock to provide a firm buttress against which structural backfill could be placed and compacted. The back-filling of the cavities was inspected and found to be adequate.

Areas of the marl exposed at final grade were inspected, approved, and protected in an adequate manner as described in the report. All foundation areas inspected were found to expose sound competent marl suitable for supporting the backfill and plant structures.

The coring and testing of the marl at selected locations in the power block yielded results which confirm the design parameters used. Results of the rebound monitoring program showed that the measured rebound was less than that predicted, giving additional evidence of the competency of the marl.

The results of the geologic work described in this report lead to the conclusion that the VEGP site is suitable for design and construction of a multiple-unit nuclear generating plant. No geologic hazards were found to exist that might affect safety and licensing considerations.

2B.3.6 REFERENCES

1. Bechtel Power Corporation, Report on Backfill Material Investigations, two volumes, January 1978.

2. Georgia Power Company, Alvin W. Vogtle Nuclear Plant Preliminary Safety Analysis Report, chapter 2.

3. Huddleston, Paul, Georgia Geological Survey, Personal Communication, 1978.

4. Bechtel Power Corporation, Report on Stratigraphic Irregularities Exposed in the Auxiliary Building Excavation, February 1978.

5. Bush, R. Y., Consulting Engineer, Dewatering Study - Alvin W. Vogtle Nuclear Plant, January 12, 1973. (See also subsequent correspondence.)

2B.3.7 BIBLIOGRAPHY

Bechtel Power Corporation, Interim Report of Geologic Conditions –Power Block Excavating, October 23, 1974.

TABLE 2B-1 (SHEET 1 OF 36)

DRILL HOLE SUMMARY

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
1[a]		91.5	98	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
2[a]		350	206	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
3[a]		220	230	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
5[a]		230	220	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
6[a]		210	198	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
7[a]		150	119	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
8[a]		225	218	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
9[a]		190		Preliminary investigation	Standard pen ASTM and rotary tricone	None	
10[a]		150	186	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
11[a]		190	215	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
13[a]		180		Preliminary investigation	Standard pen ASTM and rotary tricone	None	

TABLE 2B-1 (SHEET 2 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
14[(a)]		140		Preliminary investigation	Standard pen ASTM and rotary tricone	None	
15[(a)]		240		Preliminary investigation	Standard pen ASTM and rotary tricone	None	
12	N 1,150,535 E 620,169	240	263	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
16	N 1,148,930 E 617,946	200	233.7	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
17	N 1,144,080 E 620,200	280	264	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
18	N 1,143,403 E 621,922	225	222	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Carbonate solubility test
19	N 1,142,368 E 624,484	200	219	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Carbonate solubility test
19A		115	219	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
20	N 1,141,804 E 626,176			Preliminary investigation	Standard pen ASTM and rotary tricone	None	
21	N 1,139,810 E 621,875			Preliminary investigation	Standard pen ASTM and rotary tricone	None	Soil solubility test
22	N 1,141,735 E 622,001	260	253	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
23	N 1,142,357 E 620,273	220	215	Preliminary investigation	Standard pen ASTM and rotary tricone	None	

TABLE 2B-1 (SHEET 3 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
24	N 1,142,850 E 623,092	220	216	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Observe well/ gamma logged/ soil solubility test/ carbonate solubility test
25	N 1,142,600 E 627,657	220	230	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
26	N 1,140.963 E 629,199	200	203	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Observe well
27	N 1,143,622 E 627,931	190	210	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Observe well/ gamma logged
28	N 1,143,609 E 628,634	170	85	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
29	N 1,144,977 E 626,392	210	193	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Observe well/ gamma logged
30	N 1,145,072 E 626,534	85	91	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
31	N 1,143,764 E 625,237	210	211	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Observe well/ X-ray diffraction
32	N 1,144,784 E 623,572	210	214	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Observe well
33	N 1,146,834	220	238	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Observe well/ gamma logged
34	N 1,147,180 E 624,846	115	86	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Observe well
35A		70	94.4	Preliminary investigation	Standard pen ASTM and rotary tricone	None	

TABLE 2B-1 (SHEET 4 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
36A		70	98.3	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
36B		150	98.4	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
37	N 1,145,242 E 622,690	210	195	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Soil solubility test/ X-ray diffraction/ paleo analysis
38	N 1,143,474 E 619,772	270	257	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
39	N 1,149,703 E 622,835	90	118	Preliminary investigation	Standard pen ASTM and rotary tricone	None	X-ray diffraction
40	N 1,143,210 E 621,759	250	215	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
41	N 1,142,049 E 628,658	120	222.8	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
42	N 1,143,392 E 623,553	250	210	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Paleo analysis carbonate solubility test
42A	N 1,143,380 E 623,535	150	210.6	Hydrologic data	Standard pen ASTM and rotary tricone	None	Observe well – 150 ft
42B	N 1,143,386 E 623,544	130	210.4	Hydrologic data	Standard pen ASTM and rotary tricone	None	Gamma logged/ observe well - 130 ft
42C	N 1,143,398 E 623,563	90	210	Hydrologic data	Standard pen ASTM and rotary tricone	None	Gamma logged/ observe well - 90 ft
42D	N 1,143,403 E 623,571	70	209.7	Hydrologic data	Standard pen ASTM and rotary tricone	None	Observe well - 70 ft

TABLE 2B-1 (SHEET 5 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
42E	N 1,143,408 E 623,580	55	209.6	Hydrologic data	Standard pen ASTM and rotary tricone	None	Observe well – 55 ft
43	N 1,144,314 E 621,810	55	282.8	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
44	N 1,146,517 E 623,911	90	241.3	Preliminary investigation	Standard pen ASTM and rotary tricone	None	
45	18,300 NE of 36	370	273.52	Preliminary investigation	Standard pen ASTM and rotary tricone	None	Gamma logged/ paleo analysis
101	N 1,142,945 E 623,518	100	210.8	Plant foundation	Standard pen ASTM and rotary tricone	None	
101A	N 1,142,950 E 623,515	100	210.6	U.D. samples for reactor foundation	Rotary tricone, Denison, and Shelby	3-in. Denison-16 3-in. Shelby-12	Observe well to 200 ft
101B	20 ft North of 101	65	210.8	Bulk sample (100 lb)	24-ft auger	None	
102	N 1,142,796 E 623,727	200	211.5	Plant foundation	Standard pen ASTM and rotary tricone	None	
102A	Adjacent to No. 102	177	211.5	Plant foundation	Standard pen ASTM, Denison, and rotary tricone	Denison-15	
103	N 1,142,796 E 623,927	100	212.4	Plant foundation	Standard pen ASTM	None	
104	N 1,143,184 E 623,398	100	217.1	Plant foundation	Standard pen ASTM and rotary	None	
104A	Adjacent to No. 104	200	217.1	U.D. samples for reactor foundation	, ASTM, Shelby, and Denison	Shelby-5 Denison-6	Undisturbed samples from 100 ft to 200 ft standard pen ASTM with intermittent Denison samples

TABLE 2B-1 (SHEET 6 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
105	N 1,142,996 E 623,626	300	209.7	Reactor foundation	Standard pen ASTM and rotary tricone	None	
106	N 1,142,996 E 623,726	150	209.6	Plant foundation	Standard pen ASTM and rotary tricone	None	
107	N 1,142,996 E 623,876	300	209.4	Reactor foundation	Standard pen ASTM and rotary tricone	None	Gamma logged
107A	N 1,142,999 E 623,891	300	209.4	U.D. samples for reactor foundation	Rotary tricone, and Denison	Denison-20	
107B	20 North of107	65	209.4	Bulk sample (100 lb)	24-in. auger	None	24-in.-diameter bucket auger holes drilled adjacent to existing logged holes solely for the purpose of obtaining bulks oil samples from specific depths for testing
108	N 1,142,996 E 624,026	100	210.2	Plant foundation	Standard pen ASTM and rotary tricone	None	
109	N 1,143,405 E 623,357	200	216	Plant foundation	Standard pen ASTM and rotary tricone	None	
110	N 1,143,385 E 623,504	100	213.5	Plant foundation	Standard pen ASTM and rotary tricone	None	
111	N 1,143,256 E 623,726	200	207.2	Turbine foundation	Standard pen ASTM, Denison, Shelby, and rotary tricone	Denison-4 Shelby-10	
111A	Adjacent to111	142	207.2	Turbine foundation	Rotary tricone and Denison	4-in. Denison-3	
112	N 1,143,256 E 623,876	100	204.3	Turbine foundation	Standard pen ASTM and rotary tricone	None	

TABLE 2B-1 (SHEET 7 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
113	N 1,143,256 E 624,026	200	203.1	Turbine foundation	Standard pen ASTM and rotary tricone	None	
114	N 1,143,504 E 623,526	199	212	Plant foundation	Standard pen ASTM, Denison, Shelby, and rotary tricone	Denison-4 Shelby-10	Carbonate solubility test
114A	Adjacent to114	155	212	Plant foundation	Standard pen ASTM, Denison, and rotary tricone	4-in. Denison-3	
115	N 1,143,506 E 623,726	100	208.5	Plant foundation	Standard pen ASTM and rotary tricone	None	
116	N 1,143,503 E 623,928	200	208	Plant foundation	Standard pen ASTM and rotary tricone	None	Carbonate solubility test
117	N 1,143,940 E 624,343	100	197.8	Intake tunnel foundation	Standard pen ASTM and rotary tricone	None	
118	N 1,144,449 E 624,961	100	198	Intake tunnel foundation	Standard pen ASTM and rotary tricone	None	
119	N 1,144,966 E 625,639	100	117.76	Intake tunnel foundation	Standard pen ASTM and rotary tricone	None	
120	N 1,145,310 E 626,389	100	86.8	Intake tunnel foundation	Standard pen ASTM and rotary tricone	None	
121	N 1,145,467 E 626,195	200	88.8	Intake tunnel foundation	Standard pen ASTM and rotary tricone	None	Observe well to 88 ft
122	N 1,145,719 E 625,884	100	111.4	Intake tunnel foundation	Standard pen ASTM and rotary tricone	None	
123	N 1,146,101	200	89.3	Intake tunnel foundation	Standard pen ASTM and rotary tricone	None	

TABLE 2B-1 (SHEET 8 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
124	N 1,141,896 E 623,527	200	260.2	Cooling tower foundation	Standard pen ASTM and rotary tricone	None	Observe well to top of marl/gamma logged
125	N 1,142,156 E 624,027	100	248.1	Cooling tower foundation	Standard pen ASTM and rotary tricone	None	
126	N 1,142,997 E 625,306	100	241.4	Cooling tower	Standard pen ASTM and rotary tricone	None	
127	N 1,144,209 E 623,176	100	199.2	Switchyard foundation	Standard pen ASTM and rotary tricone	None	
128	N 1,144,206 E 623,876	100	198	Switchyard foundation	Standard pen ASTM and rotary tricone	None	
129	N 1,143,856 E 623,576	100	215.9	Switchyard	Standard pen ASTM and rotary tricone	None	Observe well to top of marl
130	N 1,142,796 E 623,527	100	209.6	Plant foundation	Standard pen ASTM and rotary tricone	None	
131	N 1,143,256 E 623,576	100	213.6	Plant foundation	Standard pen ASTM and rotary tricone	None	
132	N 1,144,988 E 626,154	150	169.5	Intake sructure bluff slope stability	Standard pen ASTM and rotary tricone	None	
133	N 1,145,145 E 626,089	150	155	Intake sructure foundation	Standard pen ASTM and rotary tricone	None	
134	N 1,146,750 E 621,024	200	191.3	Geologic fill-in section - between B16 and B37	Standard pen ASTM and rotary tricone	None	
135	N 1,143,992 E 622,742	200	200.5	Geologic-depression investigation	Cored with NWM barrel face discharge bit -	None	Observe well to below marl/carbonate solubility test

TABLE 2B-1 (SHEET 9 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
136	N 1,142,996 E 623,849	300	209.5	Geologic-center of reactor	Cored with4 x 5 ½ barrel F-D bit	None	Gamma logged/ carbonatesolublity
137	N 1,144,839 E 622,117	200	230.6	Unit 1 geologic depression investigation	Cored with NWM barrel, F-D bit	None	Observe well to below marl/gamma logged/carbonate solubility test
138	N 1,143,000 E 622,500	99.5	225.2	Reactor fan Units 3 and 4	Standard pen ASTM and rotary tricone	None	Observe well to top of marl
138A	N 1,142,966 E 622,509	200	224.9	U.D. samples for reactor foundation	Rotary tricone, Denison, and Shelby	4-in. Denison-17 3-in. Shelby-15	
139	N 1,142,996 E 623,526	300	210.9	Geologic hole edge of reactor Unit 2	Cored with4 x 5 1/2 barrel, F-D bit	None	Gamma logged/carbonate solubility test
140	N 1,142,845 E 622,702	96	222.4	Units 3 and 4	Standard pen ASTM and rotary tricone	None	Observe well to marl - 96 ft
141	N 1,142,860 E 622,292	105	230.4	Units 3 and 4	Standard pen ASTM and rotary tricone	None	Observe well to marl - 105 ft
142	N 1,143,283 E 622,262	105	231.2	Units 3 and 4	Standard pen ASTM and rotary tricone	None	Observe well to marl - 95 ft
143	N 1,143,283 E 622,738	88.5	224.5	Units 3 and 4	Standard pen ASTM and rotary tricone	None	Observe well to marl – 88.5 ft
144	N 1,145,411 E 626,127	48.5	103.2		Standard pen ASTM and rotary tricone	None	Observe well to marl – 48.5 ft
144A	N 1,145,406 E 626,133	51	103.9	Intake structure	Rotary tricone and Denison	3-in. Denison-14	

TABLE 2B-1 (SHEET 10 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
145	N 1,142,792 E 621,063	192	218.7	Geologic - in depression	Cored with NWM barrel, F-D bit		Gamma logged/observe well to marl – 82 ft
146	N 1,142,966 E 623,750	300	209.6	Seismic shot hole	Rotary tricone	None	
147	N 1,142,975 E 622,471	300	226.2	Geologic investigation Units3 and 4	Cored with4 x 5 1/2 barrel, F-D bit	None	Gamma logged/observe well to 300 ft /carbonate solubility test
148	N 1,142,996 E 623,814	300	209	Seismic shot hole	Rotary tricone	None	
149	N 1,142,996 E 623,779	300	209.2	Seismic shot hole	Rotary tricone	None	
150	N 1,142,996 E 623,556	170	210.3	Seismic shot hole	Rotary tricone	None	
151	N 1,142,946 E 623,849	300	210.4	Seismic shot hole	Rotary tricone	None	
152	N 1,133,831 E 633,344	200	152.7	Geologic hole to complete section betweenPlant site and Griffin Landing	Cored with NWM barrel, F-D bit	None	Paleo analysis
153	N 1,143,080 E 622,128	89.5	226.2	Determine depth of bearing horizon	Rotary tricone	None	
154	N 1,142,796 E 623,849	300	209.5	Seismic shot hole	Rotary tricone	None	Gamma logged
155	N 1,143,332 E 621,470	86.7	226	Determine depth of bearing horizon	Rotary tricone	None	

TABLE 2B-1 (SHEET 11 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
156	N 1,131,584 E 642,340	260	237.7	Geologic hole - Griffin Landing	Cored with NWM barrel, F-D bit	None	
157	N 1,145,605 E 621,598	184.1	207.6	Geologic hole	Cored with NWM barrel, F-D bit	None	Filled with gravel to 149.5 ft for Packer Test
158	N 1,143,838 E 622,866	72	213	Determine depth of bearing horizon	Rotary tricone	None	
159	N 1,143,931 E 622,401	80.2	222.2	Determine depth of bearing horizon	Rotary tricone	None	
160	N 1,144,157 E 622,625	78	213.7	Determine depth of bearing horizon	Rotary tricone	None	
161	N 1,144,102 E 622,899	65	201	Determine depth of bearing horizon	Rotary tricone	None	
162	N 1,144,977 E 622,318	90	235.5	Determine depth of bearing horizon	Rotary tricone	None	
163	N 1,144,748 E 621,985	95	238.6	Determine depth of bearing horizon	Rotary tricone	None	
164	N 1,145,401 E 626,120	145	103.2	Seismic shot hole	Rotary tricone	None	Gamma logged
165	N 1,145,354 E 626,138	155	112.2	Seismic shot hole	Rotary tricone	None	
166	N 1,145,215 E 626,194	185	143.1	Seismic shot hole	Rotary tricone	None	
167	N 1,145,388 E 626,087	145	104.6	Seismic shot hole	Rotary tricone	None	

TABLE 2B-1 (SHEET 12 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
168	N 1,145,375 E 626,055	147	105.8	Seismic shot hole	Rotary tricone	None	
169	N 1,145,364 E 626,027	147	106.5	Seismic shot hole	Rotary tricone	None	
170	N 1,142,988 E 622,440	180	228.3	Packer Test	Rotary tricone	None	Packer Test
171	N 1,143,420 E 621,944	90	223.1	Deep seismic shot hole	Rotary tricone	None	
172	N 1,143,452 E 621,959	90	224.1	Deep seismic shot hole	Rotary tricone	None	
173	N 1,141,664 E 626,629	80	188.6	Deep seismic shot hole	Rotary tricone	None	
174	N 1,141,691 E 626,642	89	189	Deep seismic shot hole	Rotary tricone	None	
175	N 1,143,386 E 621,363	165	233.1	Investigate geologic anomaly	Standard pen ASTM and rotary tricone	None	Gamma logged/ observe well set to 165 ft
176	N 1,142,117 E 625,423.05	80	196.4	Water observation well	Rotary tricone	None	Observe well to 75 ft
177	N 1,143,560 E 624,865	80	213	Water observation well	Rotary tricone	None	Observe well to 80 ft
178	N 1,144,958 E 622,994	93	240.4	Water observation well	Rotary tricone	None	Observe well to 91 ft
179	N 1,144,059 E 621,779	133	274.8	Water observation well	Rotary tricone	None	Observe well to 131 ft

TABLE 2B-1 (SHEET 13 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
180	N 1,142,965 E 623,724	162	210.1	Packer Test	Rotary tricone	None	Packer Test
181	N 1,143,744 E 620,833	200	258.3	Investigate geologic anomaly	Standard pen ASTM and rotary tricone	None	Observe hole 200 ft/gamma logged
182	N 1,144,232.04 E 620,820	220	260.4	Investigate geologic anomaly	Standard pen ASTM and rotary tricone	None	
183	N 1,143,026.04 E 623,526	60	210.8	Water observation well	Rotary tricone	None	Observe well to 60 ft
184	N 1,142,996 E 623,906	65	209.4	Water observation well	Rotary tricone	None	
200	N 1,142,860 E 623,560	100	209	Auxiliary building (Aux. bldg.) foundation	Standard pen ASTM and rotary tricone flight auger, and Shelby	Shelby-2	
201	N 1,142,860 E 623,740	100	211.4	Aux. bldg. foundation	Rotary tricone, standard pen ASTM, and Shelby	Shelby-1	
202	N 1,142,710 E 623,380	155.7	215.5	Emergency cooling tower foundation	Flight auger, standard pen ASTM, Shelby, and Denison	Shelby-8 Denison-8	
203	N 1,142,730 E 623,650	154.8	210.9	Railroad plant entrance foundation	Rotary tricone standard pen ASTM, Shelby, and Denison	Shelby-8 Denison-8	
204	N 1,142,710 E 623,910	156	212.8	Emergency cooling tower foundation	Rotary tricone standard pen ASTM, Shelby, and Denison	Shelby-8 Denison-8	
205	N 1,143,310 E 623,640	100	212	Turbine foundation	Rotary tricone standard pen ASTM	None	

TABLE 2B-1 (SHEET 14 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
206	N 1,143,310 E 623,900	99.5	204	Turbine foundation	Rotary tricone standard pen ASTM	None	
207	N 1,143,220 E 624,560	100.5	212.3	Cooling tower foundation	Rotary tricone standard pen ASTM	None	
208	N 1,143,220 E 625,070	90.5	218.1	Cooling tower foundation	Rotary tricone standard pen ASTM	None	
209	N 1,143,220 E 625,586	99.4	216.2	Cooling tower foundation	Rotary tricone standard pen ASTM	None	
210	N 1,142,680 E 624,560	101	216.9	Cooling tower foundation	Rotary tricone standard pen ASTM	None	
211	N 1,142,680 E 625,070	101.5	219	Cooling tower foundation	Rotary tricone standard pen ASTM	None	
212	N 1,142,680 E 625,580	96	211.1	Cooling tower foundation	Rotary tricone standard pen ASTM	None	
213	N 1,141,670 E 623,320	131	256.1	Cooling tower foundation	Rotary tricone standard pen ASTM	None	
214	N 1,141,670 E 623,830	126	248.6	Cooling tower foundation	Rotary tricone standard pen ASTM	None	
215	N 1,141,670 E 624,340	126	237.3	Cooling tower foundation	Rotary tricone standard pen ASTM	None	
216	N 1,142,930 E 623,650	142.5	210.6	Aux. bldg. foundation	Rotary tricone standard pen ASTM, Shelby, and Denison	Shelby-7 Denison-7	
217	N 1,143,130 E 623,650	141	207.5	Aux. bldg. and turbine foundation	Rotary tricone standard pen ASTM, Shelby, Denison, and Pitcher	Shelby-5 Denison-2 and Pitcher-7	

TABLE 2B-1 (SHEET 15 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
218	N 1,143,080 E 623,330	150.8	216.7	Diesel generator (gen.) bldg foundation	Rotary tricone standard pen ASTM, Shelby, Denison, and Pitcher	Shelby-4 Denison-6 and Pitcher-2	
219	N 1,143,080 E 623,950	140.3	207.8	Diesel gen. bldg foundation	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-5 Pitcher-8	
220	N 1,143,000 E 623,400	142.5	213.2	Reactor foundation	Rotary tricone standard pen ASTM, Shelby, Denison, and Pitcher	Shelby-4 Denison-8 Pitcher-4	
221	N 1,143,220 E 623,900	136	204.8	Turbine foundation	Rotary tricone standard pen ASTM, Shelby, and Denison,	Shelby-2 Denison-13	
222	N 1,143,310 E 623,370	150.5	216.5	Turbine foundation	Rotary tricone standard pen ASTM, Shelby, and Denison,	Shelby-5 Denison-8	
223	N 1,143,340 E 624,420	140.5	206.9	Intake structure foundation	Rotary tricone standard pen ASTM, Shelby, Denison, and Pitcher	Shelby-1 Denison-8 Pitcher-4	
224	N 1,142,300 E 623,180	186	250.2	Intake structure foundation	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-6 Pitcher-13	
225	N 1,142,940 E 624,560	148.7	211.6	Cooling tower foundation	Rotary tricone standard pen ASTM, Shelby, and Denison,	Shelby-4 Denison-11	
226	N 1,142,940 E 625,070	162	218.6	Cooling tower foundation	Rotary tricone standard pen ASTM, Shelby, and Denison,	Shelby-5 Denison-12	

TABLE 2B-1 (SHEET 16 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
227	N 1,142,940 E 625,580	150.7	209.5	Cooling tower foundation	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-6 Pitcher-12	
228	N 1,141,930 E 623,320	201	260.5	Cooling tower foundation	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-2 Pitcher-20	
229	N 1,141,930 E 623,830	203	255.5	Cooling tower foundation	Rotary tricone standard pen ASTM, Shelby, and Denison,	Shelby-8 Denison-12	
230	N 1,141,930 E 624,340	192	243	Cooling tower foundation	Rotary tricone standard pen ASTM, and Denison,	Denison-18	
231	N 1,142,200 E 623,320	124.7	254.2	Cooling tower foundation	Rotary tricone and standard pen ASTM	None	
232	N 1,142,200 E 623,830	125.3	250.1	Cooling tower foundation	Rotary tricone and standard pen ASTM	None	
233	N 1,142,200 E 624,340	105.5	229.8	Cooling tower foundation	Rotary tricone and standard pen ASTM	None	
234	N 1,143,760 E 621,970	105.5	245.5	Administration bldg. foundation	Rotary tricone, standard pen ASTM, and Shelby	Shelby-2	
235	N 1,143,650 E 624,450	135.5	206.2	Shops and warehouse foundation	Rotary tricone standard pen ASTM, Shelby, Pitcher, and Denison	Shelby-5 Pitcher-8 Denison-1	
236	N 1,145,550 E 626,040	81	90.5	Barge facility foundation	Rotary tricone and standard pen ASTM	None	

TABLE 2B-1 (SHEET 17 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
237	N 1,145,820 E 625,820	80	109.6	Intake structure foundation	Rotary tricone standard pen ASTM, and Denison,	Denison-6	
238	N 1,147,172 E 622,476	218	242.6	Geologic hole	Rotary tricone standard pen ASTM, and NX core,	None	
239	N 1,145,270 E 625,070	60.5	111.8	Water intake pipeline foundation	Rotary tricone and standard pen ASTM	None	
240	N 1,144,700 E 624,310	50	187.8	Makeup water pipeline foundation	Rotary tricone standard pen ASTM, Shelby, and Denison,	Shelby-3 Denison-1	
241	N 1,143,580 E 624,310	72.5	207.4	Warehouse and maintenance bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-3 Pitcher-4	
242	N 1,142,650 E 624,050	75.3	213.8	Water treatment area foundation	Rotary tricone and standard pen ASTM	None	
243	N 1,144,154 E 622,618	224	213	Geologic hole	NX core	None	
244	N 1,143,835 E 622,858	222.3	212.6	Geologic hole	5.5 in. x 4 in. core barrel	None	
245	N 1,143,491 E 623,924	150	207.6	Geologic hole	NX core	None	
246	N 1,145,532 E 620,553	400	210.4	Geologic hole	5.5 in. x 4 in. core barrel and 4.5 in. x 4 in. core barrel	None	
247	N 1,140,750 E 619,424	200	211.3	Geologic hole	5.5 in. x 4 in. core barrel	None	

TABLE 2B-1 (SHEET 18 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
248	N 1,142,466 E 619,111	200	166.8	Geologic hole	NX core	None	
249	N 1,143,826 E 624,154	183	193	Geologic hole	5.5 in. x 4 in. and 4.5 in. x 4 in. NX and core barrel	None	
301	N 1,142,692 E 622,264	161.3	227.3	Emergency cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-9 Pitcher-7	
302	N 1,142,690 E 622,390	165	229.3	Emergency cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-7 Pitcher-11	
303	N 1,142,690 E 622,605	161	229.2	Emergency cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-6 Pitcher-13	
304	N 1,142,690 E 622,776	173	231.4	Emergency cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-7 Pitcher-11	
305	N 1,142,870 E 622,505	161	227.7	Aux. bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-10 Pitcher-9	
306	N 1,143,000 E 622,335	159.9	229.8	Reactor cont.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-8 Pitcher-7	
307	N 1,143,000 E 622,675	154.5	218.7	Reactor cont.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-4 Pitcher-11	
308	N 1,143,106 E 622,740	151.3	225.9	Diesel gen. bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-9 Pitcher-7	

TABLE 2B-1 (SHEET 19 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
309	N 1,143,120 E 622,335	161.5	231.2	Control bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-6 Pitcher-14	
310	N 1,143,194 E 622,656	151.5	223.5	Control bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-6 Pitcher-9	
311	N 1,143,198 E 622,514	152.6	229.`	Control bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-4 Pitcher-5	
312	N 1,143,296 E 622,220	160.5	229.8	Turbine bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-1 Pitcher-7	
313	N 1,143,274 E 622,790	161.5	224.9	Turbine bldg	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-6 Pitcher-10	
314	N 1,143,335 E 622,494	160.5	229.5	Turbine bldg	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-5 Pitcher-7	
315	N 1,142,110 E 621,725	110.5	239.8	Unit No. 3 cooling tower	Rotary tricone and standard pen ASTM		
316	N 1,141,835 E 621,725	211.5	251.7	Unit No. 3 cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-1 Pitcher-25	
317	N 1,141,650 E 621,780	131	254.3	Unit No. 3 cooling tower	Rotary tricone and standard pen ASTM		
318	N 1,142,200 E 622,280	105.5	242.7	Unit No. 3 cooling tower	Rotary tricone and standard pen ASTM		

TABLE 2B-1 (SHEET 20 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
319	N 1,141,849 E 622,225	203	255.5	Unit No. 3 cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-6 Pitcher-14	
320	N 1,141,574	123.5	254.5	Unit No. 3 cooling tower	Rotary tricone and standard pen ASTM		
321	N 1,142,138 E 622,725	116	249.8	Unit No. 3 cooling tower	Rotary tricone and standard pen ASTM		
322	N 1,141,863 E 622,725	203	258.5	Unit No. 3 cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-4 Pitcher-16	
323	N 1,141,588 E 622,725	130.5	256.9	Unit No. 3 cooling tower	Rotary tricone and standard pen ASTM		
324	N 1,142,265 E 622,945	181.5	248.3	Unit No. 3 cooling tower structure	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-10 Pitcher-9	
325	N 1,143,276 E 620,548	105.5	236.5	Unit No. 4 cooling tower	Rotary tricone and standard pen ASTM		
326	N 1,143,000 E 620,548	206	230	Unit No. 4 cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-7 Pitcher-10	
327	N 1,142,726 E 620,548	99	228.2	Unit No. 4 cooling tower	Rotary tricone and standard pen ASTM		
328	N 1,143,262 E 621,048	104.5	234.1	Unit No. 4 cooling tower	Rotary tricone and standard pen ASTM		
329	N 1,142,987 E 621,048	165	222.4	Unit No. 4 cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-1 Pitcher-14	

TABLE 2B-1 (SHEET 21 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
330	N 1,143,248 E 621,548	85.5	221.8	Unit No. 4 cooling tower	Rotary tricone and standard pen ASTM		
331	N 1,142,873 E 621,548	161.5	220.9	Unit No. 4 cooling tower	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-5 Pitcher-9	
332	N 1,142,698 E 621,548	89.9	228.4	Unit No. 4 cooling tower	Rotary tricone and standard pen ASTM		
333	N 1,143,390 E 621,760	151.5	223.9	Unit No. 4 cooling water intake	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-6 Pitcher-9	
334	N 1,143,000 E 622,300	301	230	Reactor No. 4	Cored		
335	N 1,143,000 E 622,710	300	217.8	Reactor No. 3	Cored	Shelby-1 Pitcher-2	
336	N 1,143,535 E 622,320	170	226.8	Depression north of No. 4	Cored		
337	N 1,142,860 E 622,375	161.5	2311.2	Aux. bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-2 Pitcher-13	
338	N 1,142,860 E 622,625	153	223.5	Aux. bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-6 Pitcher-9	
339	N 1,143,070 E 622,270	160.5	229.8	Aux. bldg.	Rotary tricone standard pen ASTM, Shelby, and Pitcher	Shelby-1 Pitcher-13	
340	N 1,144,500 E 623,144	68	213.2	Groundwater monitoring	Rotary tricone and standard pen ASTM		

TABLE 2B-1 (SHEET 22 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
341	N 1,145,815 E 625,820	58	110.1	Groundwater monitoring	Rotary tricone and standard pen ASTM		
342	N 1,140,250 E 618,727	36.5	160	Dam site	Rotary tricone and standard pen ASTM		
343	N 1,140,232 E 619,064	31.5	152.2	Dam site	Rotary tricone and standard pen ASTM		
344	N 1,141,615 E 618,746	35.5	155.4	Dam site	Rotary tricone and standard pen ASTM		
345	N 1,137,248 E 628,175	31.5	118.5	Dam site	Rotary tricone and standard pen ASTM		
346	N 1,137,351 E 628,477	26.5	108	Dam site	Rotary tricone and standard pen ASTM		
347	N 1,138,000 E 627,800	25.5	114.3	Dam site	Rotary tricone and standard pen ASTM		
CT1	N 1,143,895 E 626,480	120	233	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT2	N 1,143,720 E 626,235	115	225.7	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT3	N 1,143,422 E 626,125	105	234	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT4	N 1,143,422 E 626,245	110	233.4	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT5	N 1,143,592 E 626,125	105	229.8	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT6	N 1,143,592 E 626,245	95	228.2	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	

TABLE 2B-1 (SHEET 23 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
CT7	N 1,143,848 E 626,125	95	220.4	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT8	N 1,143,848 E 626,245	100	225.2	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT9	N 1,143,935 E 626,515	100	233.4	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT10	N 1,143,935 E 626,645	100	231.4	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT11	N 1,143,820 E 626,580	95	230.8	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT12	N 1,143,620 E 626,580	101	231.8	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT13	N 1,143,505 E 626,515	102	234.6	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT14	N 1,143,505 E 626,645	110	234.2	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT15	N 1,141,077 E 627,940	120	208.3	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT16	N 1,141,247 E 627,940	100	208.5	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT17	N 1,141,417 E 627,940	95	206.3	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT18	N 1,141,672 E 627,940	115	207.7	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT19	N 1,141,077 E 628,060	100	207.7	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	

TABLE 2B-1 (SHEET 24 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
CT20	N 1,141,247 E 628,060	100	208.8	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT21	N 1,141,417 E 628,060	120	211.5	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT22	N 1,141,672 E 628,060	100	211.6	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT23	N 1,141,160 E 628,330	110	215.2	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT24	N 1,141,160 E 628,460	135	214.8	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT25	N 1,141,275 E 628,395	110	214.7	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT26	N 1,141,475 E 628,395	105	210.4	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT27	N 1,141,590 E 628,330	100	208	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
CT28	N 1,141,590 E 628,460	105	207.5	Combustion turbine foundation	Rotary tricone and standard pen ASTM	None	
RH-1	N 1,145,680 E 626,250	80	90.3	Haul road	Rotary tricone and standard pen ASTM	None	
RH-2	N 1,145,603 E 625,496	60	151.1	Haul road	Rotary tricone and standard pen ASTM	None	
RH-3	N 1,145,594 E 625,196	50	150.2	Haul road	Rotary tricone and standard pen ASTM	None	
RH-4	N 1,145,672 E 624,857	50	162.5	Haul road	Rotary tricone and standard pen ASTM	None	

TABLE 2B-1 (SHEET 25 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
OD-1	N 1,143,263 E 628,836	55	86.4	Barge dock	Rotary tricone and standard pen ASTM	None	
401	N 1,145,815 E 626,160	60	89.6	River intake	Rotary tricone and standard pen ASTM	None	
402	N 1,145,835 E 626,055	60	88.8	River intake	Rotary tricone and standard pen ASTM	None	
403	N 1,145,955 E 626,060	64	88.5	River intake	Rotary tricone and standard pen ASTM	None	
404	N 1,146,000 E 625,980	60	86.6	River intake	Rotary tricone and standard pen ASTM	None	
405	N 1,146,175 E 625,890	64	87.4	River intake	Rotary tricone and standard pen ASTM	None	
406	N 1,146,300 E 625,820	60	88.4	River intake	Rotary tricone and standard pen ASTM	None	
407	N 1,146,215 E 625,790	70	88.7	River intake	Rotary tricone and standard pen ASTM	6	
408	N 1,146,175 E 625,705	101.5	135.1	River intake	Rotary tricone and standard pen ASTM	10	
409	N 1,145,895 E 625,475	60	116.9	River intake	Rotary tricone and standard pen ASTM	None	
501	N 1,143,014 E 623,891	208	208.2	Check water loss in No. 701. Core for logging	Rotary tricone and NX split tube core barrel	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)

TABLE 2B-1 (SHEET 26 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
501A	N 1,143,109 E 623,896	141	208.2	Large core to check core loss in No. 501	Rotary ticone and 4-in. core	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
502	N 1,14,3,390 E 623,357	150	216	Check water loss in upper 5 ft of marl in No. 109	Rotary tricone and NX split tube core barrel	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
503	N 1,143,870 E 624,130	130	194.5	Check depression near No. 239. Water test for permeability	Rotary tricone and NX split tube core barrel	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
503A	N 1,143,877 E 624,126	121	194.5	Large core to check loss in No. 503	Rotary ticone and 4-in. core	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
504	N 1,144,139 E 622,611	186.5	214.6	Check water loss in No. 238. Water test for permeability. Core for logging.	Rotary tricone and NX split tube core barrel	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
505	N 1,147,040 E 622,299	201.5	241.9	Check water loss in No. 243. Water test for permeability. Check core recovery..	Rotary tricone and NX split tube core barrel	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
506	N 1,146,698 E 621,067	178	172.7	Check water loss in No. 134. Water test for permeability. Core for logging.	Rotary tricone and NX split tube core barrel	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)

TABLE 2B-1 (SHEET 27 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
506A	N 1,146,705 E 621,064	115	172.7	Core for logging to check core loss in No. 506.	Rotary tricone and 4 in. core	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
507	N 1,145,504.5 E 620,633.5	191	211.8	Check water loss in No. 246. Water test for permeability.	Rotary tricone and NX split tube core barrel	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
507A	N 1,145,503.5 E 620,628	134.5	211.8	Large core to check core loss in No. 507. Core for logging.	Rotary tricone and 4 in. core	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
508	N 1,145,605 E 621,613	163	190.5	Water test for permeability. Core for logging to check hole No. 157.	Rotary tricone and NX split tube core barrel	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
509	N 1,142,950 E 622,280	188	230.4	Menard pressure meter tests. Samples for laboratory testing.	Rotary tricone and NX split tube core barrel	14	3-D, neutron, density, and caliper geophysical logs (Birdwell)
510	N 1,143,047 E 622,353	185	230.7	Water tests for permeability. Menard tested. Samples for laboratory testing.	Rotary tricone and NX split tube core barrel	14	3-D, neutron, density, and caliper geophysical logs (Birdwell)
511	N 1,143,103 E 622,549	181	225.4	Core for logging. Samples for laboratory testing. Menard tested.	Rotary tricone and NX split tube core barrel	15	3-D, neutron, density, and caliper geophysical logs (Birdwell)

TABLE 2B-1 (SHEET 28 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
512	N 1,143,070 E 622,650	180	221.5	Core for logging. Samples for laboratory testing. Menard tested.	Rotary tricone and NX split tube core barrel	15	3-D, neutron, density, and caliper geophysical logs (Birdwell)
513	N 1,142,940 E 622,640	178	220.2	Core for logging and water tests for permeability. Menard tested. Samples for laboratory tests.	Rotary tricone and NX split tube core barrel	14	3-D, neutron, density, and caliper geophysical logs (Birdwell)
514	N 1,142,820 E 623,480	169.5	209.9	Core for logging. Menard tested. Samples for laboratory testing.	Rotary tricone and NX split tube core barrel	14	3-D, neutron, density, and caliper geophysical logs (Birdwell)
515	N 1,142,970 E 623,440	172	212.7	Core for logging. Menard tested. Samples for laboratory tests.	Rotary tricone and NX split tube core barrel	13	3-D, neutron, density, and caliper geophysical logs (Birdwell)
516	N 1,143,080 E 623,480	172	212.7	Core for logging. Menard tested. Samples for laboratory tests.	Rotary tricone and NX split tube core barrel	14	3-D, neutron, density, and caliper geophysical logs (Birdwell)
517	N 1,143,050 E 623,800	175	207.7	Core for logging. Menard tested. Samples for laboratory tests.	Rotary tricone and NX split tube core barrel	14	3-D, neutron, density, and caliper geophysical logs (Birdwell)
518	N 1,142,950 E 623,800	175	209.9	Core for logging. Menard tested. Samples for laboratory tests.	Rotary tricone and NX split tube core barrel	14	3-D, neutron, density, and caliper geophysical logs (Birdwell)

TABLE 2B-1 (SHEET 29 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
519	N 1,142,820 E 623,860	176	211.9	Core for logging. Menard tested. Samples for laboratory tests.	Rotary tricone and NX split tube core barrel	17	3-D, neutron, density, and caliper geophysical logs (Birdwell)
520	N 1,143,000 E 623,825	175	209.2	Experimental geophysical tests (high frequency sonic logging Holosonics, Inc.)	Rotary tricone	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
521	N 1,143,026.5 E 623,851.5	175	207.5	Experimental geophysical tests (high frequency sonic logging Holosonics, Inc.)	Rotary tricone	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
522	N 1,142,973 E 623,851	175	209	Experimental geophysical tests (high frequency sonic logging Holosonics, Inc.)	Rotary tricone	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
523	N 1,142,973 E 623,798	173	209.5	Experimental geophysical tests (high frequency sonic logging Holosonics, Inc.)	Rotary tricone	None	3-D, neutron, density, and caliper geophysical logs (Birdwell)
524	N 1,143,026 E 623,798	173	208.6	Experimental geophysical tests (high frequency sonic logging Holosonics, Inc.)	Rotary tricone	None	
601	N 1,143,120 E 625,035	165	218.6	Cooling tower No. 1	Standard pen ASTM, rotary tricone, and NX core		E-logged

TABLE 2B-1 (SHEET 30 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
602	N 1,143,056 E 625,191	115	217	Cooling tower No. 1	Standard pen ASTM, rotary tricone, and NX core		E-logged
603	N 1,142,900 E 625,255	170	216.7	Cooling tower No. 1	Standard pen ASTM, rotary tricone, and NX core		
604	N 1,142,744 E 625,191	119.5	217.1	Cooling tower No. 1	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
605	N 1,142,680 E 625,035	163	219	Cooling tower No. 1	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
606	N 1,142,744 E 624,879	115	216.5	Cooling tower No. 1	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
607	N 1,142,900 E 624,815	157	215.1	Cooling tower No. 1	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
608	N 1,143,056 E 624,879	115	217.9	Cooling tower No. 1	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	
609	N 1,143,320 E 625,915	160	232.1	Cooling tower No. 2	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
609A	N 1,143,320 E 625,885	187	232.6	Cooling tower No. 2	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
610	N 1,143,256 E 626,071	187.5	233.6	Cooling tower No. 2	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
611	N 1,143,100 E 626,135	185.5	235.1	Cooling tower No. 2	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
612	N 1,142,994 E 626,071	139	231.7	Cooling tower No. 2	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	
613	N 1,142,880 E 625,915	175.5	223.4	Cooling tower No. 2	Standard pen ASTM, rotary tricone, and NX core		E-logged

TABLE 2B-1 (SHEET 31 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
614	N 1,142,944 E 625,759	120	215.9	Cooling tower No. 2	Standard pen ASTM, rotary tricone, and NX core		
615	N 1,143,100 E 625,695	169	218.6	Cooling tower No. 2	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	
616A	N 1,143,256 E 625,759	135	222.4	Cooling tower No. 2	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	
617	N 1,143,320 E 620,255	203	244.4	Cooling tower No. 3	Standard pen ASTM, rotary tricone, and NX core		
618	N 1,143,256 E 620,411	138	239	Cooling tower No. 3	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	
619	N 1,143,100 E 620,475	211.5	231.9	Cooling tower No. 3	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
620	N 1,142,944 E 620,411	124.5	232.1	Cooling tower No. 3	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	
621	N 1,142,880 E 620,255	182	236.6	Cooling tower No. 3	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
622	N 1,142,944 E 620,099	153	247.3	Cooling tower No. 3	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	
623	N 1,143,100 E 620,035	219	250.3	Cooling tower No. 3	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
624	N 1,143,320 E 620,255	172	250.2	Cooling tower No. 3	Standard pen ASTM, rotary tricone, and NX core		
625	N 1,143,120 E 621,135	173.3	227.7	Cooling tower No. 4	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
626	N 1,143,056 E 621,291	114	227.4	Cooling tower No. 4	Standard pen ASTM, rotary tricone, and NX core		

TABLE 2B-1 (SHEET 32 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
627	N 1,142,900 E 621,355	168.3	221.8	Cooling tower No. 4	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
628	N 1,142,744 E 621,291	111.5	221.1	Cooling tower No. 4	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	
629	N 1,142,680 E 621,135	165.5	219.5	Cooling tower No. 4	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
630	N 1,142,744 E 620,979	110	219	Cooling tower No. 4	Standard pen ASTM, rotary tricone, and NX core		
631	N 1,142,900 E 620,915	172	223.2	Cooling tower No. 4	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	E-logged
632	N 1,143,056 E 620,979	118	225.3	Cooling tower No. 4	Standard pen ASTM, rotary tricone, and NX core	Menard Pressure Test	
633	N 1,142,363.4 E 629,540.9	165	211.2	Geology, marl at bluff	Standard pen ASTM, rotary tricone, and NX core		E-logged
701	N 1,143,245 E 625,343	110	213.88	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core		
702	N 1,143,328 E 625,500	165.2	213.27	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core	Menard Pressure Test	
703	N 1,135,127 E 625,658	125	219.13	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core		
704	N 1,142,846 E 625,500	162.1	210.73	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core	Menard Pressure Test	
705	N 1,144,032 E 625,500	172	203.83	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core	Menard Pressure Test	
705-A	10 north of boring 705	166.1	202.53	Cooling towers	Rotary tricone		

TABLE 2B-1 (SHEET 33 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
706	N 1,143,948 E 625,657	102	207.28	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core		
706-A	N 1,143,945 E 625,658	163.5	210.89	Cooling towers	Rotary tricone and NQ core		
707	N 1,143,791 E 625,741	180.5	214.43	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core	Menard Pressure Test	
708	N 1,143,634 E 625,657	116.5	214.93	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core		
709	N 1,143,550 E 625,500	165.3	213.35	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core	Menard Pressure Test	
710	N 1,143,634 E 625,343	107	213.23	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core		
711	N 1,143,791 E 625,259	161.5	208.66	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core	Menard Pressure Test	
712	N 1,143,949 E 625,343	110.5	207.3	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core		
712A	N 1,143,937 E 625,344	158.6	204.4	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core		
713	N 1,143,912 E 625,500	169	203.7	Cooling towers	Standard pen ASTM, rotary tricone, and NQ core		
800	N 1,143,851 E 625,012	94	213.7	Groundwater monitoring	Rotary tricone	None	
801	N 1,142,651 E 624,733	93.6	212.8	Groundwater monitoring	Rotary tricone	None	
802	N 1,142,196 E 624,198	94	215.8	Groundwater monitoring	Rotary tricone	None	

TABLE 2B-1 (SHEET 34 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
803	N 1,142,350 E 622,909	94	219.7	Groundwater monitoring	Rotary tricone	None	
803A	N 1,142,085 E 622,896	88	218.3	Groundwater monitoring	Rotary tricone	None	
804	N 1,141,597 E 622,227	96	224.1	Groundwater monitoring	Rotary tricone	None	
805	N 1,141,718 E 624,425	127	234.2	Groundwater monitoring	Rotary tricone	None	
805A	N 1,141,672 E 624,403	125	232.7	Groundwater monitoring	Rotary tricone	None	
VG-1	N 1,120,308.26 E 660,009.14	565	156.6	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper
VG-2	N 1,122,608.99 E 650,596.85	618	253.1	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper
VG-3	N 1,121,183.52 E 655,725.83	574.9	165.7	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper
VG-4	N 1,124,629.41 E 644,971.51	554.4	150.3	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper

TABLE 2B-1 (SHEET 35 OF 36)

Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
VG-5	N 1,116,669.12 E 665,818.68	502	94.5	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper
VG-6	N 1,110,896.34 E 669,643.15	620	217.1	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper
VG-7	N 1,127,245.60 E 640,322.37	392	250.6	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper
VG-8	N 1,104,446.34 E 678,744.09	355.4	103.7	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper
VSC-1	N 1,134,867.04 E 679,423,71	620	219	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper
VSC-2	N 1,141,512.71 E 673,492.62	600	201.7	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper
VSC-3	N 1,138,356.84 E 676,254.55	510	170.3	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper

TABLE 2B-1 (SHEET 36 OF 36)

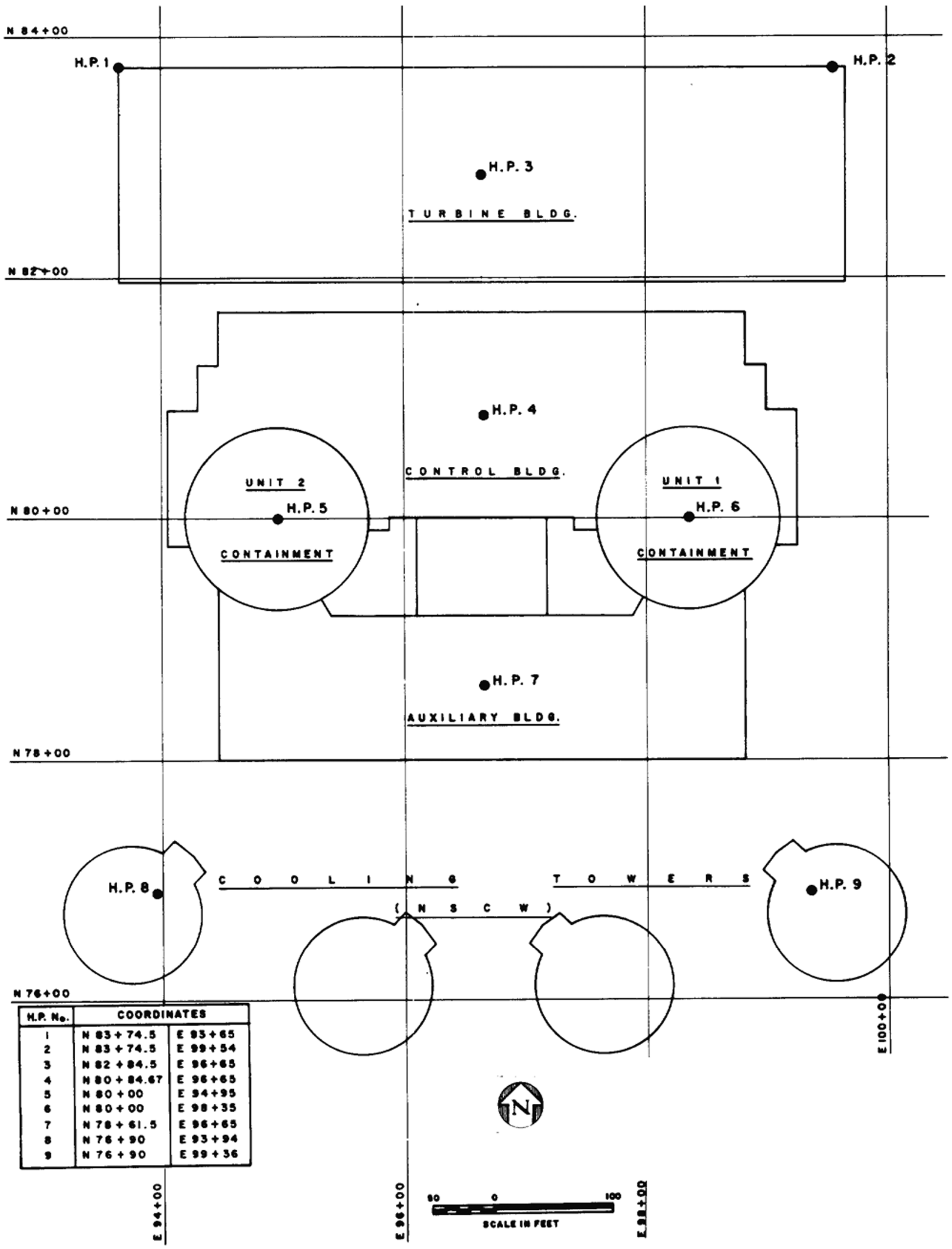
Hole No.	Depth Location	(ft)	Surface Elevation (ft)	Purpose	Type Drilling	Number and Type Undisturbed Samples Taken	Remarks
VSC-4	N 1,130,590.27 E 683,271.46	1024	156.7	Millett Fault investigation	Core	Continuous core samples taken triple-tube NQ wireline	Electric, gamma, neutron, caliper

a. Boring located outside site area, not shown on maps.

TABLE 2B-2

CORE SAMPLE INVENTORY

Hole No.	No. of Boxes	Hole No.	No. of Boxes
139	7	610	1
240	1	614	2
249	8	615	5
501	7	701	1
501B	1	703	1
502	3	704	3
503	3	705	4
503B	13	705A	3
513	2	706	2
514	4	707	5
515	2	708	2
516	4	709	3
517	4	710	2
518	4	711	3
519	2	712	3
602	2	721	1
603	5		



H.P. No.	COORDINATES	
1	N 83 + 74.5	E 93 + 65
2	N 83 + 74.5	E 99 + 54
3	N 82 + 84.5	E 96 + 65
4	N 80 + 84.67	E 96 + 65
5	N 80 + 00	E 94 + 95
6	N 80 + 00	E 98 + 35
7	N 78 + 61.5	E 96 + 65
8	N 76 + 90	E 93 + 94
9	N 76 + 90	E 99 + 36

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VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

HEAVE POINT LOCATIONS

FIGURE 2B–1

3.0 DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

This chapter identifies, describes, and discusses the principal architectural and engineering design features of those structures, components, equipment, and systems that are necessary to assure:

A. The integrity of the reactor coolant pressure boundary.

B. The capability to shut down the reactor and maintain it in a safe shutdown condition.

C. The capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline values of 10 CFR 100.

3.1 CONFORMANCE WITH NUCLEAR REGULATORY COMMISSION (NRC) GENERAL DESIGN CRITERIA (GDC)

This section briefly discusses the extent to which the design criteria for VEGP structures, systems, and components important to safety comply with Title 10, Code of Federal Regulations, Part 50 (10 CFR 50), Appendix A, General Design Criteria for Nuclear Power Plants. As presented in this section, each criterion is first quoted and then discussed in enough detail to demonstrate compliance of the VEGP with each criterion. For some criteria, additional information may be required for a complete discussion. In such cases, detailed evaluations of compliance with the various general design criteria are incorporated in more appropriate sections, but are located by reference.

3.1.1 OVERALL REQUIREMENTS

CRITERION 1 - QUALITY STANDARDS AND RECORDS

"Structures, systems, and components important to safety shall be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed. Where generally recognized codes and standards are used, they shall be identified and evaluated to determine their applicability, adequacy, and sufficiency and shall be supplemented or modified, as necessary, to assure a quality product, in keeping with the required safety function.

A quality assurance program shall be established and implemented in order to provide adequate assurance that these structures, systems, and components will satisfactorily perform their safety functions. Appropriate records of the design, fabrication, erection, and testing of structures, systems, and components important to safety shall be maintained by or under the control of the nuclear power unit licensee throughout the life of the unit."

DISCUSSION

The quality assurance program of the VEGP and Southern Nuclear Operating Company (SNC), together with the quality assurance, quality engineering, and quality control programs of the major contractors and their vendors, ensure that structures, systems, and components important to safety are designed, fabricated, erected, and tested to quality standards commensurate with the safety functions to be performed. This is accomplished through the use of recognized codes, standards, and design criteria. As necessary, additional supplemental standards, design criteria, and requirements are developed by the VEGP and the major contractors' engineering organizations. Appropriate records associated with the engineering and design, fabrication, erection, and testing which document the compliance with recognized codes, standards, and design criteria are maintained throughout the life of the units either by or under the control of SNC. Quality assurance is described in chapter 17.

The principal design criteria, design bases, codes, and standards applied to the facility are described in section 3.2. Additional detail may be found in the pertinent section of the document dealing with structures, systems, and components important to safety; e.g., the containment as described in subsection 3.8.1.

CRITERION 2 - DESIGN BASES FOR PROTECTION AGAINST NATURAL PHENOMENA

"Structures, systems, and components important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without the loss of the capability to perform their safety functions. The design bases for these structures, systems, and components shall reflect: (1) appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated, (2) appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena, and (3) the importance of the safety functions to be performed."

DISCUSSION

The structures, systems, and components important to safety are designed either to withstand the effects of natural phenomena without loss of the capability to perform their safety functions, or are designed such that their response or failure will be in a safe condition. Those structures, systems, and components vital to the shutdown capability of the reactor are designed to withstand the maximum probable natural phenomena at the site, determined from recorded data for the site vicinity, with appropriate margin to account for uncertainties in historical data. Appropriate combinations of structural loadings from normal, accident, and natural phenomena are considered in the plant design. The nature and magnitude of the natural phenomena considered in the design of this plant are discussed in chapter 2. Chapter 3 discusses the design of the plant in relationship to natural events. Seismic and quality group classifications, as well as other pertinent standards and information, are given in the sections discussing individual structures and components and in table 3.2.2-1.

CRITERION 3 - FIRE PROTECTION

"Structures, systems, and components important to safety shall be designed and located to minimize, consistent with other safety requirements, the probability and effect of fires and explosions. Noncombustible and heat-resistant materials shall be used wherever practical throughout the unit, particularly in locations such as the containment and control room. Fire detection and fighting systems of appropriate capacity and capability shall be provided and designed to minimize the adverse effects of fires on structures, systems, and components important to safety. Firefighting systems shall be designed to assure that their rupture or inadvertent operation does not significantly impair the safety capability of these structures, systems, and components."

DISCUSSION

The plant is designed to minimize the probability and effect of fires and explosions. Noncombustible and fire-resistant materials are used in the containment, control room, components of safety features systems, and throughout the unit wherever fire is a potential risk to safety-related systems. For example, electrical cables have a fire-retardant jacketing, and fire barriers and fire stops are utilized as described in subsection 9.5.1. Equipment and facilities for fire protection, including detection, alarm, and extinguishment, are provided to protect both plant equipment and personnel from fire, explosion, and the resultant release of toxic vapors.

Fire protection is provided by deluge systems (water spray), sprinklers, Halon 1301, and portable extinguishers.

Firefighting systems are designed to assure that their rupture or inadvertent operation will not prevent systems important to safety from performing their design functions.

The following codes, guides, and standards are used as guidelines in the design of the fire protection system and equipment. Where required by law, the system and equipment substantially conform to the applicable portions of the following standards:

A. National Fire Protection Association (NFPA) National Fire Codes.

B. Nuclear Mutual Limited (NML) Property Loss Prevention Standards for Nuclear Generating Stations.

C. BTP-APCSB 9.5-1, Appendix A, Guidelines for Fire Protection for Nuclear Power Plants, May 1, 1976.

D. BTP-CMEB 9.5-1, Guidelines for Fire Protection for Nuclear Power Plants, July 1981.

CRITERION 4 - ENVIRONMENTAL AND MISSILE DESIGN BASES

"Structures, systems, and components important to safety shall be designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, including loss-of-coolant accidents (LOCAs). These structures, systems, and components shall be appropriately protected against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids, that may result from equipment failures and from events and conditions outside the nuclear power unit."

DISCUSSION

Structures, systems, and components important to safety are designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, including LOCAs. Criteria are presented in chapter 3, and the environmental conditions are described in section 3.11.

These structures, systems, and components are appropriately protected against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids, that may result from equipment failures and from events and conditions outside the nuclear power unit. Details of the design, environmental testing, and construction of these systems, structures, and components are included in chapters 3, 5, 6, 7, 9, and 10. Evaluation of the performance of the safety features is contained in chapter 15.

CRITERION 5 - SHARING OF STRUCTURES, SYSTEMS, AND COMPONENTS

"Structures, systems, and components important to safety shall not be shared among nuclear power units unless it can be shown that such sharing will not significantly impair their ability to perform their safety functions, including, in the event of an accident in one unit, an orderly shutdown and cooldown of the remaining unit."

DISCUSSION

The VEGP is a two-unit plant with the following common safety-related structures:

A. Control building.

B. Auxiliary building.

C. Fuel handling building.

Within these buildings are shared spaces, such as the control room, which contain physically separated safety-related equipment. A detailed description of plant structures is provided in section 3.8.

Safety-related systems are not shared, with the exception of the fuel handling building post-accident exhaust system. (See subsection 9.4.2.) Common heating, ventilation, and air-conditioning system (HVAC) ducting headers are used in some instances for redundant HVAC units. Systems or portions of systems and spaces that are shared by units 1 and 2 are listed in paragraphs 1.2.2.1 and 1.2.2.2. Where common structures, systems, and components are utilized, such sharing has been evaluated to ensure that there are no adverse impacts on safety functions.

3.1.2 PROTECTION BY MULTIPLE FISSION PRODUCT BARRIERS

CRITERION 10 - REACTOR DESIGN

"The reactor core and associated coolant, control, and protection systems shall be designed with appropriate margin to assure that specified acceptable fuel design limits are not exceeded during any condition of normal operation, including the effects of anticipated operational occurrences."

DISCUSSION

The reactor core and associated coolant, control, and protection systems are designed to the following criteria:

A. No fuel damage will occur during normal core operation and operational transients (Condition 1) or any transient conditions arising from occurrences of moderate frequency (Condition 2) beyond a small fraction of clad defects for which various aspects of the plant are designed. Fuel damage, as used here, is defined as penetration of the fission product barrier; i.e., the fuel rod clad. Conditions 1 and 2, as used here, are defined by American National Standards Institute (ANSI) N18.2-1973. The small number of clad defects that may occur are within the capability of the plant cleanup system and are consistent with the plant design bases.

B. The reactor can be returned to a safe shutdown state following a Condition 3 event with only a small fraction of the fuel rods damaged, although sufficient fuel damage might occur to preclude the immediate resumption of operation. Condition 3, as used here, is defined by ANSI N18.2-1973.

C. The core will remain intact with acceptable heat transfer geometry following transients arising from occurrences of limiting faults (Condition 4). Condition 4, as used here, is defined by ANSI N18.2-1973.

The reactor trip system is designed to actuate a reactor trip whenever necessary to ensure that the fuel design limits are not exceeded. The core design, together with the process and decay heat removal systems, provide for this capability under all expected conditions of normal operation with appropriate margins for uncertainties and anticipated transient situations, including the effects of the loss of reactor coolant flow, trip of the turbine generator, loss of normal feedwater, and loss of both normal and preferred power sources.

Chapter 4 discusses the design bases and design evaluation of core components. Details of the control and protection systems' instrumentation design and logic are discussed in chapter 7. This information supports the accident analyses of chapter 15 which show that the acceptable fuel design limits are not exceeded for Condition 1 and 2 occurrences.

CRITERION 11 - <u>REACTOR INHERENT PROTECTION</u>

"The reactor core and associated coolant systems shall be designed so that in the power-operating range the net effect of the prompt inherent nuclear feedback characteristics tends to compensate for a rapid increase in reactivity."

<u>DISCUSSION</u>

Whenever the reactor is critical, prompt compensatory reactivity feedback effects are assured by the negative fuel temperature effect (Doppler effect) and during the initial cycle, by the nonpositive operational limit on the moderator temperature coefficient of reactivity. (The moderator temperature coefficient may be slightly positive for parts of reload cycles as discussed in chapter 15.) The negative Doppler coefficient of reactivity is assured by the inherent design, using low enrichment fuel. The nonpositive moderator temperature coefficient of reactivity at full power (100 percent) is assured by administratively controlling the dissolved absorber concentration or by using burnable poison.

Reactivity coefficients and their effects are discussed in chapter 4.

CRITERION 12 - <u>SUPPRESSION OF REACTOR POWER OSCILLATIONS</u>

"The reactor core and associated coolant, control, and protection systems shall be designed to assure that power oscillations which can result in conditions exceeding specified acceptable fuel design limits are not possible or can be reliably and readily detected and suppressed."

<u>DISCUSSION</u>

Power oscillations of the fundamental mode are inherently eliminated by negative Doppler and nonpositive moderator temperature coefficient of reactivity. During parts of reload cycles the moderator temperature coefficient may be slightly positive as discussed in chapter 15.

Oscillations, due to xenon spatial effects, in the radial, diametral, and azimuthal overtone modes are heavily damped due to the inherent design and due to the negative Doppler and nonpositive moderator temperature coefficients of reactivity.

Oscillations, due to xenon spatial effects, may occur in the axial first overtone mode. Assurance that fuel design limits are not exceeded by xenon axial oscillations is provided by reactor trip functions, using the measured axial power imbalance as an input.

If necessary to maintain axial imbalance within the limits of the Core Operating Limits Report; i.e., imbalances which are alarmed to the operator and are within the imbalance trip setpoints; the operator can suppress xenon axial oscillations by control rod motions and/or temporary power reductions.

Oscillations, due to xenon spatial effects, in axial modes higher than the first overtone are heavily damped due to the inherent design and due to the negative Doppler coefficient of reactivity.

The stability of the core against xenon-induced power oscillations and the functional requirements of instrumentation for monitoring and measuring core power distribution are discussed in chapter 4. Details of the instrumentation design and logic are discussed in chapter 7.

CRITERION 13 - INSTRUMENTATION AND CONTROL

"Instrumentation shall be provided to monitor variables and systems over their anticipated ranges for normal operation, for anticipated operational occurrences, and for accident conditions as appropriate to assure adequate safety, including those variables and systems that can affect the fission process, the integrity of the reactor core, the reactor coolant pressure boundary, and the containment and its associated systems. Appropriate controls shall be provided to maintain these variables and systems within prescribed operating ranges."

DISCUSSION

Instrumentation and controls are provided to monitor and control neutron flux, control rod position, fluid temperatures, pressures, flows, and levels, as necessary, to assure that adequate plant safety can be maintained. Instrumentation is provided in the reactor coolant system, steam and power conversion system, containment, engineered safety features systems, radioactive waste management systems, and other auxiliary systems. Parameters that must be provided for operator use under normal operating and accident conditions are indicated in the control room in proximity to the controls for maintaining the indicated parameters in their proper ranges.

The quantity and types of process instrumentation provided ensure safe and orderly operation of all systems over the full design range of the plant. These systems are described in chapters 6, 7, 8, 9, 10, 11, and 12.

CRITERION 14 - REACTOR COOLANT PRESSURE BOUNDARY

"The reactor coolant pressure boundary shall be designed, fabricated, erected, and tested so as to have an extremely low probability of abnormal leakage, of rapidly propagating failure, and of gross rupture."

DISCUSSION

The reactor coolant pressure boundary (RCPB) is designed to accommodate the system pressures and temperatures attained under the expected modes of plant operation, including anticipated transients, with stresses within applicable limits. Consideration is given to loadings under normal operating conditions and to abnormal loadings, such as pipe rupture and seismic loadings, as discussed in chapter 3. The piping is protected from overpressure by means of pressure-relieving devices, as required by American Society of Mechanical Engineers (ASME), Section III.

Reactor coolant pressure boundary materials and fabrication techniques are such that there is a low probability of gross rupture or significant leakage. (Refer to Criterion 31 for further discussion of reactor coolant pressure boundary.)

Coolant chemistry is controlled to protect from corrosion the materials of construction of the RCPB.

The RCPB welds are accessible for inservice inspections to assess the structural and leaktight integrity. The details are given in chapter 5. For the reactor vessel, a material surveillance program conforming to applicable codes is provided. Chapter 5 has additional details.

Instrumentation is provided to detect significant leakage from the RCPB with indication in the control room, as discussed in chapter 5.

CRITERION 15 - REACTOR COOLANT SYSTEM DESIGN

"The reactor coolant system and associated auxiliary, control, and protection systems shall be designed with sufficient margin to assure that the design conditions of the reactor coolant pressure boundary are not exceeded during normal operation, including anticipated operational occurrences."

DISCUSSION

Steady-state and transient analyses are performed to ensure that reactor coolant system (RCS) design conditions are not exceeded during normal operation. Protection and control setpoints are based on these analyses.

Additionally, RCPB components have a large margin of safety through application of proven materials and design codes, use of proven fabrication techniques, nondestructive shop testing, and integrated hydrostatic testing of assembled components.

The effect of radiation embrittlement is considered in reactor vessel design, and surveillance samples monitor adherence to expected conditions throughout the plant life.

Multiple safety and relief valves are provided for the RCS. These valves and their setpoints meet the ASME criteria for overpressure protection. The ASME criteria are satisfactory, based on a long history of industrial use. Chapter 5 discusses the RCS design.

CRITERION 16 - CONTAINMENT DESIGN

"The reactor containment and associated systems shall be provided to establish an essentially leak-tight barrier against the uncontrolled release of radioactivity to the environment and to assure that the containment design conditions important to safety are not exceeded for as long as postulated accident conditions require."

DISCUSSION

A steel-lined, prestressed, post-tensioned concrete containment structure encloses the entire RCS. It is designed to sustain, without loss of required integrity, the effects of LOCAs up to and including the double-ended rupture of the largest pipe in the RCS or double-ended rupture of a steam or feedwater pipe. Engineered safety features comprising the emergency core cooling system, containment spray system, and the containment air coolers serve to cool the reactor core and return the containment to near atmospheric pressure. The containment structure and engineered safety features systems are designed to ensure the required functional capability of containing any uncontrolled release of radioactivity. The concrete radiological shielding and the liner within the containment limit the uncontrolled release of radioactivity to the environment.

Refer to chapters 3, 6, and 15.

CRITERION 17 - ELECTRICAL POWER SYSTEMS

"An onsite electric power system and an offsite electric power system shall be provided to permit the functioning of structures, systems, and components important to safety. The safety function for each system (assuming that the other system is not functioning) shall be to provide sufficient capacity and capability to assure that (1) specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded as a result of anticipated operational occurrences and (2) the core is cooled and containment integrity and other vital functions are maintained in the event of postulated accidents."

"The onsite electric power supplies, including the batteries, and the onsite electric distribution system shall have sufficient independence, redundancy, and testability to perform their safety functions, assuming a single failure."

"Electric power from the transmission network to the onsite electric distribution system shall be supplied by two physically independent circuits (not necessarily on separate rights-of-way) designed and located so as to minimize, to the extent practical, the likelihood of their simultaneous failure under operating and postulated accident and environmental conditions. A switchyard common to both circuits is acceptable. Each of these circuits shall be designed to be available in sufficient time, following the loss of all onsite alternating current power sources and the offsite electric power circuit, to assure that specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded. One of these circuits shall be designed to be available within a few seconds following a LOCA to assure that core cooling, containment integrity, and other vital safety functions are maintained."

"Provisions shall be included to minimize the probability of losing electric power from any of the remaining supplies as a result of, or coincident with, the loss of power generated by the nuclear power unit, the loss of power from the transmission network, or the loss of power from the onsite electric power supplies."

DISCUSSION

An onsite electric power system and an offsite electric power system are provided to permit the functioning of structures, systems, and components important to safety. As discussed in chapter 8, each Class 1E electric power system is designed with adequate independence, capacity, redundancy, and testability to ensure the functioning of engineered safety features (ESF). Independence is provided by physical separation and electrical isolation of components and cables.

The onsite ac power system includes a Class 1E system and a non-Class 1E system. Onsite ac power is supplied from either the 230-kV switchyard through reserve auxiliary transformers (which feed the non-Class 1E and Class 1E buses), or from a 13.8-kV underground circuit emanating from Georgia Power Company Plant Wilson switchyard (which connects through the standby auxiliary transformer (SAT) located in the Vogtle low voltage switchyard) to a Class 1E 4.16-kV bus and some non-Class 1E 4.16-kV loads. The Class 1E ac power system is the power source used in (or associated with) shutting down the reactor and preventing or limiting the release of radioactive material following a design basis event. The system is divided into two independent ac power trains, train A and train B, each fed from an independent Class 1E bus.

Each Class 1E bus is provided with two (normal and alternate) offsite preferred power sources and a standby onsite power source. With at least two offsite sources available per Unit, each Class 1E bus is supplied from a separate reserve auxiliary transformer. However, one Class 1E bus may be directly connected to the SAT instead of a RAT for its offsite preferred power source feed. This direct connection is performed under administrative controls and with the use of key interlocked disconnect switches.

The Class 1E ac system distributes power to all safety-related loads. Also, the Class 1E ac system supplies power to certain selected loads which are not safety related but are important to the plant operation; however, these loads are tripped when a safety injection signal is received.

The non-Class 1E ac system supplies preferred (offsite) power to the Class 1E ac system through the reserve auxiliary transformer 4160-V windings. Each reserve auxiliary transformer has the capacity to supply all connected non-Class 1E running loads and to start and run the loads of one Class 1E train. The SAT may also supply preferred (offsite) power to either one of the Class 1E 4160 volt buses. The SAT has the capacity to start and run the loads of one Class 1E train, and to supply some additional non Class 1E loads. Non Class 1E loading of the SAT is administratively controlled.

A failure of a single component will not prevent the safety-related systems from performing their function. Each of the connected preferred offsite power circuits is designed to be available in sufficient time, following a loss of all onsite power sources and the other offsite electric power circuit, to assure that specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded.

Emergency onsite ac power is furnished by two diesel generators per unit. Each diesel generator is connected to a Class 1E bus. The ESF loads are divided between the Class 1E buses in balanced, redundant load groupings. Each diesel generator is capable of supplying sufficient power in sufficient time for the operation of the ESF required for the unit during a postulated LOCA. During a postulated LOCA, both diesel generators start automatically. If preferred power is available to the Class 1E bus following a LOCA, the ESF loads will be started sequentially. However, in the event that preferred power is lost, the load sequencing system will shed all loads, connect each diesel generator to its associated Class 1E bus, and sequentially start the ESF equipment. The diesel generators are arranged so that a failure of a single component will not prevent the safe shutdown of the reactor. The onsite Class 1E dc power supply consists of four independent battery systems. Failure of a single component in the dc power supply will not impair function of the ESF required to maintain the reactor in a safe condition.

CRITERION 18 - <u>INSPECTION AND TESTING OF ELECTRIC POWER SYSTEMS</u>

"Electric power systems important to safety shall be designed to permit appropriate periodic inspection and testing of important areas and features, such as wiring, insulation, connections, and switchboards, to assess the continuity of the systems and the condition of their components. The systems shall be designed with a capability to test periodically (1) the operability and functional performance of the components of the systems, such as onsite power sources, relays, switches, and buses, and (2) the operability of the systems as a whole and, under conditions as close to design as practical, the full operation sequence that brings the systems into operation, including operation of applicable portions of the protection system, and the transfer of power among the nuclear power unit, the offsite power system, and the onsite power system."

DISCUSSION

Class 1E electric power systems are designed as described below in order that the following aspects of the system can be periodically tested:

A. The operability and functional performance of the components of Class 1E electric power systems (diesel generators, ESF buses, dc system).

B. The operability of these electric power systems as a whole and under conditions as close to design as practical, including the full operational sequence that actuates these systems.

The non-Class 1E switchyard circuit breakers will be inspected, maintained, and tested on a routine basis without affecting the rest of the system. Protective relaying will be periodically tested, and transmission lines will be periodically inspected.

Any one of the non-Class 1E reserve auxiliary transformers or the standby auxiliary transformer, and its circuit to the Class 1E buses can be taken out of service and tested periodically. Each transformer includes the capacity to supply power to one train of Class 1E loads. The 4160-V and 480-V circuit breakers and the associated equipment will be tested one at a time only while redundant equipment is operational.

The dc system is provided with detectors to indicate and alarm when there is a ground existing on any part of the system. During plant operation, normal maintenance may be performed.

Provisions for the testing of Class 1E ac electric power systems, Class 1E dc power systems, and the standby power supplies (diesel generators) are described in chapter 8.

CRITERION 19 - CONTROL ROOM

"A control room shall be provided from which actions can be taken to operate the nuclear power unit safely under normal conditions and to maintain it in a safe condition under accident conditions, including LOCAs. Adequate radiation protection shall be provided to permit access and occupancy of the control room under accident conditions without personnel receiving radiation exposures in excess of 5 rem whole body, or its equivalent, to any part of the body, for the duration of the accident.

"Equipment at appropriate locations outside the control room shall be provided (1) with a design capability for prompt hot shutdown of the reactor, including necessary instrumentation and controls to maintain the unit in a safe condition during hot shutdown and (2) with a potential capability for subsequent cold shutdown of the reactor through the use of suitable procedures."

DISCUSSION

A shared control room is provided for the control of the VEGP units from which actions can be taken to operate each nuclear power unit safely under normal conditions and to maintain it in a safe manner under accident conditions, including LOCAs. Operator action outside of the control room to mitigate the consequences of an accident is permitted. The control room and its post-accident ventilation systems are designed to satisfy Seismic Category 1 requirements, as discussed in chapter 3. Adequate concrete shielding and radiation protection are provided against direct gamma radiation and inhalation doses resulting from a postulated release of fission products inside the containment structure based on the assumptions contained in

Regulatory Guide 1.4. The shielding and the control room standby air-conditioning system allow access to and occupancy of the control rooms under accident conditions without personnel receiving radiation exposures in excess of 5 rem whole body or its equivalent to any part of the body for the duration of the accident. (Refer to chapter 15.) Fission product removal is provided in the control room recirculation equipment to remove iodine and particulate matter, thereby minimizing the thyroid dose which could result from the accident. The control room habitability features are described in chapter 6.

In the event that the operators are forced to abandon the control room, panel-mounted instrumentation and controls are provided on the train-related shutdown panels to achieve and maintain the plant in the safe shutdown condition. (See section 7.4.)

3.1.3 PROTECTION AND REACTIVITY CONTROL SYSTEMS

CRITERION 20 - PROTECTION SYSTEM FUNCTIONS

"The protection system shall be designed (1) to initiate automatically the operation of appropriate systems, including the reactivity control systems, to assure that specified acceptable fuel design limits are not exceeded as a result of anticipated operational occurrences and (2) to sense accident conditions and to initiate the operation of systems and components important to safety."

DISCUSSION

A fully automatic protection system with appropriate redundant channels is provided to cope with transient events where insufficient time is available for manual corrective action. The design basis for all protection systems is in accordance with the guidelines of Institute of Electrical and Electronic Engineers (IEEE) Standards 279-1971 and 379-1972. The reactor protection system automatically initiates a reactor trip when any variable monitored by the system or combination of monitored variables exceeds the normal operating range. Setpoints are designed to provide an envelope of safe operating conditions with adequate margin for uncertainties to ensure that the fuel design limits are not exceeded.

Reactor trip is initiated by removing power to the rod drive mechanisms of all the rod cluster control assemblies. This causes the rods to insert by gravity, thus rapidly reducing the reactor power. The response and adequacy of the protection system have been verified by analysis of anticipated transients.

The ESF actuation system automatically initiates emergency core cooling and other safety functions by sensing accident conditions, using redundant analog channels measuring diverse variables. Manual actuation of safety features may be performed where ample time is available for operator action. The ESF actuation system automatically trips the reactor on a manual or automatic safety injection signal.

CRITERION 21 - PROTECTION SYSTEM RELIABILITY AND TESTABILITY

"The protection system shall be designed for high functional reliability and inservice testability commensurate with the safety functions to be performed. Redundancy and independence designed into the protection system shall be sufficient to assure that (1) no single failure results in the loss of the protection function and (2) removal from service of any component or channel does not result in the loss of the required minimum redundancy unless the acceptable reliability of operation of the protection system can be otherwise demonstrated. The protection system shall be designed to permit periodic testing of its functioning when the reactor is in operation, including a capability to test channels independently to determine failures and losses of redundancy that may have occurred."

DISCUSSION

The protection system is designed for functional reliability and inservice testability. The design employs redundant logic trains and measurement and equipment diversity.

The protection system, including the ESF test cabinet, is designed to meet Regulatory Guide 1.22 and conform to the requirements of IEEE Standards 279-1971 and 379-1972. Functions that cannot be tested with the reactor at power are tested during shutdown, as allowed by the regulatory guide and the above standards.

In cases where actuated equipment cannot be tested at power, the channels and logic associated with this equipment, up to the final actuation device, have the capability for testing at power. Such testing discloses failures of reduction in redundancy which may have occurred.

Removal from service of any single channel or component does not result in the loss of minimum required redundancy. For example, a two-of-three function is placed in the one-of-two mode when one channel is removed. (Note that distinction is made between channels and trains in this discussion. A train may be removed from service only during testing.) Bypassed and inoperable status indication for safety-related systems is provided in accordance with Regulatory Guide 1.47.

Semiautomatic testers are built into each of the two logic trains of the protection system. These testers have the capability of testing the system logic very rapidly while the reactor is at power. A self-testing provision is designed into each tester. (For a detailed description of reliability and testability of the protection system, refer to section 7.2.)

CRITERION 22 - PROTECTION SYSTEM INDEPENDENCE

"The protection system shall be designed to assure that the effects of natural phenomena, and of normal operating, maintenance, testing, and postulated accident conditions on redundant channels do not result in the loss of the protection function or shall be demonstrated to be acceptable on some other defined basis. Design techniques, such as functional diversity or diversity in component design and principles of operation, shall be used to the extent practical to prevent loss of the protection function."

DISCUSSION

Design of the protection systems includes consideration of natural phenomena, normal maintenance, testing, and accident conditions so that the protection functions are always available.

Protection system components are designed, arranged, and qualified for operation in the environment accompanying any emergency situation in which the components are required to function.

Functional diversity has been designed into the system. The extent of this functional diversity has been evaluated for a variety of postulated accidents. Diverse protection functions will automatically terminate an accident before intolerable consequences can occur.

Sufficient redundancy and independence are designed into the protection systems to assure that no single failure or removal from service of any component or channel of a system would result in loss of the protection function. Functional diversity and consequential location diversity are designed into the system. Automatic reactor trips are based upon neutron flux measurements, reactor coolant loop temperature measurements, pressurizer pressure and level measurements, and reactor coolant pump power supply underfrequency, undervoltage measurements, and other parameters. Trips may also be initiated manually or by a safety injection signal. See chapter 7 for details.

High quality components, conservative design and applicable quality control, inspection, calibration, and tests are utilized to guard against common-mode failure. Qualification testing and analysis is performed on the various safety systems to demonstrate functional operation at normal and post-accident conditions of temperature, humidity, pressure, and radiation for specified periods, if required. Typical protection system equipment is subjected to type tests under simulated seismic conditions, using conservatively large accelerations and applicable frequencies. The test results indicate no loss of the protection function. (Refer to sections 3.10.B, 3.10.N, 3.11.B, and 3.11.N for further details.)

CRITERION 23 - PROTECTION SYSTEM FAILURE MODES

"The protection system shall be designed to fail into a safe state or into a state demonstrated to be acceptable on some other defined basis if conditions such as disconnection of the system, loss of energy (e.g., electric power, instrument air) or postulated adverse environments (e.g., extreme heat or cold, fire, pressure, steam, water, and radiation) are experienced."

DISCUSSION

The protection system is designed with consideration of the most probable failure modes of the components under various perturbations of the environment and energy sources. Each reactor trip channel, with the exception of the reactor coolant pump (RCP) underfrequency (UF) and undervoltage (UV) trip input to the reactor trip system, is designed on the deenergize-to-trip principle so loss of power, disconnection, open channel faults, and the majority of the internal channel short circuit faults cause the channel to go into its tripped mode. The RCP UF and UV relay logic input to the reactor trip system is energize-to-trip. This meets the requirements of Criterion 23 due to the redundancy and train separation of the RCP UF and UV trip channels. Failure of one channel will not prevent a reactor trip in the event of an actual UF or UV event.

Similarly, that portion of the ESF actuation system provided for actuation of auxiliary feedwater system and containment ventilation isolation is designed to fail into a safe state, except for the final output relays. The relays are energized to actuate, as are the pumps and motor-operated valves of the actuated equipment.

For a more detailed description of the protection system, refer to chapter 7.

CRITERION 24 - SEPARATION OF PROTECTION AND CONTROL SYSTEMS

"The protection system shall be separated from the control systems to the extent that failure of any single control system component or channel, or failure or removal from service of any single protection system component or channel which is common to the control and protection systems, leaves intact a system satisfying all reliability, redundancy, and independence requirements of the protection system. Interconnection of the protection and control systems shall be limited so as to assure that safety is not significantly impaired."

DISCUSSION

The protection system is separate and distinct from the control systems, as described in chapter 7. Control systems are, in some cases, dependent on the protection system in that control signals are derived from protection system measurements, where applicable. These signals are transferred to the control system by isolation devices which are classified as protection components. The adequacy of the system isolation has been verified by testing under conditions of postulated credible faults. The failure of any single control system component or channel, or the failure or removal from service of any single protection system component or channel which is common to the control and protection system, leaves intact a system which satisfies the requirements of the protection system. The removal of a train from service is allowed only during testing of the train. Distinction between channel and train is made in the discussions.

CRITERION 25 - PROTECTION SYSTEM REQUIREMENTS FOR REACTIVITY CONTROL MALFUNCTIONS

"The protection system shall be designed to assure that specified acceptable fuel design limits are not exceeded for any single malfunction of the reactivity control systems, such as accidental withdrawal (not ejection or dropout) of the control rods."

DISCUSSION

The protection system is designed to limit reactivity transients so that the fuel design limits are not exceeded. Reactor shutdown by control rod insertion is completely independent of the normal control function since the trip breakers interrupt power to the rod mechanisms regardless of existing control signals. Thus, in the postulated accidental withdrawal of a control rod or control rod bank (assumed to be initiated by a control malfunction) neutron flux, temperature, pressure, level, and flow signals would be generated independently. Any of these signals (trip demands) would operate the breakers to trip the reactor.

Analyses of the effects of possible malfunctions are discussed in chapter 15. These analyses show that for postulated boron dilution during refueling, startup, or manual or automatic operation at power, the operator has ample time to determine the cause of dilution, terminate the source of dilution, and initiate reboration before the shutdown margin is lost. The analyses show that acceptable fuel damage limits are not exceeded even in the event of a single malfunction of either system.

CRITERION 26 - REACTIVITY CONTROL SYSTEM REDUNDANCY AND CAPABILITY

"Two independent reactivity control systems of different design principles shall be provided. One of the systems shall use control rods, preferably including a positive means for inserting the rods, and shall be capable of reliably controlling reactivity changes to assure that under conditions of normal operation, including anticipated operational occurrences, and with appropriate margin for malfunctions such as stuck rods, specified acceptable fuel design limits are not exceeded. The second reactivity control system shall be capable of reliably controlling the rate of reactivity changes resulting from planned, normal power changes (including xenon burnout) to assure that the acceptable fuel design limits are not exceeded. One of the systems shall be capable of holding the reactor core subcritical under cold conditions."

DISCUSSION

Two reactivity control systems are provided. These are rod cluster control assemblies (RCCAs) and chemical shim (boric acid). The RCCAs are inserted into the core by the force of gravity.

During operation, the shutdown rod banks are fully withdrawn. The control rod system automatically maintains a programmed average reactor temperature compensating for reactivity effects associated with scheduled and transient load changes. The shutdown rod banks, along with the control banks, are designed to shut down the reactor with adequate margin under conditions of normal operation and anticipated operational occurrences, thereby ensuring that specified fuel design limits are not exceeded. The most restrictive period in the core life is assumed in all analyses, and the most reactive rod cluster is assumed to be in the fully withdrawn position.

The boron system will maintain the reactor in the cold shutdown state independent of the position of the control rods and can compensate for xenon burnout transients.

Details of the construction of the RCCAs are presented in chapter 4, and the operation is discussed in chapter 7. The means of controlling the boric acid concentration is described in chapter 9. Performance analyses under accident conditions are included in chapter 15.

CRITERION 27 - COMBINED REACTIVITY CONTROL SYSTEMS CAPABILITY

"The reactivity control systems shall be designed to have a combined capability, in conjunction with poison addition by the emergency core cooling system, of reliably controlling reactivity changes to assure that under postulated accident conditions and with appropriate margin for stuck rods the capability to cool the core is maintained."

DISCUSSION

The facility is provided with means of making and holding the core subcritical under any anticipated conditions and with appropriate margin for contingencies. These means are discussed in detail in chapters 4 and 9. Combined use of the rod cluster control system and the chemical shim control system permits the necessary shutdown margin to be maintained during long-term xenon decay and plant cooldown. The single highest worth RCCA is assumed to be stuck full out upon trip for this determination.

CRITERION 28 - REACTIVITY LIMITS

"The reactivity control systems shall be designed with appropriate limits on the potential amount and rate of reactivity increase to assure that the effects of postulated reactivity accidents can neither (1) result in damage to the reactor coolant pressure boundary greater than limited local yielding nor (2) sufficiently disturb the core, its support structures, or other reactor pressure vessel internals to impair significantly the capability to cool the core. These postulated reactivity accidents shall include consideration of rod ejection (unless prevented by positive means), rod dropout, steam line rupture, changes in reactor coolant temperature and pressure, and cold water addition."

DISCUSSION

The maximum reactivity worth of the control rods and the maximum rates of reactivity insertion employing control rods and boron removal are limited to values that prevent any reactivity increase from rupturing the RCS boundary or disrupting the core or vessel internals to a degree that could impair the effectiveness of emergency core cooling.

The appropriate reactivity insertion rate for the withdrawal of RCCAs and the dilution of the boric acid in the reactor coolant systems are limited by design. The reactor protection system and engineered safety features actuation system provide protection for such events as a rod ejection accident and steamline break. Protection system setpoints are contained in the Technical Specifications. Reactivity insertion rates, dilution, and withdrawal limits are also discussed in chapter 4. The capability of the chemical and volume control system to avoid an inadvertent excessive rate of boron dilution is discussed in chapter 9. The relationship of the reactivity insertion rates to plant safety is discussed in chapter 15.

Core cooling capability following accidents, such as rod ejection, steam line break, etc., is assured by keeping the reactor coolant pressure boundary stresses within faulted condition limits, as specified by applicable ASME codes. Structural deformations are also checked and limited to values that do not jeopardize the operation of needed safety features.

CRITERION 29 - PROTECTION AGAINST ANTICIPATED OPERATIONAL OCCURRENCES

"The protection and reactivity control systems shall be designed to assure an extremely high probability of accomplishing their safety functions in the event of anticipated operational occurrences."

DISCUSSION

The protection and reactivity control systems have an extremely high probability of performing their required safety functions in any anticipated operational occurrences. Diversity and redundancy, coupled with a quality assurance program and analyses, support this probability as does operating experience in plants using the same basic design. Failure modes of system components are designed to be safe modes. Loss of power to the protection system results in a reactor trip. Details of system design are covered in chapters 4 and 7.

3.1.4 FLUID SYSTEMS

CRITERION 30 - QUALITY OF REACTOR COOLANT PRESSURE BOUNDARY

"Components which are part of the reactor coolant pressure boundary shall be designed, fabricated, erected, and tested to the highest quality standards practical. Means shall be provided for detecting and, to the extent practical, identifying the location of the source of reactor coolant leakage."

DISCUSSION

All RCS components are designed, fabricated, inspected, and tested in conformance with the ASME Boiler and Pressure Vessel Code, Section III.

All balance of plant components are classified according to Regulatory Guide 1.26, and all nuclear steam supply system (NSSS) components are classified according to ANSI N18.2-1973 and ANSI N18.2A-1975 (which is an acceptable alternative to Regulatory Guide 1.26) and are accorded all the quality measures appropriate to these classifications. The design bases and evaluations of the RCS are discussed in chapter 5.

A number of methods are available for detecting reactor coolant leakage. The reactor vessel closure joint is provided with a temperature monitored leakoff between double gaskets. Leakage inside the reactor containment is drained to the containment building and reactor cavity sumps, where the level is monitored. Leakage is also detected by measuring the airborne activity and humidity of the containment. Monitoring the inventory of reactor coolant in the system at the pressurizer, volume control tank, and reactor coolant drain tank provides an accurate indication of integrated leakage. Refer to chapter 5 for complete description of the RCPB leakage detection system.

CRITERION 31 - FRACTURE PREVENTION OF REACTOR COOLANT PRESSURE BOUNDARY

"The reactor coolant pressure boundary shall be designed with sufficient margin to assure that when stressed under operating, maintenance, testing, and postulated accident conditions (1) the boundary behaves in a nonbrittle manner and (2) the probability of rapidly propagating fracture is minimized. The design shall reflect consideration of service temperatures and other conditions of the boundary material under operating, maintenance, testing, and postulated accident conditions and the uncertainties in determining (1) material properties, (2) the effects of irradiation on material properties, (3) residual, steady state, and transient stresses, and (4) size of flaws."

DISCUSSION

Close control is maintained over material selection and fabrication for the RCS to assure that the boundary behaves in a nonbrittle manner. The RCS materials which are exposed to the coolant are corrosion-resistant stainless steel or Inconel. The nil ductility transition reference temperature (RT_{NDT}) of the reactor vessel structural steel is established by Charpy V-notch and drop weight tests in accordance with 10 CFR 50, Appendix G, Fracture Toughness Requirements.

The reactor vessel specification imposes the following requirements which are not specified by the ASME code.

A. The performance of a 100-percent volumetric ultrasonic shear wave test of reactor vessel plate and a post hydro-test ultrasonic map of all welds in the pressure vessel are required. Cladding bond ultrasonic inspection to more restrictive requirements than those specified in the code is also required to preclude interpretation problems during inservice inspection.

B. In the surveillance programs, the evaluation of the radiation damage is based on pre-irradiation testing of Charpy V-notch and tensile specimens and postirradiation testing of Charpy V-notch, tensile, and 1/2 T compact tension specimens. These programs are directed toward evaluation of the effect of radiation on the fracture toughness of reactor vessel steels based on the reference transition temperature approach and the fracture mechanics approach, and are in accordance with American Society of Testing Materials (ASTM) E-185-82, Standard Recommended Practice for Surveillance Tests for Nuclear Reactor Vessels, and the requirements of 10 CFR 50, Appendix H, Reactor Vessel Material Surveillance Program Requirements.

C. Reactor vessel core region material chemistry (copper, phosphorous, and vanadium) is controlled to reduce sensitivity to embrittlement due to irradiation over the life of the plant.

The fabrication and quality control techniques used in the fabrication of the RCS are equivalent to those used for the reactor vessel. The inspections of reactor vessel, pressurizer, piping, pumps, and steam generators are governed by ASME code requirements. (Refer to chapter 5 for details.)

Allowable pressure-temperature relationships for plant heatup and cooldown rates are calculated, using methods derived from the ASME Code, Section III, Appendix G, Protection Against NonDuctile Failure. The approach specifies that the allowable stress intensity factors for all vessel operating conditions do not exceed the reference stress intensity factor (KIR) for the metal temperature at any time. Operating specifications include conservative margins for predicted changes in the material reference temperatures (RT_{NDT}) due to irradiation.

CRITERION 32 - INSPECTION OF REACTOR COOLANT PRESSURE BOUNDARY

"Components which are part of the reactor coolant pressure boundary shall be designed to permit (1) periodic inspection and testing of important areas and features to assess their structural and leak-tight integrity and (2) an appropriate material surveillance program for the reactor pressure vessel."

DISCUSSION

The design of the RCPB provides accessibility to the entire internal surfaces of the reactor vessel and most external zones of the vessel, including the nozzle to reactor coolant piping welds, the top and bottom heads, and external surfaces of the reactor coolant piping, except for the area of pipe within the primary shielding concrete. The inspection capability complements the leakage detection systems in assessing the pressure boundary components' integrity. The RCPB will be periodically inspected under the provisions of the ASME Code, Section XI.

Monitoring of changes in the fracture toughness properties of the reactor vessel core region plates, forgings, weldments, and associated heat-treated zones is performed in accordance with 10 CFR 50, Appendix H. Samples of reactor vessel plate materials are retained and catalogued in case future engineering development shows the need for further testing.

The material properties surveillance program includes not only the conventional tensile and impact tests, but also fracture mechanics specimens. The observed shifts in RT_{NDT} of the core region materials with irradiation will be used to confirm the allowable limits calculated for all operational transients.

The design of the RCPB piping provides for accessibility of all welds requiring inservice inspection under the provisions of the ASME Code, Section XI. Removable insulation is provided at all welds requiring inservice inspection. The inservice inspection program is discussed in detail in chapter 6.

CRITERION 33 - REACTOR COOLANT MAKEUP

"A system to supply reactor coolant makeup for protection against small breaks in the reactor coolant pressure boundary shall be provided. The system safety function shall be to assure that specified acceptable fuel design limits are not exceeded as a result of reactor coolant loss due to leakage from the reactor coolant pressure boundary and rupture of small piping or other small components which are part of the boundary. The system shall be designed to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished using the piping, pumps, and valves used to maintain coolant inventory during normal reactor operation."

DISCUSSION

The chemical and volume control system provides a means of reactor coolant makeup and adjustment of the boric acid concentration. Makeup is added automatically if the level in the volume control tank falls below a preset level. The normal charging pump or centrifugal charging pump is used as the normal means of reactor coolant makeup. This pump is powered from the offsite power system.

The centrifugal charging pumps are a backup method of providing reactor coolant makeup. The centrifugal charging pumps are capable of supplying the required makeup and reactor coolant seal injection flow when power is available from either onsite or offsite electric power systems. Functional reliability is assured by provision of standby components assuring a safe response to probable modes of failure. Details of system design, including descriptions of the effects of small piping and component ruptures, are provided in sections 6.3 and 9.3 and in chapter 15; details of the electric power system are included in chapter 8.

CRITERION 34 - RESIDUAL HEAT REMOVAL

"A system to remove residual heat shall be provided. The system safety function shall be to transfer fission product decay heat and other residual heat from the reactor core at a rate such that specified acceptable fuel design limits and the design conditions of the reactor coolant pressure boundary are not exceeded.

"Suitable redundancy in components and features and suitable interconnections, leak detection, and isolation capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure."

DISCUSSION

The residual heat removal (RHR) system, in conjunction with the steam and power conversion system, is designed to transfer the fission product decay heat and other residual heat from the reactor core at a rate which keeps the fuel within acceptable limits. The RHR system functions when temperature and pressure are below approximately 350°F and 400 psig, respectively.

Redundancy of the RHR system is provided by two residual heat removal pumps (located in separate flood-proof compartments, with means available for draining and monitoring leakage), two heat exchangers, and associated piping, cabling, and electric power sources. (For a more detailed description of RHR system redundancy, refer to subsection 5.4.7.) The RHR system is able to operate on either the onsite or offsite electrical power system.

Redundancy of heat removal at temperatures above approximately 350°F is provided by the four steam generators, four atmospheric relief valves, and the auxiliary feedwater system.

Details of the system design are provided in subsection 5.4.7, chapter 9, and chapter 10.

CRITERION 35 - EMERGENCY CORE COOLING

"A system to provide abundant emergency core cooling shall be provided. The system safety function shall be to transfer heat from the reactor core following any loss of reactor coolant at a rate such that (1) fuel and clad damage that could interfere with continued effective core cooling is prevented and (2) clad metal-water reaction is limited to negligible amounts.

"Suitable redundancy in components and features and suitable interconnections, leak detection, isolation, and containment capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure."

DISCUSSION

The emergency core cooling system (ECCS) has the capability to mitigate the effects of any LOCA within the design bases. Cooling water is provided in an emergency to transfer heat from the core at a rate sufficient to maintain the core in a coolable geometry and to assure that clad metal-water reaction is limited to less than 1 percent. Design provisions assure performance of the required safety functions even with a postulated single failure.

Emergency core cooling is provided even if there should be a failure of any component in the system. A passive system of four accumulators which do not require any external signals or source of power to operate provide the short-term cooling requirements for large reactor coolant pipe system breaks. Three independent and redundant pumping systems are provided: the charging system, safety injection system, and residual heat removal system. The charging system is a high-pressure, low-flow system capable of providing the required emergency cooling for small breaks. The safety injection system is an intermediate-pressure, intermediate-flow system capable of providing the required emergency cooling for medium-sized breaks. The charging system can be operated to complement the safety injection system. The RHR system is a low-pressure, high-flow system capable of providing the required emergency cooling for large breaks. The charging system and safety injection system can be operated to complement the RHR system. These systems are arranged so that the single failure of any active component does not interfere with meeting the short-term cooling requirements.

The primary function of the ECCS is to deliver borated cooling water to the reactor core in the event of a LOCA. This limits the fuel-clad temperature; ensures that the core will remain intact and in place, with its essential heat transfer geometry preserved; and prevents a return to criticality. This protection is afforded for:

A. All pipe breaks sizes up to and including the hypothetical circumferential rupture of the largest pipe of a reactor coolant loop.

B. A loss-of-coolant associated with a rod ejection accident.

The ECCS is described in chapter 6. The LOCA, including an evaluation of consequences, is discussed in chapter 15.

CRITERION 36 - INSPECTION OF EMERGENCY CORE COOLING SYSTEM

"The emergency core cooling system shall be designed to permit appropriate periodic inspection of important components, such as spray rings in the reactor pressure vessel, water injection nozzles, and piping, to assure the integrity and capability of the system."

DISCUSSION

The ECCS is accessible for visual inspection and for nondestructive inservice inspection, to satisfy the ASME Code, Section XI.

Components outside the containment are accessible for leaktightness inspection during operation of the reactor.

Details of the inspection program for the ECCS are discussed in section 6.3, the Inservice Inspection Program, and the Technical Specifications.

CRITERION 37 - TESTING OF EMERGENCY CORE COOLING SYSTEM

"The emergency core cooling system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leak-tight integrity of its components, (2) the operability and performance of the active components of the system, and (3) the operability of the system as a whole and under conditions as close to design as practical, the performance of the full operational sequence that brings the system into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of the associated cooling water system."

DISCUSSION

The design of the ECCS permits periodic testing of both active and passive components of the ECCS.

Preoperational performance tests of the ECCS components are performed by the manufacturer. Initial system hydrostatic and functional flow tests demonstrate structural and leaktight integrity of components and proper functioning of the system. Thereafter, periodic tests demonstrate that components are functioning properly.

Each active component of the ECCS may be individually operated on the normal power source or transferred to standby power sources at any time during normal plant operation to demonstrate operability. The centrifugal charging pumps are available for operation as necessary during plant operation. The test of the safety injection pumps employs the minimum flow recirculation test line which connects back to the refueling water storage tank. Remote-operated valves are exercised and actuation circuits tested. The automatic actuation circuitry, valves, and pump breakers may be checked during integrated system tests performed during a planned cooldown of the RCS.

Design provisions include special instrumentation, testing, and sampling lines to perform the tests during plant shutdown to demonstrate proper automatic operation of the ECCS. (Refer to section 1.9 for a discussion of Regulatory Guide 1.22.) A test signal is applied to initiate automatic action, and verification is made that the safety injection pumps attain required discharge heads. The test demonstrates the operation of the valves, pump circuit breakers, and automatic circuitry. In addition, the periodic recirculation to the refueling water storage tank can verify the ECCS delivery capability. This recirculation test includes all but the last valve, which connects to the reactor coolant piping.

The design provides for capability to test initially, to the extent practical, the full operational sequence up to the design conditions, including transfer to alternate power sources for the ECCS to demonstrate the state of readiness and capability of the system. This functional test is performed with the water level below the safety injection signal setpoint in the pressurizer and with the RCS initially cold and depressurized. The ECCS valving is set to initially simulate the system alignment for plant power operation.

Details of the ECCS are found in chapter 6. Performance under accident conditions is evaluated in chapter 15. Surveillance requirements are identified in the Technical Specifications.

CRITERION 38 - CONTAINMENT HEAT REMOVAL SYSTEM

"A system to remove heat from the reactor containment shall be provided. The system safety function shall be to reduce rapidly, consistent with the functioning of other associated systems, the containment pressure and temperature following any LOCA and maintain them at acceptably low levels.

"Suitable redundancy in components and features and suitable interconnections, leak detection, isolation, and containment capabilities shall be provided to assure that for onsite electrical power system operation (assuming offsite power is not available) and for offsite electrical power system operation (assuming onsite power is not available) the system safety function can be accomplished, assuming a single failure."

DISCUSSION

The containment spray and containment fan cooler systems, in conjunction with the ECCS, are capable of removing sufficient energy and subsequent decay energy from the containment following the hypothesized LOCA to maintain the containment pressure below the containment design pressure. During the post-accident injection phase, water for the containment spray system and ECCS is drawn from the refueling water storage tank. During the later recirculation phase, spray water and ECCS water are pumped from the containment sump.

Each of the containment spray and containment fan cooler systems consists of two independent subsystems supplied from separate Class 1E power buses. No single failure, including loss of onsite or offsite electrical power, can cause loss of more than half of the installed 200-percent cooling capacity. The containment spray system and containment fan coolers are discussed in chapter 6. Electrical facilities are described in chapter 8. A containment pressure and temperature analysis following a LOCA is given in chapter 6, with additional results found in chapter 15.

CRITERION 39 - INSPECTION OF CONTAINMENT HEAT REMOVAL SYSTEM

"The containment heat removal system shall be designed to permit appropriate periodic inspection of important components, such as the torus, sumps, spray nozzles and piping, to assure the integrity and capability of the system."

DISCUSSION

The essential equipment of the containment spray system (CSS) is outside the containment, except for risers, distribution header piping, spray nozzles, and the containment sumps. The containment sumps, spray piping, and nozzles can be inspected during shutdown. Portions of the containment spray suction piping and the RHR suction piping from the containment recirculation sumps are embedded in concrete and are not accessible for inspection. Associated equipment outside the containment can be visually inspected.

The containment air coolers and associated cooling water system piping inside the containment can be inspected during shutdowns.

These periodic inspections assure that the capability of these heat removal systems as specified in the Technical Specifications is met.

(For details on the containment air coolers and containment spray system, see chapter 6.)

CRITERION 40 - TESTING OF CONTAINMENT HEAT REMOVAL SYSTEM

"The containment heat removal system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leak-tight integrity of its components, (2) the operability and performance of the active components of the system, and (3) the operability of the system as a whole, and, under conditions as close to the design as practical, the performance of the full operational sequence that brings the system into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of the associated cooling water system."

DISCUSSION

The containment spray system and the containment fan cooling system are designed to permit periodic testing to assure the structural and leaktight integrity of their components and to assure the operability and performance of the active components of the systems. All active components of the CSS and delivery piping up to the last powered valve before the spray nozzle have the capability to be tested during reactor power operation. In addition, when the unit is shut down, smoke or air can be blown through the test connections for visual verification of the flow path. All safety-related active components of the containment fan cooling system can be tested to verify operability during reactor power operation. In addition, since the containment fan cooling system is a normally operating system, the performance and operability of portions of the system are continuously verified during normal reactor power operation. The facility design allows, under conditions as close to the design as practicable, the performance of a full operational sequence that brings these systems into operation. More complete discussions of the testing of these systems are in chapters 6 and 8 and the Technical Specifications.

CRITERION 41 - CONTAINMENT ATMOSPHERE CLEANUP

"Systems to control fission products, hydrogen, oxygen, and other substances which may be released into the reactor containment shall be provided, as necessary, to reduce, consistent with the functioning of other associated systems, the concentration and quantity of fission products released to the environment following postulated accidents and to control the concentration of hydrogen or oxygen and other substances in the containment atmosphere following postulated accidents to assure that containment integrity is maintained.

"Each system shall have suitable redundancy in components and features and suitable interconnections, leak detection, isolation, and containment capabilities to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) its safety function can be accomplished, assuming a single failure."

DISCUSSION

The CSS serves to remove radioiodine and other airborne particulate fission products from the containment atmosphere following a LOCA. The system consists of two independent systems, each supplied from separate electrical power buses, as described in chapter 8. Either subsystem alone can provide the fission product removal capacity for which credit is taken in chapter 15, in conformance with Regulatory Guide 1.4.

The generation of hydrogen in the containment under post-accident conditions has been evaluated, using the assumptions of Regulatory Guide 1.7. (See chapter 6.) A post-accident hydrogen recombiner system is provided with redundancy of vital components so that a single failure does not prevent timely operation of the system. This system is described in subsection 6.2.5. The post-LOCA purge exhaust system is provided as a backup. No single failure causes both subsystems to fail to operate.

CRITERION 42 - <u>INSPECTION OF CONTAINMENT ATMOSPHERE CLEANUP SYSTEM</u>

"The containment atmosphere cleanup systems shall be designed to permit appropriate periodic inspection of important components, such as filter frames, ducts, and piping, to assure the integrity and capability of the systems."

<u>DISCUSSION</u>

The containment atmosphere cleanup systems are designed and located so that they can be inspected periodically, as required. The essential equipment of the CSS is outside the containment, except for risers, distribution header piping, and spray nozzles in the containment. The hydrogen recombiners are located inside the containment. The post-LOCA purge exhaust filter unit and the hydrogen monitors are located outside the containment. The equipment outside the containment may be inspected during normal power operation. Components of the CSS, the post-LOCA purge exhaust system, and the hydrogen recombiner and monitoring system located inside the containment, can be inspected during refueling shutdowns. (See chapter 6 for details on these systems.)

CRITERION 43 - <u>TESTING OF CONTAINMENT ATMOSPHERE CLEANUP SYSTEMS</u>

"The containment atmosphere cleanup systems shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leak-tight integrity of its components, (2) the operability and performance of the active components of the systems such as fans, filters, dampers, pumps, and valves, and (3) the operability of the systems as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the systems into operation, including operation of applicable portions of the protection system, the transfer between normal and emergency power sources, and the operation of associated systems."

<u>DISCUSSION</u>

The CSS which serves as the containment atmosphere cleanup system can be tested. The operation of the spray pumps can be tested by recirculation to the refueling water storage tank through a test line. The system valves can be operated through their full travel. The system is checked for leaktightness during testing. (See subsection 6.2.2.2 for details and chapter 8 for electrical power details.) The spray headers and nozzles can be smoke or air tested, as described in the response to Criterion 40.

CRITERION 44 - <u>COOLING WATER</u>

"A system to transfer heat from structures, systems, and components important to safety to an ultimate heat sink shall be provided. The system safety function shall be to transfer the combined heat load of these structures, systems, and components under normal operating and accident conditions.

"Suitable redundancy in components and features and suitable interconnections, leak detection, and isolation capabilities shall be provided to assure that for onsite electric power system operation (assuming offsite power is not available) and for offsite electric power system operation (assuming onsite power is not available) the system safety function can be accomplished assuming a single failure."

DISCUSSION

The component cooling water (CCW) and nuclear service cooling water (NSCW) systems are provided to transfer heat from plant safety-related components to the ultimate heat sink. These systems are designed to transfer their respective heat loads under all anticipated normal and accident conditions. Suitable redundancy, leak detection, systems interconnection, and isolation capabilities are incorporated in the design of these systems to assure the required safety function, assuming a single failure, with either onsite or offsite power.

Complete descriptions of the NSCW system and the CCW system are given in chapter 9.

CRITERION 45 - INSPECTION OF COOLING WATER SYSTEM

"The cooling water system shall be designed to permit appropriate periodic inspection of important components, such as heat exchangers and piping, to assure the integrity and capability of the system."

DISCUSSION

The CCW system and portions of the NSCW system are capable of being monitored during normal operation. The important components are located in accessible areas with the exception of any underground piping for the NSCW system. These components have suitable manholes, handholes, inspection ports, or other appropriate design and layout features to allow periodic inspection. The integrity of any underground piping will be demonstrated by pressure and functional tests. Piping to and from the containment air coolers is accessible for inspection during reactor shutdown and refueling periods. These systems are discussed in chapter 9.

CRITERION 46 - TESTING OF COOLING WATER SYSTEM

"The cooling water system shall be designed to permit appropriate periodic pressure and functional testing to assure (1) the structural and leak-tight integrity of its components, (2) the operability and the performance of the active components of the system, and (3) the operability of the system as a whole and, under conditions as close to design as practical, the performance of the full operational sequence that brings the system into operation for reactor shutdown and for LOCA, including operation of applicable portions of the protection system and the transfer between normal and emergency power sources."

DISCUSSION

The CCW and NSCW systems operate continuously during normal plant operation and shutdown, under flow and pressure conditions that approximate the accident conditions. These operations demonstrate the operability, performance, and structural and leaktight integrity of all cooling water system components.

These cooling water systems are designed to include the capability for testing through the full operational sequence that brings the system into operation for reactor shutdown and for LOCAs, including operation of applicable portions of the protection system and the transfer between normal and emergency power sources. The CCW system and NSCW system are capable of being tested during normal operation by alternating operation of the systems between the redundant trains.

For a detailed description of the cooling water systems, refer to section 9.2.

3.1.5 REACTOR CONTAINMENT

CRITERION 50 - CONTAINMENT DESIGN BASIS

"The reactor containment structure, including access opening, penetrations, and the containment heat removal system, shall be designed so that the containment structure and its internal compartments can accommodate, without exceeding the design leakage rate and with sufficient margin, the calculated pressure and temperature conditions resulting from any LOCA. This margin shall reflect consideration of (1) the effects of potential energy sources which have not been included in the determination of the peak conditions, such as energy in steam generators and energy from metal-water and other chemical reactions that may result from degraded emergency core cooling functioning, (2) the limited experience and experimental data available for defining accident phenomena and containment responses, and (3) the conservatism of the calculational model and input parameters."

DISCUSSION

The design of the containment structure is based on the containment design basis accidents, which include the rupture of a reactor coolant pipe in the RCS or the rupture of a main steam line. In either case, the pipe rupture is assumed to be coupled with partial loss of the redundant safety feature systems (minimum safety features). The maximum pressure and temperature reached for a containment design basis accident are presented in chapter 6. The containment design, as discussed in subsection 3.8.1, provides ample margin to the design basis limits.

See chapters 3 and 6 for details.

CRITERION 51 - FRACTURE PREVENTION OF CONTAINMENT PRESSURE BOUNDARY

"The reactor containment boundary shall be designed with sufficient margin to assure that under operating, maintenance, testing, and postulated accident conditions (1) its ferritic materials behave in a nonbrittle manner and (2) the probability of rapidly propagating fracture is minimized. The design shall reflect consideration of service temperatures and other conditions of the containment boundary material during operation, maintenance, testing, and postulated accident conditions, and the uncertainties in determining (1) material properties, (2) residual, steady-state, and transient stresses, and (3) size of flaws."

DISCUSSION

Principal load-carrying components of ferritic materials exposed to the external environment are selected (as discussed in subsection 3.8.1) so that their temperatures under normal operating and testing conditions are not less than 30°F above nil ductility transition temperature.

Refer to subsection 3.8.1 for details.

CRITERION 52 - CAPABILITY FOR CONTAINMENT LEAKAGE RATE TESTING

"The reactor containment and other equipment which may be subjected to containment test conditions shall be designed so that periodic integrated leakage rate testing can be conducted at containment design pressure."

DISCUSSION

The containment system is designed and constructed and the necessary equipment is provided to permit periodic integrated leakage rate tests during plant lifetime, in accordance with the requirements of Appendix J of 10 CFR 50. Details concerning the conduct of periodic integrated leakage rate tests are included in chapter 6.

CRITERION 53 - PROVISIONS FOR CONTAINMENT TESTING AND INSPECTION

"The reactor containment shall be designed to permit (1) appropriate periodic inspection of all important areas, such as penetrations, (2) an appropriate surveillance program, and (3) periodic testing at containment design pressure of the leak-tightness of penetrations which have resilient seals and expansion bellows."

DISCUSSION

Provisions exist for conducting individual leakage rate tests on containment penetrations. Penetrations are visually inspected and pressure tested for leaktightness at periodic intervals. Other inspections are performed as required by Appendix J of 10 CFR 50. (Refer to chapter 6.)

CRITERION 54 - PIPING SYSTEMS PENETRATING CONTAINMENT

"Piping systems penetrating the primary reactor containment shall be provided with leak detection, isolation and containment capabilities having redundancy, reliability, and performance capabilities which reflect the importance to safety of isolating these piping systems. Such piping systems shall be designed with a capability to test periodically the operability of the isolation valves and associated apparatus and to determine if valve leakage is within acceptable limits."

DISCUSSION

Piping systems penetrating the primary reactor containment are provided with containment isolation valves. Penetrations which must be closed for containment isolation have redundant valving and associated apparatus. Automatic isolation valves with air or motor operators, which do not restrict normal plant operation, are periodically tested to assure operability. Secondary system piping inside the containment is considered an extension of the containment boundary, as described in subsection 6.2.4. The isolation valve arrangements are discussed in chapter 6.

Piping that penetrates the containment has been equipped with test connections and test vents or has other provisions to allow periodic leak rate testing to ensure that leakage is within the acceptable limit as defined by the Technical Specifications and Appendix J to 10 CFR 50, as described in chapter 6.

The fuel transfer tube is not classified as a fluid system penetration. The blind flange and the portion of the transfer tube inside the containment are an extension of the containment boundary. The blind flange isolates the transfer tube at all times, except when the reactor is shutdown for refueling. This assembly is a penetration in the same sense as are equipment hatches and personnel locks.

CRITERION 55 - REACTOR COOLANT PRESSURE BOUNDARY PENETRATING CONTAINMENT

"Each line that is part of the reactor coolant pressure boundary and that penetrates the primary reactor containment shall be provided with containment isolation valves as follows, unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

1. One locked closed isolation valve inside and one locked closed isolation valve outside the containment; or
2. One automatic isolation valve inside and one locked closed isolation valve outside the containment; or
3. One locked closed isolation valve inside and one automatic isolation valve outside the containment. A simple check valve may not be used as the automatic isolation valve outside containment; or
4. One automatic isolation valve inside and one automatic isolation valve outside the containment. A simple check valve may not be used as the automatic isolation valve outside the containment.

"Isolation valves outside the containment shall be located as close to the containment as practical and, upon loss of actuating power, automatic isolation valves shall be designed to take the position that provides greater safety.

"Other appropriate requirements to minimize the probability or consequences of an accidental rupture of these lines or of lines connected to them shall be provided, as necessary, to assure adequate safety. Determination of the appropriateness of these requirements, such as higher quality in design, fabrication and testing additional provisions for inservice inspection, protection against more severe natural phenomena, and additional isolation valves and containment, shall include consideration of the population density and use characteristics and physical characteristics of the site environs."

DISCUSSION

Each line that is a part of the reactor coolant pressure boundary and penetrates the containment is provided with isolation valves meeting the intent of this criterion, except that the reactor shutdown cooling lines (RHR system) which are part of the RCPB and which penetrate the containment are provided with two isolation valves in series, both inside the containment. This system is a closed system outside the containment and is constructed to ASME Code, Section III, Class 2, specifications and is considered the second passive barrier to fission product release, as described in chapter 6. The arrangement and type of valves utilized are discussed in chapter 6. Containment penetrations are Seismic Category 1 and are protected against possible environmental effects, including missiles.

CRITERION 56 - PRIMARY CONTAINMENT ISOLATION

"Each line that connects directly to the containment atmosphere and penetrates the primary reactor containment shall be provided with containment isolation valves as follows, unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

1. One locked closed isolation valve inside and one locked closed isolation valve outside the containment; or
2. One automatic isolation valve inside and one locked closed isolation valve outside the containment; or
3. One locked closed isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment; or

4. One automatic isolation valve inside and one automatic isolation valve outside the containment. A simple check valve may not be used as the automatic isolation valve outside the containment.

"Isolation valves outside the containment shall be located as close to the containment as practical and, upon loss of actuating power automatic isolation valves shall be designed to take the position that provides greater safety."

DISCUSSION

Lines which communicate directly with the containment atmosphere and which penetrate the reactor containment are normally provided with two isolation valves in series, one inside and one outside the containment, in accordance with one of the above acceptable arrangements. Several penetrations use alternative arrangements which satisfy containment isolation on some other defined bases. Special cases are described in chapter 6.

Valving arrangements are combinations of locked-shut isolation valves and automatic isolation valves or remote-manual isolation valves. No simple check valves are utilized as automatic isolation valves outside the containment. Where necessary, provision for leak detection is provided for lines outside the containment.

Instrument lines satisfy other acceptable criteria, as described in chapter 6.

CRITERION 57 - CLOSED SYSTEM ISOLATION VALVES

"Each line that penetrates the primary reactor containment and is neither part of the reactor coolant pressure boundary nor connected directly to the containment atmosphere shall have at least one containment isolation valve which shall be either automatic, locked closed, or capable of remote manual operation. This valve shall be outside the containment and located as close to the containment as practical. A simple check valve may not be used as the automatic isolation valve."

DISCUSSION

Lines which penetrate the containment and are neither part of the RCPB nor connected directly to the containment atmosphere are considered closed systems within the containment and are equipped with at least one containment isolation valve of one of the following types:

A. An automatic isolation valve (a simple check valve is not used as this automatic valve).

B. A locked-closed valve.

C. A valve capable of remote manual operation.

This valve is located outside the containment and as close to the containment wall as practical. Valve locations are discussed in detail in subsection 6.2.4.

3.1.6 FUEL AND REACTIVITY CONTROL

CRITERION 60 - CONTROL OF RELEASES OF RADIOACTIVE MATERIALS TO THE ENVIRONMENT

"The nuclear power unit design shall include means to control suitably the release of radioactive materials in gaseous and liquid effluents and to handle radioactive solid wastes produced during normal reactor operation, including anticipated operational occurrences. Sufficient holdup capacity shall be provided for the retention of gaseous and liquid effluents containing radioactive materials, particularly where unfavorable site environmental conditions can be expected to impose unusual operational limitations upon the release of such effluents to the environment."

DISCUSSION

Means are provided to control the release of radioactive materials in gaseous and liquid effluents and to handle radioactive solid wastes produced during normal reactor operation, including anticipated operational occurrences. The radioactive waste management systems are designed to minimize the potential for an inadvertent release of radioactivity from the facility and to assure that the discharge of radioactive wastes is maintained as low as practicable below regulatory limits of 10 CFR 20 during normal operation. The radioactive waste management systems, the design criteria, and the amounts of estimated releases of radioactive effluents to the environment are described in chapter 11.

CRITERION 61 - FUEL STORAGE AND HANDLING AND RADIOACTIVITY CONTROL

"The fuel storage and handling, radioactive waste, and other systems which may contain radioactivity shall be designed to assure adequate safety under normal and postulated accident conditions. These systems shall be designed (1) with a capability to permit appropriate periodic inspection and testing of components important to safety, (2) with suitable shielding for radiation protection, (3) with appropriate containment, confinement, and filtering systems, (4) with a residual heat removal capability having reliability and testability that reflects the importance to safety of decay heat and other residual heat removal, and (5) to prevent significant reduction in fuel storage coolant inventory under accident conditions."

DISCUSSION

The spent fuel pool and associated cooling system, fuel handling system, independent spent fuel storage installation (ISFSI), and radioactive waste processing system are designed to assure adequate safety under normal and postulated accident conditions.

The spent fuel pool cooling system provides cooling to remove residual heat from the fuel stored in the spent fuel pool. The system is designed with redundancy and testability to assure continued heat removal. The spent fuel pool cooling system is described in subsection 9.1.3.

The spent fuel pool is designed so that no postulated accident could cause excessive loss-of-coolant inventory. Accidents are discussed in chapter 15.

Spent fuel in the ISFSI is stored in casks designed to remove residual heat from the fuel using passive cooling. Each cask is designed to withstand credible and postulated accidents without breaching the confinement barrier resulting in loss of coolant inventory and radiological release to the public.

Structures, components, and systems are designed and located so that appropriate periodic inspection and testing may be performed.

Adequate shielding is provided as described in chapter 12. Radiation monitoring is provided as discussed in chapters 11 and 12.

Individual components that contain significant radioactivity are in confined areas adequately ventilated through appropriate filtering systems.

CRITERION 62 - PREVENTION OF CRITICALITY IN FUEL STORAGE AND HANDLING

"Criticality in the fuel storage and handling system shall be prevented by physical systems or processes, preferably by use of geometrically safe configurations."

DISCUSSION

The restraints and interlocks provided for the safe handling and storage of new and spent fuel are discussed and illustrated in chapter 9.

The reactivity of the spent fuel rack is analyzed such that K_{eff} remains less than 1.0 under No Soluble Boron 95/95 K_{eff} conditions as described in paragraph 4.3.2.6.1. To provide safety margin in the criticality analysis of the spent fuel racks, credit is taken for the soluble boron in the spent fuel pool water as described in FSAR paragraph 4.3.2.6.1. New fuel in the new fuel storage racks is stored with enough center-to-center distance to ensure a k_{eff} <0.98, under conditions of optimum moderation.

The design of the spent fuel storage rack assembly is such that it is configurationally impossible to insert the spent fuel assemblies in other than prescribed locations, without physically modifying the rack, thereby preventing any possibility of accidental criticality.

Layout of the fuel handling area is such that the spent fuel cask cannot traverse the spent fuel storage pool.

CRITERION 63 - MONITORING FUEL AND WASTE STORAGE

"Appropriate systems shall be provided in the fuel storage and radioactive waste systems and associated handling areas (1) to detect conditions that may result in the loss of residual heat removal capability and excessive radiation levels and (2) to initiate appropriate safety actions."

DISCUSSION

Instrumentation is provided to detect and alarm, in the control room, excessive temperature or low water level in the spent fuel storage pool. Area radiation monitors are provided in the fuel storage area for personnel protection and general surveillance. These area monitors alarm locally and in the control room. Normally, the fuel building ventilation system removes radioactivity from the atmosphere above the spent fuel storage pool and discharges it by way of the plant vent. The ventilation system is continuously monitored by gaseous, particulate, and radioiodine radiation monitors.

If radiation levels reach a predetermined point, an alarm is sounded in the control room and the ventilation discharge path is automatically transferred through filter adsorber units which provide adequate filtration before discharge from the plant vent. (See chapters 7, 9, and 12 for details.)

CRITERION 64 - MONITORING RADIOACTIVITY RELEASES

"Means shall be provided for monitoring the reactor containment atmosphere, spaces containing components for recirculation of LOCA fluids, effluent discharge paths, and the plant environs for radioactivity that may be released from normal operations, including anticipated operational occurrences, and from postulated accidents."

DISCUSSION

The containment atmosphere is continually monitored during normal and transient station operations, using the containment particulate, gaseous, and radioiodine radiation monitors. Under accident conditions, samples of the containment atmosphere provide data on existing airborne radioactive concentrations within the containment. Portable radiation detection instruments are provided to periodically monitor radiation levels in the auxiliary building spaces which contain components for recirculation of LOCA fluids and components for processing radioactive wastes. Radioactivity levels contained in the facility effluent and discharge paths and in the plant environs are continually monitored during normal and accident conditions by the plant radiation monitoring systems. In addition to the installed detectors, periodic plant environmental surveillance is established. Measurement capability and reporting of effluents are based on the guidelines of Regulatory Guides 1.4 and 1.21. Radiation monitoring systems are discussed in section 11.5 and subsection 12.3.4.

3.2 CLASSIFICATION OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

This section provides a guide to the classification method of structures, components, and systems.

3.2.1 SEISMIC CLASSIFICATION

General Design Criterion 2 of Appendix A to 10 CFR 50, General Design Criteria for Nuclear Power Plants, requires that nuclear power plant structures, systems, and components important to safety be designed to withstand the effects of natural phenomena such as earthquakes without loss of capability to perform necessary safety functions. Appendix A to 10 CFR 100, Seismic and Geologic Siting Criteria for Nuclear Power Plants, sets forth the principal seismic and geologic considerations which are used in the evaluation of the suitability of plant design bases established in consideration of the site seismic and geologic characteristics.

3.2.1.1 Definitions

Seismic Category 1 structures, components, and systems are classified in accordance with Regulatory Guide 1.29. Safety-related, Seismic Category 1 structures, components, and systems are those necessary to ensure the following:

A. The integrity of the reactor coolant pressure boundary.

B. The capability to shut down the reactor and maintain it in a safe-shutdown condition.

C. The capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10 CFR 100.

Seismic Category 1 structures, components, and systems are designed to withstand the appropriate seismic loads, as discussed in section 3.7, and other applicable loads without loss of function. Seismic Category 1 structures are sufficiently isolated from non-Category 1 structures, or they are analyzed to ensure that their structural integrity is maintained during the postulated safe shutdown earthquake (SSE). Non-Seismic Category 1 systems, equipment, and components installed in Seismic Category 1 structures whose failure could result in loss of required safety function of Seismic Category 1 structures, equipment, systems, or components are either separated by distance or barrier from the affected structure, system, equipment, or component or designed together with their anchorages to maintain their structural integrity during the SSE.

Structures, equipment, and systems not classified as Seismic Category 1 are classified as Seismic Category 2.

Safety-related structures, systems, and components that are classified Seismic Category 1 are in compliance with the quality assurance requirements of 10 CFR 50, Appendix B.

The criteria used for the design of Seismic Category 1 structures, equipment, systems, and components; Seismic Category 2 items; and Seismic Category 2 items whose failure could result in the loss of required safety function of Seismic Category 1 items are discussed in section 3.7. Also discussed are the additional seismic criteria for which the radwaste buildings are designed.

3.2.1.2 Classifications

Table 3.2.2-1 provides a listing of structures, components, and systems and identifies those that are Seismic Category 1.

Where only portions of systems are identified as Seismic Category 1 in table 3.2.2-1, the boundaries of the Seismic Category 1 portions of the system are shown on the piping and instrumentation diagrams in appropriate sections of the FSAR. Conformance of the above seismic classifications with Regulatory Guide 1.29 is discussed in section 1.9.

3.2.2 VEGP CLASSIFICATION SYSTEM

Equipment, components, and structures in the VEGP are categorized according to nuclear safety, seismic category, and codes and standards by the VEGP project classification system. This system conforms to 10 CFR 50 and Regulatory Guides 1.26 and 1.29. A project classification is assigned to each item in the plant. The three element project classification indicates, in sequence, the nuclear safety class, the seismic category, and the applicable codes and standards. The project classification system provides an easily recognizable means of identifying the extent to which components, equipment, and structures are related to nuclear safety and seismic qualification requirements. In addition, the project classification system provides the means whereby the codes and/or standards that govern the design of a component or structure can be located. Table 3.2.2-1 provides a listing of the principal VEGP structures, systems, components, and the associated project classifications. Table 3.2.2-3 summarizes the construction codes and standards for VEGP components.

3.2.2.1 Nuclear Safety Classifications

The first element of a project classification identifies the nuclear safety class. The nuclear safety class designators used on VEGP are as follows:

Designator	Definition
0	Nuclear Safety Class 0 is assigned to safety-related mechanical components not within the purview of Regulatory Guide 1.26. Nuclear Safety Class 0 is also assigned to safety-related structures and structural components.
1	Nuclear Safety Class 1 parallels Group A as defined in 10 CFR 50.55a and 10 CFR 50.2(v) and is assigned to components of the reactor coolant pressure boundary. Nuclear Safety Class 1 is also assigned to safety-related instruments, controls, and electrical components.
2	Nuclear Safety Class 2 parallels Group B as defined in Regulatory Guide 1.26 and is assigned to emergency and auxiliary systems serving the reactor coolant system.
3	Nuclear Safety Class 3 parallels Group C as defined in Regulatory Guide 1.26 and is assigned to other safety-related auxiliary systems.
4	Nuclear Safety Class 4 parallels Group D as defined in

Designator	Definition
	Regulatory Guide 1.26 and is assigned to nonsafety-related systems.
6	Nuclear Safety Class 6 is assigned to nonsafety-related components not within the purview of Regulatory Guide 1.26. This designation is also assigned to nonsafety-related structures and structural components, instrumentation, controls, and electrical components.

3.2.2.2 Seismic Classification

The second element of a project classification is either 1 or 2, which designates the appropriate seismic category. Seismic classification is discussed in subsection 3.2.1.

3.2.2.3 Codes and Standards

The third element of a project classification indicates the primary codes and/or standards applicable to plant equipment, components, and structures. The codes and standards designators used on VEGP are as follows:

Designator	Codes and Standards
1	American Society of Mechanical Engineers Boiler and Pressure Vessel (ASME B&PV) Code, Section III, Class 1.
2	ASME B&PV Code, Section III, Class 2.
3	ASME B&PV Code, Section III, Class 3.
4	Regulatory Guide 1.26 - Table 1 – Quality Group D.
5	Special designator associated with Nuclear Safety Classes 0 or 4. (Refer to paragraph 3.2.2.3.1.)
6	Special designator associated with Nuclear Safety Class 6. (Refer to paragraph 3.2.2.3.2.)
7	Regulatory Guide 1.143 - Table 1. (Refer to paragraph 3.2.2.3.3.)
8	ASME B&PV Code, Section I.
9	Applicable National Fire Protection Association (NFPA) codes per the guidelines of Branch Technical Position CMEB 9.5-1, section C4. (Refer to paragraph 3.2.2.3.4.)
C	Structures or structural components designed to codes and standards as defined in the design bases.[d]
E	Electrical equipment designed to codes and standards as defined in the design bases.[a]
J	Instrumentation and control equipment designed to codes

Designator	Codes and Standards
	and standards as defined in the design bases.[d]

[a] For fire protection-related features, BTP CMEB 9.5-1, section C4 applies as described in section 9.5.1.1.4.

A more complete listing of applicable codes and standards is provided in table 3.2.2-2.

3.2.2.3.1 Codes and Standards Designator 5

Codes and standards designator 5 is associated only with Nuclear Safety Class 0 or 4.

When associated with Nuclear Safety Class 0, codes and standards designator 5 requires standards for construction which are of sufficient quality to ensure acceptable performance for the intended safety-related function of the component, as defined in the design bases for the particular system.

When associated with Nuclear Safety Class 4, codes and standards designator 5 specifies that materials, components, parts, appurtenances, and piping subassemblies shall be procured in accordance with the ASME B&PV Code, Section III, Class 3; however, the system shall be designed and installed in accordance with American National Standards Institute (ANSI) B31.1. Conformance with these aspects of the ASME code is required only for initial procurement. Subsequent conformance for maintenance and replacement will be subject to the owner's discretion.

3.2.2.3.2 Codes and Standards Designator 6

Codes and standards designator 6 is associated only with Nuclear Safety Class 6. Codes and standards designator 6 requires standards for construction which are of sufficient quality to ensure acceptable performance for the intended nonsafety-related function of the component, as defined in the design bases for the particular system.

3.2.2.3.3 Codes and Standards Designator 7

The codes and standards used for the construction of radioactive waste management and steam generator blowdown systems are provided in Regulatory Guide 1.143. Quality assurance requirements are to be applied to radioactive waste management systems as described in Regulatory Guide 1.143.

3.2.2.3.4 Codes and Standards Designator 9

The design, fabrication, construction, and testing of fire protection systems are performed in accordance with the applicable portions of the NFPA codes, which invoke ANSI B31.1, American Water Works Association (AWWA), American Petroleum Institute (API), and other codes, depending upon service. Quality assurance program requirements are implemented to ensure that the requirements for design, procurement, installation, testing, and administrative controls for the fire protection program are satisfied. The quality assurance requirements that apply to the fire protection program are described in section 9.5.1.1.4. These quality assurance requirements also apply to fire protection-related features under codes and standards designators C, E, J, 4, and 6.

3.2.2.4 Clarification to VEGP Classification System

Instrumentation and control project classes 11J, 61J, and 62J are normally assigned to instrumentation. Piping, tubing, fittings, control valves, instrument valves, and other mechanical components associated with instrumentation are assigned mechanical project classifications consistent with the project class assigned to process equipment (tanks, piping, vessels, etc.) to which the instrumentation is connected.

Equipment which is classified 61J and 61E is seismically designed not to fail in a manner that would compromise the functioning of safety-related equipment during or after a safe shutdown earthquake. However, the 61J and 61E equipment will not necessarily remain functional.

TABLE 3.2.2-1 CONTENTS

CLASSIFICATION OF STRUCTURES, COMPONENTS, AND SYSTEMS

TABLE 3.2.2-1 CONTENTS

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TABLE 3.2.2-1 (SHEET 1 OF 98)

CLASSIFICATION OF STRUCTURES, COMPONENTS, AND SYSTEMS

Principal System and Components		(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
REACTOR COOLANT SYSTEM													
1.	Reactor vessel and head	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
2.	Vessel internals	C	C	W	B	2	1	2	III-CS	Y	Y	1-A	Note af
3.	Fuel assemblies and appurtenances	C	C	W	NA	1	1	1	mfg	Y	Y	1-A	
4.	CRDM housing	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
5.	CRDM head adapter plug	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
6.	Steam generator	C	C	W	A	1,2	1	1,2	III-1,2	Y	Y	1-A	
7.	Pressurizer	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
8.	Pressurizer surge line	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
9.	Pressurizer relief lines (upstream of relief valves)	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
10.	Pressurizer relief lines (downstream) of relief valve	C	C	B	D	4	1	5	III-3	N	N		Note r
11.	Pressurizer safety and relief valves	C	C	W	A	1	1	1	III-1	Y	y	1-A	
12.	Pressurizer relief tank	C-171′	C-171′	W	D	4	2	5	III-3	N	N		Note r
13.	Normal pressurizer heaters	C	C	W	NA	6	2	E	mfg	N	N		
14.	Backup pressurizer heaters	C	C	W	NA	6	1	E	mfg	N	N		Note m
15.	Reactor coolant pump casing	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
16.	RCP seal standpipe	C	C	W	D	4	2	4	VIII	N	N		
17.	(This line has been intentionally left blank.)												
18.	Seal 1 housing	C	C	W	A	1	1	1	III-1	Y	Y	1-A	

TABLE 3.2.2-1 (SHEET 2 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
19. Thermal barrier	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
20. Seal 2 housing	C	C	W	B	2	1	2	III-2	Y	Y	1-A	
21. Pressure retaining bolts	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
22. Main flange	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
23. RCP motor	C	C	W	NA	6	2	E	NEMA MG1	N	N		
24. Shaft coupling	C	C	W	NA	2	1	NA	NA	Y	Y	1-A	
25. Spool piece	C	C	W	B	2	1	2	Mfg	Y	Y	1-A	
26. Armature	C	C	W	B	2	1	2	NEMA MG1	Y	Y	1-A	
27. Flywheel	C	C	W	NA	2	1	NA	mfg	Y	Y	1-A	
28. Motor bolting	C	C	W	NA	2	1	NA	NEMA MG1	Y	Y	1-A	
29. Upper oil cooler (tube side - ACCW)	C	C	W	C	3	1	3	III-3	Y	Y		
30. Upper oil cooler (shell side - oil)	C	C	W	C	3	1	3	III-3	Y	Y	1-A	
31. Lower oil cooling coil	C	C	W	C	3	1	3	III-3	Y	Y		
32.	(This line has been intentionally left blank.)											
33. RTD thermowells	C	C	W	A	1	1	1	III-1	Y	Y	1-A	
34. RCS loop piping	C	C	W	A	1	1	1	III-1	Y	Y		
35. Valves >O_2 in.	C	C	W	A	1	1	1	III-1	Y	Y		
36. Other piping and valves ≤2 in.	C	C	B	A	1	1	1	III-1	Y	Y		
37. Safety-related valve operators	C	C	W	NA	1	1	E	NEMA MG1	Y	Y		
38. Reactor vessel supports	C	C	W	NA	0	1	C	III-NF	Y	Y		
39. Steam generator supports	C	C	W	NA	0	1	C	III-NF	Y	Y		
40. Pressurizer supports	C	C	W	NA	0	1	C	III-NF	Y	Y		

TABLE 3.2.2-1 (SHEET 3 OF 98)

	Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	((g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
41.	Reactor coolant pump supports	C	C	W	NA	0	1	C	III-NF	Y	Y		
42.	Other safety-related piping supports and hangers	C	C	W,B	NA	See Note 4		III-NF	III-NF	Y	Y		
43.	Lube oil drain tanks	C-171′	C-171′	B	D	4	1	4	API-650	N	N		Note ac
44.	Bottom-mounted instrument tubing	C	C	W	A	1	1	1	III-1	Y	Y		
45.	Control rods	C	C	W	NA	0	1	5	mfg	Y	Y		
46.	Reactor coolant pump articulated arm	C	C	W	NA	6	2	6	mfg	N	N		
REACTOR HEAD VENT SYSTEM													
1.	Piping and valves through second isolation valve			W	A	1	1	1	III-1	Y	Y		
2.	All other piping and valves			W,B	B	2	1	2	III-2	Y	Y		
3.	Instrumentation			W,B	NA	1	1	J	mfg	Y	Y		Note s
SAFETY INJECTION SYSTEM													
1.	Accumulators	C-171′	C-171′	W	B	2	1	2	III-2	Y	Y	VIII	
2.	Boron injection tank	AB-B11	Deleted	W	B	2	1	2	III-2	Y	Y	VIII	Note ak
3.						(This line has been intentionally left blank.)							
4.						(This line has been intentionally left blank.)							
5.						(This line has been intentionally left blank.)							
6.						(This line has been intentionally left blank.)							
7.	Safety injection pumps	AB-B15 AB-B19	AB-B117 AB-B119	W	B	2	1	2	III-2	Y	Y	VIII	
8.	Safety injection pump motors	AB-B15 AB-B19	AB-B117 AB-B119	W	NA	1	1	E	NEMA MG1	Y	Y	VIII	

TABLE 3.2.2-1 (SHEET 4 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
9. Cold leg injection piping and valves downstream of 8 check valves, 143-150		W,B	A	1	1	1	III-1	Y	Y		
10. Hot leg injection piping and valves downstream of 6 check valves, 120-123, 128, 129		W,B	A	1	1	1	III-1	Y	Y		
11. Accumulator discharge piping and valves downstream of MOVs 8808A-D		W,B	A	1	1	1	III-1	Y	Y		
12. Piping and valves from RWST and containment sumps to 8 check valves, 139-142 and 120-123		W,B	B	2	1	2	III-2	Y	Y		
13. Piping from accumulators to MOVs 8808A-D		B	B	2	1	2	III-2	Y	Y		
14. Boron injection piping and valves downstream of check valve 013		W,B	A	1	1	1	III-1	Y	Y		
15. Boron injection piping and valves from charging pumps to check valve 013		W,B	B	2	1	2	III-2	Y	Y		
16. Safety-related instrumentation		W	NA	1	1	J	mfg	Y	Y		Note s
17. Safety-related valve operators		W	NA	1	1	E	NEMA MG1	Y	Y		

TABLE 3.2.2-1 (SHEET 5 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
18. Sludge mixing pump	O	O	B	D	4	2	4	mfg	N	N		
19. Electric circulation heater	O	O	B	D	4	2	4	mfg	N	N		
20. Mixing eductors	O	O	B	D	4	2	4	mfg	N	N		
21. Safety injection pump lube oil coolers	AB-B15 AB-B19	AB-B117 AB-B119	W	NA	0	1	5	III-3	Y	Y	VIII	
22. Miniflow orifices			W	B	2	1	2	III-2	Y	Y		
RESIDUAL HEAT REMOVAL SYSTEM												
1. RHR pumps	AB-D48 AB-D49	AB-D21 AB-D22	W	B	2	1	2	III-2	Y	Y	VIII	
2. RHR pump Motors	AB-D48 AB-D49	AB-D21 AB-D22	W	NA	1	1	E	NEMA MG1	Y	Y	VIII	
3. RHR HXs:												
Tube side, RHR	AB-C90 & C91	AB-C25 & C26	W	B	2	1	2	III-2, TEMA-R	Y	Y	VIII	Note t
Shell side, CCW	AB-C90 AB-C91	AB-C25 AB-C26	W	C	3	1	3	III-3, TEMA-R	Y	Y	VIII	
4. Piping and valves from hot legs to pump suction MOVs 8701A and 8702A			W,B	A	1	1	1	III-1	Y	Y		
5. Piping and valves from pump suction MOVs 8701A and 8702A to cold leg injection check valves 147-150 and to hot leg injection check valves 128 and 129			W,B	B	2	1	2	III-2	Y	Y		
6. Safety-related instrumentation			W	NA	1	1	J	mfg	Y	Y		Note s

TABLE 3.2.2-1 (SHEET 6 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
7. Safety-related valve operators			W	NA	1	1	E	NEMA MG1	Y	Y		
8. RHR pump seal coolers	AB-D48 AB-D49	AB-D21 AB-D22	B	B	2	1	2	III-2	Y	Y	VIII	
9. Emergency sump screen	C	C	SNC	NA	0	1	C	MFG	Y	Y		
10. Encapsulation vessels	FB-C07 AB-C105	FB-C01 AB-C04	B	NA	0	1	C	III-MC	Y	Y	VII	
CONTAINMENT SPRAY SYSTEM												
1. Containment spray pumps	AB-D76 AB-D77	AB-D04 AB-D05	W	B	2	1	2	III-2	Y	Y	VIII	
2. Containment spray pump motors	AB-D76 AB-D77	AB-D04 AB-D05	W	NA	1	1	E	NEMA MG1	Y	Y	VIII	
3. Spray nozzles	C	C	W	B	2	1	2	III-2	Y	Y	VIII	
4. Spray additive tank	AB-D74	AB-D04	W	C	3	1	3	III-3	Y	Y	VIII	Note aq
5. Spray eductors	AB-D76 AB-D77	AB-D04 AB-D05	W	B	2	1	2	III-2	Y	Y	VIII	Note aq
6. Process valves and piping from RWST, containment sumps, and educator inlet check valves to spray nozzles			W,B	B	2	1	2	III-2	Y	Y		Note aq
7. Process valves and piping downstream of spray additive tank to eductor inlet check valves			W,B	C	3	1	3	III-3	Y	Y		
8. Safety-related instrumentation			W	NA	1	1	J	mfg	Y	Y		Note s
9. Safety-related valve operators			W	NA	1	1	E	NEMA MG1	Y	Y		
10. Encapsulation vessels	FB-C08 AB-C105	FB-C02 AB-C04	B	NA	0	1	C	III-MC	Y	Y	VIII	
11. Trisodium phosphate baskets	C	C	S	NA	6	1	C	AISC	N	N		

TABLE 3.2.2-1 (SHEET 7 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
CHEMICAL AND VOLUME CONTROL SYSTEM												
1. Volume control tank	AB-A48	AB-A80	W	B	2	1	2	III-2	Y	Y	VIII	
2. Boric acid storage tank	AB-D69	AB-D09	B	C	3	1	3	III-3	Y	Y	VIII	
3. Boric acid batching tank	AB-C113	shared	W	D	4	2	4	API-650	N	N		
4. Boric acid batching tank agitator	AB-C113	shared	W	D	4	2	4	mfg	N	N		
5. Boric acid transfer pumps	AB-D72 & D119	AB-D106 & D123	W	C	3	1	3	III-3	Y	Y	VIII	
6. Boric acid transfer pump motors	AB-D72 & D119	AB-D106 AB-D123	W	NA	1	1	E	NEMA MG1	Y	Y	VIII	Note m
7. Centrifugal charging pumps	AB-C115 AB-C118	AB-C16 AB-C17	W	B	2	1	2	III-2	Y	Y	VIII	
8. Centrifugal charging pump motors	AB-C115 & C118	AB-C16 & C117	W	NA	1	1	E	NEMA MG1	Y	Y	VIII	
9. Deleted												
10. Deleted												
11. Regenerative HX	C-171′	C-171′	W	B	2	1	2	III-2 TEMA-R	Y	Y	1-A	
12. Letdown HX:			W									
Tube side, CVCS	AB-A07	AB-A100		B	2	1	2	III-2, TEMA-R	Y	Y	VIII	
Shell side, ACCW	AB-A07	AB-A100		D	4	1	5	III-3, TEMA-R	N	N		Note t
13. Excess letdown HX:			W									
Tube side, CVCS	C-171′	C-171′		B	2	1	2	III-2, TEMA-R	Y	Y	1-A	
Shell side, ACCW	C-171′	C-171′		D	4	1	5	III-2, TEMA-R	N	N		Note t

TABLE 3.2.2-1 (SHEET 8 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
14. Seal water HX:			W									
Tube side, CVCS	AB-B20	AB-B116		B	2	1	2	III-2, TEMA-R	Y	Y	VIII	
Shell side, ACCW	AB-B20	AB-B116		D	4	1	5	III-3, TEMA-R	N	N		Note t
15. Letdown reheat HX:			W									
Tube side, CVCS	AB-A07	AB-A100		B	2	1	2	III-2, TEMA-R	Y	Y	VIII	
Shell side, CVCS	AB-A07	AB-A100		C	3	1	3	III-3, TEMA-R	Y	Y		
16. Moderating HX	AB-106	AB-154	W	C	3	1	3	III-3, TEMA-R	Y	Y	VIII	
17. Letdown chiller HX:			W									
Tube side	AB-135	AB-160		C	3	1	3	III-3, TEMA-R	Y	Y	VIII	
Shell side	AB-135	AB-160		D	4	2	4	VIII, TEMA-C	N	N		
18. Reactor coolant filter housing	AB-B	AB-B	B	B	2	1	2	III-2	Y	Y	VIII	
19. Seal water return backflushable filter housing	AB-B	AB-B	B	B	2	1	2	III-2	Y	Y	VIII	
20. Boric acid filter	AB-D90	AB-D81	W	C	3	1	3	III-3	Y	Y	VIII	
21. Seal injection filter housings	AB-B	AB-B	B	B	2	1	2	III-2	Y	Y	VIII	
22. Letdown orifices			W	C	2	1	2	III-3	Y	Y		
23. Boric acid transfer pump orifices			B	C	3	1	3	III-3	Y	Y		
24. RCP seal bypass orifice			W	B	2	1	2	III-2	Y	Y		
25. Centrifugal charging pump miniflow orifice			W	B	2	1	2	III-2	Y	Y		

TABLE 3.2.2-1 (SHEET 9 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
26. CVCS chillers	AB-124		G	D	4	2	4	VIII	N	N		
27. CVCS chiller surge tank	AB-124		W	D	4	2	4	API-650	N	N		
28. CVCS chiller pump	AB-124		W	D	4	2	4	mfg	N	N		
29. CVCS chiller pump motor	AB-124		W	NA	6	2	E	NEMA MG1	N	N		
30. Deleted												
31. Thermal regenerative demineralizers	AB-A	AB-A	W	C	3	1	3	III-3	Y	Y	VIII	
32. CVCS cation bed demineralizer	AB-A	AB-A	W	C	3	1	3	III-3	Y	Y	VIII	
33. Mixed bed demineralizers	AB-A	AB-A	W	C	3	1	3	III-3	Y	Y	VIII	
34. Boron meter	AB-C80	AB-C33	W	D	4	2	4	B31.1	N	N		
35. This line has been intentionally left blank.												
36. This line has been intentionally left blank.												
37. Chemical mixing tank	AB-A39	AB-A69	W	D	4	2	4	VIII	N	N		
38. Safety-related CVCS instrumentation			W,B	NA	1	1	J	mfg	Y	Y		Note s
39. Piping from valve 273 to boric acid storage tank			B	C	3	1	3	III-3	Y	Y		
40. Piping and valves from boric acid storage tank to MOV HV-8104, boric acid mixing tee valve 188 and valve 505			W,B,S	C	3	1	3	III-3	Y	Y		

TABLE 3.2.2-1 (SHEET 10 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
41. Chemical mixing tank inlet and discharge piping from valve 176 to valve 181		B	D	4	2	4	B31.1	N	N		
42. Piping and valves from reactor makeup water supply valve 177 to valves 183, 176, and FV-0110B		B	C	3	1	3	III-3	Y	Y		
43. RCP seal injection piping and valves from check valves 006, 359, 360, and 361 to RCPs		B	A	1	1	1	III-1	Y	Y		
44. Letdown piping from valve LV-0459 to volume control tank		W,B	B	2	1	2	III-2	Y	Y		
45. Charging piping from volume control tank to valves 035 and 037		W,B	B	2	1	2	III-2	Y	Y		
46. Mixed bed demineralizer piping and valves from valve TV-0129 to check valve 083		W,B	C	3	1	3	III-3	Y	Y		
47. Mixed bed demineralizer piping from check valve 083 to reactor coolant filter		W,B	B	2	1	2	III-2	Y	Y		

TABLE 3.2.2-1 (SHEET 11 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
48. Moderating HX, letdown chiller HX (tube side) and letdown reheat HX(shell side) piping and valves			W,B	C	3	1	3	III-3	Y	Y		
49. Thermal regenerative demineralizer piping and valves			W,B	C	3	1	3	III-3	Y	Y		
50. Letdown reheat HX (tube side) piping and valves			W,B	B	2	1	2	III-2	Y	Y		
51. Letdown chiller HX (shell side) piping and valves			W,B	D	4	2	4	B31.1	N	N		
52. Piping and valves from boric acid batching tank to valve 304			W,B	D	4	2	4	B31.1	N	N		
53. Safety-related valve operators			W	NA	1	1	E	NEMA MG1	Y	Y		
54. Seal return piping and valves from RCP to seal water HX			W,B	B	2	1	2	III-2	Y	Y		
55. Piping and valves between seal water HX and volume control tank discharge piping			W,B	B	2	1	2	III-2	Y	Y		
56. Centrifugal charging pump lube oil coolers	AB-C115 & C118	AB-C16 & C17	W	NA	0	1	5	III-3	Y	Y	VIII	
57. Normal charging pump	AB-C111	AB-C12	S	B	2	1	2	III-2	Y	Y	VIII	
58. Normal charging pump motor	AB-C111	AB-C12	S	NA	6	2	E	NEMA MG1	N	N		
59. Normal charging pump minimum flow orifice	AB-C112	AB-C09	S	B	2	1	2	III-2	Y	Y		

TABLE 3.2.2-1 (SHEET 12 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
BORON RECYCLE SYSTEM												
1. Recycle evaporator feed pumps	AB-D34 & D35	shared	W	D	4	1	4	mfg	N	N		au
2. Recycle evaporator feed pump motors	AB-D34 & D35	shared	W	NA	6	2	E	NEMA MG1	N	N		au
3. Boron recycle holdup tanks	AB-D57 & D33	shared	B	C	3	1	3	III-3	Y	Y	VIII	au
4. Recycle evaporator feed backflushable filter housings	AB-B	shared	B	D	4	1	4	VIII	N	N		au
6. Recycle evaporator feed demineralizers	AB-C141	shared	W	D	4	1	4	VIII	N	N		au
7. Recycle evaporator condensate demineralizer	AB-C140	shared	W	D	4	2	4	VIII	N	N		au
10. Recycle evaporator condensate filter	AB-D96	shared	W	D	4	2	5	III-3	N	N		au
11. Piping and valves between recycle holdup tanks and recycle evaporator package			W,B	D	4	1	4	B31.1	N	N		au

TABLE 3.2.2-1 (SHEET 13 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
12. Condensate piping and valves between recycle evaporator package and reactor makeup water storage tank inlet check valve 002		W,B	D	4	2	4	B31.1	N	N		
13. Piping and valves between recycle evaporator feed demineralizers and recycle holdup tanks		W,B	D	4	1	4	B31.1	N	N		au
14. Letdown piping and valves from valve LV-0112A to recycle evaporator feed demineralizers		W,B	D	4	1	4	B31.1	N	N		
15. Piping and valves from recycle evaporator to boric acid storage tank inlet valve 273		B	D	4	2	4	B31.1	N	N		au
16. Recycle holdup tank vent eductor		W	D	4	1	7	B31.1	N	N		Note ag
17. Valve operators		W	NA	6	1	E	NEMA MG1	N	N		
18. Instrumentation		W	NA	6	2	J	mfg	N	N		Note s
19. Electronic metering pump		G	NA	4	2	4	mfg	N	N	VIII	
20. Zinc addition batch tank		G	NA	4	2	4	mfg	N	N	VIII	
CONTAINMENT ISOLATION SYSTEM											
1. Valves and piping		W,B	B	2	1	2	III-2	Y	Y		
2. Valve operators		W,B	NA	1	1	E	NEMA MG1	Y	Y	VIII	

TABLE 3.2.2-1 (SHEET 14 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
3.	Instrumentation and controls			W,B	NA	1	1	J	mfg	Y	Y		Note s
NUCLEAR SERVICE COOLING WATER SYSTEM													
1.	NSCW pumps	NSW	NSW	B	C	3	1	3	III-3	Y	Y	III	
2.	NSCW pump motors	NSW	NSW	B	NA	1	1	E	NEMA MG1	Y	Y	III	
3.	NSCW transfer pumps	NSW	NSW	B	C	3	1	3	III-3	Y	Y	III	
4.	NSCW transfer pump motors	NSW	NSW	B	NA	1	1	E	NEMA MG1	Y	Y	III	Note m
5.	(This line has been intentionally left blank.)												
6.	(This line has been intentionally left blank.)												
7.	(This line has been intentionally left blank.)												
8.	Tower fans	NSW	NSW	B	NA	0	1	5	AMCA	Y	Y	III	
9.	Fan motors	NSW	NSW	B	NA	1	1	E	NEMA MG1	Y	Y	III	
10.	Valves and piping (outside containment)			B	C	3	1	3	III-3	Y	Y		
11.	Valves and piping inside containment)			B	B	2	1	2	III-2	Y	Y		
12.	Safety-related valve operators			B	NA	1	1	E	NEMA MG1	Y	Y		
13.	Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
14.	Containment auxiliary air coolers (tube side)	C-261′	C-261′	B	B	2	1	2	III-2	Y	Y	1-B	
15.	Containment cavity coolers (tube side)	C-206′	C-206′	B	B	2	1	2	III-2	Y	Y	1-A	
16.	Piping penetration area coolers (tube side)	AB-209′	AB-206′	B	C	3	1	3	III-3	Y	Y	VIII	

TABLE 3.2.2-1 (SHEET 15 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
17. Containment spray pump motor coolers	AB-D76 & D77	AB-D04 & D05	W	C	3	1	3	III-3	Y	Y	VIII	
18. Safety injection pump motor coolers	AB-215 & B19	AB-B117 & B119	W	C	3	1	3	III-3	Y	Y	VIII	
19. RHR pump motor coolers	AB-D48 & D49	AB-D21 & D22	W	C	3	1	3	III-3	Y	Y	VIII	
20. CCW pump motor coolers	AB-A03 & A05	AB-A96 & A98	B	C	3	1	3	III-3	Y	Y	VIII,B4	
21. Centrifugal charging pump motor coolers	AB-C115 & C118	AB-C16 & C17	W	C	3	1	3	III-3	Y	Y	VIII	
22. Containment air coolers (tube side)	C-238′	C-238′	B	B	2	1	2	III-2	Y	Y	1-B	
23. NSCW pump motor coolers	NSW	NSW	B	NA	0	1	5	mfg	Y	Y	VI	
24. NSCW tower basin transfer line (buried pipe)			B	C	3	1	3	III-3	Y	Y		
COMPONENT COOLING WATER SYSTEM												
1. CCW surge tanks	AB-213 & 214	AB-201 &202	B	C	3	1	3	III-3	Y	Y	VIII	
2. CCW pumps	AB-A03 & A05	AB-A96 & A98	B	C	3	1	3	III-3	Y	Y	VIII,B4	
3. CCW HXs	AB-213 & 214	AB-201 & 202	B	C	3	1	3	III-3, TEMA-R	Y	Y	VIII	
4. CCW pump motors	AB-A03 & A05	AB-A96 & A98	B	NA	1	1	E	NEMA MG1	Y	Y	VIII,B4	
5. CCW chemical addition tanks	AB-A03 & A05	AB-A96 & A98	B	D	4	2	4	VIII	N	N		
6. Chemical addition tank valves and piping			B	D	4	2	4	B31.1	N	N		
7. Other process valves and piping			B	C	3	1	3	III-3	Y	Y	VIII	

TABLE 3.2.2-1 (SHEET 16 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
8. Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
AUXILIARY COMPONENT COOLING WATER SYSTEM												
1. ACCW surge tank	AB-132	AB-105	B	D	4	1	5	III-3	N	N		Note z for 415 components
2. ACCW pumps	AB-B23 & B24	AB-B112 & B113	B	D	4	1	5	III-3	N	N		
3. ACCW pump motors	AB-B23 & B24	AB-B112 & B113	B	NA	1	1	E	NEMA MG1	Y	Y	VIII,B4	Note m
4. ACCW HXs	AB-132 & 134	AB-103 & 105	B	C	3	1	3	III-3, TEMA-R	Y	Y	VIII	
5. Containment penetration piping and valves			B	B	2	1	2	III-2	Y	Y		
6. Chemical addition feeder tank	AB-B23	AB-B112	B	D	4	2	4	VIII	N	N		
7. Other safety-related valves and piping			B	C	3	1	3	III-3	Y	Y		
8. All other valves and piping			B	D	4	1	5	III-3	N	N		
9. Safety-related valve operators			B	NA	1	1	E	NEMA MG1	Y	Y		
10. Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
11. All other instrumentation			B	NA	6	2	J	mfg	N	N		
SPENT FUEL COOLING AND PURIFICATION SYSTEM												
1. SFP HXs	AB-A53 FB-A07	AB-A91 FB-A04	W	C	3	1	3	III-3, TEMA-R	Y	Y	VIII,VII	
2. SFP pumps	AB-A53 FB-A07	AB-A91 FB-A04	W	C	3	1	3	III-3	Y	Y	VIII	
3. SFP pump motors	AB-A53 FB-A07	AB-A91 FB-A04	W	NA	1	1	E	NEMA MG1	Y	Y	VIII	Note m

TABLE 3.2.2-1 (SHEET 17 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
4.	Refueling water purification pump	AB-A40	AB-A89	B	D	4	2	4	mfg	N	N		
5.	Refueling water purification pump motor	AB-A40	AB-A89	B	NA	6	2	E	NEMA MG1	N	N		
6.	SFP demineralizer	AB-A	AB-A	W	D	4	2	4	III-3	N	N		Note r
7.	SFP strainers			W	D	4	2	4	mfg	N	N	VIII	
8.	SFP filter housing	AB-B	AB-B	B	D	4	2	4	VIII	N	N		Note r
9.	SFP skimmer pump	AB-A53	AB-A91	W	D	4	2	4	mfg	N	N		
10.	SFP skimmer pump motor	AB-A53	AB-A91	W	NA	6	2	E	NEMA MG1	N	N		
11.	SFP skimmer filter	AB-D94	AB-D83	W	D	4	2	4	VIII	N	N		
12.	SFP skimmer strainers	FB	FB	W	D	4	2	4	VIII	N	N		
13.	Purification and skimmer-related valves and piping			B	D	4	2	4	B31.1	N	N		
14.	Cooling-related valves and piping W2 in.			W,B	C	3	1	3	III-3	Y	Y		
15.	All other cooling-related valves and piping K2 in.			B	C	3	1	3	III-3	Y	Y		
16.	Safety-related instrumentation			W,B	NA	1	1	J	mfg	Y	Y		Note s
17.	Safety-related valve operators			W	NA	1	1	E	NEMA MG1	Y	Y		

REACTOR MAKEUP WATER SYSTEM

1.	Makeup pumps	AB-B22	AB-B114	B	D	4	1	5	III-3	N	N		Note r

TABLE 3.2.2-1 (SHEET 18 OF 98)

Principal System and Components		(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
2.	Makeup pump motors	AB-B22	AB-B114	B	NA	6	2	E	NEMA MG1	N	N		Note m
3.	Degasifier vacuum pumps	O	O	B	D	4	2	4	mfg	N	N		
4.	Degasifier feed pump	O	O	B	D	4	2	4	mfg	N	N		
5.	Degasifier feed/transfer pump	O	O	B	D	4	2	4	mfg	N	N		
6.	Degasifier transfer pump	O	O	B	D	4	2	4	mfg	N	N		
7.	Vacuum degasifier	O	O	B	D	4	2	4	mfg	N	N		
8.	Makeup water process piping and valves to SFP			B	D	4	1	5	III-3	N	N		Note r
9.	Degasifier piping and valves			B	D	4	2	4	B31.1	N	N		
10.	Instrumentation			B	NA	6	2	J	mfg	N	N		
11.	Degasifier pump motors	O	O	B	NA	6	2	E	NEMA MG1	N	N		
12.	All other piping			B	D	4	2	4	B31.1	N	N		
WASTE PROCESSING SYSTEM - LIQUID													Note w Note r for all 417 and 427 components designated as III-3.
1.	Waste holdup tank	AB-D63	AB-D13	W	D	4	1	7	III-3	N	N		
2.	Waste evaporator feed pump	AB-D62	AB-D14	W	D	4	1	7	III-3	N	N		
3.	Waste evaporator-feed pump motor	AB-D62	AB-D14	W	NA	6	2	E	NEMA MG1	N	N		

TABLE 3.2.2-1 (SHEET 19 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
4. Waste evaporator feed backflushable filter housing	AB-B	AB-B	B	D	4	1	7	III-3	N	N		Note an
5. Deleted												
6. Deleted												
7. Deleted												
8. Deleted												
9. Waste evaporator reagent tank	AB-C63	AB-C40	W	D	4	2	4	VIII	N	N		av
10. Waste evaporator condensate demineralizer	AB-C146	AB-C149	W	D	4	2	7	III-3	N	N		
11. Waste evaporator condensate filter	AB-D91	AB-D86	W	D	4	2	7	III-3	N	N		
12. Waste evaporator condensate pump	AB-D64	AB-D12	W	D	4	2	7	III-3	N	N		
13. Waste evaporator condensate pump motor	AB-D64	AB-D12	W	NA	6	2	E	NEMA MG1	N	N		
14. Waste evaporator condensate tank	AB-C80	AB-C33	W	D	4	2	7	III-3	N	N		
15. Chemical drain tank	AB-D45	shared	W	D	4	2	7	VIII	N	N		
16. Chemical drain tank pump	AB-D47	shared	W	D	4	2	7	VIII	N	N		
17. Chemical drain tank pump motor	AB-D47	shared	W	NA	6	2	E	NEMA MG1	N	N		
18. Spent resin storage tank	AB-D36	AB-D37	W	D	4	1	7	III-3	N	N		

TABLE 3.2.2-1 (SHEET 20 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
19.	Spent resin sluice pump	AB-D39	AB-D40	W	D	4	1	7	mfg	N	N		
20.	Spent resin sluice pump motor	AB-D39	AB-D40	W	NA	6	2	E	NEMA MG1	N	N		
21.	Sample vessels	AB-C63	AB-C40	B	D	4	2	7	VIII	N	N		
22.	Solidification strainer			W	D	4	2	7	B 31.1	N	N		
23.	Spent resin sluice back-flushable filter housing	AB-B	AB-B	B	D	4	1	7	III-3	N	N		
24.	Floor drain tank	AB-D46	AB-D24	W	D	4	2	7	III-3	N	N		
25.	Floor drain		AB-D29	S	D	4	2	7	mfg	N	N		
	tank pump	AB-D50		S	D	4	2	7	mfg	N	N		
26.	Floor drain tank pump motor	AB-D50	AB-D29	W	NA	6	2	E	NEMA MG1	N	N		
27.	Floor drain tank pump suction strainer	AB-D	AB-D	W	D	4	2	7	mfg	N	N		
28.	Floor drain tank backflushable filter housing	AB-B	AB-B	B	D	4	2	7	III-3	N	N		
29.	Waste monitor tank	AB-C81 & C82	AB-C34 & C35	W	D	4	2	7	III-3	N	N		
30.	Waste monitor tank pump	AB-D58 & D59	AB-D17 & D18	W	D	4	2	7	III-3	N	N		
31.	Waste monitor tank pump motor	AB-D58 & D59	AB-D17 & D18	W	NA	6	2	E	NEMA MG1	N	N		
32.	Waste monitor tank backflushable filter housing	AB-B	AB-B	B	D	4	2	7	III-3	N	N		
33.	Waste monitor tank demineralizer	AB-C143	AB-C139	W	D	4	2	7	III-3	N	N		
33a.	Auxiliary waste monitor tank	N/A	AB-D08	B	D	4	2	7	API-650	N	N		
33b.	Auxiliary waste monitor tank pump	N/A	AB-D109	B	D	4	2	7	mfg	N	N		

TABLE 3.2.2-1 (SHEET 21 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
33c.	Auxiliary waste monitor tank pump motor	N/A	AB-D109	B	N/A	6	2	E	mfg	N	N		
34.	Laundry and hot shower tank	AB-D25	shared	W	D	4	2	7	VIII	N	N		
35.	Laundry and hot shower tank pump	AB-D26	shared	W	D	4	2	7	mfg	N	N		
36.	Laundry and hot shower tank pump motor	AB-D26	shared	W	NA	6	2	E	NEMA MG1	N	N		
37.	Laundry and hot shower tank strainer	AB-D26	shared	W	D	4	2	7	B31.1	N	N		
38.	Laundry and hot shower tank filter	AB-D88	shared	W	D	4	2	7	B31.1	N	N		Note ao
39.	Reactor coolant drain tank	C-171′	C-171′	W	D	4	2	7	III-3	N	N		
40.	Reactor coolant drain tank pump	C-171′	C-171′	W	D	4	2	7	III-3	N	N		
41.	Reactor coolant drain tank pump motor	C-171′	C-171′	W	NA	6	2	E	NEMA MG1	N	N		
42.	Reactor coolant drain tank HX:			W									
	Tube side	C-171′	C-171′		D	4	2	7	III-3, TEMA-R	N	N		
	Shell side, ACCW	C-171′	C-171′		D	4	1	5	III-2, TEMA-R	N	N		Note t
43.	Piping and valves from waste evaporator and evaporator condensate demineralizer through evaporator condensate pump discharge valve119			B	D	4	2	7	B31.1	N	N		av

TABLE 3.2.2-1 (SHEET 22 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
44. Piping and valves from waste holdup tank to waste evaporator		B	D	4	1	7	III-3	N	N		
45. Piping and valves from spent resin storage tank to valves HV-7325 and 143 and from valves HV-7305 and 058 to spent resin storage tank		W,B	D	4	1	7	III-3	N	N		
46. Process piping and valves downstream of spent resin tank valves HV-7325 and 143		W,B	D	4	2	7	B31.1	N	N		
47. Piping and valves from chemical drain tank to RPF		B	D	4	2	7	B31.1	N	N		
48. Process piping and valves from laundry and hot shower tank to waste monitor tank, from floor drain tank to waste monitor tank and demineralizer, from demineralizer to waste monitor tank, and discharge from waste monitor tanks		W,B	D	4	2	7	B31.1	N	N		

TABLE 3.2.2-1 (SHEET 23 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
49. Reactor coolant drain tank inlet and discharge piping and valves to valve LV-1003			W,B	D	4	2	7	B31.1	N	N		
50. RPF Demineralizers	RPF		S	D	4	2	7	B31.1	N	N		
51. Instrumentation			W,B	NA	6	2	J	mfg	N	N		Note s
52. RPF Demineralizers Filters			S	D	4	2	7	B31.1	N	N		
WASTE PROCESSING SYSTEM - GASEOUS												
1. Gas decay tanks	AB-B33 B36,B38 B42,B43 B40,B41	AB-B78 B82,B83 B88,B90 B84,B85	W	C	3	1	3	III-3	Y	Y	VIII	
2. Gas decay tank drain pump	AB-B35	AB-B89	W	D	4	2	4	mfg	N	N		
3. Gas decay tank drain pump motor	AB-B35	AB-B89	W	NA	6	2	E	NEMA MG1	N	N		
4. Waste gas drain filter	AB-D95	AB-D82	W	D	4	2	4	VIII	N	N		
5. Sample vessel	AB-B45	AB-B79	B	D	4	2	4	VIII	N	N		
6. Waste gas compressor package	AB-B64 & B68	AB-B100 & B101	W	D	4	1	7	B31.1	N	N	VIII	Note ag
7. Catalytic H_2 recombiner and gas analyzer package	AB-B58 & B59	AB-B74 & B76	W	D	4	1	7	B31.1	N	N	VIII	Note ag
8. Waste gas decay shutdown tank	AB-B46 & B47	shared	W	C	3	1	3	III-3	Y	Y	VIII	
9. Process piping and valves			W,B	C	3	1	3	III-3	Y	Y		Note ay
10. Drain piping and valves			B	D	4	2	4	B31.1	N	N		
11. Safety-related valve operators			W	NA	1	1	E	NEMA MG1	Y	Y		
12. Safety-related instrumentation			W	NA	1	1	J	mfg	Y	Y		Note s
13. Gas traps			S,W	D	4	2	4	mfg	N	N		

TABLE 3.2.2-1 (SHEET 24 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
STEAM GENERATOR BLOWDOWN SYSTEM												
1. Blowdown HXs	AB-C108	AB-C02	W	D	4	2	7	VIII, TEMA-C	N	N		
2. Steam generator drain pump	AB-C108	AB-C02	W	D	4	2	7	mfg	N	N		
3. Blowdown backflushable filters	AB-B	AB-B	B	D	4	2	7	VIII	N	N		
4. Demineralizers	AB-A	AB-A	W	D	4	2	7	VIII	N	N		
5. Spent resin storage tank	AB-C75	AB-C54	W	D	4	2	7	VIII	N	N		
6. Spent resin sluice pump	AB-C74	AB-C55	W	D	4	2	7	mfg	N	N		
7. Spent resin sluice pump motor	AB-C74	AB-C55	W	NA	6	2	E	NEMA MG1	N	N		
8. Spent resin sluice filter	AB-A58	AB-A58	W	D	4	2	7	VIII	N	N		
9. Process valves and piping b2 in. outside containment			W,B	D	4	2	7	B31.1	N	N		
10. Process valves and piping K2 in.			B	D	4	2	7	B31.1	N	N		
11. Process valves and piping inside containment through outer isolation valves			W,B	B	2	1	2	III-2	Y	Y	VIII	
12. Blowdown trim HX	AB-B03	AB-B124	W	D	4	2	7	VIII, TEMA-C	N	N		
13. Steam generator drain pump motor	AB-C108	AB-C02	W	NA	6	2	E	NEMA MG1	N	N		
14. Blowdown outlet filters	AB-A58	AB-A58	B	D	4	2	7	VIII	N	N		
15. Instrumentation			W,B	NA	6	2	J	mfg	N	N		
16. Steam generator blowdown cartridge filter	AB-A49		S	D	4	2	7	VIII	N	N		

TABLE 3.2.2-1 (SHEET 25 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
BACKFLUSHABLE FILTER SYSTEM												
1. Crud tank	AB-C	AB-C	B	D	4	2	7	VIII	N	N		
2. Crud tank pumps	AB-C93 & C94	AB-C28 & C29	B	D	4	2	7	mfg	N	N		
3. Crud tank pump motors	AB-C93 & C94	AB-C28 & C29	B	NA	6	2	E	NEMA MG1	N	N		
4. N_2 accumulator	AB-B63	AB-B103	B	D	4	2	4	VIII	N	N		
5. N_2 system valves and piping			B	D	4	2	4	B31.1	N	N		
6. Flush valves and piping			B	D	4	2	4	B31.1	N	N		
7. Instrumentation			B	NA	6	2	J	mfg	N	N		
MAIN STEAM SYSTEM												
1. Steam generator (shell side)	C	C	W	B	2	1	2	III-2	Y	Y	1-A	
2. Steam diffuser	C	C	W	B	2	1	2	III-2	Y	Y	1-A	
3. Piping from SG to weld 1 after 5-way restraint			B	B	2	1	2	III-2	Y	Y	1-A	
4. Piping downstream of 5-way restraint to turbines			B	D	4	2	4	B31.1	N	N		
5. Safety valves	AB-108 EB-123 & 122	AB-159 EB-123 & 122	B	B	2	1	2	III-2	Y	Y	II	
6. Atmospheric power-operated relief valves	AB-108 EB-123 & 122	AB-159 EB-123 & 122	B	B	2	1	2	III-2	Y	Y	II	
7. Atmospheric power-operated relief valves operators	AB-108 EB-123 & 122	AB-159 EB-123 & 122	B	NA	1	1	E	NEMA MG1	Y	Y	II	
8. Main steam isolation valves	AB-108 EB-123 & 122	AB-159 EB-123 & 122	B	B	2	1	2	III-2	Y	Y	II	

TABLE 3.2.2-1 (SHEET 26 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
9.	Main steam isolation valve actuators	AB-108 EB-123 & 122	AB-159 EB-123 & 122	B	NA	1	1	E	mfg	Y	Y	II	Note bb
10.	Steam packing exhauster condenser	TB-245′	TB-245′	S	D	4	2	4	mfg TEMA-C	N	N		
11.	Safety-related valve operators			B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
12.	Drain and test valves and piping upstream of forged section			B	B	2	1	2	III-2	Y	Y		
13.	Wet layup pumps	C-180′	C-180′	B	D	4	2	4	mfg	N	N		
14.	Wet layup pump motors	C-180′	C-180′	B	NA	6	2	E	NEMA MG1	N	N		
15.	Wet layup piping and valves			B	D	4	2	4	B31.1	N	N		
16.	Mechanical pressure, flow, and level instruments			W	B	2	1	2	III-2	Y	Y		
17.	Safety-related instrumentation			W	NA	1	1	J	mfg	Y	Y		Note s
20.	Steam flow limiters			W	B	2	1	2	III-2	Y	Y		
21.	Steam packing exhauster blower	TB-245′	TB-245′	S	D	4	2	4	mfg	N	N		
22.	Steam packing exhauster blower motor	TB-245′	TB-245′	S	NA	6	2	E	NEMA MG1	N	N		
23.	Main steam isolation valve bypass valves	AB-108 EG-123 & 122	AB-159 EB-123 & 122	B	B	2	1	2	III-2	Y	Y	II	
24.	Main steam isolation valve bypass actuators	AB-108 EB-123 & 122	AB-159 EB-123 & 122	B	NA	1	1	E	mfg	Y	Y	II	

TABLE 3.2.2-1 (SHEET 27 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
AUXILIARY FEEDWATER SYSTEM												
1. Auxiliary feed pump turbine	AFP	AFP	B	NA	0	1	5	NEMA SM23	Y	Y	VI	
2. Auxiliary feed pumps	AFP	AFP	B	C	3	1	3	III-3	Y	Y	VI	
3. Auxiliary feed pump motors	AFP	AFP	B	NA	1	1	E	NEMA MG1	Y	Y	VI	
4. Auxiliary feed turbine steam valve (auto)	AFP	AFP	B	C	3	1	3	III-3	Y	Y	VI	
5. Auxiliary feed turbine steam supply valve motor	AFP	AFP	B	NA	1	1	E	NEMA MG1	Y	Y	VI	
6. AFW piping up to AFW flow control MOVs			B	C	3	1	3	III-3	Y	Y		
7. AFW piping and valves including AFW flow control MOVs			B	B	2	1	2	III-2	Y	Y		
8. AFW flow control MOV motors			B	NA	1	1	E	NEMA MG1	Y	Y		
9. AFW pump suction valves and piping			B	C	3	1	3	III-3	Y	Y		
10. AFW pump suction MOV motors			B	NA	1	1	E	NEMA MG1	Y	Y		
11. Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
12. Flow limiting orifices			B	B	2	1	2	III-2	Y	Y		
13. Degasifier feed pump	O	O	B	D	4	2	4	mfg	N	N		
14. Degasifier feed/transfer pump	O	O	B	D	4	2	4	mfg	N	N		
15. Degasifier transfer pump	O	O	B	D	4	2	4	mfg	N	N		
16. Vacuum degasifier	O	O	B	D	4	2	4	mfg	N	N		
17. Degasifier silencer/separator	O	O	B	D	4	2	4	mfg	N	N		

TABLE 3.2.2-1 (SHEET 28 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
18. Degasifier ejector	O	O	B	D	4	2	4	VIII	N	N		
19. Degasifier piping and valves			B	D	4	2	4	B31.1	N	N		
20. Degasifier pump motors	O	O	B	NA	6	2	E	NEMA MG1	N	N		
21. AFW turbine lube oil cooler:												
Oil side	AFP	AFP	B	NA	0	1	5	III-3	Y	Y	II	
Water side			B	C	3	1	3	III-3	Y	Y	II	
CONDENSATE AND FEEDWATER SYSTEM												
1. Main feed line isolation valves			B	B	2	1	2	III-2	Y	Y		
2. Main feed line isolation valve actuators			B	NA	1	1	E	NEMA MG1	Y	Y		
3. Main and auxiliary feed inlet check valves			B	B	2	1	2	III-2	Y	Y		
4. Piping from forged section to SG			B	B	2	1	2	III-2	Y	Y		
5. Piping upstream of forged section			B	D	4	2	4	B31.1	N	N		
6. Main feed regulating valves			W	C	3	1	3	III-3	Y	Y		
7. Main feed regulating bypass valves			B	C	3	1	3	III-3	Y	Y		
8. Condensate pumps	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
9. Condensate pump motors	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
10. Feed pumps	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
11. Feed pump turbines	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
12. Speed control valves			S	D	4	2	4	B31.1	N	N		
13. Other piping and valves			S	D	4	2	4	B31.1	N	N		

TABLE 3.2.2-1 (SHEET 29 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
14. Feed isolation instrumentation and controls			B	NA	1	1	J	mfg	Y	Y		Note s
15. Other instrumentation and controls			S	NA	6	2	J	mfg	N	N		
16.				(This line has been intentionally left blank.)								
17. Feedwater isolation bypass valves			B	B	2	1	2	III-2	Y	Y	II	
18. Feedwater isolation bypass valves actuator			B	NA	1	1	E	mfg	Y	Y	II	
POWER CONVERSION SYSTEM												
1. Main turbines	TB-270′	TB-270′	S	D	4	2	4	mfg	N	N		
2. Generator	TB-270′	TB-270′	S	D	4	2	4	mfg	N	N		
3. Main condensers	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
4. Feedwater heaters	TB-220′ 245′ & 290′	TB-220′ 245′ & 290′	S	D	4	2	4	VIII	N	N		
5. Moisture separator reheaters	TB-270′	TB-270′	S	D	4	2	4	VIII	N	N		
6. Moisture separator drain tanks	TB-245′	TB-245′	S	D	4	2	4	VIII	N	N		
7. Heater drain tanks	TB-220′	TB-220′		D	4	2	4	VIII	N	N		
8. Reheater drain tanks	TB-245′	TB-245′	S	D	4	2	4	VIII	N	N		
9. Heater drain pumps	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
10. Heater drain pump motors	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
11. Extraction steam valves and piping			S	D	4	2	4	B31.1	N	N		
12. Other valves and piping			S	D	4	2	4	B31.1	N	N		
13. Instrumentation			S	NA	6	2	J	mfg	N	N		
14. Generator H_2 cooler	TB-270′	TB-270′	S	D	4	2	4	mfg	N	N		
15. Generator stator coolant pump	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		

TABLE 3.2.2-1 (SHEET 30 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
16. Generator stator coolant pump motor	TB-220′	TB-220′	S	NA	6	2	E	mfg	N	N		
17. Generator stator cooler	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
CONDENSER AIR EJECTOR SYSTEM												
1. Air ejectors	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
2. Air ejector condensers	TB-220′	TB-220′	S	D	4	2	4	mfg TEMA-C	N	N		
3. Vacuum pumps	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
4. Vacuum pump motors	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
5. Vacuum pump exhaust silencers	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
6. Vacuum pump seal water HXs	TB-195′	TB-195′	S	D	4	2	4	VIII, TEMA-C	N	N		
7. Seal water pumps	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
8. Valves and piping			S	D	4	2	4	B31.1	N	N		
9. Seal water pump motors	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
10. Instrumentation			S	NA	6	2	J	mfg	N	N		
CIRCULATING WATER SYSTEM												
1. Circulating water pumps	O	O	S	D	4	2	4	mfg	N	N		
2. Circulating water pump motors	O	O	S	NA	6	2	E	NEMA MG1	N	N		
3. Condenser water box drain pump	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
4. Condenser water box drain pump motor	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
5. Piping			S	NA	6	2	6	B31.1	N	N		
6. Valves			S	NA	6	2	6	B31.1	N	N		
7. Instrumentation			S	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 31 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
RIVER INTAKE STRUCTURE SYSTEMS												
1. Mixing chamber	VB	shared	S	NA	6	2	C	mfg	N	N		
2. River makeup and dilution pumps	O	shared	S	NA	6	2	6	mfg	N	N		
3. Mixing chamber sample unit	VB	shared	S	NA	6	2	6	mfg	N	N		
4. Mixing chamber sample pump	VB	shared	S	NA	6	2	6	mfg	N	N		
5. Traveling and stationary screens	O	shared	S	NA	6	2	6	mfg	N	N		
6. Process piping and valves			S	NA	6	2	6	B31.1	N	N		
7. Pump motors	VB	shared	S	NA	6	2	E	NEMA MG1	N	N		
8. Instrumentation			S	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 32 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
CONDENSATE CHEMICAL INJECTION SYSTEM												
1. Methoxypropylamine (MPA) storage tank	O	shared	S	D	4	2	4	VIII	N	N		
2. MPA transfer pumps	O	O	S	D	4	2	4	mfg	N	N		
3. Hydrazine day tank	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
4. MPA day tank	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
5. Hydrazine storage tank	O	shared	S	D	4	2	4	mfg	N	N		
6. Hydrazine transfer pump	O	O	S	D	4	2	4	mfg	N	N		
7. Hydrazine mixing pump	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
8. Deleted												
9. Hydrazine dispensing pump (low volume)	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
10. Hydrazine dispensing pump (high volume)	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
11. Continuous condensate feed pumps (Hydrazine and MPA)	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
12. Batch tank	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
13. Batch mixing pump	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
14. Steam generator layup pump	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
15. Feedwater layup pump	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
16. Pump motors	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
17. Piping and valves			S	D	4	2	4	B31.1	N	N		
18. Instrumentation			S	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 33 OF 98)

	Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
	CONDENSATE FILTER DEMINERALIZER SYSTEM												
1.	Spent resin transfer pump	RTB	RTB	B	D	4	2	4	mfg	N	N		
2.	Spent resin transfer pump motor	RTB	RTB	B	NA	6	2	E	NEMA MG1	N	N		
3.	Backwash recovery pump	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
4.	Backwash recovery pump motor	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
5.	Backwash pump	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
6.	Backwash pump motor	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
7.	Decant transfer pump	RTB	RTB	B	D	4	2	4	mfg	N	N		
8.	Decant transfer pump motor	RTB	RTB	B	NA	6	2	E	NEMA MG1	N	N		
9.	Spent resin pump	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
10.	Spent resin pump motor	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
11.	Precoat tank	TB-220′	TB-220′	S	D	4	2	4	API 650	N	N		
12.	Precoat tank agitator	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
13.	Overlay tank	TB-220′	TB-220′	S	D	4	2	4	API 650	N	N		
14.	Overlay tank agitator	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
15.	Holding pumps	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
16.	Holding pump motors	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
17.	Resin traps	TB-220′	TB-220′	S	D	4	2	4	VIII	N	N		
18.	Precoat pump	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		

TABLE 3.2.2-1 (SHEET 34 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
19. Precoat pump motor	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
20. Overlay pump	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
21. Overlay pump motor	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
22. Decant collection tank	RTB	RTB	B	D	4	2	4	API 650	N	N		
23. Backwash recovery tank	TB-195′	TB-195′	S	D	4	2	4	API 650	N	N		
24. Backwash recovery tank scraper	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
25. Phase separator tank	RTB	RTB	B	D	4	2	4	API 650	N	N		
26. Powdex vessels	TB-220′	TB-220′	S	D	4	2	4	VIII	N	N		
27. Air receiver	TB-195′	TB-195′	S	D	4	2	4	VIII	N	N		
28. Valves and piping			S,B	D	4	2	4	B31.1	N	N		
29. Instrumentation			S	NA	6	2	J	mfg	N	N		
30. Dirty spent resin holding tank	TB-195′	shared	G	D	4	2	4	API 650	N	N		
31. Clean spent resin holding tank	TB-195′	shared	G	D	4	2	4	API 650	N	N		
32. Recirculation pump	TB-195′	shared	G	D	4	2	4	mfg	N	N		
33. Recirculation pump motor	TB-195′	shared	G	N/A	6	2	E	NEMA MG1	N	N		
34. Dewatering pump	TB-195′	shared	G	D	4	2	4	mfg	N	N		
35. Dewatering pump motor	TB-195′	shared	G	N/A	6	2	E	NEMA MG1	N	N		
36. Dewatering filters	TB-195′	shared	G	D	4	2	4	mfg	N	N		
37. pressure filter skid	TB-195′	shared	G	D	4	2	4	mfg	N	N		
PLANT MAKEUP WATER WELL SYSTEM												
1. Demineralizer booster pumps	VB	shared	S	NA	6	2	6	mfg	N	N		
2. Demineralizer backwash pump	VB	shared	S	NA	6	2	6	mfg	N	N		ba
3. Plant makeup well pumps	O	shared	S	NA	6	2	6	mfg	N	N		

TABLE 3.2.2-1 (SHEET 35 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
4. Makeup well water storage tank	O	shared	S	NA	6	2	6	API 650	N	N		
5. Pump motors			S	NA	6	2	E	NEMA MG1	N	N		
6. Piping and valves			S	NA	6	2	6	B31.1	N	N		
7. Instrumentation			S	NA	6	2	J	mfg	N	N		
8. Flushing water storage tank Pump (Note 1)	O	shared	G	NA	6	2	6	mfg	N	N		
9. Flushing water storage tank (Note 1)	O	shared	G	NA	6	2	6	API 650	N	N		
PLANT MAKEUP WATER TREATMENT SYSTEM												
1. Vendor supplied water treatment system	VB	shared	S	NA	6	2	6	mfg	N	N		

Notes:

1. The flushing water storage tank and pump have been retired in place.

TABLE 3.2.2-1 (SHEET 36 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
2. All other pump motors	VB	shared	S	NA	6	2	E	NEMA MG1	N	N		
3. Piping and valves	VB	shared	S	NA	6	2	6	B31.1	N	N		
4. Instrumentation	VB	shared	S	NA	6	2	J	mfg	N	N		
PLANT MAKEUP WATER TREATMENT SYSTEM WASTE NEUTRALIZATION SYSTEM												
1. Neutralizer transfer pumps	O	shared	S	NA	6	2	6	mfg	N	N		
2. Pump motors	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
3. Valves and piping			S	NA	6	2	6	B31.1	N	N		
4. Instrumentation			S	NA	6	2	J	mfg	N	N		
DEMINERALIZED WATER SYSTEM												
1. Demineralized water storage tank	O	shared	S	NA	6	2	4	API 650	N	N		
2. Transfer pumps	VB	shared	S	NA	6	2	4	mfg	N	N		
3. Transfer pump motors	VB	shared	S	NA	6	2	E	NEMA MG1	N	N		
4. Valves and piping			S,B	NA	6	2	4	B31.1	N	N		
5. Transfer booster pumps	AB-A17	shared	B	NA	6	2	4	mfg	N	N		
6. Transfer booster pump motors	AB-A17	shared	B	NA	6	2	E	NEMA MG1	N	N		
7. Instrumentation			S,B	NA	6	2	J	mfg	N	N		
DIESEL GENERATOR SYSTEMS												
1. Day tanks	DB	DB	B	NA	0	1	3	III-3	Y	Y	IV	
2. Fuel transfer pumps	VB	VB	B	NA	0	1	3	III-3	Y	Y	IV	

TABLE 3.2.2-1 (SHEET 37 OF 98)

	Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	((g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
3.	Fuel transfer pump motors	VB	VB	B	NA	1	1	E	NEMA MG1	Y	Y	IV	
4.	Duplex fuel filters	DB	DB	B	NA	0	1	5	mfg	Y	Y	IV	
5.	Duplex fuel strainers	DB	DB	B	NA	0	1	5	mfg	Y	Y	IV	
6.	Engine driven fuel pump	DB	DB	B	NA	0	1	5	mfg	Y	Y	IV	
7.	Engine driven lube oil pump	DB	DB	B	NA	0	1	5	mfg	Y	Y	IV	
8.	Lube oil HX	DB	DB	B	NA	0	1	3	III-3	Y	Y	IV	
9.	Lube oil heater	DB	DB	B	NA	0	1	5	mfg	Y	Y		Casing is ASME VIII
10.	Lube oil keep warm pump	DB	DB	B	NA	0	1	3	III-3	Y	Y	IV	
11.	Lube oil keep warm pump motor	DB	DB	B	NA	6	2	E	mfg	N	N		
12.	Lube oil strainers	DB	DB	B	NA	0	1	5	mfg	Y	Y	IV	
13.	Lube oil filters	DB	DB	B	NA	0	1	3	III-3	Y	Y	IV	
14.	Cooling jacket water heater	DB	DB	B	NA	0	1	5	mfg	Y	Y		Casing is ASME VIII
15.	Engine driven jacket water pump	DB	DB	B	NA	0	1	5	mfg	Y	Y	IV	
16.	Jacket water keep warm pump	DB	DB	B	NA	0	1	3	III-3	Y	Y	IV	
17.	Jacket water keep warm pump motor	DB	DB	B	NA	6	2	E	mfg	N	N		
18.	Jacket water HX	DB	DB	B	NA	0	1	3	III-3	Y	Y	IV	
19.	Jacket water standpipe	DB	DB	B	NA	0	1	3	III-3	Y	Y	IV	
20.	Air compressors	DB	DB	B	NA	6	2	6	mfg	N	N		
21.	Air receivers	DB	DB	B	NA	0	1	3	III-3	Y	Y	IV	
22.	Air dryers	DB	DB	B	NA	6	2	6	mfg	N	N		
23.	Air compressor after coolers	DB	DB	B	NA	6	2	6	mfg	N	N		
24.	Intake and exhaust silencers	DB	DB	B	NA	0	1	5	mfg	Y	Y	IV	
25.	Intake air filter	DB	DB	B	NA	0	1	5	mfg	Y	Y	IV	

TABLE 3.2.2-1 (SHEET 38 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
26.	Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y	IV	Note s
27.	Diesel generators	DB	DB	B	NA	0	1	5	DEMA, NEMA	Y	Y	IV	
28.	Fuel oil pressure regulating valve	DB	DB	B	NA	0	1	5	mfg	Y	Y	IV	
29.	Engine boundary piping and valves			B	NA	0	1	5	mfg	Y	Y		
30.	Engine auxiliaries piping and valves			B	NA	0	1	3	III-3	Y	Y		
31.	Flame arresters	VB	VB	B	NA	6	2	6	mfg	N	N		
32.	Lube oil sump	DB	DB	B	NA	0	1	3	III-3	Y	Y		
33.	Fuel oil storage tanks	VB	VB	B	NA	0	1	3	III-3	Y	Y		
34.	Exhaust piping			B	NA	0	1	5	mfg	Y	Y		
35.	Category 1 buried pipe			B	NA	0	1	3	III-3	Y	Y		
FIRE PROTECTION SYSTEMS													Note v
1.	Diesel fire pumps	FPH	FPH	B	NA	6	2	9	NFPA	N	N		
2.	Diesel engines	FPH	FPH	B	NA	6	2	9	mfg	N	N		
3.	Motor driven fire pump	FPH	FPH	B	NA	6	2	9	NFPA	N	N		
4.	Pump motors	FPH	FPH	B	NA	6	2	E	NEMA MG1	N	N		
5.	Jockey pumps	FPH	FPH	B	NA	6	2	9	NFPA	N	N		
6.	Diesel fuel oil tanks	O	shared	B	NA	6	2	9	NFPA	N	N		
7.	Water storage tanks	O	shared	B	NA	6	2	9	NFPA	N	N		
8.	Water system piping and valves			B,S	NA	6	2	9	NFPA	N	N		
9.	Halon system piping, valves, and components			B	NA	6	2	9	NFPA	N	N		
10.	Instrumentation			B	NA	6	2	J	mfg	N	N		
11.	Hydrant, hose-houses	O	shared	S	NA	6	2	9	NFPA	N	N		

TABLE 3.2.2-1 (SHEET 39 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
12.	Hose cabinet and fire extinguishers	VB	VB	S	NA	6	2	9	NFPA	N	N		
13.	Local suppression indication panel	VB	VB	B	NA	6	2	9	mfg	N	N		
FIRE PROTECTION SYSTEMS - SEISMIC CATEGORY 1													
1.	Valves and piping			B	C	3	1	3	III-3	N	N		Note x
AUXILIARY GAS SYSTEMS (N_2 , H_2 , AND 0_2)													
1.	N_2 low pressure vaporizer	O	shared	S	NA	6	2	6	mfg	N	N		
2.	N_2 high pressure vaporizer	O	shared	S	NA	6	2	6	mfg	N	N		
3.	N_2 cryogenic pump	O	shared	S	NA	6	2	6	mfg	N	N		
4.	N_2 cryogenic pump motor	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
5.	Liquid N_2 storage tank	O	shared	S	NA	6	2	6	VIII	N	N		
6.	Gaseous N_2 active storage tanks	O	shared	S	NA	6	2	6	VIII	N	N		
7.						(This line has been intentionally left blank.)							
8.	O_2 active storage tanks	O	shared	S	NA	6	2	6	VIII	N	N		
9.	O_2 reserve storage tanks	O	shared	S	NA	6	2	6	VIII	N	N		
10.	H_2 active (liquid storage tanks)			REMOVED									Note ap
11.						(This line has been intentionally left blank.)							
12.	Piping and valves	O	shared	S	NA	6	2	6	B31.1	N	N		
13.	Instrumentation	O	shared	S	NA	6	2	J	mfg	N	N		
NSSS LIQUID SAMPLING SYSTEM													
1.	Process sample valves and piping			B	D	4	2	4	B31.1	N	N		See note u

TABLE 3.2.2-1 (SHEET 40 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
2. RCS sample valves and piping through outside containment isolation valve			B	B	2	1	2	III-2	Y	Y		
3. Sample vessels	CB-144	CB-144	B	D	4	2	4	VIII	N	N		
4. Sample coolers	FB-A10	FB-A01	B	D	4	2	4	VIII	N	N		
5. Deleted												
6. Instrumentation			B	NA	6	2	J	mfg	N	N		
NSSS GAS SAMPLING SYSTEM												
1. Sample vessels	CB-144	CB-144	B	D	4	2	4	VIII	N	N		
2. Valves and piping inside vent hood			B	D	4	2	4	B31.1	N	N		
3. Instrumentation			B	NA	6	2	J	mfg	N	N		
POST-ACCIDENT SAMPLING SYSTEM												
1. Deleted												See paragraph 9.3.2.2.5
2. Deleted												
3. Deleted												
4. Sample piping and valves			B	D	4	1	4	B31.1	N	N		
5. Containment penetration piping and valves			B	B	2	1	2	III-2	Y	Y		
6. Other piping and valves			B	D	4	2	4	B31.1	N	N		
7. Deleted												
TURBINE PLANT SAMPLING SYSTEM												
1. Hotwell sample pumps	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
2. Hotwell sample pump motors	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		

TABLE 3.2.2-1 (SHEET 41 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
3. Sample coolers	TB-220′	TB-220′	S	D	4	2	4	mfg, TEMA C	N	N		
4. Process valves and piping			S	D	4	2	4	B31.1	N	N		
5. Chilled water circulating pump	TB-220′	TB-220′	S	NA	6	2	6	mfg	N	N		
6. Chilled water circulating pump motor	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
7. Condenser circulating water sample pump	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
8. Condenser circulating water sample pump motor	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
9. Instrumentation			S	NA	6	2	J	mfg	N	N		
CONTROL BUILDING DRAIN SYSTEM												
1. Sump pumps	CB-C	CB-C	B	D	4	2	4	mfg	N	N		
2. Sump pump motors	CB-C	CB-C	B	NA	6	2	E	NEMA MG1	N	N		
3. Piping, valves, and floor drain boxes			B	D	4	1	4	B31.1, API 650	N	N		
4. Instrumentation			B	NA	6	2	J	mfg	N	N		
AUXILIARY BUILDING FLOOD RETAINING ROOMS, ALARMS, AND DRAINS												
1. ESF pump room: Alarm units			B	NA	1	1	J	mfg	Y	Y		Notes as, at
Piping to room isolation valve			B	D	4	1	4	B31.1	N	N		
Isolation valves			B	C	3	1	3	III-3	Y	Y		
2. Penetration rooms:												
Alarm units			B	NA	6	2	J	mfg	N	N		
Valves and piping			B	D	4	1	4	B31.1	N	N		
3. Instrumentation			B	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 42 OF 98)

	Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
CONTAINMENT, AUXILIARY BUILDING, AND MISCELLANEOUS DRAIN SYSTEMS													
1.	Radioactive drain sump pumps	AB-D55	AB-D20	B	D	4	2	4	mfg	N	N		
2.	Reactor cavity sump pumps	C-171′	C-171′	B	D	4	1	4	mfg	N	N		
3.	Containment sump pumps	C-171′	C-171′	B	D	4	1	4	mfg	N	N		
4.	Penetration room sump pumps	AB-D73	AB-D107	B	D	4	1	4	mfg	N	N		
5.	Auxiliary building sump pumps	AB-D51	AB-D28	B	D	4	2	4	mfg	N	N		
6.	Component cooling water drain tank	AB-D75	AB-D06	B	D	4	2	4	API 650	N	N		
7.	Component cooling water drain tank pump	AB-D75	AB-D06	B	D	4	2	4	mfg	N	N		
8.	Clean water sump pumps	AB-D36	AB-D32	B	D	4	2	4	mfg	N	N		
9.	NSCW pumphouse sump pumps	NSW	NSW	B	D	4	2	4	mfg	N	N		
10.	Diesel electrical tunnel sump pumps	VB	VB	B	D	4	2	4	mfg	N	N		
11.	Main steam and feedwater tunnel sump pumps	VB	VB	B	D	4	2	4	mfg	N	N		
12.	Diesel building oily waste sump pumps	DB	DB	B	D	4	2	4	mfg	N	N		
13.	Deleted												
14.	Deleted												
15.	Auxiliary feedwater pumphouse sump pumps	AFP	AFP	B	D	4	2	4	mfg	N	N		
16.	Electric boiler building sump pumps	VB	Shared	B	D	4	2	4	mfg	N	N		

TABLE 3.2.2-1 (SHEET 43 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
17. Containment, reactor cavity, and penetration room sump pump motors			B	NA	6	1	E	mfg	N	N		
18. All other pump motors			B	NA	6	2	E	NEMA MG1	N	N		
19. Instrumentation			B	NA	6	2	J	mfg	N	N		
20. Drain isolation valves for ESF pump, HX, and valve rooms			B	C	3	1	3	III-3	Y	Y		Notes as, at
21. Drain piping valves inside containment			B	D	4	2	4	B31.1	N	N		
22. All other valves and all piping			B	D	4	1	4	B31.1	N	N		
23. Tendon Gallery Sump Pumps	C-148′	C-148′	G	D	4	2	4	mfg	N	N		
24. Tendon Gallery Sump Pump Motors	C-148′	C-148′	G	NA	6	2	E	mfg	N	N		
25. Tendon Gallery Sump Pump discharge valves and piping			G	D	4	2	4	B31.1	N	N		

TABLE 3.2.2-1 (SHEET 44 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
TURBINE BUILDING DRAIN SYSTEM												
1. Turbine building drain tanks	TB-195′	TB-195′	S	D	4	2	4	API-650	N	N		
2. Turbine building drain transfer pumps	TB-195′	TB-195′	S	D	4	2	4	mfg	N	N		
3. Turbine building drain transfer pump motors	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
4. Turbine building sump pumps	TB-195′	TB-195′	S	D	4	2	4	API-610	N	N		
5. Turbine building sump pump motors	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
6. Turbine building drain discharge filter	AB-D93	AB-D84	W	D	4	2	4	VIII	N	N		
7. Turbine building mixed bed demineralizers	AB-C144 & C145	AB-C147 & C148	B	D	4	2	4	VIII	N	N		
8. Turbine building drain feed filter	AB-D92	AB-D85	B	D	4	2	4	VIII	N	N		
9. Turbine building drain oil separator	AB-D89	AB-D87	W	D	4	2	4	VIII	N	N		
10. Valves and piping			S,B	D	4	2	4	B31.1	N	N		
11. Instrumentation			S,B	NA	6	2	J	mfg	N	N		
TURBINE LUBE OIL STORAGE AND FILTRATION SYSTEM												
1. Main lube oil reservoir package	TB-245′	TB-245′	S	NA	6	2	6	mfg	N	N		
2. FW pump turbine lube oil reservoir packages	TB-220′	TB-220′	S	NA	6	2	6	mfg	N	N		
3. Clean lube oil storage tank	O	shared	S	NA	6	2	6	API 650	N	N		
4. Dirty lube oil storage tank	O	shared	S	NA	6	2	6	API 650	N	N		
5. Lube oil storage tank transfer pump	O	shared	S	NA	6	2	6	mfg	N	N		

TABLE 3.2.2-1 (SHEET 45 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
6. Vapor extractors	TB-220′	TB-220′	S	NA	6	2	6	mfg	N	N		
7. Lube oil filter pumps	TB-220′	TB-220′	S	NA	6	2	6	mfg	N	N		
8. Pump motors			S	NA	6	2	E	NEMA MG1	N	N		
9. Main lube oil conditioner package	TB-220′	TB-220′	S	NA	6	2	6	B31.1	N	N		
10. FW pump lube oil conditioner package	TB-195′	TB-195′	S	NA	6	2	6	B31.1	N	N		
11. Oil coolers	TB-270′	TB-270′	S	NA	6	2	6	mfg	N	N		
12. Main lube oil conditioner backflow tank			S	NA	6	2	6	mfg	N	N		
13. Lube oil drain pump	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
14. Valves and piping			S	NA	6	2	6	B31.1	N	N		
15. Instrumentation			S	NA	6	2	J	mfg	N	N		
AUXILIARY STEAM SYSTEM												
1. Deleted												
2. Deleted												
3. Other mechanical equipment	VB	shared	S	NA	6	2	6	mfg	N	N		
4. Electrical equipment			S	NA	6	2	E	mfg	N	N		
5. Instrumentation			S	NA	6	2	J	mfg	N	N		
6. Valves and piping			S	NA	6	2	6	B31.1	N	N		
TURBINE GENERATOR H_2 AND H_2 SEAL OIL SYSTEM												
1. Main seal oil pump	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
2. Recirculating seal oil pump	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
3. Seal oil vacuum pump	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
4. Emergency seal oil pump	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
5. Pump motors	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		

TABLE 3.2.2-1 (SHEET 46 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
6.	Valves and piping			S	D	4	2	4	B31.1	N	N		
7.	Instrumentation			S	NA	6	2	J	mfg	N	N		
TURBINE GENERATOR CO_2 SYSTEM													
1.	All mechanical components	TB-220′	TB-220′	S	D	4	2	4	mfg	N	N		
TURBINE PLANT CLOSED COOLING WATER SYSTEM													
1.	Closed cooling HXs	TB-195′	TB-195′	S	NA	6	2	6	VIII	N	N		
2.	Closed cooling circulating pumps	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
3.	Closed cooling water makeup surge tank	TB-270′	TB-270′	S	NA	6	2	6	VIII	N	N		Note ac
4.	Pump motors	TB-195′	TB-195′	S	NA	6	2	E	NEMA MG1	N	N		
5.	Piping and valves			S	NA	6	2	6	B31.1	N	N		
6.	Chemical addition pot	TB	TB	S	NA	6	2	6	mfg	N	N		
7.	Instrumentation			S	NA	6	2	J	mfg	N	N		
TURBINE PLANT COOLING WATER SYSTEM													
1.	Turbine plant cooling water pumps	VB	VB	S	D	4	2	4	mfg	N	N		
2.	Turbine plant cooling water pump motors	VB	VB	S	NA	6	2	E	NEMA MG1	N	N		
3.	Piping and valves			S,B	NA	6	2	6	mfg	N	N		
4.	Instrumentation			S	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 47 OF 98)

	Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
	WASTE EVAPORATOR STEAM SUPPLY SYSTEM												
1.	Electric boiler feed-water pumps	AB-D52	shared	B	D	4	2	4	mfg	N	N		av
2.	Electric boiler condensate receiver tank	AB-D52	shared	B	D	4	2	4	API 650	N	N		av

TABLE 3.2.2-1 (SHEET 48 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
3. Steam isolation valves		shared										av
4. Steam isolation piping		shared										av
5. Manual valve upstream of steam isolation valves		shared										av
WASTE WATER EFFLUENT SYSTEM												
1. Waste water retention basin transfer pumps	O	shared	S	NA	6	2	6	mfg	N	N		
2. Waste water retention basin transfer pump motors	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
3. Low voltage transformer area sump pump	O	shared	S	NA	6	2	6	mfg	N	N		
4. Low voltage transformer area sump pump motor	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
5. Firewater pumphouse oily waste separator pumps	O	shared	S	NA	6	2	6	mfg	N	N		
6. Firewater pumphouse oily waste separator pump motors	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
7. Auxiliary boiler room sump pumps	VB	shared	S	NA	6	2	6	mfg	N	N		

TABLE 3.2.2-1 (SHEET 49 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
8.	Auxiliary boiler room sump pump motors	VB	shared	S	NA	6	2	E	NEMA MG1	N	N		
9.	Waste water sample booster pump	VB	shared	S	NA	6	2	6	mfg	N	N		
10.	Waste water sample booster pump motor	VB	shared	S	NA	6	2	E	NEMA MG1	N	N		
11.	Water treatment building floor drain sump pump	O	shared	S	NA	6	2	6	mfg	N	N		
12.	Water treatment building floor drain sump pump motor	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
13.	Lube oil storage area sump pump	O	shared	S	NA	4	2	4	mfg	N	N		
14.	Lube oil storage area sump pump motor	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
15.	Station service transformer area sump pumps	O	O	S	NA	6	2	6	mfg	N	N		
16.	Station service transformer area sump pumps motor	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
17.	Outdoor pipe trench sump pump	O	shared	S	NA	6	2	6	mfg	N	N		
18.	Outdoor pipe trench sump pump motor	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
19.	Automatic transformer area sump pump	O	shared	S	NA	6	2	6	mfg	N	N		
20.	Automatic transformer area sump pump motor	O	shared	S	NA	6	2	E	NEMA MG1	N	N		

TABLE 3.2.2-1 (SHEET 50 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
21. Switch house station service transformer area sump pump	O	shared	S	NA	6	2	6	mfg	N	N		
22. Switch house station service transformer area sump pump motor	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
23. Lined startup pond discharge pump	O	shared	S	NA	6	2	6	mfg	N	N		
24. Lined startup pond discharge pump motor	O	shared	S	NA	6	2	E	NEMA MG1	N	N		
25. Waste water dechlorination package	VB	shared	S	NA	6	2	6	mfg	N	N		
26. Piping and valves			S	NA	6	2	6	mfg	N	N		
27. Instrumentation			S	NA	6	2	J	mfg	N	N		
INSTRUMENT AND SERVICE AIR SYSTEM												
1. Rotary air compressors	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
2. Rotary air compressor motors	TB-195′	TB-195	S	NA	6	2	E	NEMA MG1	N	N		
3. Air receivers	TB-195′	TB-195′	S	NA	6	2	6	VIII	N	N		
4. Intake filter silencers	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
5. Aftercoolers	TB-195′	TB-195′	S	NA	6	2	6	VIII	N	N		
6. Moisture separators	TB-195′	TB-195′	S	NA	6	2	6	VIII	N	N		

TABLE 3.2.2-1 (SHEET 51 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
9. Coolant heat exchangers	TB-195′	TB-195′	S	NA	6	2	6	VIII	N	N		
10. Air coolant receivers	TB-195′	TB-195′	S	NA	6	2	6	VIII	N	N		
11. Deleted												
12. Deleted												
13. Service air prefilters	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
14. Service air afterfilters	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
15. Instrument air prefilters	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
16. Instrument air afterfilters	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
17. Service air dryers	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
18. Instrument air dryers	TB-195′	TB-195′	S	NA	6	2	6	mfg	N	N		
19. Safety-related piping and valves (other than containment isolation)			B	NA	0	1	3	III-3	Y	Y		Note ad
20. Containment penetration piping and valves			B	B	2	1	2	III-2	Y	Y		
21. All other piping and valves			S	NA	6	2	6	B31.1	N	N		
22. Instrumentation			S	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 52 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
POTABLE WATER SYSTEM												
1. Hot water recirculating pumps	CB	shared	B	NA	6	2	6	mfg	N	N		
2. Hot water recirculating pump motors	CB	shared	B	NA	6	2	E	NEMA MG1	N	N		
3. Water heaters	CB	shared	B	NA	6	2	E	UL	N	N		
4. Piping and valves			B	NA	6	2	6	B31.1	N	N		
5. Instrumentation			B	NA	6	2	J	mfg	N	N		
6. Potable water booster pumps	VB	shared	S	NA	6	2	6	mfg	N	N		
7. Potable water jockey pump	VB	shared	S	NA	6	2	6	mfg	N	N		
8. Chlorine injection pump	VB	shared	S	NA	6	2	6	mfg	N	N		
9. Chlorine injector	VB	shared	S	NA	6	2	6	mfg	N	N		
10. Chlorine dispenser	VB	shared	S	NA	6	2	6	mfg	N	N		
11. Potable water storage tank	O	shared	S	NA	6	2	6	mfg	N	N		
12. Pump motors	VB	shared	S	NA	6	2	E	NEMA MG1	N	N		
REACTOR COOLANT SYSTEM LEAK DETECTION SYSTEM												
1. Radiation monitors	C	C	W	NA	6	1	J	mfg	N	N		
2. Condensate measuring instruments	C	C	B	NA	6	2	J	mfg	N	N		
3. Condensate measuring piping and valves	C	C	B	D	4	2	4	B31.1	N	N		
4. Humidity instruments	C	C	B	NA	6	2	J	mfg	N	N		
5. Tank and sump level instruments	C	C	B	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 53 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
MISCELLANEOUS LEAK DETECTION SYSTEM												
1. Piping and valves			B	D	4	2	4	B31.1	N	N		
FUEL HANDLING BUILDING DRAIN SYSTEM												
1. Sump pumps	FB-C09	FB-C03	B	D	4	2	4	mfg	N	N		
2. Sump pump motors	FB-C09	FB-C03	B	NA	6	2	E	NEMA MG1	N	N		
3. Piping and valves			B	D	4	1	4	B31.1	N	N		
4. Instrumentation			B	NA	6	2	J	mfg	N	N		
UTILITY WATER SYSTEM												
1. Utility water booster pumps	VB	shared	S	NA	6	2	6	mfg	N	N		
2. Utility water booster pump motors	VB	shared	S	NA	6	2	E	NEMA MG1	N	N		
3. Utility water loop header	O	shared	S	NA	6	2	C	mfg	N	N		
4. Valves and all other piping			S,B	NA	6	2	6	mfg	N	N		
5. Instrumentation			S	NA	6	2	J	mfg	N	N		
CONTAINMENT AIR COOLING SYSTEM												Note ah
1. Cooling fans	C-238′	C-238′	B	NA	0	1	5	AMCA	Y	Y	1-B	
2. Fan motors	C-238′	C-238′	B	NA	1	1	E	NEMA MG1	Y	Y	1-B	
3. Cooling coils	C-238′	C-238′	B	B	2	1	2	III-2	Y	Y	1-B	
4. Ductwork	C	C	B	NA	0	1	5	ANSI N509	Y	Y		
5. Dampers	C	C	B	NA	0	1	5	ANSI N509	Y	Y	1-B	
6. Damper motors	C	C	B	NA	1	1	E	NEMA MG1	Y	Y		
7. Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
CONTAINMENT LOWER LEVEL AIR CIRCULATING SYSTEM												Note ah
1. Fans	C-176′	C-176′	B	NA	6	2	6	AMCA	N	N		
2. Fan motors	C-176′	C-176′	B	NA	6	2	E	NEMA MG1	N	N		
3. Ductwork			B	NA	6	2	6	SMACNA	N	N		
4. Instrumentation			B	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 54 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
CONTAINMENT PREACCESS FILTER SYSTEM												Note ah
1. Fans	C-268′	C-268′	B	NA	6	2	6	AMCA	N	N		
2. Fan motors	C-268′	C-268′	B	NA	6	2	E	NEMA MG1	N	N		
3. HEPA filters	C-261′	C-261′	B	NA	6	2	6	ANSI N509	N	N		
4. Charcoal filters	C-261′	C-261′	B	NA	6	2	6	ANSI N509	N	N		
5. Heaters	C-261′	C-261′	B	NA	6	2	E	UL	N	N		
6. Moisture eliminators	C-261′	C-261′	B	NA	6	2	6	ANSI N509	N	N		
7. Instrumentation			B	NA	6	2	J	mfg	N	N		
CONTAINMENT NORMAL PREACCESS PURGE EXHAUST SYSTEM												Note ah
1. Preaccess purge fan	EB-227′	EB-227′	B	NA	6	2	6	AMCA	N	N		
2. Preaccess purge fan motor	EB-227′	EB-227′	B	NA	6	2	E	NEMA MG1	N	N		
3. Minipurge fan	EB-227′	EB-227′	B	NA	6	2	6	AMCA	N	N		
4. Minipurge fan motor	EB-227′	EB-227′	B	NA	6	2	E	NEMA MG1	N	N		
5. Moisture eliminators	EB-220′	EB-220′	B	NA	6	2	6	ANSI N509	N	N		
6. Heaters	EB-220′	EB-220′	B	NA	6	2	E	UL	N	N		
7. HEPA filters	EB-220′	EB-220′	B	NA	6	2	6	ANSI N509	N	N		
8. Charcoal filters	EB-220′	EB-220′	B	NA	6	2	6	ANSI N509	N	N		
9. Preaccess purge exhaust unit housing	EB-220′	EB-220′	B	NA	6	1	6	ANSI N509	N	N		
10. Containment penetration ducting			B	B	2	1	2	III-2	Y	Y		
11. All other ductwork			B	NA	6	2	6	SMACNA	N	N		
12. Isolation dampers			B	B	2	1	2	III-2	Y	Y		
13. Isolation damper motors			B	NA	1	1	E	NEMA MG1	Y	Y		
14. Discharge dampers			B	NA	6	2	6	ANSI N509	N	N		
15. Instrumentation for containment isolation			B	NA	1	1	J	mfg	Y	Y		Note s

TABLE 3.2.2-1 (SHEET 55 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
16.	All other instrumentation			B	NA	6	2	J	mfg	N	N		
CONTAINMENT NORMAL PREACCESS PURGE SUPPLY SYSTEM													Note ah
1.	Preaccess purge fan	EB-223′	EB-223′	B	NA	6	2	6	AMCA	N	N		
2.	Preaccess purge fan motor	EB-223′	EB-223′	B	NA	6	2	E	NEMA MG1	N	N		
3.	Minipurge fan	EB-223′	EB-223′	B	NA	6	2	6	AMCA	N	N		
4.	Minipurge fan motor	EB-223′	EB-223′	B	NA	6	2	E	NEMA MG1	N	N		
5.	Prefilters	EB-220′	EB-220′	B	NA	6	2	6	ASHRAE	N	N		
6.	Heaters	EB-220′	EB-220′	B	NA	6	2	E	UL	N	N		
7.	Containment penetration ducting			B	B	2	1	2	III-2	Y	Y		
8.	All other ductwork			B	NA	6	2	6	SMACNA	N	N		
9.	Isolation dampers			B	B	2	1	2	III-2	Y	Y		
10.	Isolation damper motors			B	NA	1	1	E	NEMA MG1	Y	Y		
11.	Inlet damper			B	NA	6	2	6	ANSI N509	N	N		
12.	Instrumentation for containment isolation			B	NA	1	1	J	mfg	Y	Y		Note s
13.	All other instrumentation			B	NA	6	2	J	mfg	N	N		
CONTAINMENT POST-LOCA PURGE EXHAUST SYSTEM													Note ah
1.	Containment penetration ducting			B	B	2	1	2	III-2	Y	Y		
2.	Welded ductwork			B	NA	6	1	6	ANSI N509	N	N		
3.	All other ductwork			B	NA	6	2	6	SMACNA	N	N		
4.	Isolation dampers			B	B	2	1	2	III-2	Y	Y		
5.	Isolation damper motors			B	NA	1	1	E	NEMA MG1	Y	Y		
6.	Heaters	EB-222′	EB-222′	B	NA	6	2	E	UL	N	N		
7.	Moisture eliminators	EB-222′	EB-222′	B	NA	6	2	6	ANSI N509	N	N		
8.	HEPA filters	EB-222′	EB-222′	B	NA	6	2	6	ANSI N509	N	N		

TABLE 3.2.2-1 (SHEET 56 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
9. Charcoal filters	EB-222′	EB-222′	B	NA	6	2	6	ANSI N509	N	N		
10. Post-LOCA purge exhaust unit housing	EB-222′	EB-222′	B	NA	6	1	6	ANSI N509	N	N		
11. Inlet globe valve and discharge check valve			B	NA	6	1	6	B31.1	N	N		
12. Instrumentation			B	NA	6	2	J	mfg	N	N		
CONTAINMENT CRDM COOLING SYSTEM												Note ah
1. Fans	C-240′	C-240′	W	NA	6	2	6	AMCA	N	N		
2. Fan motors	C-240′	C-240′	W	NA	6	2	E	NEMA MG1	N	N		Note m
3. Ductwork	C	C	W	NA	6	2	6	SMACNA	N	N		
4. Instrumentation			W	NA	6	2	J	mfg	N	N		
CONTAINMENT CAVITY COOLING SYSTEM												Note ah
1. Fans	C-206′	C-206′	B	NA	6	1	6	AMCA	N	N		
2. Fan motors	C-206′	C-206′	B	NA	6	1	E	NEMA MG1	N	N		Note m
3. Ductwork	C	C	B	NA	6	2	6	ANSI N509	N	N		
4. Dampers	C	C	B	NA	6	2	6	ANSI N509	N	N		
5. Cooling coils	C-206′	C-206′	B	B	2	1	2	III-2	Y	Y	1-B	
6. Instrumentation			B	NA	6	2	J	mfg	N	N		
CONTAINMENT REACTOR SUPPORT COOLING SYSTEM												Note ah
1. Fans	C-171′	C-171′	B	NA	6	1	6	AMCA	N	N		
2. Fan motors	C-171′	C-171′	B	NA	6	1	E	NEMA MG1	N	N		Note m
3. Ductwork			B	NA	6	1	6	ANSI N509	N	N		Note ae
4. Dampers			B	NA	6	1	6	ANSI N509	N	N		
5. Damper motors			B	NA	6	1	E	NEMA MG1	N	N		
6. Instrumentation			B	NA	6	2	J	mfg	N	N		
H_2 RECOMBINER AND MONITORING SYSTEM												
1. Penetration piping and valves			B	B	2	1	2	III-2	Y	Y		
2. Other piping and valves			B	B	2	1	2	III-2	Y	Y		
3. H_2 monitors			B	NA	1	1	J	mfg	Y	Y		

TABLE 3.2.2-1 (SHEET 57 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
4. H_2 recombiners	C-261′	C-261′	W	NA	0	1	5	III-2	Y	Y		Note m
5. Safety-related recombiner instrumentation			W,B	NA	1	1	J	mfg		Y	Y	Note s
6. Safety-related valve operators			B	NA	1	1	E	NEMA MG1	Y	Y		
7. All other instrumentation			W,G	NA	6	1	J	mfg	N	N		
CONTAINMENT AUXILIARY AIR COOLING SYSTEM												Note ah
1. Fans	C-261′	C-261′	B	NA	6	1	6	AMCA	N	N		
2. Fan motors	C-261′	C-261′	B	NA	6	1	E	NEMA MG1	N	N		
3. Cooling coils	C-261′	C-261′	B	B	2	1	2	III-2	Y	Y	1-A	
4. Instrumentation			B	NA	6	2	J	mfg	N	N		
CONTAINMENT POST-LOCA CAVITY PURGE SYSTEM												Note ah
1. Fans	C-199′	C-199′	B	NA	0	1	5	AMCA	Y	Y	1-A	
2. Fan motors	C-199′	C-199′	B	NA	1	1	E	NEMA MG1	Y	Y	1-A	
3. Ductwork and backdraft damper			B	NA	0	1	5	ANSI N509	Y	Y		
4. Embedded ductwork			B	NA	0	1	5	B31.1	Y	Y		
5. Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
EQUIPMENT BUILDING HVAC SYSTEM												Note ah
1. Equipment building ventilation fans	EB-236′	EB-236′	B	NA	6	2	6	AMCA	N	N		
2. Equipment building ventilation fan motors	EB-236′	EB-236′	B	NA	6	2	E	NEMA MG1	N	N		
3. Tendon gallery ventilation fans	EB-177′	EB-177′	B	NA	6	2	6	AMCA	N	N		
4. Tendon gallery ventilation fan motors	EB-177′	EB-177′	B	NA	6	2	E	NEMA MG1	N	N		
5. Ductwork			B	NA	6	2	6	ANSI N509	N	N		
6. Dampers			B	NA	6	2	6	ANSI N509	N	N		
7. Damper motors			B	NA	6	2	E	NEMA MG1	N	N		
8. Instrumentation			B	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 58 OF 98)

Principal System and Components		(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
CONTROL ROOM HVAC SYSTEM (ESSENTIAL PORTION)													Note ah
1.	Filter unit fans	CB-312 & 321	CB-305 & 311	B	NA	0	1	5	AMCA	Y	Y	IX	
2.	Filter unit fans motors	CB-312 & 321	CB-305 & 311	B	NA	1	1	E	NEMA MG1	Y	Y	IX	
3.	Return air fans	CB-312 & 321	CB-305 & 311	B	NA	0	1	5	AMCA	Y	Y	IX	Note ar
4.	Return air fans motor	CB-312 & 321	CB-305 & 311	B	NA	1	1	E	NEMA MG1	Y	Y	IX	Note ar
5.	Moisture eliminators	CB-312 & 321	CB-305 & 311	B	NA	0	1	5	ANSI N509	Y	Y	IX	
6.	Heaters	CB-312 & 321	CB-305 & 311	B	NA	1	1	E	UL, ANSI N509	Y	Y	IX	
7.	HEPA filters	CB-312 & 321	CB-305 & 311	B	NA	0	1	5	ANSI N509	Y	Y	IX	
8.	Charcoal filters	CB-312 & 321	CB-305 & 321	B	NA	0	1	5	ANSI N509	Y	Y	IX	
9.	Cooling coils	CB-312 & 321	CB-305 & 311	B	C	3	1	3	III-3	Y	Y	IX	
10.	Dampers			B	NA	0	1	5	ANSI N509	Y	Y		
11.	Damper motors			B	NA	1	1	E	NEMA MG1	Y	Y		
12.	Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
13.	Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
14.	ESF chiller room exhaust fans	CB-313 & 320	CB-308 & 310	B	NA	0	1	5	AMCA	Y	Y	IX	
15.	ESF chiller room exhaust fan motors	CB-313 & 320	CB-308 & 310	B	NA	1	1	E	NEMA MG1	Y	Y	IX	
16.	ESF chiller room electric heaters	CB-313 & 320	CB-308 & 310	B	NA	6	2	E	UL	N	N		
17.	ESF chiller room electric heater motors	CB-313 & 320	CB-308 & 310	B	NA	6	2	E	NEMA MG1	N	N		
18.	Deleted												
19.	Duct silencers	CB-313 & 320	CB-305, 308, 310 & 123	B	NA	0	1	5	ASTM E-477	Y	Y		

TABLE 3.2.2-1 (SHEET 59 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
CONTROL ROOM HVAC SYSTEM (NORMAL PORTION)												Note ah
1. Return and exhaust fans	CB-403	shared	B	NA	6	2	6	AMCA	N	N		
2. Return and exhaust fan motors	CB-403	shared	B	NA	6	2	E	NEMA MG1	N	N		
3. Kitchen, toilet, and conference room fan	CB-325	shared	B	NA	6	2	6	AMCA	N	N		
4. Kitchen, toilet, and conference room fan motor	CB-325	shared	B	NA	6	2	E	NEMA MG1	N	N		
5. A/C fans	CB-403	shared	B	NA	6	2	6	AMCA	N	N		
6. A/C fan motors	CB-403	shared	B	NA	6	2	E	NEMA MG1	N	N		
7. Cooling coils	CB-403	shared	B	NA	6	2	6	ARI	N	N		
8. High efficiency and prefilters	CB-403	shared	B	NA	6	2	6	ANSI N509, ASHRAE	N	N		
9. Suction and discharge duct-work including isolation dampers			B	NA	0	1	5	ANSI N509	Y	Y		
10. All other ductwork and dampers			B	NA	6	1	6	ANSI N509	N	N		Note ae
11. Electric duct heaters	CB-220′	shared	B	NA	6	2	E	UL	N	N		
12. Instrumentation			B	NA	6	2	J	mfg	N	N		
13. Filter room electric heaters	CB-260′	CB-260′	B	NA	6	2	E	UL	N	N		
14. Filter room electric heater motors	CB-260′	CB-260′	B	NA	6	2	E	NEMA MG1	N	N		
15. Normal A/C equipment room exhaust fan	CB-260′	shared	B	NA	6	2	6	AMCA	N	N		
16. Normal A/C equipment room exhaust fan motor	CB-260′	shared	B	NA	6	2	E	NEMA MG1	N	N		

TABLE 3.2.2-1 (SHEET 60 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
17. A/C equipment room electric heater	CB-260′ & 280′	CB-260′ & 280′	B	NA	6	2	E	UL	N	N		
18. A/C equipment room electric heater motor	CB-260′ & 280′	CB-260′ & 280′	B	NA	6	2	E	NEMA MG1	N	N		
19. Normal chiller room exhaust fan	CB-320′	CB-308′	B	NA	6	2	6	AMCA	N	N		
20. Normal chiller exhaust fan motor	CB-320	CB-308	B	NA	6	2	E	NEMA MG1	N	N		
21. Normal chiller room electric heater	CB-320	CB-308	B	NA	6	2	E	UL	N	N		
22. Normal chiller room electric heater motor	CB-320	CB-308	B	NA	6	2	E	NEMA MG1	N	N		
SAFETY FEATURE ELECTRICAL EQUIPMENT ROOM HVAC SYSTEM												Note ah
1. Battery room exhaust fans	CB-180′	CB-180′	B	NA	0	1	5	AMCA	Y	Y	IX	
2. Battery room exhaust fan motors	CB-180′	CB-180′	B	NA	1	1	E	NEMA MG1	Y	Y	IX	
3. A/C unit fans	CB-B60 & B62	CB-B16 & B17	B	NA	0	1	5	AMCA	Y	Y	IX	
4. A/C unit fan motors	CB-B60 & B62	CB-B16 & B17	B	NA	1	1	E	NEMA MG1	Y	Y	IX	
5. Cooling coils	CB-B60 & B62	CB-B16 & B17	B	C	3	1	3	III-3	Y	Y	IX	
6. Prefilters	CB-B60 & B62	CB-B16 & B17	B	NA	0	1	5	ASHRAE	Y	Y	IX	
7. Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
8. Dampers			B	NA	0	1	5	ANSI N509	Y	Y		
9. Damper motors			B	NA	1	1	E	NEMA MG1	Y	Y		
10. Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
CONTROL BUILDING LEVELS A, B, 1, AND 2 NORMAL SYSTEM												Note ah
1. Wing area A/C fan	CB-B05	CB-B75	B	NA	6	2	6	AMCA	N	N		
2. Wing area A/C fan motor	CB-B05	CB-B75	B	NA	6	2	E	NEMA MG1	N	N		

TABLE 3.2.2-1 (SHEET 61 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
3. Switchgear A/C fan	CB-B40	CB-B38	B	NA	6	2	6	AMCA	N	N		
4. Switchgear A/C fan motor	CB-B40	CB-B38	B	NA	6	2	E	NEMA MG1	N	N		
5. Prefilters	CB-B05 & B40	CB-B75 & B38	B	NA	6	2	6	ASHRAE	N	N		
6. Cooling coils	CB-B05 & B40	CB-B75 & B38	B	NA	6	2	6	ARI	N	N		
7. Ductwork and dampers			B	NA	6	2	6	SMACNA/ ANSI N509	N	N		
8. Instrumentation			B	NA	6	2	J	mfg	N	N		
9. Wing area exhaust fan	CB-180′	CB-180′	B	NA	6	2	6	AMCA	N	N		
10. Wing area exhaust fan motor	CB-180′	CB-180′	B	NA	6	2	E	NEMA MG1	N	N		
11. Service area normal return and exhaust fan	CB-240′	CB-240′	B	NA	6	2	6	AMCA	N	N		
12. Service area normal return and exhaust fan motor	CB-240′	CB-240′	B	NA	6	2	E	NEMA MG1	N	N		
13. Service area normal A/C fan	CB-249′	shared	B	NA	6	2	6	AMCA	N	N		
14. Service area normal A/C fan motor	CB-249′	shared	B	NA	6	2	E	NEMA MG1	N	N		
15. Cooling coils	CB-249′	shared	B	NA	6	2	6	ARI	N	N		
16. Prefilter	CB-249′	shared	B	NA	6	2	6	ASHRAE	N	N		
17. Service area electric wall heater	CB-220′ CB-240′	CB-220′ CB-240′	B	NA	6	2	E	UL	N	N		
18. Service area electric duct heater	CB-240′	CB-240′	B	NA	6	2	E	UL	N	N		
19. Central alarm station standby fan	CB-240′	shared	B	NA	6	2	6	AMCA	N	N		
20. Central alarm station standby fan motor	CB-240′	shared	B	NA	6	2	E	NEMA MG1	N	N		
21. Cooling coils	CB-240′	shared	B	NA	6	2	6	ARI	N	N		
22. Prefilter	CB-240′	shared	B	NA	6	2	6	ASHRAE	N	N		

TABLE 3.2.2-1 (SHEET 62 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
23. Smoke exhaust fan	CB-280′	shared	B	NA	6	2	6	AMCA	N	N		
24. Smoke exhaust fan motor	CB-280′	shared	B	NA	6	2	E	NEMA MG1	N	N		
CONTROL BUILDING LAB HOOD VENT SYSTEM												Note ah
1. Fumehoods supply fan	CB-240′	shared	B	NA	6	2	6	AMCA	N	N		
2. Fumehoods supply fan motor	CB-240′	shared	B	NA	6	2	E	NEMA MG1	N	N		
3. Fumehood filter unit fan	CB-248′	shared	B	NA	6	2	6	AMCA	N	N		
4. Fumehood filter unit fan motor	CB-248	shared	B	NA	6	2	E	NEMA MG1	N	N		
5. Moisture eliminator	CB-248	shared	B	NA	6	2	6	ANSI N509	N	N		
6. Heater	CB-248	shared	B	NA	6	2	E	UL	N	N		
7. HEPA filters	CB-248	shared	B	NA	6	2	6	ANSI N509	N	N		
8. Charcoal filter	CB-248	shared	B	NA	6	2	6	ANSI N509	N	N		
9. Ductwork and dampers			B	NA	6	2	6	SMACNA/ ANSI N509	N	N		
10. Instrumentation			B	NA	6	2	J	mfg	N	N		
CONTROL BUILDING LOCKER AND TOILET EXHAUST SYSTEM												Note ah
1. Exhaust fan	CB-240′	shared	B	NA	6	2	6	AMCA	N	N		
2. Exhaust fan motor	CB-240′	shared	B	NA	6	2	E	NEMA MG1	N	N		
3. Ductwork			B	NA	6	2	6	SMACNA	N	N		
4. Instrumentation			B	NA	6	2	J	mfg	N	N		
CONTROL BUILDING CABLE SPREADING ROOM HVAC SYSTEM												Note ah
1. Cable spreading room A/C fans	CB-A32 CB-325	shared	B	NA	6	2	6	AMCA	N	N		
2. Cable spreading room A/C fan motors	CB-A32 CB-325	shared	B	NA	6	2	E	NEMA MG1	N	N		
3. Auxiliary relay room ESF A/C fans	CB-226	CB-223	B	NA	0	1	5	AMCA	Y	Y	IX	

TABLE 3.2.2-1 (SHEET 63 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
4. Auxiliary relay room ESF A/C fan motors	CB-226	CB-223	B	NA	1	1	E	NEMA MG1	Y	Y	IX	
5. Prefilters	CB-A32 CB-325′	shared	B	NA	6	2	6	ASHRAE	N	N		
6. Auxiliary relay room ESF A/C	CB-200′ CB-240′	CB-200′ CB-240′										
(cooling coils)			B	C	3	1	3	III-3	Y	Y	IX	
(ductwork) fan coolers			B	NA	0	1	5	mfg	Y	Y		
7. All other coolers	CB-A32 CB-325′	shared	B	NA	6	2	6	ARI	N	N		
8. Electric duct heaters			B	NA	6	2	E	UL	N	N		
9. All other ductwork			B	NA	6	2	6	SMACNA	N	N		
10. Auxiliary relay room ductwork and dampers			B	NA	0	1	5	ANSI N509	Y	Y		
11. Smoke exhaust fan	CB-325	shared	B	NA	6	2	6	AMCA	N	N		
12. Smoke exhaust fan motor	CB-325	shared	B	NA	6	2	E	NEMA MG1	N	N		
13. Computer room A/C fans	CB-A38	CB-A30	G	NA	6	2	6	AMCA	N	N		
14. Computer room A/C fan motors	CB-A38	CB-A30	G	NA	6	2	E	NEMA MG1	N	N		
15. Computer room A/C unit humidifier	CB-A38	CB-A30	G	NA	6	2	E	ANSI N509	N	N		
16. Computer room A/C unit cooling coils	CB-A38	CB-A30	G	NA	6	2	6	ARI	N	N		
17. Auxiliary relay room ESF A/C instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
18. All other instrumentation			B	NA	6	2	J	mfg	N	N		
19. Normal AC room ESF AC vent	CB-325	CB-325	B	NA	0	1	5		Y	Y		
Normal chilled water cooling coil	CB-325	-	B	NA	6	2	6	ARI	N	N		
ESF chilled water cooling coils	CB-325	CB-325	B	C	3	1	3	III-3	Y	Y		

TABLE 3.2.2-1 (SHEET 64 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
20. Electric equipment room ESF AC unit	CB-322	-	B	NA	0	1	5		Y	Y		
Normal chilled water cooling coil	CB-322	-	B	NA	6	2	6	ARI	N	N		
ESF chilled water cooling coil	CB-322	-	B	C	3	1	3	III-3	Y	Y		
ELECTRICAL TUNNEL VENTILATION SYSTEM												Note ah
1. Electrical tunnel supply fan (normal)	VB	VB	B	NA	6	2	6	AMCA	N	N		
2. Electrical tunnel supply fan motor	VB	VB	B	NA	6	2	E	NEMA MG1	N	N		
3. Turbine building chase to control building tunnel fan	CB-180′	CB-180′	B	NA	6	2	6	AMCA	N	N		
4. Turbine building chase to control building tunnel fan motor	CB-180′	CB-180′	B	NA	6	2	E	NEMA MG1	N	N		
5. Turbine and auxiliary building tunnel supply fan	AB-260′	AB-260′	B	NA	0	1	5	AMCA	Y	Y	VIII	
6. Turbine and auxiliary building tunnel supply fan motor	AB-260′	AB-260′	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
7. Diesel power cable tunnel exhaust fans	VB	VB	B	NA	0	1	5	AMCA	Y	Y	III	
8. Diesel power cable tunnel exhaust fan motors	VB	VB	B	NA	1	1	E	NEMA MG1	Y	Y	III	

TABLE 3.2.2-1 (SHEET 65 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
9. NSCW tower cable tunnel fans	VB	VB	B	NA	0	1	5	AMCA	Y	Y	III	
10. NSCW tower cable tunnel fan motors	VB	VB	B	NA	1	1	E	NEMA MG1	Y	Y	III	
11. Prefilters	VB	VB	B	NA	6	2	6	ASHRAE	N	N		
12. Turbine building chase and electrical tunnel ductwork			B	NA	6	2	6	SMACNA	N	N		
13. All other ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
14. Turbine building chase and electrical tunnel instrumentation			B	NA	6	2	J	mfg	N	N		
15. All other instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
FUEL HANDLING BUILDING HVAC SYSTEM												Note ah
1. Fuel pool recirculating air handling unit fans	FB-220′	FB-220′	B	NA	6	2	6	AMCA	N	N		
2. Fuel pool recirculating fan motors	FB-220′	FB-220′	B	NA	6	2	E	NEMA MG1	N	N		
3. Recirculating fan coolers	FB-220′	FB-220′	B	NA	6	2	6	ARI	N	N		
4. Fuel pool electric duct fan heaters	FB-220′	FB-220′	B	NA	6	2	E	UL	N	N		
5. Railroad corridor recirculating fan	AB-127′	shared	B	NA	6	2	6	AMCA	N	N		
6. Railroad corridor recirculating fan motor	AB-127	shared	B	NA	6	2	E	NEMA MG1	N	N		
7. Railroad corridor recirculating fan cooler	AB-127	shared	B	NA	6	2	6	ARI	N	N		

TABLE 3.2.2-1 (SHEET 66 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
8. Railroad corridor infrared heaters	AB-127	shared	B	NA	6	2	E	UL	N	N		
9. FHB normal A/C fan	CB-403	shared	B	NA	6	2	6	AMCA	N	N		
10. FHB normal A/C fan motor	CB-403	shared	B	NA	6	2	E	NEMA MG1	N	N		
11. FHB normal A/C prefilters	CB-403	shared	B	NA	6	2	6	ASHRAE	N	N		
12. FHB normal A/C heaters	CB-403	shared	B	NA	6	2	E	UL	N	N		
13. FHB normal A/C coolers	CB-403	shared	B	NA	6	2	6	ARI	N	N		
14. FHB exhaust A/C fan	FB-301	shared	B	NA	6	2	6	AMCA	N	N		
15. FHB exhaust A/C fan motor	FB-301	shared	B	NA	6	2	E	NEMA MG1	N	N		
16. FHB exhaust A/C HEPA filters	FB-301	shared	B	NA	6	2	6	ANSI N509	N	N		
17. FHB exhaust A/C heaters	FB-301	shared	B	NA	6	2	E	UL	N	N		
18. FHB exhaust A/C moisture eliminators	FB-301	shared	B	NA	6	2	6	ANSI N509	N	N		
19. FHB exhaust A/C charcoal filters	FB-301	shared	B	NA	6	2	6	ANSI N509	N	N		
20. HEPA filters	FB-301	shared	B	NA	6	2	6	ANSI N509	N	N		
21. Process duct-work and dampers			B	NA	6	2	6	SMACNA/ ANSI N509	N	N		
22. Damper motors			B	NA	6	2	E	NEMA MG1	N	N		
23. Negative pres-sure boundary penetration ductwork and dampers			B	NA	0	1	5	ANSI N509	Y	Y		
24. Instrumentation			B	NA	6	2	J	mfg	N	N		
25. FHB normal equipment reheat coil	FB-301	shared	B	NA	6	2	E	UL	N	N		

TABLE 3.2.2-1 (SHEET 67 OF 98)

	Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
26.	FHB accident equipment reheat coil	FB-260′	shared	B	NA	6	2	E	UL	N	N		
FUEL HANDLING BUILDING POST-ACCIDENT EXHAUST SYSTEM													Note ah
1.	Fans	FB-303, 304	shared	B	NA	0	1	5	AMCA	Y	Y	VII	
2.	Fan motors	FB-303, 304	shared	B	NA	1	1	E	NEMA MG1	Y	Y	VII	
3.	Moisture eliminators	FB-303, 304	shared	B	NA	0	1	5	ANSI N509	Y	Y	VII	
4.	Heaters	FB-303, 304	shared	B	NA	1	1	E	UL, ANSI N509	Y	Y	VII	
5.	HEPA filters	FB-303, 304	shared	B	NA	0	1	5	ANSI N509	Y	Y	VII	
6.	Charcoal filters	FB-303, 304	shared	B	NA	0	1	5	ANSI N509	Y	Y	VII	
7.	Ductwork			B	NA	0	1	5	ANSI N509	Y	Y	VII	
8.	Dampers			B	NA	0	1	5	ANSI N509	Y	Y	VII	
9.	Dampers motors			B	NA	1	1	E	NEMA MG1	Y	Y	VII	
10.	Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y	VII	Note s
AUXILIARY BUILDING OUTSIDE AIR SUPPLY, NORMAL HVAC, RADIOACTIVE FILTER EXHAUST, AND CONTINUOUS EXHAUST SYSTEMS													Note ah
1.	Exhaust unit fans	AB-212	AB-221	B	NA	6	2	6	AMCA	N	N		
2.	Exhaust unit fan motors	AB-212	AB-221	B	NA	6	2	E	NEMA MG1	N	N		
3.	A/C fans	AB-212	AB-221	B	NA	6	2	6	AMCA	N	N		
4.	A/C fan motors	AB-212	AB-221	B	NA	6	2	E	NEMA MG1	N	N		
5.	HEPA filters	AB-212	AB-221	B	NA	6	2	6	ANSI N509	N	N		
6.	Charcoal filters	AB-212	AB-221	B	NA	6	2	6	ANSI N509	N	N		
7.	Moisture eliminators	AB-212	AB-221	B	NA	6	2	6	ANSI N509	N	N		
8.	Electric heaters	AB-212	AB-221	B	NA	6	2	E	UL	N	N		
9.	Cooling coils	AB-212	AB-221	B	NA	6	2	6	ARI	N	N		
10.	Ductwork			B	NA	6	2	6	SMACNA	N	N		
11.	Dampers			B	NA	6	2	6	ANSI N509	N	N		
12.	Damper motors			B	NA	6	2	E	NEMA MG1	N	N		
13.	Instrumentation			B	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 68 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
AUXILIARY BUILDING ESF ROOM COOLERS												Note ah
1. Electrical, switchgear, and MCC room cooler A, level D:												
Fan	AB-D79	AB-D02	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-D79	AB-D02	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-D79	AB-D02	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-D79	AB-D02	B	C	3	1	3	III-3	Y	Y	VIII	
Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
2. Electrical, switchgear, and MCC room cooler B, level 2:												
Fan	AB-212	AB-221	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-212	AB-221	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-212	AB-221	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-212	AB-221	B	C	3	1	3	III-3	Y	Y	VIII	
Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
3. Electrical, switchgear, and MCC room cooler A, level C:												
Fan	AB-B13	AB-B123	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-B13	AB-B123	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-B13	AB-B123	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-B13	AB-B123	B	C	3	1	3	III-3	Y	Y	VIII	
Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
4. Electrical, switchgear, and MCC room cooler B, level B:												
Fan	AB-B16	AB-B122	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-B16	AB-B122	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	

TABLE 3.2.2-1 (SHEET 69 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
Normal chilled water cooling coils	AB-B16	AB-B122	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-B16	AB-B122	B	C	3	1	3	III-3	Y	Y	VIII	
Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
5. Electrical, switchgear, and MCC room cooler A, level 1:												
Fan	AB-118	AB-149	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-118	AB-149	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-118	AB-149	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-118	AB-149	B	C	3	1	3	III-3	Y	Y	VIII	
Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
6. Electrical, switchgear, and MCC room cooler B, level 1:												
Fan	AB-116	AB-147	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-116	AB-147	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-116	AB-147	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-116	AB-147	B	C	3	1	3	III-3	Y	Y	VIII	
Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
7. RHR pump room cooler A:												
Fan	AB-D122	AB-D112	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-D122	AB-D112	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-D122	AB-D112	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-D122	AB-D112	B	C	3	1	3	III-3	Y	Y	VIII	
Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		

TABLE 3.2.2-1 (SHEET 70 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
8. RHR pump room cooler B:												
Fan	AB-D122	AB-D112	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-D122	AB-D112	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-D122	AB-D112	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-D122	AB-D112	B	C	3	1	3	III-3	Y	Y	VIII	
Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
9. Containment spray pump room cooler A:												
Fan	AB-D79	AB-D02	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-D79	AB-D02	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
ESF cooling coils	AB-D79	AB-D02	B	C	3	1	3	III-3	Y	Y	VIII	
Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
10. Containment spray pump room cooler B:												
Fan	AB-D77	AB-D04	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-D77	AB-D04	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
ESF cooling coils	AB-D77	AB-D04	B	C	3	1	3	III-3	Y	Y	VIII	
11. CCW pump room cooler A:												
Fan	AB-A05	AB-A98	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-A05	AB-A98	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
ESF cooling coils	AB-A05	AB-A98	B	C	3	1	3	III-3	Y	Y	VIII	
12. CCW pump room cooler B:												
Fan	AB-A03	AB-A96	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-A03	AB-A96	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
ESF cooling coils	AB-A03	AB-A96	B	C	3	1	3	III-3	Y	Y	VIII	

TABLE 3.2.2-1 (SHEET 71 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
13. Charging pump room cooler A:												
Fan	AB-C115	AB-C16	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-C115	AB-C16	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-C115	AB-C16	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-C115	AB-C16	B	C	3	1	3	III-3	Y	Y	VIII	
14. Charging pump room cooler B:												
Fan	AB-C118	AB-C17	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-C118	AB-C17	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-C118	AB-C17	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-C118	AB-C17	B	C	3	1	3	III-3	Y	Y	VIII	
15. SI pump room cooler A:												
Fan	AB-B15	AB-B119	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-B15	AB-B119	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
ESF cooling coils	AB-B15	AB-B119	B	C	3	1	3	III-3	Y	Y	VIII	
16. SI pump room cooler B:												
Fan	AB-B19	AB-B117	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-B19	AB-B117	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
ESF cooling coils	AB-B19	AB-B117	B	C	3	1	3	III-3	Y	Y	VIII	
17. SFP pump and HX room cooler A:												
Fan	AB-A53	AB-A91	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	AB-A53	AB-A91	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	AB-A53	AB-A91	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	AB-A53	AB-A91	B	C	3	1	3	III-3	Y	Y	VIII	

TABLE 3.2.2-1 (SHEET 72 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
18. SFP pump and HX room cooler B:												
Fan	FB-A07	FB-A04	B	NA	0	1	5	AMCA	Y	Y	VIII	
Fan motor	FB-A07	FB-A04	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
Normal chilled water cooling coils	FB-A07	FB-A04	B	C	3	1	3	III-3	Y	Y	VIII	
ESF chilled water cooling coils	FB-A07	FB-A04	B	C	3	1	3	III-3	Y	Y	VIII	
19. Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
PIPING PENETRATION AND MSIV VENTILATION SYSTEM												Note ah
1. Restraint cooling fans	VB	VB	B	NA	6	2	6	AMCA	N	N		
2. Restraint cooling fan motors	VB	VB	B	NA	6	2	E	NEMA MG1	N	N		Note m
3. Restraint cooling ductwork			B	NA	6	2	6	SMACNA	N	N		
4. Restraint cooling backdraft dampers			B	NA	6	2	6	ANSI N509	N	N		
5. Restraint cooling instrumentation			B	NA	6	2	J	mfg	N	N		
6. MSIV AHU fans	VB	VB	S	NA	6	2	6	AMCA	N	N		
7. MSIV AHU fan motors	VB	VB	S	NA	6	2	E	NEMA MG1	N	N		
8. MSIV ductwork			S	NA	6	2	6	SMACNA	N	N		
9. MSIV instrumentation		S	NA	6	2	J	mfg	N	N			
10. MSIV ventilation instrumentation			S	NA	6	2	J	mfg	N	N		
PIPING PENETRATION FILTER EXHAUST SYSTEM												Note ah
1. Fans	AB-209 & 220	AB-219 & 220	B	NA	0	1	5	AMCA	Y	Y	VIII	
2. Fan motors	AB-209 & 220	AB-219 & 220	B	NA	1	1	E	NEMA MG1	Y	Y	VIII	
3. Moisture eliminators	AB-209 & 220	AB-219 & 220	B	NA	0	1	5	ANSI N509	Y	Y	VIII	
4. Electrical heaters	AB-209 & 220	AB-219 & 220	B	NA	1	1	E	UL, ANSI N509	Y	Y	VIII	

TABLE 3.2.2-1 (SHEET 73 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
5. Infrared heaters	AB-209 & 220	AB-219 & 220	B	NA	6	2	E	UL	N	N		
6. HEPA filters	AB-209 & 220	AB-219 & 220	B	NA	0	1	5	ANSI N509	Y	Y	VIII	
7. Charcoal filters	AB-209 & 220	AB-219 & 220	B	NA	0	1	5	ANSI N509	Y	Y	VIII	
8. Dampers			B	NA	0	1	5	ANSI N509	Y	Y		
9. Damper motors			B	NA	1	1	E	NEMA MG1	Y	Y		
10. Ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
11. Area coolers	AB-209 & 220	AB-219 & 220	B	C	3	1	3	III-3	Y	Y	VIII	
12. Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
DIESEL GENERATOR BUILDING HVAC SYSTEM												Note ah
1. ESF supply fans	DB-270′	DB-270′	B	NA	0	1	5	AMCA	Y	Y	IV	
2. ESF supply fan motors	DB-270′	DB-270′	B	NA	1	1	E	NEMA MG1	Y	Y	IV	
3. Non-ESF exhaust fans	DB-255′	DB-255′	B	NA	6	2	6	AMCA	N	N		
4. Non-ESF exhaust fan motors	DB-255′	DB-255′	B	NA	6	2	E	NEMA MG1	N	N		
5. Building unit heaters	DB-234′	DB-234′	B	NA	6	2	E	UL	N	N		

TABLE 3.2.2-1 (SHEET 74 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
6. ESF ductwork			B	NA	0	1	5	ANSI N509	Y	Y		
7. ESF dampers			B	NA	0	1	5	ANSI N509	Y	Y		
8. Non-ESF ductwork			B	NA	6	2	6	SMACNA	N	N		
9. ESF damper motors			B	NA	1	1	E	NEMA MG1	Y	Y		
10. ESF instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
11. Non-ESF instrumentation			B	NA	6	2	J	mfg	N	N		
TURBINE BUILDING CONDENSER VACUUM EXHAUST FILTRATION SYSTEM												Note ah
1. HEPA filters	TB-245′	TB-245′	S	NA	6	2	6	ANSI N509	N	N		
2. Charcoal filter	TB-245′	TB-245′	S	NA	6	2	6	ANSI N509	N	N		
3. Demister	TB-245′	TB-245′	S	NA	6	2	6	ANSI N509	N	N		
4. Heater	TB-245′	TB-245′	S	NA	6	2	E	UL	N	N		
5. Piping			S	D	4	2	4	B31.1	N	N		
6. Dampers			S	NA	6	2	6	ANSI N509	N	N		
7. Instrumentation			S	NA	6	2	J	mfg	N	N		
TURBINE BUILDING HVAC SYSTEM												Note ah
1. Turbine building supply fans	TB-220′	TB-220′	S	NA	6	2	6	AMCA	N	N		
2. Turbine building supply fan motors	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
3. Toilet exhaust fans	TB-220′	TB-220′	S	NA	6	2	6	AMCA	N	N		
4. Toilet exhaust fan motors	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
5. Water analysis A/C fan	TB-220′	TB-220′	S	NA	6	2	6	AMCA	N	N		
6. Water analysis A/C fan motor	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
7. Battery room A/C fan	TB-220′	TB-220′	S	NA	6	2	6	AMCA	N	N		
8. Battery room A/C fan motor	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
9. Switchgear room level 1 A/C fan	TB-220′	TB-220′	S	NA	6	2	6	AMCA	N	N		
10. Switchgear room level 1 A/C fan motor	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
11. Switchgear room level 2 A/C fan	TB-245′	TB-245′	S	NA	6	2	6	AMCA	N	N		

TABLE 3.2.2-1 (SHEET 75 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
12. Switchgear room level 2 A/C fan motor	TB-245′	TB-245′	S	NA	6	2	E	NEMA MG1	N	N		
13. Stairwell exhaust fans	TB-270′	TB-270′	S	NA	6	2	6	AMCA	N	N		
14. Stairwell exhaust fan motors	TB-270′	TB-270′	S	NA	6	2	E	NEMA MG1	N	N		
15. Turbine building exhaust fans	TB-270′	TB-270′	S	NA	6	2	6	AMCA	N	N		
16. Turbine building exhaust fan motors	TB-270′	TB-270′	S	NA	6	2	E	NEMA MG1	N	N		
17. Prefilters	TB	TB	S	NA	6	2	6	ASHRAE	N	N		
18. Cooling coils	TB	TB	S	NA	6	2	6	ARI	N	N		
19. Heaters	TB	TB	S	NA	6	2	E	UL	N	N		
20. Heater fans	TB	TB	S	NA	6	2	6	AMCA	N	N		
21. Ductwork and dampers			S	NA	6	2	6	SMACNA	N	N		
22. Damper motors			S	NA	6	2	E	NEMA MG1	N	N		
23. Instrumentation			S	NA	6	2	J	mfg	N	N		
24. Battery room exhaust fans	TB-220′	TB-220′	S	NA	6	2	6	AMCA	N	N		
25. Battery room exhaust fan motors	TB-220′	TB-220′	S	NA	6	2	E	NEMA MG1	N	N		
TECHNICAL SUPPORT CENTER HVAC SYSTEM												Note ah
1. TSC filter fan	TSC	shared	B	NA	6	2	6	AMCA	N	N		
2. TSC filter fan motor	TSC	shared	B	NA	6	2	E	NEMA MG1	N	N		
3. HEPA filters	TSC	shared	B	NA	6	2	6	ANSI N509	N	N		
4. Charcoal filter	TSC	shared	B	NA	6	2	6	ANSI N509	N	N		
5. Prefilter	TSC	shared	B	NA	6	2	6	ASHRAE	N	N		
6. Filter unit heater	TSC	shared	B	NA	6	2	E	UL	N	N		
7. TSC A/C fan	TSC	shared	B	NA	6	2	6	AMCA	N	N		
8. TSC A/C fan motor	TSC	shared	B	NA	6	2	E	NEMA MG1	N	N		
9. Moisture eliminator	TSC	shared	B	NA	6	2	6	ANSI N509	N	N		
10. TSC toilet exhaust fan	TSC	shared	B	NA	6	2	6	AMCA	N	N		

TABLE 3.2.2-1 (SHEET 76 OF 98)

	Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
11.	TSC toilet exhaust fan motor	TSC	shared	B	NA	6	2	E	NEMA MG1	N	N		
12.	TSC battery room exhaust fan	TSC	shared	B	NA	6	2	6	AMCA	N	N		
13.	TSC battery room exhaust fan motor	TSC	shared	B	NA	6	2	E	NEMA MG1	N	N		
14.	Electric duct heaters	TSC	shared	B	NA	6	2	E	UL	N	N		
15.	Ductwork	TSC	shared	B	NA	6	2	6	SMACNA	N	N		
16.	Dampers	TSC	shared	B	NA	6	2	6	ANSI N509	N	N		
17.	Damper operators	TSC	shared	B	NA	6	2	E	mfg	N	N		
18.	Instrumentation	TSC	shared	B	NA	6	2	J	mfg	N	N		
	FIRE PUMP AND WELL PUMPHOUSE HVAC SYSTEM												Note ah
1.	Exhaust fans	VB	shared	S	NA	6	2	6	AMCA	N	N		
2.	Exhaust fan motors	VB	shared	S	NA	6	2	E	NEMA MG1	N	N		
3.	Heaters	VB	shared	S	NA	6	2	E	UL	N	N		
4.	Instrumentation	VB	shared	S	NA	6	2	J	mfg	N	N		
	NORMAL CHILLED WATER SYSTEM												
1.	Expansion tank	CB-410	shared	B	D	4	2	4	VIII	N	N		
2.	Chemical feed pot	CB-410	shared	B	D	4	2	4	VIII	N	N		
3.	Air separator/ strainer	CB-410	shared	B	D	4	2	4	VIII	N	N		
4.	Chilled water pumps	CB-410	shared	B	D	4	2	4	mfg	N	N		
5.	Chilled water pump motors	CB-410	shared	B	NA	6	2	E	NEMA MG1	N	N		
6.	Chillers	CB-410	shared	B	NA	6	2	6	VIII	N	N		
7.	Piping and valves in safety-related equipment room			B	D	4	2	4	B31.1	N	N		
8.	Other piping and valves			B	D	4	2	4	B31.1	N	N		
9.	Instrumentation			B	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 77 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
ESSENTIAL CHILLED WATER SYSTEM												
1. Expansion tanks	CB-320 & 313	CB-310 & 308	B	C	3	1	3	III-3	Y	Y	IX	
2. Chemical feed pots	CB-320 & 313	CB-320 & 313	B	NA	4	2	4	VIII	N	N		
3. ESF chilled water pumps	CB-320 & 313	CB-310 & 308	B	C	3	1	3	III-3	Y	Y	IX	
4. ESF chilled water pump motors	CB-320 & 313	CB-310 & 308	B	NA	1	1	E	NEMA MG1	Y	Y	IX	
5. Valves, piping, and cooling coils			B	C	3	1	3	III-3	Y	Y		
6. Chillers	CB-320 & 313	CB-310 &308									IX	
(tube side)				C	3	1	3	III-3	Y	Y		
(shell side)				NA	0	1	5	VIII	Y	Y		
7. Safety-related instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
SPECIAL CHILLED WATER SYSTEMS												
1. TSC chilled water pump			B	NA	6	2	6	mfg	N	N		
2. TSC chilled water pump motor			B	NA	6	2	E	NEMA MG1	N	N		
3. TSC chilled water expansion tank			B	D	4	2	4	VIII	N	N		
4. TSC chilled water air separator/strainer			B	D	4	2	4	B31.1	N	N		
5. TSC chilled water chemical feed pot			B	D	4	2	4	VIII	N	N		
6. TSC chillers			B	NA	6	2	6	UL	N	N		
7. TSC chilled water buffer tank			B	D	4	2	4	VIII	N	N		
8. CAS chilled water pump			B	NA	6	2	6	mfg	N	N		
9. CAS chilled water pump motor			B	NA	6	2	E	NEMA MG1	N	N		
10. CAS chilled water expansion tank			B	D	4	2	4	VIII	N	N		
11. CAS chilled water air separator strainer			B	D	4	2	4	B31.1	N	N		

TABLE 3.2.2-1 (SHEET 78 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
12. CAS chilled water chemical feed pot			B	D	4	2	4	VIII	N	N		
13. CAS chillers			B	NA	6	2	6	VIII	N	N		
14. Piping and valves			B	D	4	2	4	B31.1	N	N		
15. Instrumentation			B	NA	6	2	J	mfg	N	N		
AUXILIARY FEEDWATER PUMPHOUSE HVAC SYSTEM												Note ah
1. ESF supply fan	AFP	AFP	B	NA	0	1	5	AMCA	Y	Y	VI	
2. ESF supply fan motors	AFP	AFP	B	NA	1	1	E	NEMA MG1	Y	Y	VI	
3. Non-ESF supply fan	AFP	AFP	B	NA	6	2	6	AMCA	N	N		
4. Non-ESF supply fan motor	AFP	AFP	B	NA	6	2	E	NEMA MG1	N	N		
5. Building unit heaters	AFP	AFP	B	NA	6	2	E	UL	N	N		
6. ESF Dampers			B	NA	0	1	5	ANSI N509	Y	Y		
7. ESF instrumentation			B	NA	1	1	J	mfg	Y	Y		Note s
8. Non-ESF instrumentation			B	NA	6	2	J	mfg	N	N		
MISCELLANEOUS HVAC SYSTEMS												Note ah
1. Mechanical components	VB	VB	B	NA	6	2	6	mfg	N	N		
2. Electrical equipment	VB	VB	B	NA	6	2	E	NEMA MG1	N	N		
3. Instrumentation			B	NA	6	2	J	mfg	N	N		
MAIN CONTROL BOARD												Notes l, p and s
1. PAMS instrumentation			W	NA	1	1	J	mfg	Y	Y		
2. Hand switches and controls for safety-related equipment			W	NA	1	1	J	mfg	Y	Y		
3. All other instruments and controls			W	NA	6	1	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 79 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
NUCLEAR INSTRUMENTATION SYSTEM											
1. All instruments inputting to reactor protection system		W	NA	1	1	J	mfg	Y	Y		
PROCESS CONTROL SYSTEM											
1. NSSS BOP safety-related instrumentation and controls		W,B	NA	1	1	J	mfg	Y	Y		
2. NSSS BOP nonsafety-related instrumentation and controls		W,B	NA	6	1	J	mfg	N	N		
PROTECTION SYSTEM NSS											
1. Protection instrumentation and controls		W	NA	1	1	J	mfg	Y	Y		Note s
ROD CONTROL POWER SYSTEM											
1. Reactor trip switchgear		W	NA	1	1	E	mfg	Y	Y		
2. Other switchgear		W	NA	6	2	E	mfg	N	N		
FULL LENGTH ROD CONTROL SYSTEM											
1. Rod control equipment		W	NA	6	1	J	mfg	N	N		
ROD POSITION INDICATION SYSTEM											
1. Rod position instrumentation		W	NA	6	1	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 80 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
RADIATION MONITORING SYSTEM											
1. Safety-related portions		W	NA	1	1	J	mfg	Y	Y		
2. Nonsafety-related, seismic Category 1 portions		W	NA	6	1	J	mfg	N	N		
3. Other portions		W	NA	6	2	J	mfg	N	N		
ESF ACTUATION SYSTEM											
1. All portions		W	NA	1	1	J	mfg	Y	Y		
REACTOR INSTRUMENTATION											
1. All portions inputting to reactor protection		W	NA	1	1	J	mfg	Y	Y		
2. Other portions		W	NA	6	2	J	mfg	N	N		
REACTOR CONTROL SYSTEM											
1. Protection-related portions		W	NA	1	1	J	mfg	Y	Y		
2. Other portions		W	NA	6	2	J	mfg	N	N		
POST-ACCIDENT MONITORING SYSTEM											Note p
1. Safety-related portions		W,B	NA	1	1	J	mfg	Y	Y		
2. Nonsafety-related, seismic Category 1 portions		W,B	NA	6	1	J	mfg	N	N		
3. Other portions		W,B	NA	6	2	J	mfg	N	N		
PLANT AUXILIARY CONTROL BOARDS											
1. Safety-related portions		W,B	NA	1	1	J	mfg	Y	Y		Note s
2. Nonsafety-related portions		W,B	NA	6	2	J	mfg	N	N		

TABLE 3.2.2-1 (SHEET 81 OF 98)

Principal System and Components	(a) Location Unit 1	(a) Location Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
SAFETY-RELATED SYSTEMS BYPASS/INOPERABLE STATUS AND TRIP/MONITORING INDICATING LIGHTS												
1. Trip monitoring lights			W	NA	6	1	J	mfg	N	N		
2. All other portions			W,B	NA	1	1	J	mfg	Y	Y		
INCORE INSTRUMENTATION												
1. All portions			W	NA	6	1	J	mfg	N	N		
TURBINE PROTECTION SYSTEM												
1. All portions			S	NA	6	2	J	mfg	N	N		
TURBINE SUPERVISORY INSTRUMENTATION												
1. All portions			S	NA	6	2	J	mfg	N	N		
ELECTROHYDRAULIC CONTROL SYSTEM												
1. DEHC Mark Vle			SNC	NA	6	2	J	mfg	N	N		
2. All other I & C portions			S	NA	6	2	J	mfg	N	N		
3. Hydraulic fluid power units	TB-220′	TB-220′	S	NA	6	2	6	mfg	N	N		
4. Control coolers	TB-220′	TB-220′	S	NA	6	2	6	mfg	N	N		
5. Piping & valves			S	NA	6	2	6	B31.1	N	N		
AMSAC												
1. Nonsafety-related portion			W	NA	6	1	J	mfg	N	N		
2. Safety-related portion			W	NA	1	1	J	mfg	Y	Y		
COMPUTER SYSTEM												
1. All portions			S	NA	6	2	J	mfg	N	N		
ANNUNCIATOR SYSTEM												
1. All portions			W	NA	6	1	J	mfg	N	N		
TELEPHONE PAGE SYSTEM												
1. All portions			B	NA	6	2	E	mfg	N	N		

TABLE 3.2.2-1 (SHEET 82 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
PABX SYSTEM											
1. All portions		G	NA	6	2	E	mfg	N	N		
SOUND-POWERED SYSTEM											
1. Maintenance and refueling		B	NA	6	2	E	mfg	N	N		
2. Shutdown		B	NA	6	2	E	mfg	N	N		Note v
SEISMIC MONITORING EQUIPMENT											
1. “Free Field” accelerograph near river intake structure		B	NA	6	2	J	mfg	N	N		Note al
2. All other portions		B	NA	6	1	J	mfg	N	N		Note ax
PLANT SECURITY SYSTEM											
1. Fencing		B	NA	6	2	6	mfg	N	N		
2. Instrumentation		B	NA	6	2	J	mfg	N	N		
3. Electrical equipment		B	NA	6	2	E	mfg	N	N		
OFFSITE POWER SYSTEM											Note n
1. All portions		S	NA	6	2	E	mfg	N	N		
ac SYSTEM - 480 V (Class 1E portion)											
1. 4160/480 V transformers		B	NA	1	1	E	mfg	Y	Y		
2. Load centers		B	NA	1	1	E	mfg	Y	Y		
3. Motor control centers		B	NA	1	1	E	mfg	Y	Y		
4. Instrumentation and control		B	NA	1	1	E	mfg	Y	Y		
ac SYSTEM, 480 V (Non-class 1E portions)											
1. 4160/480V transformers		B	NA	6	2	E	mfg	N	N		
2. Load centers		B	NA	6	2	E	mfg	N	N		
3. Motor control centers		B	NA	6	2	E	mfg	N	N		
4. Instrumentation and control		B	NA	6	2	E	mfg	N	N		

TABLE 3.2.2-1 (SHEET 83 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
ac SYSTEM - 4160 V (Class 1E portions)											
1. 4.16 kV buses and switchgear		B	NA	1	1	E	mfg	Y	Y		
2. Instrumentation and controls		B	NA	1	1	E	mfg	Y	Y		
ac SYSTEM - 4160 V (Non-class 1E portion)											
1. 4.16 kV buses and switchgear		B	NA	6	2	E	mfg	N	N		
2. Instrumentation and control		B	NA	6	2	E	mfg	N	N		
dc SYSTEM - CLASS 1E											
1. Batteries		B	NA	1	1	E	mfg	Y	Y		
2. Chargers		B	NA	1	1	E	mfg	Y	Y		
3. Breakers, buswork, and switchgear		B	NA	1	1	E	mfg	Y	Y		
4. Instrumentation and controls		B	NA	1	1	E	mfg	Y	Y		
5. Motor control center		B	NA	1	1	E	mfg	Y	Y		
6. Distribution panels		B	NA	1	1	E	mfg	Y	Y		
120-V ac POWER SYSTEM - CLASS 1E											
1. Transformers		B	NA	1	1	E	mfg	Y	Y		
2.		(This line has been intentionally left blank.)									
3. dc-ac inverters		B,W	NA	1	1	E	mfg	Y	Y		
4. Instrumentation and control		B	NA	1	1	E	mfg	Y	Y		Note s
5. Distribution panels		B	NA	1	1	E	mfg	Y	Y		
LIGHTING SYSTEM											
1. Emergency lighting (control room, shutdown, diesel, and auxiliary feedwater panels including access and egress route to these areas)		B	NA	6	1	E	Mfg	N	N		Note o, v

TABLE 3.2.2-1 (SHEET 84 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
2. Other emergency lighting		B	NA	6	2	E	mfg	N	N		Note v
3. Essential lighting		B	NA	6	2	E	mfg	N	N		
4. Other lighting		B	NA	6	2	E	mfg	N	N		
5. Lighting isolation transformers		B	NA	1	1	E	mfg	Y	Y		
CABLE SYSTEM											
1. Safety-related power, control, and instrument cables		B	NA	1	2	E	mfg	Y	Y		
2. Nonsafety-related portions		B	NA	6	2	E	mfg	N	N		
FIRE DETECTION SYSTEM											Note v
1. Detector and alarm panels		B	NA	6	2	E	mfg	N	N		
2. Signaling systems		B	NA	6	2	E	mfg	N	N		
3. Local zone indicating panel		B	NA	6	2	E	mfg	N	N		
4. Local display cabinets		B	NA	6	2	E	mfg	N	N		
HEAT TRACING SYSTEMS											
1. Boric acid injection heat tracing, sensors and controls		B	NA	6	2	E	mfg	N	N		
2. Other heat tracing, sensors, and controls		B,S	NA	6	2	E	mfg	N	N		
ELECTRICAL PENETRATION SYSTEM											
1. Penetration assemblies		B	NA	1	1	E	IEEE-317, III-MC	Y	Y		

TABLE 3.2.2-1 (SHEET 85 OF 98)

Principal System and Components	(a) Location Unit 1 Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
STANDBY POWER SYSTEM											
1. Diesel generator package		B	NA	1	1	E	DEMA	Y	Y		
2. Instrumentation and controls		B	NA	1	1	E	mfg	Y	Y		Note s
3. Diesel generator jacket water chemical addition system		S	NA	6	2	6	mfg	N	N		
ac SYSTEM - 25 kV											
1. All portions		B	NA	6	2	E	mfg	N	N		
dc SYSTEM - NON-CLASS 1E											
1. All portions		B	NA	6	2	E	mfg	N	N		
120-V ac POWER SYSTEM - NON-CLASS 1E											
1. All portions		B,S	NA	6	2	E	mfg	N	N		
SWITCHYARD INTERFACES											
1. All portions		B,S	NA	6	2	E	mfg	N	N		
MULTISYSTEM PANELS AND BOARDS											
1. Safety-related portions		B	NA	1	1	E	mfg	Y	Y		
2. Other portions		B,S	NA	6	1,2	E	mfg	N	N		
ISO-PHASE BUS SYSTEM											
1. All portions		S	NA	6	2	E	mfg	N	N		
ac SYSTEM - 13.8 kV											
1. 13.8-kV buses and switchgear		B,S	NA	6	2	E	mfg	N	N		
2. 13.8-kV RCP 1E breakers		B	NA	1	1	E	mfg	Y	Y		

TABLE 3.2.2-1 (SHEET 86 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
HIGH VOLTAGE SWITCHYARD												
1. All portions			S	NA	6	2	E	mfg	N	N		
STRUCTURES AND BUILDINGS												
1. Containment building			B	NA	0	1	C	ASME III Div. 2, CC-3000	Y	Y		Code identified is for design only. See section 3.8.1 for details. Material, fabrication, and erection only.
2. Equipment hatch and personnel locks	C	C	B	NA	0	1	C	III-MC	Y	Y		
3. Liner plate system	C	C	B	NA	0	1	C	ASME III	Y	Y		
4. Penetration assemblies	C	C	B	NA	0	1	C	ASME III Div. 1,	Y	Y		
5. Fuel transfer tube housing expansion bellows and supports	C,FB	C,FB	B	NA	0	1	C	III-MC, NF	Y	Y		
6. Equipment building			B	NA	6	2	C	AISC-69, ACI 318-71	N	N		
7. NSCW cooling towers			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
8. Diesel generator building			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
9. Auxiliary building			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
10. Fuel handling building			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
11. Control building			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
12. Refueling water storage tank and dike	O	O	B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
13. Condensate storage tank and dike	O	O	B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
14. Diesel fuel oil storage tank pumphouse			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
15. Category 1 tunnels			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		

TABLE 3.2.2-1 (SHEET 87 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
16. Auxiliary feedwater pumphouse			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
17. Spent fuel pool and refueling canal liner plate	FB	FB	B	NA	0	1	C	AISC-69	Y	Y		
18. Fuel pool gate	FB	FB	B	NA	0	1	C	AISC-69	Y	Y		
19. Turbine building			S	NA	6	2	C	AISC-69, ACI 318-71, UBC-76	N	N		
20. River intake structure			S	NA	6	2	C	AISC-69, ACI 318-71, UBC-76	N	N		
21. Plant water makeup wells			S	NA	6	2	C	AISC-69, ACI 318-71, UBC-76	N	N		
22. Reactor makeup water storage tank and dike			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
23. Circulating water intake structure			S	NA	6	2	C	AISC-69, ACI 318-71, UBC-76	N	N		
24. Natural draft cooling tower			S	NA	6	2	C	AISC-69, ACI 318-71, UBC-76	N	N		
25. Circulating water canals			S	NA	6	2	C	AISC-69, ACI 318-71, UBC-76	N	N		
26. Circulating water piping			B	NA	6	2	C	AWWA C-301	N	N		
27. Minor plant structures and pads			S	NA	6	2	C	AISC-69, ACI 318-71, UBC-76	N	N		
28. Containment internal structures			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		
29. Turbine generator pedestal			S	NA	6	2	C	AISC-69, ACI 318-71, UBC-76	N	N		
30. Storm drain system			S	NA	6	2	C	mfg	N	N		Note Y
31. River makeup water piping			S	NA	6	2	C	AWWA C-200	N	N		
33. NSCW tower valve house			B	NA	0	1	C	AISC-69, ACI 318-71	Y	Y		

TABLE 3.2.2-1 (SHEET 88 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
34. Radwaste transfer building			B	NA	6	2	C	AISC-69 ACI 318-71, UBC-76	N	N		
35. Category 1 electrical cable tray and conduit supports			B	NA	0	1	C	AISC-69, AISI-68	Y	Y		Note 2
36. Category 1 HVAC duct supports			B	NA	0	1	C	AISC-69	Y	Y		Note 2
37. Category 1 pipe supports			B		See Note 4			AISC-69 III-NF	Y	Y		Note 2
38. Pipe whip restraints			W,B	NA	0	1	C	AISC-69	Y	Y		
39. Water tight doors and seals			B	NA	0	1	C	mfg	Y	Y		
40. Waterproofing and water stops			B	NA	6	2	C	mfg	N	N		
41. Category 1 backfill			B	NA	0	1	C	See DC-1000-C	Y	Y		
42. Category 1 tank liner plate	O	O	B	NA	0	1	C	AISC-69	Y	Y		
43. Underground Category 1 conduits			B	NA	0	1	C	AISC-69,	Y	Y		
44. Alternate radwaste building, control room and dress out area	shared		B	NA	6	2	C	UBC-76 AISC-69 ACI-318-71	N	N		
45. Fire dampers	VB	VB	B	NA	6	1,2	6	mfg	N	N		Note v
46. Fire doors	VB	VB	B	NA	6	2	C	mfg	N	N		Note v
47. Fire-rated penetration seals	VB	VB	B	NA	6	2	C	mfg	N	N		Note v
48. Structural steel fire proofing	VB	VB	B	NA	6	2	C	mfg	N	N		Note v
49. Radiant energy shields	C	C	B	NA	6	2	9	mfg	N	N		Note v
50. Electrical raceway fireproofing	VB	VB	B	NA	6	2	9	mfg	N	N		Note v
51. 3-hour plaster walls	VB	VB	B	NA	6	2	C	mfg	N	N		Note v

TABLE 3.2.2-1 (SHEET 89 OF 98)

Principal System and Components		(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
52.	Seismic gap fire seals	VB	VB	B	NA	6	2	C	mfg	N	N		Note v
53.	Alternate radwaste bldg. dressout area and control room HVAC	Shared		S	NA	6	2	6	mfg	N	N		
54.	Alternate radwaste bldg bridge crane	Shared		G	NA	6	2	C	mfg	N	N		
55.	Deleted.												
56.	Deleted.												
57.	Deleted.												
58.	HELB doors	AB	AB	B,S	NA	6	2	C	mfg	N	N		Note am
59.	RPF bridge crane	Shared		S	NA	6	2	C	mfg	N	N		
60.	RPF building	Shared		S	NA	6	2	C	UBC-76 ACI-318-71	N	N		
CONTAINMENT BUILDING POLAR BRIDGE CRANE													
1.	Mechanical components	C	C	B	NA	6	1	6	mfg	N	N		Note q and ab
2.	Motors	C	C	B	NA	6	2	E	NEMA MG1	N	N		Note ab
3.	Instrumentation and controls			B	NA	6	2	J	mfg	N	N		Note ab
FUEL HANDLING SYSTEM													
1.	New and spent fuel storage racks	FB	FB	W,B	NA	0	1	5	mfg	Y	Y	VII	
2.	Refueling machine	C	C	W,G	NA	6	2	6	mfg	N	N		Note ab
3.	RCC storage station	C	C	W	NA	6	2	6	mfg	N	N		
4.	Thimble plug storage rack	C	C	W	NA	6	2	6	mfg	N	N		
5.	Integrated head cable assembly	C	C	W	NA	6	2	6	mfg	N	N		
6.	Integrated head cable tray	C	C	W	NA	6	2	6	mfg	N	N		
7.	Integrated head lifting rig	C	C	W	NA	6	2	6	mfg	N	N		
8.	Integrated head lift rods	C	C	W	NA	0	1	5	III-NF	Y	Y		

TABLE 3.2.2-1 (SHEET 90 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
9. Integrated head missile shield	C	C	W	NA	0	1	5	III-NF	Y	Y		
10. Integrated head operator support stand	C	C	W	NA	6	2	6	mfg	N	N		
11. Integrated head package stud support collars	C	C	W	NA	6	2	6	mfg	N	N		
12. Radial arm hoist assembly	C	C	W	NA	6	2	6	mfg	N	N		
13. Radial arm stud tensioner hoist			S	NA	6	2	6	mfg	N	N		
14. Reactor internals lifting rig	C	C	W	NA	6	2	6	mfg	N	N		
15. Fuel handling machine	FB	FB	W	NA	0	1	5	mfg	Y	Y	VII	
16. Spent fuel handling tool	FB	FB	W	NA	0	1	5	mfg	Y	Y	VII	
17. New fuel handling tool	FB	FB	W	NA	6	2	6	mfg	N	N		
18. Fuel transfer tube			W	B	2	1	2	III-MC	Y	Y		
19. Fuel transfer system			W	NA	6	2	6	mfg	N	N		Note ab
20. New fuel elevator	FB	FB	W	NA	6	2	6	mfg	N	N		Note ab
21. Spent fuel cask bridge crane	FB	FB	B	NA	0	1	5	mfg	Y	Y		Note q
RACEWAY, RACEWAY ACCESSORIES, AND FITTINGS FOR												
1. Safety-related power, control, and instrument circuits, in seismic structures			B	NA	6	1	E	mfg	N	N		
2. Nonsafety-related circuits			B	NA	6	2	E	mfg	N	N		

TABLE 3.2.2-1 (SHEET 91 OF 98)

Principal System and Components	(a) Location Unit 1	Unit 2	(b) Source of Supply	(c) Quality Group	(d) VEGP Safety Class	(e) Seismic Category	(f) Codes and Standards Designator	(g) Principal Construction Code	(h) Q-List	(i) Safety Related	(j) Environmental Designator	(k) Comments
3. Safety-related power, control, and instrumentation circuits in nonseismic structures (Reactor trip on turbine trip, steam dump solenoid valves, and turbine impulse chamber pressure transmitters only.)	TB	TB	B	NA	1	1	E/J	mfg	Y	Y		az

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FOOTNOTES AND COMMENTS

a. Location

AB	Auxiliary building
AFP	Auxiliary feedwater pumphouse
C	Containment building
CB	Control building
DB	Diesel generator building
EB	Equipment building
FB	Fuel handling building
FPH	Fire protection pumphouses
NSW	Nuclear service cooling water valve house
O	Outdoors onsite
RTB	Radwaste transfer building
TB	Turbine building
TSC	Technical support center
VB	Various building
RPF	Radwaste processing facility

Equipment is located by either room number, elevation, or building. Major buildings and their abbreviations are listed above. These abbreviations are used in the location column; e.g., AB is the auxiliary building, as shown in the following examples: AB-A48 indicates the equipment is located in the auxiliary building, on level A, room number 48; C-171' indicates the equipment is located in the containment building at elevation 171 ft.

For equipment that is common to both units, the location is indicated in the column for Unit 1 with "shared" indicated in the column for Unit 2. Some major valves, such as main steam isolation valves, have been located, but in general, piping, valves, ductwork, instrumentation, etc., have not been located.

b. Source of Supply

G	Georgia Power Company
B	Bechtel
W	Westinghouse
S	Southern Company Services, Inc.
SNC	Southern Nuclear Operating Company, Inc.

The organization which has the principal procurement responsibility is identified as the source of supply. *(Historical) This is historical information to indicate the organization which at the time of construction of Vogtle 1 and 2 had principle procurement responsibility. Updates to this information for future equipment changes will not be maintained.*

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c. Quality Group

The quality group classification corresponds to those provided in Regulatory Guide 1.26. NA indicates not applicable and is used for equipment and structures that do not fall under the purview of Regulatory Guide 1.26.

d. VEGP Safety Class

The VEGP nuclear safety class corresponds to the quality group classifications in Regulatory Guide 1.26; i.e., Nuclear Safety Classes 1, 2, 3, and 4 correspond to Quality Groups A, B, C, and D, respectively. Nuclear Safety Class 1 is also assigned to safety-related instruments, controls, and electric components.

e. Seismic Category

Seismic Category 1 is applied to those safety-related structures, systems, and components that must remain functional during and after a safe shutdown earthquake (SSE) according to Regulatory Guide 1.29 and to those nonsafety-related structures, systems, and components that are designed to Seismic Category 1 requirements.

f. Codes and Standards Designator

See paragraph 3.2.2.3.

g. Principal Construction Code

The codes referenced are primary codes only and are defined in table 3.2.2-2. Detailed construction codes are listed in the component specification.

h. Q-List

Y Yes; requires compliance with 10 CFR 50, Appendix B, as implemented in the VEGP and Bechtel/Westinghouse/SCS/SNC quality assurance programs.

N No; not within the scope of 10 CFR 50, Appendix B.

i. Safety Related

Y Yes; safety related.

N No; not safety related.

j. Environmental Designator

The environmental designators are defined in table 3.11.B.1-1. Environmental designators are provided here for only principal safety-related equipment such as pumps, tank, fans, etc.

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k. Comments

This column contains a listing of applicable design criteria and other amplifying information.

l. The main control board will be qualified by Westinghouse to Institute of Electrical and Electronics Engineers (IEEE) 344 and 323. Installation of 62J instruments will be equivalent to project Class 61J.

m. The load can be manually applied to the diesel generator by the operator after sequencer loads have been energized. Refer to diesel generator loading table in drawings 1X3D-AA-K02A, 2X3D-AA-K02A, 1X3D-AA-K02B, and 2X3D-AA-K02B.

n. Offsite power equipment is designed and arranged to provide redundancy and separation for increased reliability.

o. Emergency lighting for the control room suspended ceiling, shutdown panels, and diesel and auxiliary feedwater pumphouse panels is non-Class 1E but shall be powered from 1E supply through isolation devices, maintain necessary redundancy, and meet single failure criteria. The emergency lighting distribution panels are non-Class 1E, but are Seismic Category 1. The control room suspended ceiling is designed and constructed to ensure that the ceiling will not fall or compromise the functioning of safety-related equipment during or after an SSE. Emergency lighting for fire protection consists of fixed 8-h-rated sealed beam fixtures with self-contained battery and charger units powered from normal lighting system. Fixtures which only provide for life safety and are not required for safe shutdown or to support station blackout (SBO) are 1 ½-h rated (minimum). Refer to paragraph 8.4.1.1.2.G for SBO emergency lighting requirements and paragraph 9.5.3.2.3.C for fire protection lighting requirements.

p. Post-accident monitoring system (PAMS) instruments are assigned a project classification based on their category as defined in Regulatory Guide 1.97, Revision 2, as indicated below:

Category 1 - All instruments in this category are classified 11J.

Category 2 - Instruments in this category are qualified from the sensor up to and including the channel isolation device as delineated in table 7.5-1.

Category 3 - All instruments in this category are classified 62J.

PAMS recorders are assigned a project classification of 61J regardless of their category, although they are not designed to function during an SSE, only after an SSE.

q. The crane is designed to retain and prevent dropping its design load during and after an SSE.

r. Selected materials, components, parts, appurtenances, and piping subassemblies are procured in accordance with ASME Code, Section III, Class 3; however, the system is designed and installed in accordance with ANSI B31.1. Conformance with these aspects of the ASME code is required only for initial procurement.

s. Instrumentation and controls which can affect the operation or actuation of a safety-related function must be qualified to requirements of project Class 11J. Other instrumentation and controls, even in safety systems, need not be qualified to project Class 11J requirements if their function is associated with normal plant operations only and serves no safety-related function. Refer to the project instrument index for individual instrument listing. Refer to note p for further clarification.

t. The entire heat exchanger is constructed to ASME Section III requirements to ensure the integrity of the safety-related portion.

u. Portions of the sample system that are part of the pressure boundary of the system being sampled must meet the same quality and code requirements as that sampled system up to and including the first normally shut isolation valve in the sample line.

v. The quality assurance program to be applied to fire protection systems is described in paragraph 9.5.1.1.4.

w. The quality assurance program to be applied to radioactive waste management systems is described in Regulatory Guide 1.143.

x. The Seismic Category 1 fire protection standpipe system serves no safety function but is classified as project class 313 to ensure the implementation of a Seismic Category 1, ASME III-3 design and installation. ASME Code stamping and inclusion in the Final Design Verification Program is not required.

y. Changes to the site grading will be done on an engineered basis so as to assure acceptability of the drainage analysis for the probable maximum precipitation (PMP) event as described in paragraph 2.4.2.3.

z. Selected materials, components, parts, appurtenances, and piping subassemblies are procured in accordance with ASME Code, Section III, Class 3. This conformance with the ASME Code is only required for initial procurement. The system is designed and installed in accordance with ANSI B31.1. Post-installation nondestructive examination of the system in accordance with ASME III, Class 3 is required for the initial installation. Final "N" stamping and associated documentation are not required.

aa. These components are manufactured under the appropriate provisions of WCAP 8370.

ab. The pertinent provisions of the QA program will be applied to these items.

ac. The principal construction code for this tank is ASME Section VIII; however, no code stamp is required since the tank operates at atmospheric pressure.

ad. This portion of the instrument and service air system identifies upgraded piping and valves that function as part of the nonsafety-related post-LOCA containment purge

TABLE 3.2.2-1 (SHEET 96 OF 98)

system. As described in section 6.2.5, the post-LOCA containment purge system is not required following an accident, but provides additional combustible gas control capabilities.

To assure the availability of this function, the associated portions of the instrument and service air system inside the containment have been upgraded to satisfy Seismic Category 1 and ASME III design requirement.

ae. Duct work in this system was upgraded to Seismic 1 requirement to provide additional margin in the structural design. The quality assurance program is not applicable to the duct work.

af. Design and installation to the technical requirement of 1971-S73 ASME Code without "CS" symbol stamp.

ag. These components were originally procured as safety related but do not perform a safety function, are not required to respond to any accident, and are not included in any evaluation of an accident in the UFSAR. A portion of the waste gas processing system, and also the RHT vent eductor, were reclassified to project class 417 as defined on P&IDs 1/2X4DB128, 1/2X4DB129, 1X4DB141, 1X6AK09-176, 1X6AK09-177, 2X6AK09-178, 2X6AK09-179, AX6AK09-180, and AX6AK09-181. The pertinent provisions of the quality assurance program will be applied to the waste gas processing system.

ah. Leak testing of nonfiltration systems and portions of filtration systems (i.e., flexible connections and terminating duct pieces and accessories at inlets, outlets, and exhaust shafts) is performed per SMACNA requirements using ANSI N510 requirements that address methods for performing leak tests.

ai. Duct stiffener angles may be attached to the duct using huck bolts in safety-related nonfiltration systems or any portion of nonsafety-related systems.

aj. Deleted.

ak. The boron injection tank is not provided for Unit 2.

al. The accelerograph is purchased 61J but powered and mounted 62J.

am. The quality assurance program to be applied to HELB doors is the same as that applied to fire protection systems described in paragraph 9.5.1.1.4.

an. The internals have been removed from filter. Filter housing remains in piping system.

ao. The filter element may be removed by cutting element in half and replacing cover assembly. Note that the cover assembly seal includes the upper portion of the filter element basket.

TABLE 3.2.2-1 (SHEET 97 OF 98)

ap. The liquid hydrogen storage tank has been removed and the hydrogen requirements are provided from a gaseous tube trailer through the discharging stanchion.

aq. The spray additive portion of the containment spray system has been eliminated. The associated components are abandoned in place.

ar. The control room HVAC system (essential portion) return air fans are retired in place.

as. The drain isolation valves for ESF equipment rooms and negative pressure boundary areas are procured as project class 313, but are installed as project class 414. These valves are required to be locked closed to ensure validity of the offsite dose exposure analysis as tabulated in table 15.6.5-6.

at. Drain isolation valves for the ESF equipment rooms may be left open during modes 5 and 6. The exceptions for CVCS and RHR equipment rooms are described below.

   a. For RHR equipment rooms, the drain isolation valves of one train may remain open during modes 5 and 6 if that train is out of service and the drain valves of the other train remain locked closed. This will ensure a potential flood in one train room would not affect the other train.

   b. For CVCS centrifugal charging pump rooms during modes 5 and 6, the equipment drain isolation valves of one train may remain open if that train is out of service and the equipment drain isolation valves of the other train remain locked in the closed position. For these rooms during modes 5 and 6, the floor drain isolation valves of one train may remain open if that train is out of service and the floor drain isolation valves of the other train remain locked closed. This will ensure a potential flood in one train room would not affect the other train.

   c. For the CVCS normal charging pump room during modes 5 and 6, the equipment drain isolation valve(s) may remain open if the normal charging pump is out of service and the equipment drain isolation valves of the inservice CVCS centrifugal charging pump train(s) remain locked in the closed position. For these rooms during modes 5 and 6, the floor drain isolation valves may remain open if the normal charging pump is out of service and the floor drain isolation valves of the inservice CVCS centrifugal charging pump train(s) remain locked closed. This will ensure a potential flood in the normal charging pump room would not affect the inservice CVCS centrifugal charging pump train(s).

au. Recycle evaporator package is abandoned in place.

av. Abandoned in place.

aw. Waste evaporator is abandoned in place.

ax. The seismic category of the cabinet is nonseismic 2. However, through analysis the cabinet can withstand a safe shutdown earthquake.

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ay. A portion of the waste gas processing system was reclassified to project class 417 as defined on P&IDs 1/2X4DB128, 1/2X4DB129, 1X4DB141, 1X6AK09-176, 1X6AK09-177, 2X6AK09-178, 2X6AK09-179, AX6AK09-180, and AX6AK09-181. The pertinent provisions of the quality assurance program will be applied to the waste gas processing system.

az. Procured as class 11E/11J but installed as class 12E/12J. See paragraph 7.2.1.1.2.F for additional detail.

ba. Pump has been removed from the system process. Pump remains in place for future use.

bb. Unit 2 Train A actuator is an extension of the pressure boundary and is therefore Quality Group B, Class 212, ASME III-2. Electrical components associated with actuation are as indicated in the table.

GENERAL NOTES

1. For systems under the Westinghouse scope of supply, all piping and all manual valves 2 in. and smaller are supplied by Bechtel, except for the reactor coolant loop piping, the pressurizer surge line, the pressurizer relief piping complex, reactor vessel bottom mounted instrument tubing, reactor vessel head vent piping to refueling disconnect flange, and reactor vessel seal leak detection leakoff appurtenance.

2. Hangers and supports for Seismic Category 1 systems and components are designed as Seismic Category 1. In general hangers and supports for Seismic Category 2 piping, cable tray, and ducting in Seismic Category 1 buildings are designed to maintain their structural integrity under the postulated earthquake conditions; however, exceptions to this requirement are permitted when it is demonstrated that their failure will not adversely affect adjacent Seismic Category 1 equipment or systems.

3. All "Q" listed coatings are assigned a project classification of 02C. Q listed coatings are not seismically qualified but will not fail in a manner that would compromise the function of safety-related equipment in the event of an earthquake since they are applied to Seismic Category 1 structures.

4. The safety class, seismic category, and codes and standards designators of hangers and supports of the Category 1 piping systems are the same as the piping system they support.

TABLE 3.2.2-2 (SHEET 1 OF 2)

PRINCIPAL CODES AND STANDARDS

I	ASME Boiler and Pressure Vessel Code, Section I.
III-1,2,3, MC, NF,	ASME Boiler and Pressure Vessel Code, CS Section III, Subsections NB, NC, ND, NE, NF, and NG.
VIII	ASME Boiler and Pressure Vessel Code, Section VIII, Division 1.
B31.1	ANSI B31.1.0, Power Piping.
AISC-69	American Institute of Steel Construction, Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, adopted February 12, 1969, with Supplements 1, 2, and 3.
AISI	American Iron and Steel Institute, Specification for the Design of Cold-Formed Steel Structural Members, 1968, Design of Light Gage Cold-Formed Stainless Steel Structural Members, 1968.
AMCA	Air Moving and Conditioning Association.
ACI 318-71	American Concrete Institute, Building Code Requirements for Reinforced Concrete, including 1974 Supplement.
ANSI N509	American National Standard Institute, Nuclear Power Plant Air Cleaning Units and Components.
API-620	American Petroleum Institute, Recommended Rules for Design and Construction of Large, Low Pressure Storage Tanks.
API-650	American Petroleum Institute, Welded Steel Tanks for Oil Storage.
ARI	Air Conditioning and Refrigeration Institute.
ASHRAE	American Society of Heating, Refrigerating and Air Conditioning Engineers.
DEMA	Diesel Engine Manufacturer Association, Standard Practices for Stationary Diesel and Gas Engines, 1971.
mfg	Manufacturer's standard. Design requirements specified by designer with appropriate consideration of the intended service and operating conditions.
NEMA MG1	National Electric Manufacturers Association, 1972, Motors and Generators.
NFPA	National Fire Protection Association.

TABLE 3.2.2-2 (SHEET 2 OF 2)

SMACNA	Sheet Metal and Air Conditioning Contractors National Association, Inc.
TEMA C,R	Tubular Exchanger Manufacturers Association, Class C or R.
UBC	Uniform Building Code.
UL	Underwriters' Laboratories.

TABLE 3.2.2-3

SUMMARY OF CONSTRUCTION[a] CODES AND STANDARDS FOR VEGP COMPONENTS
BY NRC QUALITY CLASSIFICATION SYSTEM[b]

Components	Quality Group A	Quality Group B	Quality Group C	Quality Group D
Pressure vessels	ASME Boiler and Pressure Vessel Code, Section III, Division 1, Subsection NB, Class 1, nuclear power plant components	ASME Boiler and Pressure Vessel Code, Section III, Division 1, Subsection NC, Class 2, nuclear power plant components	ASME Boiler and Pressure Vessel Code, Section III, Division 1, Subsection ND, Class 3, nuclear power plant components[c]	ASME Boiler and Pressure Vessel Code, Section VIII, Division 1
Piping	As above	As above	As above	ANSI B31.1 power piping
Pumps	As above	As above	As above	Manufacturer's standards
Valves	As above	As above	As above	ANSI B31.1 power piping
Atmospheric storage tanks	Not applicable	As above	As above	API-650
0- to 15-psig storage tanks	Not applicable	As above	As above	API-620
Supports	As above except Subsection NF	As above except Subsection NF	As above except Subsection NF	Manufacturer's standards
Metal containment components	Not applicable	As above except Subsection NE, Class MC	Not applicable	Not applicable
Core support structures	Not applicable	As above except Subsection NG	Not applicable	Not applicable

a. As defined in Subarticle NCA-1110 of Section III of the ASME Boiler and Pressure Vessel Code, construction is an all-inclusive term comprising materials, design, fabrication, examination, testing, inspection, and certification required in the manufacture and installation of components.

b. As defined in Regulatory Guide 1.26, the NRC quality classification system identifies on a functional basis components of fluid systems by Quality Groups A, B, C, and D.

c. The specific applicability of ASME Code Cases is covered separately in tables 1.9-1 through 1.9-3.

3.3 WIND AND TORNADO LOADINGS

3.3.1 WIND LOADINGS

The wind loadings for Seismic Category 1 structures are in accordance with American National Standards Institute (ANSI) A58.1, Building Code Requirements for Minimum Design Loads in Buildings and Other Structures.[1]

3.3.1.1 Design Wind Velocity

The design wind velocity for all Seismic Category 1 structures is 110 mph at 30 ft above grade for a 100-year mean recurrence interval. The vertical velocity profiles and gust factors are in accordance with subsection 6.3.4 of reference 1.

3.3.1.2 Determination of Applied Forces

The procedures utilized in transforming the wind velocity into an effective pressure to be applied to structures and parts and portions of structures follow the guidelines of reference 1. For a design wind velocity of V_{30} mph specified at a height of 30 ft above grade the velocity pressure, q_{30}, is given by:

$$q_{30} = 0.00256\ (V_{30})^2\ \text{lb/ft}^2$$

The design wind pressure and the pressure distribution are obtained using the provisions of reference 1 for Exposure C, which is applicable for flat open terrain and BC-TOP-3A.[2]

3.3.1.3 References

1. "The American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures," ANSI A58.1-72.

2. Topical Report, "Tornado and Extreme Wind Design Criteria for Nuclear Power Plants," Bechtel Power Corporation, BC-TOP-3A, Rev. 3, San Francisco, California, August 1974.

3.3.2 TORNADO LOADINGS

Seismic Category 1 structures, housing safety-related equipment, systems, and components, are designed to withstand the effects due to the design basis tornado as described in the following paragraphs.

3.3.2.1 Applicable Design Parameters

The design parameters applicable to the design basis tornado are as follows:

- Maximum peripheral tangential velocity - 290 mph.

- Translational velocity - 70 mph maximum/5 mph minimum.
- Maximum wind velocity - 360 mph.
- Radius from the center of the tornado, where the maximum wind velocity occurs - 150 ft.
- Atmospheric pressure drop - 3 psi.
- Rate of pressure drop - 2 psi/s.

These parameters conform to those given in Regulatory Guide 1.76 for Region I. The design basis tornado missiles are discussed in paragraph 3.5.1.4.

3.3.2.2 Determination of Forces on Structures

The procedures specified in BC-TOP-3A[(1)] are used to transform the tornado wind loading and differential pressure loading into effective loads on structures.

The dynamic wind pressure is applied to the structure in the same manner as the wind loads described in paragraph 3.3.1.2, with the exception that the gust factor and the variation of wind speed with height do not apply. Loading combinations and load factors used are as specified in reference 1 and are as follows:

$W_t = W_w$

$W_t = W_p$

$W_t = W_m$

$W_t = W_w + 0.5\ W_p$

$W_t = W_w + W_m$

$W_t = W_w + 0.5\ W_p + W_m$

where:

W_t = total tornado load.

W_w = total wind load.

W_p = total differential pressure load.

W_m = total missile load.

The maximum pressure drop of 3 psi, applicable to a nonvented structure, is used for W_p, unless a lower value is justified using the provisions of reference 1, for partially vented structures. When the tornado loading includes the missile load, the structure locally may go in the plastic range due to the missile impact. The procedure for analyzing local missile effects is presented in appendix 3C.

The analysis of the nonsafety-related equipment building shows that it will not collapse on adjacent Seismic Category 1 structures, equipment, systems, or components due to tornado loading.

3.3.2.3 Effect of Failure of Structures or Components Not Designed for Tornado Loads

The non-Seismic Category 1 structures, equipment, systems, and components not designed for tornado loadings are investigated to ensure the following:

A. These structures, equipment, systems, and components cannot produce missiles that have more severe effects than the tornado-generated missiles discussed in paragraph 3.5.1.4.

B. Their failure will not affect the integrity of adjacent Seismic Category 1 structures. This design ensures that Seismic Category 1 structures, equipment, systems, and components required for safe shutdown after a tornado will perform their intended functions.

The analyses of the turbine building, radwaste transfer building, radwaste tunnel, alternate radwaste building, and radwaste processing facility (Seismic Category 2 structures) show that they will not jeopardize adjacent Seismic Category 1 structures when subjected to the tornado loads described in paragraph 3.3.2.2.

3.3.2.4 Reference

1. Topical Report, "Tornado and Extreme Wind Design Criteria for Nuclear Power Plants," Bechtel Power Corporation, BC-TOP-3A, Rev. 3, San Francisco, California, August 1974.

3.4 WATER LEVEL (FLOOD) DESIGN

The flooding of a nuclear power plant from natural causes can be attributed to probable maximum flood (PMF), site and adjacent area probable maximum precipitation (PMP) runoff, and ground water. Criteria for the design basis flood conform to the requirements of Regulatory Guide 1.59, Design Basis Floods for Nuclear Power Plants, and Regulatory Guide 1.102, Flood Protection for Nuclear Power Plants.

3.4.1 FLOOD PROTECTION

3.4.1.1 Flood Protection Measures for Seismic Category 1 Structures

The Seismic Category 1 structures, systems, and components identified in table 3.2.2-1 are designed to withstand the effects of natural phenomena, such as flooding and ground water level. A description of the structures is provided in sub-sections 3.8.1, 3.8.3, and 3.8.4.

3.4.1.2 Flood Protection from Natural Causes

No flooding due to the PMF can occur, because the finished grade levels of the VEGP are located above the PMF level of el 165 ft msl. The PMF results from river flooding, upstream dam failure, and other natural causes. (See subsection 2.4.3.) No Seismic Category 1 structures, systems, or components can be affected by this maximum flood condition, because the plant nominal finished grade elevation is 219 ft 6 in.

No flooding can occur from the PMP. Water from roof drains and/or scuppers and runoff from the plant site and adjacent areas is conveyed to catch basins, underground pipes, or directly to open ditches by sloping the tributary surface area. The site is graded to offer protection to the Seismic Category 1 structures by a minimum of 1-percent surface slope.

A high ground water level has been established at el 165 ft msl (paragraph 2.5.1.2). The basement levels of the containment, auxiliary and fuel handling buildings, and electrical and piping tunnels, which are located below the high ground water table, are protected against ground water flooding. Prevention of ground water entry is primarily afforded by the thick (minimum 24 in.) concrete walls and floors. In addition, nonsafety-related waterproofing treatments are provided on exterior walls of safety-related structures up to plant grade elevation. Two types of waterproofing are used:

- Waterproof membrane.
- A chemical waterproofing treatment.

One waterstop is provided at each construction joint below el 170 ft, except in the nuclear service cooling water towers where two waterstops are provided at each construction joint below el 220 ft. Two waterstops are provided at each seismic separation joint below el 170 ft, and one waterstop is provided in between el 170 and 220 ft. A typical waterstop in a seismic separation joint is described in subsection 3.8.5. The seismic joints are covered by a flashing to prevent water from entering the joint from above. There are no penetrations through exterior walls or basemats into the soil below the high ground water level. The only exterior personnel or equipment access to the Seismic Category 1 structures is at the finished grade level or above.

3.4.1.2.1 Flood Protection From Component Failures

Each area of the plant is reviewed to determine the postulated fluid system failure, including non-Seismic Category 1 and nontornado-protected tanks, vessels, and other process equipment, which results in the most adverse flooding conditions. Flooding due to failure of non-Seismic Category 1 tanks and vessels located in outside areas will not impact safety-related equipment, since flood water would be directed away from buildings to catch basins and open ditches. In addition, no safety-related equipment would be affected by flooding due to failure of non-Seismic Category 1 tanks and vessels in the auxiliary building. Waterproof doors, curbs, wall penetrations, seals, or drainage systems are provided for safety-related equipment to mitigate the consequences of such failures. The criteria used in the evaluation of the most adverse failure of a fluid system are discussed in appendix 3F.

Failure of the natural draft cooling tower or circulating water system in the plant yard will not cause adverse effects to any safety-related components required to mitigate the consequences of the event and/or safely shutdown the plant (i.e., essential equipment). Part of the cooling tower basin is below ground level. In case of failure, the water above ground level (~310,000 ft^3) will empty on the surroundings and flow away from the power block, any Category 1 structures, or any essential equipment due to the slope of the yard grade. All the water will eventually flow into the yard drainage system. A postulated worst condition crack on the circulating water system pipeline outside the turbine building would result in a conservatively calculated leakage rate of 10,621 ft^3/min. At 45 psig it is assumed that the water will break the backfill and flow in the direction of least resistance, up toward ground level.

Once on ground level, the water will be channeled away from the power block due to the slope of the yard grade, and all water will flow into the yard drainage system. There are no essential components in that area. At this time appropriate measures would be taken to stop the waterflow from the cooling tower basin to the crack.

In the unlikely event that a crack on the circulating water system outside the turbine building is not detected by plant operations, the integrity of the safety-related seismic Category 1 structures, buried piping, and tunnels will not be impaired through soil erosion, since they are remotely located, and there are other intervening nonsafety-related structures in between.

3.4.1.2.2 Flood Protection Procedures

The VEGP is designed so that the maximum water levels considered due to natural phenomena do not jeopardize the safety of the plant or the ability to conduct a safe shutdown.

3.4.1.3 Permanent Dewatering System

Only the lower levels of the containment, the auxiliary building, and the tunnels beneath the fuel handling building and control building are subject to inseepage due to their location below the high ground water level. Ground water entry is precluded by the thick concrete walls and floors, waterproofing, and waterstops. No permanent dewatering system is required.

3.4.2 ANALYTICAL AND TEST PROCEDURES

The foundation slabs and exterior walls of the structures are designed to resist the upward and the lateral pressures caused by the high ground water level. The vertical hydrostatic pressure acting uniformly at the bottom of the structures is the product of the height to the high ground water level and the unit weight of water assumed as 62 lb/ft^3. The horizontal hydrostatic pressure acting on the exterior walls varies with height, from the maximum at the bottom of the wall to zero at the maximum ground water level. Minimum factors of safety for overturning, sliding, and flotation are described in subsection 3.8.5. There are no dynamic water forces associated with the high ground water level. Dynamic forces associated with the probable maximum flood or probable maximum precipitation are not factors in the analysis or design of Category 1 structures, since the finished grade is adequately sloped and is located at an elevation above the maximum flood level.

There are no safety-related hydraulic structures at VEGP.

3.5 MISSILE PROTECTION

In accordance with the requirements of 10 CFR 50, Appendix A, GDC-2 and 4, adequate missile protection is provided to ensure that those portions of the safety-related structures, systems, or components whose failure would result in the failure of the integrity of the reactor coolant system pressure boundary, reduce to an unacceptable level the functioning of any plant feature required for a safe shutdown, or lead to unacceptable offsite radiological consequences, are designed and constructed so as not to fail or cause such a failure in the event of a postulated credible missile impact. Conformance to the requirements of Regulatory Guides 1.13, 1.14, 1.27, 1.76, 1.115, and 1.117 is discussed in section 1.9. The following sections provide the bases for the selection of the postulated missiles, protection requirements for external missiles, and details of the missile barrier design. Safety-related systems or components as described above are protected by locating them within missile-proof structures, by providing separation, or by providing missile shields or barriers. Nonsafety-related structures, systems, and components are protected from internally generated missiles if their failure by postulated missile impact could prevent the required safety function of other safety-related structures, systems, and components.

3.5.1 MISSILE SELECTION AND DESCRIPTION

The following sources are considered for the generation of missiles:

- Internally generated missiles:
  - Internally generated missiles outside containment.
  - Internally generated missiles inside containment.
- Turbine missiles.
- Externally generated missiles:
  - Missiles generated by natural phenomena.
  - Missiles generated by events near the site.
  - Aircraft hazards.
  - Gravity-generated missiles.

The systems located both inside and outside of the containment have been examined to identify and classify potential missiles.

The basic approach is to ensure design adequacy against generation of missiles rather than to allow missile formation and design plant features to contain their effects. In those cases where missile formation does occur, plant features are designed to contain their effects.

3.5.1.1 Internally Generated Missiles (Outside Containment)

There are two general sources of postulated missiles outside the containment which are potentially generated as a result of plant operation:

- Rotating component failures.
- Pressurized component failures.

Excluded as sources of postulated missiles are:

- Those components which operate approximately 2 percent or less of the time (a tabulation of these components is provided in table 3.5.1-8), or;
- Those components, provided in the line designation list, located in high energy fluid systems which are at high energy conditions for 2 percent or less of the time the system is in operation, or;
- Those components, provided in the line designation list, located in high energy fluid systems which are at high energy conditions when in operation but operate for less than 1 percent of the plant operating time.

A tabulation of safety-related structures, systems, and components and their locations, seismic categories, and quality group classifications is given in table 3.2.2-1. General arrangement and section detail drawings are located in section 1.2. The results of the analysis of the effects of missiles are given in table 3.5.1-1.

3.5.1.1.1 Rotating Component Failure Missiles

A tabulation of missiles generated by postulated failures of rotating components, their sources and characteristics, location, and provided missile protection is given in table 3.5.1-1.

Missile selection is based on the following conditions:

A. All rotating components that are operated during normal operating plant conditions are considered to be potential missiles if the energy of the missile is sufficient to perforate the housing.

B. The energy in a rotating part associated with component failure is assumed to occur at 120-percent overspeed for turbine-driven components and at maximum operating speed for electrically-driven components.

C. Components within one train of a redundant system are not protected from potential rotating missiles originating from the same train. Components within the other train are protected by complete separation and compartmentalization.

3.5.1.1.2 Pressurized Component Failure Missiles

Based on the design features noted below and review of the plant areas outside the containment containing pressurized components, it is concluded that there are no pressurized components whose failure will result in postulated missiles affecting the safety-related systems, structures, and components required for safe shutdown of the reactor. The design features of the pressurized components and the basis for the missile selection are described below.

A. Pressurized components in systems which qualify as high-energy systems (as defined in section 3.6) are evaluated as to their potential for becoming missiles.

B. Temperature or other detectors installed in high-energy piping are evaluated as potential missiles if failure of a threaded connection would cause their ejection. Thermowells retained by circumferential, pressure-retaining welds are not considered credible missiles because of their conservative design and weld quality inspections.

C. Where auxiliary fittings such as thermocouple wells, pressure gages, vents, drains, and test connections are attached to piping or process equipment by threaded connections only, they are postulated as missiles. When such fittings are attached by welding and the completed joint has a greater design strength than the parent metal, they are not postulated as missiles.

D. Valves of American National Standards Institute (ANSI) rating 900 psig and above, constructed in accordance with Section III of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel (B&PV) Code, are primarily pressure seal, bonnet-type valves with the exception that certain small bore valves in this ANSI rating range may be the bolted-bonnet style. For pressure seal bonnet valves, valve bonnets are prevented from becoming missiles by the retaining ring, which would have to fail in shear, and by the yoke, which would capture the bonnet or reduce bonnet energy. Because of the highly conservative design of the retaining ring (safety factors in excess of eight may be used), bonnet ejection is highly improbable; hence, bonnets are not considered credible missiles for these valves.

E. Most valves of ANSI rating 600 psig and below are valves with bolted bonnets or pressure seal, bonnet-type valves. Valve bonnets are prevented from becoming missiles by limiting stresses in the bonnet-to-body bolting material by rules set forth in the ASME B&PV Code, Section III, and by designing flanges in accordance with applicable code requirements. Even if bolt failure were to occur, the likelihood of all bolts experiencing a simultaneous complete severance failure is very remote. The widespread use of valves with bolted bonnets and the low historical incidence of complete severance valve bonnet failures confirm that bolted valve bonnets need not be considered as credible missiles.(1)

F. Valve stems are not considered as potential missiles if at least one feature, in addition to the stem threads, is included in their design to prevent ejection. For example, valves with backseats are prevented from becoming missiles by this feature. In addition, air-or motor-operated valve stems are effectively restrained by the valve operators.

G. Nuts, bolts, nut and bolt combinations, and nut and stud combinations have only a small amount of stored energy and thus are not considered potential missiles.

H. Normally closed gate valves are not considered as potential missile sources, since the force of the fluid acts perpendicularly to the disc, stem, and operator.

I. Components within one train of a system containing redundant trains are not protected from potential pressurized missiles originating from the same train due to complete separation and compartmentalization.

The conclusion, based on design features noted above, that valve bonnets are not credible missiles is also supported by industry experience.

It is necessary to consider the possible modes of rupture of valves to estimate the likelihood that, given a rupture, a missile would be ejected. As has been shown by Nuclear Plant Reliability Data System data and general industry experience, rupture is most likely to take the form of a through-wall crack, which would be detected as a leak long before it could propagate into a serious loss of fluid or missile-generating failure. To be a source of a significant missile, such a crack would have to occur in the bonnet area of a valve and would have to be a circumferential crack. With the above probability for any such rupture, it is not reasonably credible that such a particular crack could occur and remain undetected for a sufficient time to propagate into a missile-generating condition.

Stem ejection is a possible source of missiles, but because the stem is attached firmly to the valve internals as well as the driving and pressure-retaining mechanism in the majority of large valves, it is highly unlikely.

3.5.1.2 Internally Generated Missiles (Inside Containment)

The general sources and exclusions (see table 3.5.1-8) of postulated missiles outside the containment (paragraph 3.5.1.1) also apply to inside the containment. The results of the analysis of the effects of the missiles are given in tables 3.5.1-2 and 3.5.1-3. For the reactor coolant pressure boundary (RCPB), the selection of potential missiles is based on the application of single-failure criteria to the normal retention features of plant equipment for which there is a source of energy capable of creating a missile in the event of the postulated removal of the normal retention features. Where redundancy is provided by the normal retention features, such that sufficient retention capability remains to prevent creation of a missile in the event of a postulated failure of a single retention feature, no potential missile is postulated.

3.5.1.2.1 Control Rod Drive Mechanisms

Gross failure of a control rod drive mechanism (CRDM) housing sufficient to allow a control rod to be rapidly ejected from the core is not considered credible for the following reasons:

A. Control rod drive mechanisms are shop hydrotested at 4100 ±75 psi.

B. Control rod drive mechanism housings are individually hydrotested to 3107 psi after they are installed on the reactor vessel to the head adapters and are checked again during the hydrotest of the completed reactor coolant system.

C. Control rod drive mechanism housings are made of type 304 stainless steel. This material exhibits excellent notch toughness at all temperatures that will be encountered.

D. Stress levels in the mechanisms are not affected by system transients at power or by thermal movement of the coolant loops.

However, it is postulated that the top plug on the CRDM will become loose and will be forced upward by the water jet. The following sequence of events is assumed:

1. The drive shaft and control rod cluster are forced out of the core by the differential pressure of 2500 psi across the drive shaft. The drive shaft and control rod cluster, latched together, are assumed fully inserted when the accident starts.

2. After approximately 12 ft of travel, the rod cluster control spider hits the underside of the upper support plate (part of the integrated head).

3. Upon impact the flexure arms in the coupling joining the drive shaft and control cluster fracture, completely freeing the drive shaft from the control rod cluster. It is assumed that the control cluster would be completely stopped by the upper support plate; however, the drive shaft would continue to be accelerated upward, hitting the missile shield provided.

The CRDM missiles are summarized in table 3.5.1-2. The velocity of the missiles has been calculated by balancing the forces due to the water jet. No spreading of the water jet has been assumed. These missiles are contained by the integrated head missile shield.

3.5.1.2.2 Valves

Valves have been examined to identify potential missiles. As a result of this review, there are no credible failures that could result in missile formation. Therefore, valves are not considered as credible sources of missiles. Motor-operated and air-operated valves contain design features which effectively preclude the ejection of valve stems.

Valves with a nominal diameter larger than 2 in. are designed against bonnet-to-body connection failure and subsequent bonnet ejection by means of the following:

A. Compliance with the ASME Code, Section III.

B. Control of load during tightening of bonnet-to-body bolted connections.

Reactor coolant pressure retaining parts are constructed in accordance with the ASME B&PV Code, Section III, Class 1. The valves are hydrostatically tested in accordance with the ASME Code, Section III.

In the special case of those valves located on the top of the pressurizer, which extends above the operating deck, certain vertical missiles, although not considered credible, are postulated; and protection is provided by the 3-ft-thick concrete roof slab, which prevents potential damage to the containment liner, engineered safeguards pipes, and components located outside the pressurizer compartments.

The missile characteristics of the valves in the region where the pressurizer extends above the operating deck are given in table 3.5.1-3.

3.5.1.2.3 Temperature and Pressure Sensors

The only credible source of jet-propelled missiles from the reactor coolant piping and piping systems connected to the reactor coolant system is that represented by the temperature and pressure sensor assemblies. The resistance temperature sensor assemblies can be of two types, with well and without well. Two rupture locations have been postulated: one around the welding between the boss and the pipe wall; another at the welding (or thread) between the temperature element assembly and the boss for the without-well element and the welding (or thread) between the well and the boss for the with-well element.

A temperature sensor is installed on the reactor coolant pumps close to the radial bearing assembly. A hole is drilled in the gasket and sealed on the internal end of a steel plate. In evaluating missile potential, it is assumed that this plate could break and the pipe plug on the external end of the hole could become a missile.

The missile characteristics of the piping temperature sensor assemblies are given in table 3.5.1-3. A 10° expansion, half- angle water jet has been assumed. The missile characteristics of the piping pressure element assemblies are less severe than those of table 3.5.1-3 and are not of concern from a penetration standpoint.

3.5.1.2.4 Other Missiles

The missile characteristics of the reactor coolant pump temperature sensor, the instrumentation well of the pressurizer, and the pressurizer heaters are given in table 3.5.1-3.

Pressurizer heaters are potential missiles; but inasmuch as they would be ejected in a downward direction, no damage to safety-related structures, systems, and components inside the containment would occur.

The pressurizer relief tank rupture discs are designed such that their failure will not result in the formation of missiles. With rupture, the disc will split into quadrants that will be retained by the disc circumference. The tank is located low in the containment outside the secondary shield wall, and disc rupture will not cause failure to either the primary or secondary systems.

Based on the design features and the analysis presented in the preceding sections, it is concluded that because of compartmentalization, protective barriers, redundancy, and low kinetic energy associated with missiles, the intended safety function of the essential structures, systems, or components will not be impaired by any type of rotating or pressurized missile source.

3.5.1.3 Turbine Missiles

The turbine-generator stores large amounts of rotational kinetic energy in its rotor. In the unlikely event of a major mechanical failure, this energy may be transformed into both rotational and translational energy of rotor fragments. These fragments may impact the surrounding stationary parts. If the energy-absorbing capability of these stationary turbine- generator parts is insufficient, external missiles will be released. These ejected missiles may impact various plant structures, including those housing safety-related equipment. Paragraphs 10.2.3.6 and 10.2.4.6 describe the inspection requirements and the testing of valves which prevent turbine overspeed that would cause the missile generation.

3.5.1.3.1 Turbine Placement and Orientation

The placement and orientation of the turbine-generators is shown on drawing AX6DD303.

3.5.1.3.2 Target Description

Drawings AX6DD304, AX6DD305, AX6DD306, AX6DD307, and AX6DD308 show the physical location of all targets considered in this analysis. The safety-related components that are protected from turbine missiles include the components that are protected from tornado missiles in accordance with Regulatory Guide 1.117.

3.5.1.3.3 Low-Pressure Turbine Rotor Types

The turbine-generators for VEGP are manufactured by General Electric (GE) and are described in section 10.2. General Electric's experience and calculations show that, in the improbable event of a rotor fracture, the substantial fragments of the high-pressure turbine and generator rotors will be contained within their respective casings.[(2)]

The low-pressure turbine rotors for Unit 1 are manufactured from monoblock forgings. The monoblock rotors have bucket attachment areas integral to the shaft rather than keyed wheels.

The Unit 2 low-pressure rotors are built-up rotors with shrunk-on wheels with axial keyways.

3.5.1.3.4 Unit 1 Low-Pressure Turbine Missile Probability Analysis

The methodology for missile generation probability includes consideration of the probability of unit overspeed, wheel materials, in-service inspection capabilities, and the potential for wheel containment by stationary turbine structures. The analysis methodology considers two fundamental failure modes that can lead to missile generation, brittle fracture failures, and ductile tensile failures. These two failure modes are statistically independent.[(3)]

The brittle-fracture failure mechanism is due to the growth of keyway stress corrosion cracks to critical size. Since the monoblock rotors on Unit 1 contain no wheel keyways, no missile generation will occur due to brittle fracture. The Unit 1 monoblock rotors do not have shrunk-on wheels, therefore, only the ductile failure portion of the methodology is used.

The probability of ductile failure is a function of speed, temperature, and material tensile strength. The turbine control system is designed to limit peak overspeed to 120 percent of rated. Under 120 percent speed, rotor design stresses are below the ultimate material strength, thus the probability of a ductile failure is negligible at speeds under 120 percent. The GE probabilistic analysis of turbine overspeed was documented in the GE 1984 NRC report and referenced in Supplementary Report GET-8039 dated September 1993, and is applicable to units with low-pressure monoblock rotors. The overspeed analysis considers the characteristics of the turbine control system, the unit configuration, and test requirements for the steam valves and other overspeed protection devices. This overspeed analysis showed that the probability of attaining a given overspeed decreases rapidly as the overspeed value increases. As long as the control system is maintained in accordance with GE’s recommendations, the annual probability of attaining an overspeed of 120 percent or greater is less than 2×10^{-6}.

Unit 1 main turbine controls was upgraded with a General Electric Mark VIe control system using triple modular redundancy for increased reliability over the original GE Mark II controls. Overspeed protective trips are generated by two sets of triple redundant speed pickups using a highly reliable and redundant trip manifold assembly (TMA). GE analysis “Control System Upgrade Impact on the Probability of Turbine Missile Generation” (reference 5) concluded that the probability of an overspeed event caused by a control system failure to be less than the original Mark II control system. Since the control system overspeed failure probability is only a small portion of the total overspeed probability, the overall annual probability of attaining an overspeed of 120 percent or greater remains less than 2×10^{-6}.

All of the components of the monoblock rotors have sufficient margin to tensile strength (at design component temperatures) to support operating speeds well in excess of 120 percent of rated. The limiting components, per design, for the low-pressure rotors are the last stage buckets, which have overspeed capability of 168 percent.

Since ductile failure is only possible at speeds significantly greater than 120 percent and since the turbine control system keeps the probability of speeds over 120 percent below 2×10^{-6}, the probability of missile generation is well below 2×10^{-6}.

The turbine missile analysis considers the following probabilities:

P1 = The probability of missile genesis due to turbine failure which causes fragment ejection through turbine casing.

P2 = The probability that a fragment strikes a specified target given its generation and ejection.

P3 = The probability that the fragment strike damages its target in a manner leading to unacceptable consequences.

P4 = The overall probability that a particular target suffers unacceptable consequences because of turbine failure.

The probability that a particular safety-related or important-to-safety target suffers unacceptable consequences because of turbine failure is:

P4 = P1 x P2 x P3

Therefore, for the monoblock rotor design, the value of P1 is:

P1 = 2 x 10-6

Since the probability of missile generation (P_1) is sufficiently low that when combined with P_2 and P_3 would result in a value of P_4 that meets the NRC acceptance criteria of 1×10^{-7} for target damage with unacceptable consequences, GE does not calculate values for P_2 and P_3. In NUREG-1048, Safety Evaluation Report related to the operation of Hope Creek Generating Station, Supplement 6, July 1986, the NRC makes the following statements in Appendix U, Probability of Missile Generation in General Electric Nuclear Turbines:

> "…in the evaluation of P_4 ($P_1 \times P_2 \times P_3$), the probability of unacceptable damage to safety-related systems from potential turbine missiles, the NRC staff is giving credit for the product of the strike and damage probabilities of $10^{-3}yr^{-1}$ for a favorably oriented turbine and $10^{-2}yr^{-1}$ for an unfavorably oriented turbine, and is discouraging the elaborate calculation of these values.
>
> The NRC staff believes that maintaining an initial small value of P_1 through turbine testing and inspection is a reliable means of ensuring that the objectives precluding turbine missiles and unacceptable damage to safety-related structures, systems, and components can be met. It simplifies and improves procedures for evaluating turbine missile risks and ensures that the public health and safety is maintained."

The above reference provides generic NRC acceptance regarding the use of a nominal value in lieu of plant specific values for the strike and damage probability in the calculation of the overall probability of unacceptable damage due to a turbine missile. Therefore, the value of P_4 for the monoblock rotors will be:

P_4 = $P_1 (1 \times 10^{-2})$

P_4 = 2×10^{-8}

3.5.1.3.5 Unit 2 Low-Pressure Turbine Missile Probability Analysis

In the low-pressure turbine design, the energy stored in the hypothetical fragments of the wheels is of the same order of magnitude as the energy-absorbing capability of the stationary parts. The probability of missile generation is based on three major components: 1) probability of overspeed, 2) estimation of wheel burst probability as function of speed, and 3) probability of a wheel fragment penetrating the casing.(3,4) Appropriate information regarding missile fragment nomenclature, size, shape, weight, energy, and velocity is presented in table 5-2 of the GE report.(3)

Turbine missiles are ejected fragments of the turbine wheels or surrounding casing which originate because of brittle fracture at normal rated speed (low-speed burst) or due to ductile fracture during turbine runaway (high-speed burst). Missiles may be ejected at any angle of the 360° arc about the turbine axis. The ejection path will not always be perfectly normal to the turbine axis but may vary from -5° to +5° of the normal for the interior turbine wheels, according to GE data. For the two outer turbine wheels, GE postulates a range of missile ejection angles from -25° to +25° of the normal to the turbine axis.

This probability analysis considers both low-trajectory and high-trajectory missiles. The missile ejection angles are illustrated in figure 3.5.1-1. The vertical angle ϕ is measured about the turbine axis from the horizontal plane; ψ is the angle measured from the normal to the turbine axis; and θ is the projection of ψ on the horizontal plane. The horizontal angle θ is related to ϕ and ψ by the formula:

$$\theta = \tan^{-1}\left(\frac{\tan\psi}{\cos\phi}\right) \qquad (1)$$

From equation 1 it is seen that the angular range of missile ejection measured on the horizontal plane increases with increasing ϕ, the vertical angle of ejection. For example, for a missile ejected in the horizontal plane, at ϕ = 0°, the horizontal range of ejection angles varies from -5° to +5° for interior turbine wheels. At ϕ = 45°, the angular range measured at the horizontal plane varies from -7° to +7°; and at ϕ = 90°, it ranges from -90° to +90°. It is theoretically possible for a missile to strike a target located in line with the turbine axis, although the probability of strike is much lower than for targets located on either side of the axis.

3.5.1.3.5.1 Probabilities Considered. The following probabilities are considered in the determination of the likelihood of a turbine missile accident leading to damage of structures, systems, or components required for safe plant shutdown:

P_1 = The probability of missile genesis due to turbine failure which causes fragment ejection through turbine casing.

P_2 = The probability that a fragment strikes a specified target given its generation and ejection.

P_3 = The probability that the fragment strike damages its target in a manner leading to unacceptable consequences.

P_4 = The overall probability that a particular target suffers unacceptable consequences because of turbine failure.

The probability analysis is performed for the layout of the two generating units as shown on drawing AX6DD303.

3.5.1.3.5.2 Probability of Missile Genesis (P1). The probability of missile genesis has been determined by GE for each low pressure turbine wheel. Turbine overspeed probability, wheel burst probability, and casing penetration probability are the major components of the missile genesis probability.[3,4] Turbine overspeed probability considers abnormal events such as full load rejection and control system failures which could cause an overspeed. Wheel burst probability considers turbine speed, wheel temperature, and wheel keyway stress corrosion cracking.

Probability of casing penetration considers the kinetic energy of the wheel fragment at the instant of burst as well as the energy absorbing capability of the low pressure turbine stationary components.

The NRC missile genesis is $P_1 = 1 \times 10^{-4}$/year and is used in the turbine missile analysis to satisfy the Regulatory Guide 1.115 requirement.

3.5.1.3.5.3 Probability of Missile Strike (P2). Calculation of P_2: Neglecting the effect of air resistance, a missile trajectory is determined by the initial ejection vector from the turbine casing. The direction of the ejection vector is defined by two angles: ϕ, which is measured about the turbine axis, and ψ, which is measured from the plane normal to the turbine axis. The magnitude of the ejection vector is V, the ejection velocity from the casing. Functions must be specified, $P(\phi)$, $P(\psi)$, and $P(V)$, which determine the distribution of the missile ejection probability over the range of three variables.

The ejection probability distribution $P(\phi)$ is assumed to be uniform over the 360° arc about the turbine axis:

$$P(\phi)\, d\phi = \frac{d\phi}{2\pi} \qquad (2)$$

The probability distribution $P(\psi)$ is considered to be uniform within some specified angular limits:

$$P(\psi)\, d\psi = \frac{d\psi}{\psi\max - \psi\min},\ \psi \min < \psi < \psi \max \qquad (3)$$

The limits ψ_{min} to ψ_{max} are typically -5° to +5° for missiles ejected from the interior turbine wheel and -25° to +25° for missiles ejected from the end turbine wheels.

The ejection probability distribution P(V) is normally assumed to be uniform over the specified range of ejection velocities V_{min} to V_{max}:

$$P(V)dV = \frac{dV}{V\max - V\min},\ V_{min} < V < V_{max} \qquad (4)$$

The principle of the P_2 calculation is to determine, using the basic equations of missile ballistics, the ranges of the variables ϕ, ψ, and V which determine trajectories intersecting the specified target structure. The strike probability is determined by integrating the product of the three ejection probability distributions over the ranges of the variables corresponding to target strike:

$$P_2 = \int_{\phi_1}^{\phi_2} \int_{\psi_1^{(\phi)}}^{\psi_2^{(\phi)}} \int_{V_{1(\phi,\psi)}}^{V_{2(\phi,\psi)}} P(\phi)\, P(\psi)\, P\,(V)\, dV\, d\psi \pi d\phi \qquad (5)$$

The integral is evaluated by the Bechtel computer code TURMIS (turbine missile). Discrete ejection directions are evaluated by first specifying a ϕ_i, for which the limits ψ_1 (ϕ_i) and ψ_2 (ϕ_i) may be computed corresponding to target strike. Discrete values ψ_j within the range $\psi_i(\phi_i)$ to $\psi_2(\phi_i)$ are then specified. Given the values ϕ_i and ψ_j, the limits $V_i(\phi_i, \psi_j)$ and $V_2(\phi_i, \psi_j)$ corresponding to target strike are then computed, and the integral over V may be evaluated analytically. The range of velocity V_{min} to V_{max} is illustrated in figure 3.5.1-2.

3.5.1.3.5.4 Probability of Damage to Target Structure (P_3). Missiles striking a target structure may cause damage to safety-related systems contained inside. The damage incurred may be due to missile penetration or spalling of concrete fragments from the interior surface of the structures. The Ballistic Research Laboratory equations with a safety factor of 1.2 have been adopted to determine whether or not spalling or perforation will occur upon missile impact of a specified concrete slab to ensure that the safety-related functions of the systems are not impaired.

3.5.1.3.5.5 Analytical Results. Calculation of P_4: The value of P_4 for a particular target structure and a particular turbine failure mode is taken as $P_1 \times P_2 \times P_3$ for the worst missile.

The P_4 value for the plant is determined by summation of P_4 values corresponding to the critical failure mode for all targets on the plant site. Table 3.5.1-4 lists missile targets, strike probabilities, and turbine missile damage probabilities, in case the turbine in Unit 2 fails. Table 3.5.1-4 shows the P_4 values, which include both the high- and low-trajectory missiles (0.40×10^{-7} versus 1.0×10^{-7} allowable).

3.5.1.3.6 Turbine Overspeed Protection

A description of the turbine overspeed protection system, in terms of redundancy, diversity, component reliability, and testing procedures, is provided in subsection 10.2.2.

3.5.1.3.7 Turbine Valve Testing

A discussion of the turbine valve testing is provided in subsection 10.2.3.

3.5.1.3.8 Turbine Characteristics

Turbine data pertinent to the evaluation of its failure characteristics, including a description of its overall configuration, major components (e.g., steam valves, reheaters, etc.), rotor materials and their properties, steam environment (e.g., pressure, temperature, quality, chemistry), and other appropriate properties, are provided in section 10.2. Turbine operational and transient characteristics, including turbine startup and trip environments as well as their overspeed parameters, also are provided in section 10.2.

3.5.1.4 Missiles Generated by Natural Phenomena

The credible missiles at VEGP created by natural phenomena are those generated by tornadoes.

At the construction permit stage, VEGP was required to postulate the tornado missiles described in table 3.5.1-5. VEGP was not required to design to the missile spectra specified in paragraph 3.5.1.4 of the Standard Review Plan dated November 24, 1975. However, to further demonstrate the adequacy of the VEGP design for tornado missiles, the missile characteristics of the steel rod and the utility pole, as specified in the Standard Review Plan (November 24, 1975) for missiles C and F, respectively, are considered. The tornado missiles considered in the VEGP design are provided in table 3.5.1-6.

The methodology used to design the Category 1 structures to provide adequate protection for the safety-related equipment, system, and components is described in appendix 3C.

Safety-related systems and components are protected by missile barriers. The barriers provided are listed in table 3.5.1-7. Where concrete exterior walls and roofs are used as barriers to offer missile protection, such walls have a 24-in. minimum thickness, while the roofs are at least 21 in. thick. The concrete has a 28-day compressive strength of at least 4000 psi (91-day strength for concrete containing pozzolan). Where the interior walls and slabs having concrete compressive strength of 5000 psi are used as missile barriers, such walls have an 18-in. minimum thickness, while the slabs are at least 14 in. thick.

3.5.1.5 Missiles Generated by Events Near the Site

As described in subsection 2.2.3, there are no credible site proximity missiles created by events near the site.

3.5.1.6 Aircraft Hazards

There are no airports or airport approaches within 10 miles of the site; there are no airways within 2 miles of the site. For the airports greater than 10 miles from the site, none has projected operations per year greater than 1000 d^2 movements, where d is the distance in miles from the site. Available military aerial navigation charts for Fort Gordon (U.S. Army)show no low-level flight or landing patterns near the plant. Thus, there are no credible aircraft hazards to the VEGP site.

3.5.1.7 Gravity-Generated Missiles

The occurrence of falling objects as a result of seismic events is prevented by adequately supporting equipment in areas where the possibility of interaction exists. The occurrence of falling objects as a result of the failure of a crane or hoist is discussed in subsections 9.1.4 and 9.1.5.

3.5.1.8 Standard Review Plan Evaluation

The tornado missile spectrum used for VEGP differs from that of the Standard Review Plan.

At the construction permit stage, VEGP was required to postulate the tornado missiles described in table 3.5.1-5. VEGP was not required to design to the missile spectra specified in paragraph 3.5.1.4 of the Standard Review Plan dated November 24, 1975. However, to further demonstrate the adequacy of the VEGP design for tornado missiles, the missile characteristics of the steel rod and the utility pole, as specified in the Standard Review Plan (November 24,

1975) for missiles C and F, respectively, are considered. The tornado missiles considered in the VEGP design are provided in table 3.5.1-6.

3.5.1.9 References

1. Kilsby, E. R. Jr., "Reactor Primary - Piping-System Rupture Studies," Nuclear Safety, Vol 7, Winter 1965-1966, p 185.
2. Downs, J. E., "Hypothetical Turbine Missiles – Probability of Occurrence," General Electric Company Memo Report, March 14, 1973. Data cited applies to 43-inch last stage blading.
3. General Electric, "Probability of Missile Generation in General Electric Nuclear Turbines," January 1984. Proprietary Document.
4. General Electric, "Probability of Missile Generation in General Electric Nuclear Turbines, Supplementary Report: Steam Valve Surveillance Test Interval Extension," September 1993. Proprietary Document.
5. General Electric, “Control System Upgrade Impact on the Probability of Turbine Missile Generation,” drawing number AX5AA11-00029.

3.5.2 STRUCTURES, SYSTEMS, AND COMPONENTS TO BE PROTECTED FROM EXTERNALLY GENERATED MISSILES

3.5.2.1 General

The sources of missiles which, if generated, could affect the safety of the plant are considered in subsection 3.5.1. Safety-related structures, systems, and components are designed to withstand the impact of postulated missiles, are physically separated from the source of missiles, or are protected by a missile barrier.

3.5.2.2 Missile Barriers Within Containment

The secondary shield walls, the refueling canal walls, the various structural beams, and the operating floor act as missile barriers separating reactor coolant loops from other protected components and missile sources. These barriers also protect the reactor coolant pressure boundary (RCPB) in each loop from those identified missiles generated elsewhere in the containment building while protecting the RCPB in each loop from externally generated missiles. The feedwater system is routed so that it is not affected by potential missiles.

Except for short piping runs in the safety injection system (SIS), which must supply cooling water to the reactor coolant system after a loss of coolant accident, the emergency safety features are located outside the secondary shield. The SIS lines which penetrate the secondary shield do so in the vicinity of the loop segment to which they are attached.

A missile shield structure is provided over the control rod drive mechanisms (CRDMs) to block any identified missiles generated in that location. The design of the missile shield is discussed in subsection 3.5.3. The control rod drives are protected from horizontal missiles by the refueling canal walls that extend vertically above the CRDMs. The head vent and letdown system piping is the only high-energy piping located close to the CRDMs. (No potential missile sources exist in the system.) A roof slab is provided to protect against identified missiles that originate in the region where the pressurizer extends above the operating floor.

Missile barriers are provided, as required, to prevent missiles generated by the failure of main steam or feedwater components inside the containment from causing loss of integrity to the containment liner, isolation system, or steam system associated with another steam generator, or from causing loss of function to other required systems or components inside the containment in accordance with the missile protection design criteria previously listed in subsection 3.5.1.

3.5.2.3 Barriers for Missiles Generated Outside of Plant Structures

The protective structures, shields, and missile barriers designed to provide protection against identified missiles generated outside these structures, shields, and missile barriers are listed in table 3.5.1-6. The missile barriers listed are designed for the tornado and accident missiles described in subsection 3.5.1, utilizing the procedures stated in subsection 3.5.3.

3.5.2.4 Missile Barriers Within Plant Structures Other Than Containment

Missile barriers are provided within plant structures outside the containment in conformance with the missile protection design criteria discussed in section 3.5. For the pressurized and rotating component failure missiles that originate outside the containment, identified in subsection 3.5.1, the following steps are taken to assure that the missile protection design criteria are met.

A. Missiles are categorized according to the system in which they originate.

B. The components that must be protected from a missile are identified in accordance with the missile protection design criteria given in subsection 3.5.1.

3.5.3 BARRIER DESIGN PROCEDURES

Missile barriers and protective structures are designed to withstand and absorb missile impact loads in order to prevent damage to safety-related components.

With the exception of the nuclear service cooling water (NSCW) tower fan cells, main steam safety valve exhausts, atmospheric relief valve (loop 2), atmospheric relief valve exhaust stacks, turbine-driven auxiliary feedwater pump exhaust, and condensate storage tank vents, protection of essential safety-related systems or components against tornado missiles that could enter through any openings in the exterior walls or roofs of Category 1 structures is provided as follows.

Barriers are provided for the openings to ensure that safety-related systems and components are protected from postulated credible missile impact. The following design features provide missile protection:

1. Missile proof doors.

2. Steel plate missile shields.

3. Concrete missile shields.

4. For small openings, the space within the opening being sufficiently occupied by piping, pipe support, or other substantial intervening commodities.

Interior walls and slabs are treated as barriers for systems or components located in the interior rooms, if missile protection is not provided by exterior walls and slabs or other barriers. Any openings in the exterior walls or slabs and the interior walls or slabs that may be credible paths for missile entry are investigated to ensure that the appropriate level of localized protection is provided if necessary.

The NSCW towers are inherently protected against direct horizontal missiles by the towers' concrete construction. The minimum height a missile would have to obtain to enter the cooling tower vertically and strike a fan is approximately 45 ft above grade, which eliminates heavier missiles (such as a utility pole or automobile) from consideration. Since each fan has its own opening, a single missile can damage only a single fan. A detailed probabilistic study was performed to determine the risk of the NSCW towers not being available during and following a tornado. This study demonstrated that even with all incorporated conservatisms the frequency of tornado missiles disabling the NSCW system (loss of one tower for maintenance, and missiles disabling two or more fans in the single operating tower) is lower than the acceptance criterion of 10^{-7} per year given in Standard Review Plan Section 2.2.3. Therefore, additional tornado missile protection is not required for the NSCW tower fans.

The NRC approved a license amendment for VEGP that authorized use of the Tornado Missile Risk Evaluator (TMRE) methodology.[1,2] TMRE is a risk-informed methodology for identifying and evaluating the safety significance associated with structures, systems, and components (SSCs) that are exposed to potential tornado-generated missiles and demonstrating compliance with tornado missile protection requirements if the importance to safety is sufficiently low. The main steam safety valve exhausts, atmospheric relief valve (loop 2), atmospheric relief valve exhaust stacks, turbine-driven auxiliary feedwater pump exhaust, and condensate storage tank vents were evaluated using TMRE. The TMRE evaluation demonstrates that tornado missile protection is not required for the main steam safety valve exhausts, atmospheric relief valve (loop 2), atmospheric relief valve exhaust stacks, turbine-driven auxiliary feedwater pump exhaust, and condensate storage tank vents.

The procedures by which each structure or barrier is designed to resist the tornado missile hazards described in paragraph 3.5.1.4 are presented in appendix 3C. Appendix 3C is also applicable to the other missile hazards of subsection 3.5.1, provided such missiles display parameters similar to those created by tornadoes. For those possible missiles not similar to tornado-borne missiles, the design is accomplished using similar principles.

In general, Westinghouse-supplied equipment is not designed to withstand the impact of postulated missiles; therefore, Bechtel has considered the effects of postulated missiles and provided the necessary protection to safety-related components as determined by the design bases provided in subsection 3.5.1.

The exception is the control rod drive mechanism (CRDM) missile shield, which is supplied by Westinghouse as part of the integrated head.

A missile shield structure is provided over the CRDMs to block missiles that might be associated with a fracture of the pressure housing of any mechanism. This missile shield is a reinforced steel structure attached to the reactor vessel head and located above the CRDMs. Each CRDM housing is terminated with a small tapered pin that penetrates the missile shield through a slightly larger diameter hole to direct the ejected CRDM missile into the shield. This prevents any missile from missing or ricocheting from the shield to strike the containment liner or other CRDMs.

Missile shield penetrations are given in tables 3.5.1-2 and 3.5.1-3 using the Ballistic Research Laboratories formula for steel. The steel missile shield has an effective thickness of approximately 3 in.

For the case of housing plug and drive shaft impact, which is the design case, it is assumed that the plug partially perforates the missile shield. The drive shaft then hits the plug and further penetrates the steel missile shield. The resultant penetration into the shield is 0.773 in.; therefore, the effective thickness of the steel missile shield is more than three times the combined penetration for the design case.

The CRDM missile shield is also designed to withstand the dynamic impact loads due to the missile and the water jet.

3.5.3.1 References

1. Letter from NRC to SNC, "Vogtle Electric Generating Plant, Units 1 and 2 Regarding Issuance of Amendments," ML18304A394, January 11, 2019.
2. NEI 17-02, Rev. 1A, "Tornado Missile Risk Evaluator (TMRE) Industry Guidance Document," July 2018.

TABLE 3.5.1-1 (SHEET 1 OF 5)

INTERNALLY GENERATED MISSILES OUTSIDE CONTAINMENT ROTATING COMPONENT FAILURES[(c)]

Missile Identification	Source of Missile	Location[(d)]	Missile Characteristics: Velocity (ft/s)	Missile Characteristics: Equiv. Dia. (in.)	Missile Characteristics: Mass (lbm)	Calculated Maximum Steel Perforation Depth (in.)	Casing Thickness (in.)	Casing Perforation	Missile Residual Velocity After Casing Perforation	Calculated Thickness of Surrounding Material to Prevent: Concrete Spalling (in.)	Calculated Thickness of Surrounding Material to Prevent: Steel Perforation (in.)	Missile Protection Provided
Impeller	CCW drain tank pump	Aux. bldg. level D room 75	107.0	5.65	6.9	0.17	0.25	No	None	None	None	None
Fan blade	Air handling unit cooling	Aux. bldg. level D	91.8	0.43	0.22	0.033	0.049	No	None	None	None	None
Fan blade	Air handling unit cooling	Aux. bldg. level D room 79	106.9	0.62	0.36	0.038	0.111	No	None	None	None	None
Fan blade	Air handling unit cooling	Aux. bldg. level D room 128/ 130	54	0.16	0.42	0.0143	0.041	No	None	None	None	None
Impeller	CVCS cent. charging pump train A	Aux. bldg. level C room 115	144	2.25	32.72	0.32	0.35	No	None	None	None	None
Impeller	CVCS cent. charging pump train B	Aux. bldg. level C room 118	144	2.25	32.72	0.32	0.35	No	None	None	None	None
Impeller	CVCS cent. normal charging pump	Aux. bldg. level C room 111	128	3.66	10.00	0.08	3.18	No	None	None	None	None
Fan blade	SGBD heat exchanger room cooler fan	Aux. bldg. level C room 108/125	98	0.86	0.20	0.017	0.048	No	None	None	None	None

TABLE 3.5.1-1 (SHEET 2 OF 5)

Missile Identification	Source of Missile	Location[(d)]	Velocity (ft/s)	Missile Characteristics Equiv. Dia. (in.)	Missile Characteristics Mass (lbm)	Calculated Maximum Steel Perforation Depth (in.)	Casing Thickness (in.)	Casing Perforation	Missile Residual Velocity After Casing Perforation	Calculated Thickness of Surrounding Material to Prevent Concrete Spalling (in.)	Calculated Thickness of Surrounding Material to Prevent Steel Perforation (in.)	Missile Protection Provided
Fan blade	Air handling unit train A	Aux. bldg. level C room UC-C14	105	0.62	0.36	0.038	0.111	No	None	None	None	None
Fan blade	Air handling unit train B	Aux. bldg. level C room UC-C14	105	0.62	0.36	0.038	0.111	No	None	None	None	None
Impeller	SGB drain pump	Aux. bldg. level C room 108	112	1.75	1.6	0.19	0.25	No	None	None	None	None
Impeller	Boron inj. recirc. pump train A	Aux. bldg. level B room R-B05	65.5	0.365	1	0.07	0.375	No	None	None	None	None
Impeller	Boron inj. recirc. pump train B	Aux. bldg. level B room R-B06	65.5	0.365	1	0.07	0.375	No	None	None	None	None
Fan blade	Air handling unit train B	Aux. bldg. level B room R-B17	44.8	0.16	0.042	0.014	0.041	No	None	None	None	None
Fan blade	Air handling unit	Aux. bldg. room R-B13	44.8	0.16	0.042	0.014	0.041	No	None	None	None	None
Impeller	Aux. comp. cooling water pump	Aux. bldg. level B room R-B23	98	1.14	75	0.65	0.75	No	None	None	None	None

TABLE 3.5.1-1 (SHEET 3 OF 5)

Missile Identification	Source of Missile	Location[d]	Missile Characteristics Velocity (ft/s)	Equiv. Dia. (in.)	Mass (lbm)	Calculated Maximum Steel Perforation Depth (in.)	Casing Thickness (in.)	Casing Perforation	Missile Residual Velocity After Casing Perforation	Calculated Thickness of Surrounding Material to Prevent Concrete Spalling (in.)	Steel Perforation (in.)	Missile Protection Provided
Impeller	Aux. comp. cooling water pump	Aux. bldg. level B room R-B24	98	1.14	75	0.65	0.75	No	None	None	None	None
Fan blade	Air handling unit	Aux. bldg. level A room R-A03/ R-A05	106.9	0.62	0.36	0.038	0.111	No	None	None	None	None
Impeller	Refueling water purification pump	Aux. bldg. level A room A40A	85.8	0.888	3.09	0.08	0.375	No	None	None	None	None
Fan blade	Air handling unit train A	Aux. bldg. level A room R-A53	96.7	0.68	0.60	0.046	0.049	No	None	None	None	None
Impeller	Spent fuel pool skimmer pump	Aux. bldg. level A room R-A53	137.31	2.925	7.635	0.087	0.3125	No	None	None	None	None
Impeller	Spent fuel pool pump train A	Aux. bldg. level A room A-53	77.7	0.98	16.5	0.2	0.25	No	None	None	None	None
Fan blade	MCC room cooler train A	Aux. bldg. level 1 room 116, 118	44.2	16	0.42	0.013	0.041	No	None	None	None	None
Fan blade	Rail corr. ac unit[e]	Aux. bldg. level 1	101	1.83	4.5	0.065	0.078	No	None	None	None	None
Fan blade	AB cont. exhaust units	Aux. bldg. level 2 room 212	254	1.58	4.44	0.255	0.1875	Yes	154.2	1.26[a]	0.131[b]	Yes[f]

TABLE 3.5.1-1 (SHEET 4 OF 5)

Missile Identification	Source of Missile	Location[d]	Velocity (ft/s)	Missile Characteristics Equiv. Dia. (in.)	Mass (lbm)	Calculated Maximum Steel Perforation Depth (in.)	Casing Thickness (in.)	Casing Perforation	Missile Residual Velocity After Casing Perforation	Calculated Thickness of Surrounding Material to Prevent Concrete Spalling (in.)	Steel Perforation (in.)	Missile Protection Provided
Fan blade	AB cont. exhaust units	Aux. bldg. level 2 room 221 (Unit 2)	263	1.58	4.44	0.267	0.1875	Yes	168.6	1.42[a]	0.147[b]	Yes[f]
Fan blade	Elect. swgr and MCC room cooler	Aux. bldg. level 2 room 212	109.5	0.591	0.242	0.032	0.028	Yes	46	0.081[a]	0.01[b]	None
Impeller	Turbine-driven pump	Aux. feedwater pumphouse room 106	150.0	0.67	21	0.84	2.0	No	None	None	None	None
Impeller	Motor-driven pumps	Aux. feedwater pumphouse room 101/ 102	164.85	0.513	21	1.26	1.44	No	None	None	None	None
Turbine disk	Steam turbine	Aux. feedwater pumphouse room 106	297.8	3.7	30	0.48	0.812	No	None	None	None	None
Impeller	Fan - a/c unit [e]	Control building room 226 relay room	49.7	0.164	0.021	0.008	0.041	No	None	None	None	None
Impeller	Centrifugal fan-filter unit [e]	Control building room 248 (HVAC)	222.916	1.066	1.532	0.157	0.165	No	None	None	None	None
Fan blade	Normal exhaust unit [e]	Fuel handling bldg. room 301	209.6	2.02	6	0.188	0.375	No	None	None	None	None

TABLE 3.5.1-1 (SHEET 5 OF 5)

Missile Identification	Source of Missile	Location[d]	Velocity (ft/s)	Missile Characteristics Equiv. Dia. (in.)	Missile Characteristics Mass (lbm)	Calculated Maximum Steel Perforation Depth (in.)	Casing Thickness (in.)	Casing Perforation	Missile Residual Velocity After Casing Perforation	Calculated Thickness of Surrounding Material to Prevent Concrete Spalling (in.)	Calculated Thickness of Surrounding Material to Prevent Steel Perforation (in.)	Missile Protection Provided
Fan blade	Elevator room chiller unit	Fuel handling building room 127 railroad corridor	66	0.71	1.04	0.04	0.049	No	None	None	None	None

a. Missile protection is provided by the structural concrete.

b. Thickness of the piping prevents failure.

c. This table lists the rotating component failures for which a specific casing perforation calculation was performed.

d. The locations provided are for unit 1. The unit 2 locations are similar.

e. The missile source is common to both units.

f. Missile protection is provided for essential chilled water lines 1/2-1592-109-4, 1-1592-054-4, and 2-1592-110-4.

TABLE 3.5.1-2

INTERNALLY GENERATED MISSILES INSIDE CONTAINMENT
ROTATING COMPONENT FAILURES[b]

Postulated Missile	Weight (lb)	Thrust$_2$ Area (in.)	Impact$_2$ Area (in.)	Impact Velocity (ft/s)	Kinetic Energy (ft-lb)	Protection Provided
CRDM housing plug	50	4.91	0.87	40	1242	Integrated head missile shield
Control rod drive shaft	165	2.40	3.56	100	25,620	
Control rod drive shaft and mechanism	1610	12.57	1.37	12	3600	
Reactor coolant drain tank pump impeller	2.55	-	-	209.6	-	None[a] Casing is not perforated
Containment preaccess filter unit fan blade	6	-	-	118	-	None[a] Casing is not perforated

a. The analysis performed on these rotating components is similar to the analysis presented in table 3.5.1-1.

b. This table lists the rotating component failures for which a specific calculation was performed.

TABLE 3.5.1-3 (SHEET 1 OF 2)

INTERNALLY GENERATED MISSILES INSIDE CONTAINMENT
PRESSURIZED COMPONENT FAILURES

		Valve Missile Characteristics				
Missile Identification	Weight (lb)	Flow Discharge Area (in.^{2})	Thrust Area (in.^{2})	Impact Area (in.^{2})	Velocity (ft/s)	Protection Provided
Safety-relief valve bonnet	350	2.86	80	24	110	3 ft 0 in. concrete pressurizer roof slab
3-in. motor-operated isolation valve bonnet (plus motor and stem)	400	5.5	113	28.3	135	3 ft 0 in. concrete pressurizer roof slab
2-in. air-operated valve bonnet (plus stem)	75	1.8	20.7	20	115	3 ft 0 in. concrete pressurizer roof slab
3-in. air-operated spray valve bonnet (plus stem)	120	5.5	50.3	50	190	3 ft 0 in. concrete pressurizer roof slab
4-in. air-operated spray valve bonnet (plus stem)	200	9.3	50.3	50.0	190	3 ft 0 in. concrete pressurizer roof slab

TABLE 3.5.1-3 (SHEET 2 OF 2)

Piping Temperature Element Assembly Missile Characteristics

Characteristics	Without Well	With Well
For a tear around the weld between the boss and the pipe		
Flow discharge area (in.^{2})	0.11	0.60
Thrust area (in.^{2})	7.1	9.6
Missile weight (lb)	11.0	15.2
Area of impact (in.^{2})	3.14	3.14
(psi)	3.15	4.84
Velocity (ft/s)	20	120
For a tear at the junction between the temperature element assembly and the boss for the without-well element and at the junction between the boss and the well for the with-well element		
Flow discharge area (in.^{2})	0.11	0.60
Thrust area (in.^{2})	3.14	3.14
Missile weight (lb)	4.0	6.1
Area of impact (in.^{2})	3.14	3.14
Weight to impact area ratio (psi)	1.27	1.94
Velocity (ft/s)	75	120

Characteristics of Other Missiles Postulated Within Reactor Containment

	Reactor Coolant Pump Temperature Element	Instrument Well of Pressurizer	Pressurizer Heaters
Weight (lb)	0.25	5.5	15
Discharge area (in.^{2})	0.50	0.442	0.61
Thrust area (in.^{2})	0.50	1.35	2.4
Impact area (in.^{2})	0.50	1.35	2.4
Velocity (ft/s)	260	100	55

TABLE 3.5.1-4 (SHEET 1 OF 2)

TURBINE MISSILE STRIKE AND DAMAGE PROBABILITIES
PER MISSILE FRAGMENT FROM UNIT 2

I. Turbine Missile Genesis Probability (P_1) = 1 x 10^{-4} per year.

II. Missile Damage Probabilities (P_4) per year.

Location Unit	Target Building or Structure	Strike and Damage Probabilities (P_2) x (P_3) x 10^{-3} per year	Missile Damage Probabilities (P_4) x (10^{-7}) per year
2	Containment	0.130	0.130
1 & 2	Control building	0.167	0.167
2	Main steam valve room (north)	0.005	0.005
1 & 2	Fuel handling building	0.080	0.080
1 & 2	Auxiliary building	0.180	0.180
2	Main steam valve room (south)	0.012	0.012
2	Comp. cooling heat exch. A & B	0.037	0.037
2	Condenser storage tank	0	0
2	Condenser storage tank	0	0
2	Auxiliary feedwater line	0.020	0.020
2	Refueling water storage tank	0	0
2	Reactor makeup water storage tank	0	0

TABLE 3.5.1-4 (SHEET 2 OF 2)

Location Unit	Target Building or Structure	Strike and Damage Probabilities $(P_2) \times (P_3) \times 10^{-3}$ per year	Missile Damage Probabilities $(P_4) \times (10^{-7})$ per year
2	Diesel generator building	0.123	0.123
2	Auxiliary feedwater pumphouse	0.035	0.035
1	Containment	0	0
1	Main steam valve room (north)	0.005	0.005
1	Main steam valve room (south)	0.010	0.010
1	Comp. cooling heat exchange.	0.032	0.032
1	Condenser storage tank	0	0
1	Condenser storage tank	0	0
1	Auxiliary feedwater line	0.012	0.012

TABLE 3.5.1-5

TORNADO MISSILES POSTULATED
AT THE CONSTRUCTION PERMIT STAGE

Missile	Weight (lb)	Design Missile Velocity (ft/s)	Height at Which Attained (ft)
Wooden plank, 4 in. x 12 in. x 12 ft	200.0	200	216
Steel pipe, 3-in. diameter, schedule 40, 10 ft long	78.5	200	212
Steel rod, 1-in. diameter, 3 ft long	8.0	160	114
Steel pipe, 6-in. diameter, schedule 40, 15 ft long	285.0	160	101
Steel pipe, 12-in. diameter, schedule 40, 15 ft long	744.0	150	46
Utility pole, 13-1/2 in. diameter, 35 ft long	1490.0	100	Ground level
Automobile, frontal area 20 ft^2	4000.0	75	Ground level

TABLE 3.5.1-6

TORNADO MISSILES CONSIDERED
IN THE VEGP DESIGN

Description of Missile	Weight (lb)	Height Limit (ft)	End-On Horizontal Velocity (ft/s)	End-On Vertical Velocity (ft/s)
Wooden plank, 4 in. x 12 in. x 12 ft	200	216	200	160
Steel pipe, 3-in. diameter, schedule 40, 10 ft long	78.5	212	200	160
Steel rod, 1-in. diameter, 3 ft long	8	Unlimited	317	254
Steel pipe, 6-in. diameter, schedule 40, 15 ft long	285	101	160	128
Steel pipe, 12-in. diameter, schedule 40, 15 ft long	744	46	150	120
Utility pole, 13-1/2-in. diameter, 35 ft long	1490	30[(a)]	211	169
Automobile, frontal area 20 ft^2	4000	Ground Level	75	60

a. To 30 ft above all grade levels within 1/2 mile of facility structures.

TABLE 3.5.1-7 (SHEET 1 OF 3)

PROTECTED SYSTEM AND COMPONENT BARRIERS AGAINST EXTERNALLY GENERATED MISSILES

Protected Systems and Components	Missile Barrier	Minimum Concrete Thickness (in.) Walls	Roof	Floor	Design Concrete Strength (psi)
Reactor equipment, reactor coolant system, containment piping and valves, containment electrical, instrumentation, and control systems and containment engineered safety features actuation systems, transfer tube	Containment shell and dome	45	45	-	6000
	Containment basemat	-	-	126	5000
	Internal structures				
	Primary shield wall	Varies 102 min			5000
	Secondary shield wall	36	-	-	5000
	Floor at el 220 ft	-	-	24	5000
Penetrations in containment shell	Missile shield wall/ building walls	24	21	-	5000
Control room and protected electrical, instrumentation control, and ventilation equipment in control building	Control building	24	21	-	4000
Safety injection, containment spray cooling water, ventilation, electrical, instrumentation and control equipment	Auxiliary building	24	21	-	5000
Reactor auxiliaries, e.g., chemical and volume control system, boric acid storage tanks and transfer pumps, and CCW System	Auxiliary building	24[a]	21	-	5000
Spent fuel pool, transfer tube	Fuel handling building	24	21	-	4000
	Fuel pool walls	60	-	-	4000

TABLE 3.5.1-7 (SHEET 2 OF 3)

Protected Systems and Components	Missile Barrier	Minimum Concrete Thickness (in.) Walls	Roof	Floor	Design Concrete Strength (psi)
Diesel generators, diesel generator fuel oil system, combustion air intake	Diesel generator building	24	21	-	4000
Diesel generator combustion air exhaust	Concrete barrier (horizontal missile) Steel Plate (vertical missile)	24		-	6000
Diesel fuel storage tank, diesel fuel transfer pumps and pump motors	Diesel fuel storage tank pumphouse	24	21	-	4000
Main steam line isolation valves	Auxiliary and control building main steam valve rooms	24	21	-	5000-auxiliary building 4000-control building
Category 1 water storage tanks	Cylindrical walls and sloping roof	24	21	-	4000
Category 1 water storage pumps, valves, and piping	Enclosures adjoining tanks	24	21	-	4000
Nuclear service cooling water tower fan motors	Enclosures	24	21	-	4000
Nuclear service cooling water pumps	Nuclear service cooling tower valve houses	24	21	-	4000
Auxiliary feedwater pumps, motors, valves, and piping	Auxiliary feedwater pumphouse	24	21	-	4000
Category 1 piping and electrical cables	Category 1 tunnels, or buried a minimum 6 ft backfill cover or 21 in. concrete cover or a total 6 ft combination of backfill and concrete	24[b]	21[b]	-	4000
Auxiliary building HVAC intakes and exhausts	Auxiliary building	24	21	-	5000

TABLE 3.5.1-7 (SHEET 3 OF 3)

Protected Systems and Components	Missile Barrier	Minimum Concrete Thickness (in.) Walls	Roof	Floor	Design Concrete Strength (psi)
Control building HVAC intakes and exhausts	Control building	24	21	-	4000
Fuel handling building HVAC intakes and exhausts	Fuel handling building	24	21	-	4000
Diesel generator building HVAC intakes and exhausts	Diesel generator building	24	21	-	4000
Auxiliary feedwater pumphouse HVAC intakes and exhausts	Auxiliary feedwater pumphouse	24	21	-	4000
Auxiliary feedwater piping	Control building main steam valve room entrance	24	21	-	4000

a. Interior walls and roof slabs credited with providing missile protection each have a minimum concrete thickness of 18 in. and 14 in. respectively and design strength of 5000 psi.

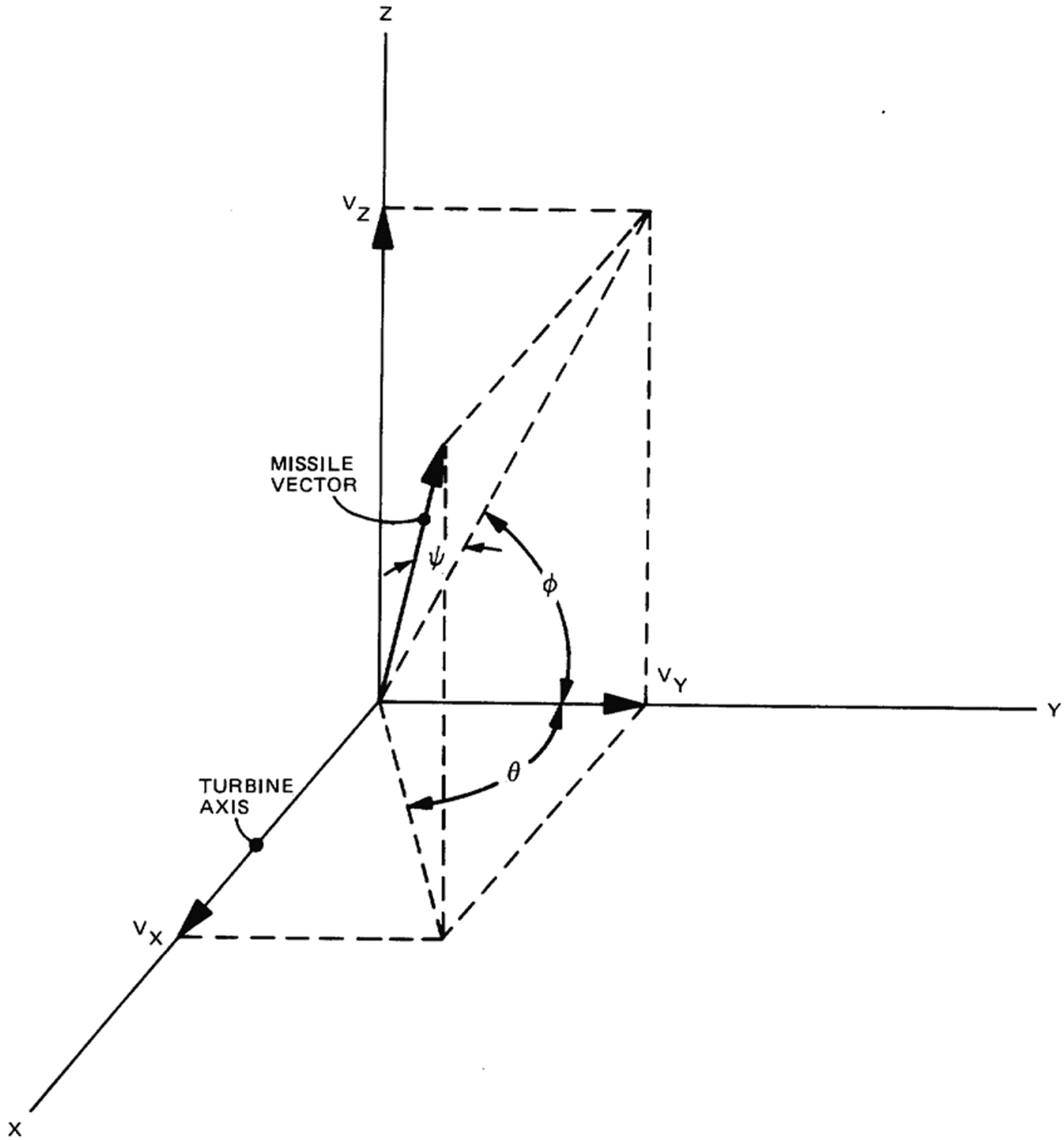
b. Category 1 piping and electrical cables may be embedded within the specified minimum thickness provided the local effects have been demonstrated to verify the component is adequately protected.

c. As discussed in subsection 3.5.3, the NRC approved TMRE, a risk-informed methodology to determine whether tornado missile protection is required. The SSCs where the TMRE methodology demonstrated that tornado missile protection is not required are described in subsection 3.5.3.

TABLE 3.5.1-8

INTERNAL ROTATING COMPONENTS EXCLUDED
AS MISSILE GENERATION SOURCES

TAG NUMBER	DESCRIPTION	LOCATION
1301-P4-010 through 013	Wet Layup Recirculation Pump	Containment
1516-B7-001 & -002	Containment Post LOCA Cavity Purge Fans	Containment
2203-P6-001	Fuel Transfer System Hydraulic Pump	Containment
1204-P6-003 & -004	Safety Injection Pumps	Auxiliary Bldg
1206-P6-001 & -002	Containment Spray Pumps	Auxiliary Bldg
1208-P6-006 & -007	Boric Acid Transfer Pumps	Auxiliary Bldg
1555-A7-015 & -016	Safety Injection Pump Room Cooler Fans	Auxiliary Bldg
1561-N7-001 & -002	Piping Penetration Filtration Unit Fans	Auxiliary Bldg
None Provided	Safety Injection Lube Oil Pump	Auxiliary Bldg
2203-P6-002	Fuel Transfer System Hydraulic Pump	Fuel Handling Bldg
2403-G4-001-P27& -002-P27	Jacket Water Chemical Addition Pump	Diesel Generator Bldg



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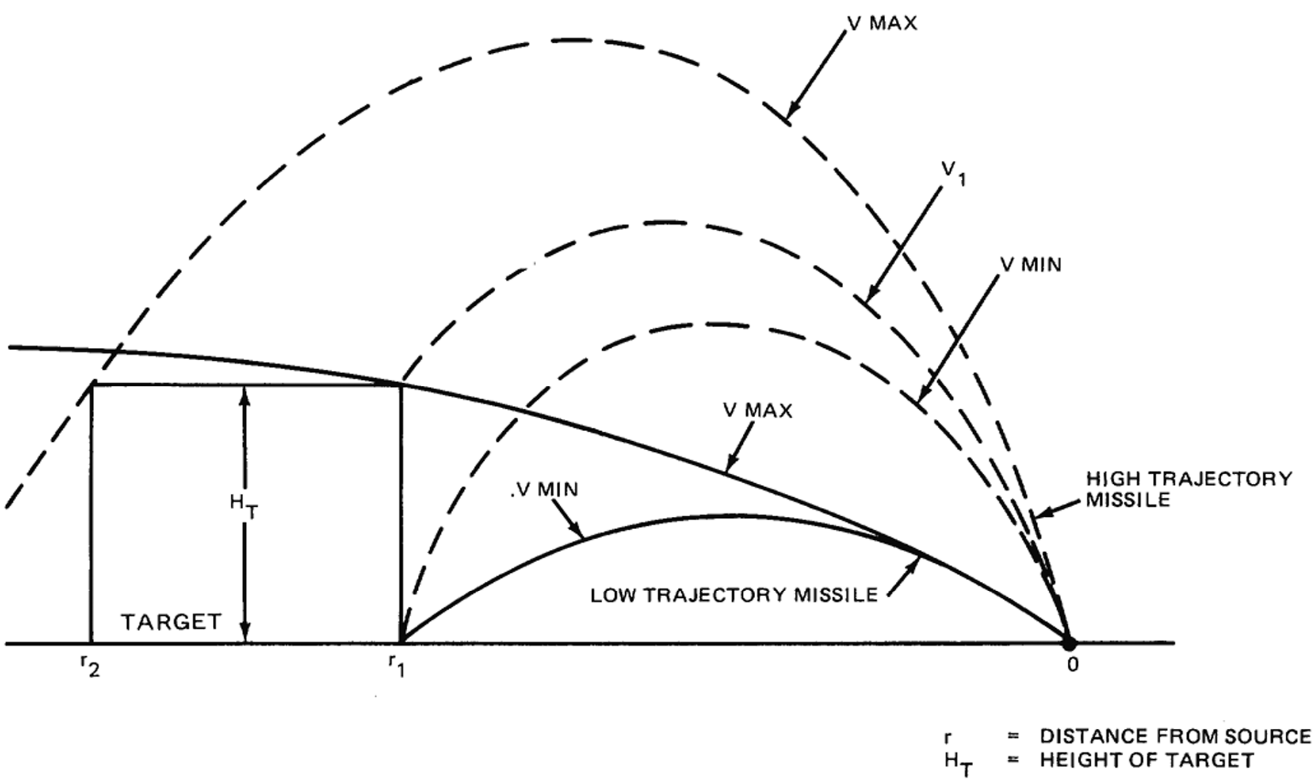
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

MISSILE EJECTION ANGLES

FIGURE 3.5.1–1



REV 14 10/07

SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	MISSLE TRAJECTORIES FIGURE 3.5.1–2

3.6 PROTECTION AGAINST THE DYNAMIC EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING

Pipe failure protection is provided in accordance with the requirements of 10 CFR 50, Appendix A, General Design Criterion 4.

Postulated breaks in the reactor coolant loop (RCL), except for branch line connections, have been eliminated for both Unit 1 and Unit 2 by reference 3. Subsequent to the General Design Criterion 4 final rule change (52 FR 41288, October 27, 1987), postulated breaks in the RCL branch lines (pressurizer surge line, accumulator line, and residual heat removal (RHR) line for Unit 2 and the pressurizer surge line for Unit 1) were eliminated by application of leak-before-break technology as presented in references 4, 5, 6, 9, and 10. Approval of the elimination of breaks in these Unit 2 branch lines is given in the Vogtle Safety Evaluation Report, Supplement 7, dated November 19, 1987. The necessary information supporting the elimination of breaks in the Unit 1 pressurizer surge line was submitted to the NRC via reference 11.

In the event of a high- or moderate-energy pipe failure within the plant, adequate protection is provided to ensure that those essential structures, systems, or components are not impacted by the effects of postulated piping failure. Essential systems and components are those required to shut down the reactor and mitigate the consequences of the postulated piping failure.

Appendix 3F, Hazards Analysis, provides several examples of the evaluations made of the effects of postulated pipe failures within the plant. The following sections provide the bases for selection of the pipe failures, the determination of the resultant effects, and details of the protection requirements.

3.6.1 POSTULATED PIPING FAILURES IN FLUID SYSTEMS INSIDE AND OUTSIDE CONTAINMENT

Table 3.6.1-1 provides a matrix of plant systems that indicates their classification: high-energy, moderate-energy, essential, or nonessential. Selection of pipe failure locations and evaluation of the consequences on nearby essential systems, components, and structures are presented in subsection 3.6.2 and are in accordance with the requirements of 10 CFR 50, Appendix A, General Design Criterion, and Nuclear Regulatory Commission (NRC) Branch Technical Positions ASB 3-1 and MEB 3-1.

For the reactor coolant loop, reference 1 provides the original criteria for postulating breaks in the reactor coolant loop. Subsequent elimination of postulated pipe breaks in the RCL and Class 1 branch lines is discussed above.

3.6.1.1 Design Bases

The following design bases relate to the evaluation of the effects of the pipe failures determined in subsection 3.6.2:

A. The selection of the failure type is based on whether the system is high or moderate energy during normal operating conditions of the system.

   High-energy piping includes those systems or portions of systems in which the maximum normal operating temperature exceeds 200°F or the maximum normal operating pressure exceeds 275 psig.

Piping systems or portions of systems pressurized above atmospheric pressure during normal plant conditions and not identified as high energy are considered moderate energy.

Piping systems that exceed 200°F or 275 psig for about 2 percent or less of the time the system is in operation or that experience high-energy pressures or temperatures for less than 1 percent of the plant operation time are considered moderate energy.

B. The following assumptions are used to determine the thermodynamic state in the piping system for the calculation of fluid reaction forces:

1. For those portions of piping systems normally pressurized during operation at power, the thermodynamic state in the pipe and associated reservoirs are those of full-power operation.

2. For those portions of piping systems only pressurized during other normal plant conditions (e.g., startup, hot standby, reactor cooldown), the thermodynamic state and associated operating condition is determined as the mode giving the highest enthalpy.

C. Moderate-energy pipe cracks are evaluated for spray wetting, flooding, and other environmental effects.

D. Where postulated, each longitudinal or circumferential break in high-energy fluid system piping or leakage crack in moderate-energy fluid system piping is considered separately as a single initial event occurring during normal plant conditions.

E. Offsite power is assumed to be unavailable if an automatic trip of the turbine-generator system or reactor protection system is a direct consequence of the postulated piping failure.

F. A single active component failure is assumed in systems used to mitigate the consequences of the postulated piping failure or to safely shut down the reactor, except as noted in paragraph G below. The single active component failure is assumed to occur in addition to the postulated piping failure and any direct consequences of the piping failure, such as unit trip and loss of offsite power.

G. When the postulated piping failure occurs in one of two or more redundant trains of a dual-purpose, moderate-energy essential system, single failures of components in the other train or trains (and associated supporting train) are not assumed, because the system is designed to Seismic Category 1 standards; powered from both offsite and onsite sources; and constructed, operated, and inspected to quality assurance, testing, and inservice inspection standards appropriate for nuclear safety systems.

H. All available systems, including those actuated by operator actions, are employed to mitigate the consequences of a postulated piping failure to the extent clarified in the following paragraphs:

1. In determining the availability of the systems, account is taken of the postulated failure and its direct consequences, such as unit trip and loss of offsite power, and of the assumed single active component failure and its direct consequences. The feasibility of carrying out operator actions is determined on the basis of ample time and adequate access to

equipment being available for the proposed actions. Although a postulated high/moderate-energy line failure outside the containment may ultimately require a cold shutdown, operation at hot standby is allowed in order for plant personnel to assess the situation and make repairs.

2. The use of non-Seismic Category 1 piping in mitigating the consequence of postulated piping failure outside the containment is clarified in the following paragraphs:

   a. For non-Seismic Category 1 piping failures, it is assumed that a safe shutdown earthquake could be the cause of the failure. Therefore, only Seismic Category 1 equipment can be used to mitigate the consequences of the failure and bring the plant to a safe shutdown.

   b. Category 1 and seismically supported non-Category 1 piping systems located outside the containment are assumed to fail nonmechanistically (i.e., failure is produced by some mechanism other than an earthquake) for the purpose of pipe break hazard analysis. Therefore, non-Category 1 equipment can be used to bring the plant to a safe shutdown following a postulated pipe break event, subject to the power being available to operate such equipment and provided that the radiological consequences are insignificant in comparison to 10 CFR 100 dose guidelines. For example, non-Category 1 equipment may be used in the mitigation of a charging header piping failure, since this event does not cause a unit trip or significant radiological consequences.

I. A whipping pipe is not considered capable of rupturing impacted pipes of equal or greater nominal pipe diameter and equal or greater wall thickness. This is based on the assumption that only piping is determined to do the impacting. A whipping pipe is considered capable of developing a through-wall leakage crack in a pipe of larger nominal pipe size with thinner wall thickness, assuming that only piping is determined to do the impacting. The above criterion is not utilized where the potential exists for valves or other components in the whipping pipe to impact the targets, since these are treated on a case-by-case basis.

J. Pipe whip is assumed to occur in the plane defined by the piping geometry and to cause movement in the direction of the jet reaction.

If unrestrained, a whipping pipe having a constant energy source sufficient to form a plastic hinge is considered to form a plastic hinge and rotate about the nearest rigid pipe whip restraint, anchor, or wall penetration capable of resisting the pipe whip loads. If the direction of the initial pipe movement caused by the thrust force is such that the whipping pipe impacts a flat surface normal to its direction of travel, it is assumed that the pipe comes to rest against that surface, with no pipe whip in other directions.

In general, whipping ends from a pipe break are restrained so that plastic hinge formation is not allowed to occur. Where plastic hinge could be formed, the effects are evaluated. Pipe whip restraints are provided wherever postulated pipe breaks could impair the ability of any essential system or component to perform its intended safety functions listed in section 3.6.

K. The calculation of thrust and jet impingement forces considers any line restrictions (e.g., flow limiter) between the pressure source and break location and the absence of energy reservoirs, as applicable.

L. Pipe breaks are not postulated to occur in pump and valve bodies since the wall thickness exceeds that of connecting pipe.

M. Components impacted by jets from breaks in piping containing high pressure (870 to 2465 psia) steam or subcooled liquid that would flash at the break, such as piping connected to the steam generators or reactor coolant loops, are evaluated as follows:

1. Impacted components within 10 piping diameters of the broken pipe are assumed to fail. Specific jet loads are calculated and evaluated only when failure of the component, when combined with a single active failure, could adversely affect safe shutdown or accident mitigation capability. These jet loads will be calculated in accordance with FSAR paragraph 3.6.2.3.

3. Components beyond 10 diameters of the broken pipe are considered to be undamaged by the jet and are not analyzed. The basis for these criteria is contained in reference 4.

3.6.1.2 Description

Systems, components, and equipment required to perform the functions discussed in section 3.6 (essential systems) are reviewed to ensure conformance with the design bases and to determine their susceptibility to the failure effects. The break and crack locations are determined in accordance with subsection 3.6.2.

A design comparison to NRC Branch Technical Positions ASB 3-1 and MEB 3-1 is provided in tables 3.6.1-2 and 3.6.1-3.

Pressure response analyses are performed for subcompartments containing high-energy piping. For a detailed discussion of the pipe breaks selected and pressure results, refer to paragraph 6.2.1.2 for selected subcompartments inside the containment and to appendix 3F for selected subcompartments outside the containment. Effects of both internal reactor pressure vessel asymmetric pressurization loads and asymmetric compartment pressurization loads inside containment are addressed in paragraph 6.2.1.2. The analytical methods used for pressure response analysis are in accordance with reference 2.

Appendix 3F provides a typical hazards analysis for the effects of postulated pipe breaks on essential systems, components, and structures.

There are no high-energy lines in the proximity of the control room; therefore, there are no effects upon the habitability of the control room resulting from postulated pipe breaks. Further discussion of the control room habitability systems is provided in section 6.4.

3.6.1.3 Safety Evaluation

3.6.1.3.1 General

An analysis of postulated pipe failures is performed to determine the impact of such piping failures on those safety-related systems or components which provide protective actions and are required to mitigate the consequences of the failure. By means of protective measures, such as separation, barriers, and pipe whip restraints, the effects of breaks and cracks are prevented from damaging essential items to an extent that would impair their essential function or necessary component operability. Typical measures used for protecting the essential systems, components, and equipment are outlined below and are discussed in detail in subsection 3.6.2. The ability of specific safety-related systems to withstand a single active failure concurrent with the postulated event is discussed, as applicable. When the results of the pipe failure effects analysis show that the effects of a postulated pipe failure are isolated, physically remote, or restrained by protective measures from essential systems or components, no further dynamic hazards analysis is performed.

3.6.1.3.2 Protection Mechanisms

The plant layout arrangement is based on maximizing the physical separation of redundant or diverse safety-related components and systems from each other and from nonsafety-related items. Therefore, in the event a pipe failure occurs, there is a minimal effect on other essential systems or components required for safe shutdown of the plant or to mitigate the consequences of the failure.

The effects associated with a particular pipe failure must be mechanistically consistent with the failure. Thus, pipe dimensions, piping layouts, material properties, and equipment arrangements are considered in defining the specific measures for protection against the consequences of postulated failures.

Protection against the dynamic effects of pipe failures is provided in the form of physical separation of systems and components, barriers, equipment shields, and pipe whip restraints. The precise method chosen depends largely upon considerations such as accessibility and maintenance.

A. Separation

The plant arrangement provides separation, to the extent practicable, between redundant safety systems (including their appurtenances) to prevent loss of safety function as a result of hazards for which the system is required to be functional. Separation between redundant safety systems, with their related appurtenances, therefore, is the basic protective measure incorporated in the design to protect against the dynamic effects of postulated pipe failures.

In general, layout of the facility follows a multi-step process to ensure adequate separation:

1. Safety-related systems are located remotely from high-energy piping, where practicable.
2. Redundant safety systems are located in separate compartments.
3. As necessary, specific components are enclosed to retain the redundancy required for those systems that must function as a consequence of specific piping failure.
4. Drainage systems are reviewed to ensure their adequacy for flooding control.

B. Barriers and Shields

Protection requirements are met through the protection afforded by walls, floors, columns, abutments, and foundations. Where adequate protection does not already exist as a result of separation, additional barriers, deflectors, or shields are provided to meet the functional protection requirements.

Inside the containment, the secondary shield wall serves as a barrier between the reactor coolant loops and the containment liner. In addition, the refueling cavity walls, operating floor, and secondary shield walls minimize the possibility of an accident which may occur in any one reactor coolant loop affecting another loop or the containment liner. Those portions of the steam and feedwater lines located within the containment are routed in such a manner that possible interaction between these lines and the reactor coolant piping is minimized. The barriers described above will withstand loadings caused by jet forces and pipe whip impact forces.

Further discussion of barriers and shields is provided in paragraph 3.6.2.4.

C. Piping Restraint Protection

Measures for protection against pipe whip are provided where the unrestrained pipe movement of either end of the ruptured pipe could cause damage at an unacceptable level to any structure, system, or component required to meet the criteria outlined in section 3.6.

The design criteria for and description of pipe whip restraints are given in paragraph 3.6.2.3.

3.6.1.3.3 Specific Protection Considerations

A. Nonessential systems, structures, and components are not required to meet the criteria outlined in section 3.6. However, while none of the above are needed during or following a pipe break event, pipe whip protection is evaluated where a high-energy nonessential system component or nonessential steel failure could initiate a pipe break event in an essential system or component or in another nonessential system whose failure could affect an essential system.

B. High-energy containment penetrations are subject to special protection mechanisms. As discussed in paragraph 3.6.2.1.1.D, isolation restraints are located as close as practical to the containment isolation valves associated with these penetrations. These restraints are provided to maintain the operability of the isolation valves and the integrity of the penetration due to a break either upstream or downstream of the respective isolation restraints.

C. Instrumentation that is required to function following a pipe rupture is protected.

D. High-energy fluid system pipe whip restraints and protective measures are designed so that a postulated break in one pipe cannot, in turn, lead to a rupture of other nearby pipes or components, if the secondary rupture will result in consequences that would be considered unacceptable for the initial postulated break.

E. For any postulated loss-of-coolant accident, the structural and leaktight integrity of the containment is maintained.

F. The escape of steam, water, combustible or corrosive fluids, gases, and heat in the event of a pipe rupture will not preclude:

  1. Subsequent access to any areas, as required, to cope with the postulated pipe rupture.
  2. Habitability of the control room.

3. The ability of essential instrumentation, electric power supplies, components, and controls to perform their safety functions to the extent necessary to meet the criteria outlined in section 3.6.

3.6.1.4 References

1. "Pipe Breaks for the LOCA Analysis of the Westinghouse Primary Coolant Loop," WCAP-8082-P-A (proprietary) and WCAP-8172-A (nonproprietary), January 1975.
2. "Subcompartment Pressure Analyses," BN-TOP-4, Revision 1, Bechtel Power Corporation, October 1977.
3. Federal Register, Vol. 50, No. 27, February 8, 1985.
4. NUREG/CR-2913, "Two-Phase Jet Loads," January 1983.
5. Federal Register, Vol. 50, FR 5454, February 8, 1985.
6. "Technical Bases for Eliminating Pressurizer Surge Line Rupture as the Structural Design Basis for Vogtle Unit 2," WCAP-11531 (proprietary) and WCAP-11532 (nonproprietary), July 1987 plus Addenda 1 and 2 dated August and September, 1987.
7. "Technical Basis for Eliminating Accumulator Line Rupture as the Structural Design Basis for Vogtle Unit 2, " WCAP-11583 (proprietary) and WCAP-11584 (nonproprietary), October 1987.
8. "Technical Basis for Eliminating RHR Line Rupture as the Structural Design Basis for Vogtle Unit 2," WCAP-11599 (proprietary) and WCAP-11600 (nonproprietary), September 1987.
9. "Evaluation of Thermal Stratification for the Vogtle Unit 2 Pressurizer Surge Line," WCAP-12218 (proprietary) and WCAP-12219 (nonproprietary), dated March 1989.
10. "Supplementary Analysis to Address Thermal Stratification for Vogtle Unit 1 Pressurizer Surge Line," WCAP-12218 Supplement 1 (proprietary) and WCAP-12219 Supplement 1 (nonproprietary), dated December 1989.
11. GPC letter to NRC transmitting WCAP-12218 Supplement 1 and WCAP-12219 Supplement 1.

3.6.2 DETERMINATION OF BREAK LOCATIONS AND DYNAMIC EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING

This subsection describes the design bases for locating postulated breaks and cracks in high- and moderate-energy piping systems inside and outside of the containment; the procedures used to define the jet thrust reaction at the break location; the procedures used to define the jet impingement loading on adjacent essential structures, systems, or components; pipe whip restraint design; and the protective assembly design.

3.6.2.1 Criteria Used To Define High/Moderate-Energy Break/Crack Locations and Configurations

Nuclear Regulatory Commission (NRC) Branch Technical Position (BTP) MEB 3-1[(1)] is used as the basis of the criteria for the postulation of high-energy pipe breaks except for the reactor coolant loop piping of Units 1 and 2 and the Class 1 branch line piping of Units 1 and 2 as discussed in section 3.6. Specific moderate-energy pipe crack locations are not ascertained; and, therefore, they are assumed to occur at any location, as described in paragraph 3.6.2.1.2.4.

A postulated high-energy pipe break is defined as a sudden, gross failure of the pressure boundary of a pipe either in the form of a complete circumferential severance (i.e., a guillotine break) or as a sudden longitudinal, uncontrolled crack. For moderate-energy fluid systems, pipe failures are confined to postulation of controlled cracks in piping. The effects of these cracks on the safety-related equipment are analyzed for flooding and wetting only. These cracks do not result in jet impingement or whipping of the cracked piping.

3.6.2.1.1 High-Energy Break Locations

With the exception of those portions of the piping identified in paragraph 3.6.2.1.1.D, breaks are postulated in high-energy piping at the following locations:

A. American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel (B&PV) Code, Section III, Division 1 - Class 1 Piping

1. The original design criteria postulates a limited number of pipe break locations in the reactor coolant loop[2]. Eight of these break locations were eliminated from the plant design basis because of the consideration of the detailed fracture mechanics evaluation of reference 10. This new design was approved by the NRC in reference 9. For the breaks at the residual heat removal line nozzle and accumulator line nozzle, the breaks are postulated on terminal end criteria for Unit 1 only. The pressurizer surge line nozzle break for Units 1 and 2 and the residual heat removal line nozzle and accumulator line nozzle breaks for Unit 2 are eliminated as discussed in section 3.6.

2. Except for pipe breaks which are not postulated in the Class 1 branch lines as discussed above, pipe breaks are postulated to occur at the following locations in Class 1 piping runs or branch runs outside the primary reactor coolant loops (RCL) as follows:

   a. At terminal ends of the piping, including:

      (1) Piping connected to structures, components, or anchors that act as essentially rigid restraints to piping translation and rotational motion due to static or dynamic loading.

      (2) High/moderate-energy boundary such as piping runs which are maintained pressurized during normal plant conditions for only a portion of the run; i.e., up to the first normally closed valve. The terminal end of such piping is the piping connection to the closed valve.

      (3) Branch intersection points are considered a terminal end for the branch line unless the following are met: the branch and the main piping systems are modeled in the same static, dynamic, and thermal analyses; and the branch and main run are of comparable size and fixity; i.e., the nominal size of the branch is at least one-half of that of the main.

b. At all intermediate locations where the following conditions are satisfied:

(1) Any intermediate locations where the maximum stress range as calculated by equation (10) and either (12) or (13) exceeds 2.4 S_m, (where S_m is the design stress intensity) as described in paragraph NB-3653 of the ASME B&PV Code, Section III.

(2) Any intermediate locations where the cumulative usage factor exceeds 0.1.[(3)] If two intermediate locations cannot be determined by the above criteria, two highest stress locations based on equation (10) in paragraph NB-3653 of the ASME B&PV Code, Section III[(3)] are selected. If the piping run has only one change or no change of direction, only one intermediate break location is postulated.

c. For those high energy systems identified in references 11 and 12, postulated intermediate break locations selected in accordance with item b.(2) above may be eliminated from design consideration if the following conditions are satisfied:

- Possibility of stress corrosion cracking has been minimized.
- Thermal and vibration induced piping fatigue has been minimized.
- Steam/waterhammer effects have been minimized.

d. As a result of piping reanalysis, the highest stress locations may be shifted. However, once a high-energy piping system has been analyzed and break locations have been identified and evaluated, the original intermediate break locations (selection based on the methodology specified in item b above) remain the same unless one of the following conditions exists:

(1) Maximum stress ranges or cumulative usage factors exceed the threshold levels specified in item b above.

(2) A change is required in pipe parameters, such as a major difference in pipe size, wall thickness, and routing.

B. ASME B&PV Code, Section III - Class 2 and 3 Piping Systems

1. Pipe breaks are postulated to occur at terminal ends.

2. Pipe breaks are postulated at intermediate locations between terminal ends where the maximum stress value, as calculated by the sum of equations (9) and (10) in Subarticle NC-3652 of the ASME B&PV Code, Section III,[(3)] considering normal and upset plant conditions (i.e., sustained loads, occasional loads, thermal expansion, and an operating basis earthquake (OBE) event) exceed 0.8 (1.2 S_h + S_A).

S_h and S_A are the allowable stress at maximum hot temperature and allowable stress range for thermal expansion, respectively, for Class 2 and 3 piping, as defined in Subarticle NC-3600 of the ASME B&PV Code, Section III.

3. In piping systems where the stresses are lower than the limits in item 2 above, a minimum of two intermediate break locations are postulated on the basis of the highest calculated stress levels. Where the piping consists of one change or a straight run with no fittings, welded attachments, or valves, only one location is chosen, based on the highest stress.

4. For those high energy systems identified in references 11 and 12, postulated intermediate break locations selected in accordance with item 3 above may be eliminated from design consideration if the following conditions are satisfied:

   - Possibility of stress corrosion cracking has been minimized.

   - Thermal and vibration-induced piping fatigue has been minimized.

   - Steam/waterhammer effects have been minimized.

   - Welded attachments (W/A) are not located in the vicinity of intermediate break locations as described below:

     a. No W/A is within 5 nominal piping diameters of the highest stress location(s) on high energy main steam and main feedwater systems.

     b. No W/A is within $\sqrt[3]{Rt}$ of the highest stress location(s) on remaining high energy piping systems other than main steam and main feedwater systems.

        R = The mean pipe radius based on ½ nominal pipe diameter and thickness.

        t = The pipe nominal thickness.

   If condition (a) above is not met for high energy main steam and main feedwater systems, a weak link analysis of the W/A support structure shall be made. Pipe break(s) will not be postulated if this analysis shows that the pipe wall at the W/A to pipe interface is not the weak link.

   For all high energy piping systems, if condition (a) or (b) above is not met, additional intermediate pipe breaks shall be postulated if the combined stress (dissipated local plus general) within $\sqrt[3]{Rt}$ of the high stress location exceeds the threshold of 0.8 of the combined equation 9 and 10 allowables.

4. As a result of piping reanalysis, the highest stress locations may be shifted. However, once a high-energy piping system has been analyzed and break locations have been identified and evaluated, the original intermediate break locations (selection based on the methodology specified in item 2 above) remain the same unless one of the following conditions exists:

a. Maximum stress exceeds the threshold level specified in item 2 above.

b. A change is required in pipe parameters, such as a major difference in pipe size, wall thickness, and routing.

Breaks are postulated as stated above in each piping and branch run adjacent to a protective structure or compartment containing essential systems and components required for safe shutdown. Such piping is considered as located adjacent to a protective structure if the distance between the piping and structure is insufficient to preclude impairment of the structure's integrity from the effects of a postulated piping failure, assuming that the piping is unrestrained.

C. Nonnuclear Piping (i.e., not ASME Section III Class 1, 2, or 3)

Breaks in nonnuclear piping are postulated at the following locations in each run:

1. At the locations specified for ASME, Section III(3) Class 2 and 3 piping (refer to paragraph 3.6.2.1.1.B), if the nonnuclear piping is analyzed and supported to withstand full safe shutdown earthquake loadings.
2. In the absence of stress analysis, breaks in nonnuclear piping are postulated at the following locations in each run or branch run:

   a. Terminal ends.

   b. Each intermediate fitting; e.g., short- and long-radius elbows, tees, and reducers; welded attachments; and valves.

D. High-Energy Piping in Containment Penetration Areas

Breaks are not postulated in the portions of Class 2 piping between the containment penetration flued-head and five-way restraints (i.e., break exclusion zone) provided subject piping meets the following provisions:

1. Stresses do not exceed those specified in paragraph 3.6.2.1.1.B.
2. The maximum stress in this piping as calculated by equation (9), per paragraph NC-3652 of ASME Section III when subjected to the combined loadings of internal pressure, deadweight, and pipe rupture outside the protective restraints, does not exceed 1.8 S_h .
3. The number of circumferential and longitudinal piping welds and branch connections is minimized.

Areas of system piping where no breaks are postulated are as follows:

   a. The main steam piping, from the containment penetration flued head outboard weld, to the upstream weld of the five-way restraint, which is downstream of the main steam isolation valves, including the main steam safety valves and branch piping to the main steam safety valves. This includes approximately 33 ft of piping for each steam line.

b. The main feedwater piping from the containment penetration to the five-way restraint which is upstream of the isolation valve (approximately 33 ft of piping for each feedwater line).

When required for isolation valve operability, structural integrity, or containment integrity, five-way restraints capable of resisting torsional and bending moments produced by a postulated pipe break, either upstream or downstream of the piping and valves which form the containment isolation boundary, are located reasonably close to the isolation valves or penetration.

The five-way restraints do not prevent the access required to conduct inservice inspection examinations specified in Section XI of the ASME Code. Inservice examinations completed during each inspection interval provide 100-percent volumetric examination of circumferential and longitudinal pipe welds within the boundary of these portions of piping during each inspection interval, as described in section 6.6.

Welded attachments to these portions of piping for pipe supports or other purposes are avoided. Where welded attachments are necessary, detailed stress analyses are performed to demonstrate compliance with the limits of paragraph 3.6.2.1.1.

The five-way restraints outside the containment on the main steam and main feedwater lines are located as close as possible to the containment to accommodate the design for the auxiliary building steam tunnel and still minimize stresses.

3.6.2.1.2 Types of Breaks/Cracks Postulated

3.6.2.1.2.1 ASME Section III, Class 1 RCL Piping - High-Energy. The types of breaks postulated in the ASME Section III,[3] Class 1 primary RCL are discussed in paragraph 3.6.2.1.1.A.1.

3.6.2.1.2.2 Piping Other than RCL Piping - High-Energy. The following types of breaks are postulated to occur at the locations determined in accordance with paragraph 3.6.2.1.1.

A. In piping whose nominal diameter is greater than or equal to 4 in., both circumferential and longitudinal breaks are postulated at each selected break location unless eliminated by comparison of longitudinal and axial stresses with the maximum stress as follows:

1. If the maximum stress range exceeds the limits specified in paragraphs 3.6.2.1.1.A.2.b and 3.6.2.1.1.B.2 but the circumferential stress range is at least 1.5 times the axial stress range, only a longitudinal break is postulated.

2. If the maximum stress range exceeds the limits specified in paragraphs 3.6.2.1.1.A.2.b and 3.6.2.1.1.B.2 but the axial stress is at least 1.5 times

the circumferential stress range, only a circumferential break is postulated.

Longitudinal breaks, however, are not postulated at the following locations:

1. Terminal ends.
2. Intermediate points of Class 1 piping systems where the stress range as calculated by equations (10) and either (12) or (13) does not exceed 2.4 S_m as described in paragraph NB-3653 of the ASME B&PV Code, Section III,(3) and/or if the cumulative usage factor does not exceed 0.1.
3. Intermediate points of Class 2 and 3 piping systems where the maximum stress value, as calculated by the sum of equations (9) and (10) described in paragraph NC-3652 of the ASME B&PV Code, Section III,(3) does not exceed 0.8 (1.2 S_h + S_A).

B. In piping whose nominal diameter is greater than 1 in. but less than 4 in., only circumferential breaks are postulated at each selected break location.

C. No breaks are postulated for piping whose nominal diameter is 1 in. or less.

D. In the absence of mechanistic break locations, as described in paragraph 3.6.2.1.1. Breaks or critical cracks are postulated in high energy ASME Code, Section III, Class 1, 2, and 3 and ANSI B31.1 piping at locations that result in the most severe environmental consequences.

3.6.2.1.2.3 Nonnuclear Piping - High-Energy. The types of breaks for nonnuclear piping are postulated as discussed in paragraph 3.6.2.1.2.2; the corresponding break locations are determined in accordance with paragraph 3.6.2.1.1.C.

3.6.2.1.2.4 ASME Section III and Nonnuclear Piping - Moderate-Energy. Through-wall leakage cracks are postulated in moderate-energy piping including branch runs larger than 1-in. nominal diameter as clarified below:

A. Through-wall leakage cracks are not required to be postulated in those portions of piping between containment isolation valves, provided they meet the requirements of ASME Code, Section III, Sub-article NE-1120, and are designed so that the maximum stress range does not exceed 0.4 (1.2 S_h + S_A).

B. Through-wall leakage cracks are not required to be postulated in moderate-energy fluid system piping located in an area where a break in the high-energy fluid system is postulated, provided that such cracks do not result in environmental conditions more limiting than the high-energy pipe break.

C. Subject to paragraph D below, through-wall leakage cracks are required to be postulated in:

(1) ASME, B&PV Code, Section III, Division 1 - Class 1 piping where the maximum stress range in the piping is greater than 1.2 S_m.

(2) ASME, B&PV Code, Section III, Division 1 - Class 2 or 3 piping and seismically supported nonnuclear class piping at locations where the maximum stress range in the piping is greater than 0.4 (1.2 S_h + S_A).

D. Individual cracks are not required to be postulated at specific locations determined by stress analyses when a review of the piping layout and plant arrangement drawings shows that the effects of through-wall leakage cracks at any location in the piping designed to seismic or nonseismic standards are isolated or physically remote from structures, systems, and components required for safe shutdown.

To simplify analysis, cracks may be postulated to occur everywhere in moderate-energy piping regardless of the stress analysis results to determine the maximum damage from fluid spraying and flooding, with the consequent hazards or environmental conditions. Flooding effects are determined on the basis of a 30-min operator time required to effect corrective actions. Further discussion of flooding effects is provided in appendix 3F.

3.6.2.1.3 Break/Crack Configuration

3.6.2.1.3.1 High-Energy Break Configuration. Following a circumferential break, the two ends of the broken pipe are assumed to move clear of each other unless physically limited by piping restraints, structural members, or piping stiffness. The effective cross-sectional (inside diameter) flow area of the pipe is used in the jet discharge evaluation. Movement is assumed to be in the direction of the jet reaction initially, with the total path controlled by the piping geometry.

The orientation of a longitudinal break, except when otherwise justified by a detailed stress analysis, is assumed to be at opposing points on a line perpendicular to the plane of a fitting for a nonaxisymmetric fitting and anywhere around the circumference of the fitting for axisymmetric fittings. The flow area of such a break is equal to the cross-sectional flow area of the pipe. Longitudinal and circumferential breaks are not postulated concurrently.

3.6.2.1.3.2 Moderate-Energy Crack Configuration. Moderate-energy crack openings are assumed to be a circular orifice with cross-sectional flow area equal to that of a rectangle one-half the pipe inside diameter in length and one-half pipe wall thickness in width.

3.6.2.2 Analytical Methods To Define Forcing Functions and Response Models

3.6.2.2.1 Forcing Functions for Jet Thrust

To determine the forcing function, the fluid conditions at the upstream source and at the break exit dictate the analytical approach and approximations that are used. For most applications, one of the following situations exists:

- Superheated or saturated steam.
- Saturated or subcooled water.
- Cold water (nonflashing).

Analytical methods for calculation of jet thrust for the above- described situations are discussed in references 4 and 5. For a discussion of the jet thrust forcing functions from RCL breaks, see paragraph 3.6.2.2.1.1.

3.6.2.2.1.1 Time Functions of Jet Thrust Force on Ruptured and Intact RCL Piping. To determine the thrust and reactive force loads to be applied to the RCL during the postulated loss-of-coolant accident (LOCA), it is necessary to have a detailed description of the hydraulic transient. Hydraulic forcing functions are calculated for the intact RCLs as a result of a postulated LOCA. These forces result from the transient flow and pressure histories in the reactor coolant system (RCS). The calculation is performed in two steps. The first step is to calculate the transient pressure, mass flowrates, and thermodynamic properties as a function of time. The second step uses the results obtained from the hydraulic analysis, along with input of areas and direction coordinates, and calculates the time-history of forces at appropriate locations (e.g., elbows) in the RCLs.

The hydraulic model represents the behavior of the coolant fluid within the entire RCS. Key parameters calculated by the hydraulic model are pressure, mass flowrate, and density. These are supplied to the thrust calculation, together with plant layout information, to determine the time-dependent loads exerted by the fluid on the loops. In evaluating the hydraulic forcing functions during a postulated LOCA, the pressure and momentum flux terms are dominant. The inertia and gravitational terms are taken into account in the evaluation of the local fluid conditions in the hydraulic model.

The blowdown hydraulic analysis is required to provide the basic information concerning the dynamic behavior of the reactor core environment for the loop forces. This requires the ability to predict the flow, quality, and pressure of the fluid throughout the reactor system. The MULTIFLEX code[6] was developed with a capability to provide this information.

The MULTIFLEX computer code calculates the hydraulic transients within the entire primary coolant system. This hydraulic program considers a coupled, fluid-structure interaction by accounting for the deflection of the core support barrel. The depressurization of the system is calculated using the method of characteristics applicable to transient flow of a homogenous fluid in thermal equilibrium.

The ability to treat multiple flow branches and a large number of mesh points gives the MULTIFLEX code the flexibility required to represent the various flow passages within the primary RCS. The system geometry is represented by a network of one-dimensional flow passages.

The THRUST computer program was developed to compute the transient (blowdown) hydraulic loads resulting from a LOCA.

The blowdown hydraulic loads on primary loop components are computed from the equation:

The symbols and units are as follows:

$$F = 144A \left[(P - 14.7) + \left(\frac{\dot{m}^2}{\rho g A_m 2_{144}} \right) \right]$$

F = Force (lb_f).

A = Aperture area (ft^2).

P = System pressure (psia).

m = Mass flowrate (lbm/s).

ρ = Density (lbm/ft^3).

g = Gravitational constant = 32.174 ft-lbm/lb -s^2.

A_m = Mass flow area (ft^2).

In the model to compute forcing functions, the RCL system is represented by a model similar to that employed in the blowdown analysis. The entire loop layout is represented in a global coordinate system. Each node is fully described by:

A. Blowdown hydraulic information.

B. The orientation of the streamlines of the force nodes in the system, which includes flow areas, and projection coefficients along the three axes of the global coordinate system.

Each node is modeled as a separate control volume with one or two flow apertures associated with it. Two apertures are used to simulate a change in flow direction and area. Each force is divided into its x, y, and z components using the projection coefficients. The force components are then summed over the total number of apertures in any one node to give a total x force, a total y force, and a total z force. These thrust forces serve as input to the piping/restraint dynamic analysis.

The THRUST code calculates forces exactly the same way as the STHRUST code, which is described in reference 7.

3.6.2.2.2 Dynamic Analysis of the Reactor Coolant Loop Piping, Equipment Supports

The dynamic analysis of the RCL for LOCA loadings is described in section 3.9.

3.6.2.3 <u>Dynamic Analysis Methods To Verify Integrity and Operability</u>

3.6.2.3.1 Dynamic Analysis Methods To Verify Integrity and Operability for Other than RCL

The analytical methods of references 4 and 5 are used to determine the jet impingement effects and loading effects applicable to components and systems resulting from postulated pipe breaks and cracks.

3.6.2.3.2 Dynamic Analysis Methods To Verify Integrity and Operability for the RCL

3.6.2.3.2.1 General. A LOCA is assumed to occur for a branch line break down to the restraint of the second normally open automatic isolation valve (case II, figure 3.6.2-1) on outgoing lines(1) and down to and including the second check valve (case III, figure 3.6.2-1) on incoming lines normally with flow. A pipe break beyond the restraint or second check valve does not result in an uncontrolled loss of reactor coolant if either of the two valves in the line closes.

Accordingly, both of the automatic isolation valves are suitably protected and restrained as close to the valves as possible so that a pipe break beyond the restraint does not jeopardize the integrity and operability of the valves. Further, periodic testing capability of the valves to perform their intended function is essential. This criterion takes credit for only one of the two valves performing its intended function. For normally closed isolation or incoming check valves (cases I and IV, figure 3.6.2-1), a LOCA is assumed to occur for pipe breaks on the reactor side of the valve.

Branch lines connected to the RCL are defined as large strictly for the purpose of pipe break criteria if they have an inside diameter greater than 4 in. up to the largest connecting line. Rupture of these lines results in a rapid blowdown from the RCL, and protection is basically provided by the accumulators and the low-head safety injection pumps (residual heat removal pumps).

Branch lines connected to the RCL are defined as small for the purpose of pipe break analysis if they have an inside diameter equal to or less than 4 in. This size is such that emergency core cooling system analyses, using realistic assumptions, show that no clad damage is expected for a break area of up to 12.5 in.^{2} corresponding to 4 in. inside diameter piping.

Engineered safety features are provided for core cooling and boration, pressure reduction, and activity confinement in the event of a LOCA or steam or feedwater line break accident to ensure that the public is protected in accordance with 10 CFR 100 guidelines. These safety systems are designed to provide protection for an RCS pipe rupture of a limited flow area severance of an RCL, as identified in paragraph 6.2.1.2.

To assure the continued integrity of the essential components and the engineered safety systems, consideration is given to the consequential effects of the pipe break itself to the extent that:

A. The minimum performance capabilities of the engineered safety systems are not reduced below that required to protect against the postulated break.

B. The containment leaktightness is not decreased below the design value if the break leads to a LOCA.(2)

C. Propagation of damage is limited in type and/or degree to the extent that:

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(1) It is assumed that motion of the unsupported line containing the isolation valves can cause failure of the operators of both valves to function.

(2) The containment is here defined as the containment structure liner and penetrations and the steam generator shell, the steam generator steam side instrumentation connections, the steam, feedwater, blowdown, and steam generator drain pipes within the containment structure.

1. A pipe break which is not a LOCA will not cause a LOCA or steam or feedwater line break. However, pipe breaks on the nonreactor side of an RCS pressure boundary may cause the failure of the reactor side of the same pipe, provided the combined failures are evaluated for impact on system performance.

2. An RCS pipe break will not cause a steam or feedwater system pipe break, and vice versa.

3.6.2.3.2.2 Large Branch Lines. Large branch line piping, as defined in paragraph 3.6.2.3.2.1, is restrained[3] to meet the following criteria in addition to items A through C of paragraph 3.6.2.3.2.1 for a pipe break resulting in a LOCA:

A. Propagation of the break to the unaffected loops (except pressurizer spray on Unit 1) is prevented to ensure the delivery capacity of the accumulators and low head pumps.

B. Propagation of the break in the affected loop is permitted to occur but does not exceed 20 percent of the flow area of the line which initially ruptured (except pressurizer spray, safety, and relief lines on Unit 1). The capacity of the accumulators and low head safety injection system establishes the limit for propagation of large break LOCAs (lines 6 in. and greater) to other branch lines in the loop containing the initially postulated pipe break. The limit selected (total area of additional failed lines to be less than 20 percent of initial break area) ensures that the severity of the initial break is not increased to an unacceptable level. For large branch line breaks, the actual capacity of the accumulators and the low head safety injection system is well in excess of that needed for propagation of this magnitude. For this reason, exceptions to the above criteria are allowed on a case basis, provided that the failure of the broken lines is shown to be enveloped by the design basis LOCAs.

3.6.2.3.2.3 Small Branch Lines. Should one of the small pressurized lines, as defined in paragraph 3.6.2.3.2.1, fail and result in a LOCA, the piping is restrained or arranged to meet the following criteria in addition to items A through C of paragraph 3.6.2.3.2.1:

A. Break propagation is limited to the affected leg; i.e., propagation to the other leg of the affected loop and to the other loops is prevented. Damage to the high-head safety injection lines connected to the other leg of the affected loop or to the other loops is prevented.

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[3] The Class 1 branch lines in Unit 2 are not restrained because the dynamic effects of pipe rupture in these lines are not in the plant's design basis as discussed in section 3.6.

B. Propagation of the break in the affected leg is permitted but must be limited to a total break area of 12.5 in.^{2} (4-in. inside diameter). The exception to this case is when the initiating small break is a cold leg high-head safety injection line. Further propagation is not permitted for this case. The capacity of the high head safety injection system establishes the limit for propagation of small break LOCAs (lines 4 in. and smaller) to other branch lines in the leg containing the initially postulated pipe break. The limit selected (total area of additional failed lines to be limited to 12.5 in ^{2}) ensures that the sum of the piping failures will not require safety injection delivery in excess of the capacity of the high head system. Although the severity of such an event is significantly less than that of a large break LOCA, the limit is applied to prevent a small LOCA from becoming a large LOCA. Since an initial break in the cold leg high head safety injection line reduces the capacity of the high head injection system, additional propagation for these breaks is not permitted.

C. Propagation of the break to a high-head safety injection line connected to the affected leg is prevented if the line break results in a loss of core cooling capability due to a spilling injection line.

The above criteria (paragraphs 3.6.2.3.2.2 and 3.6.2.3.2.3) are primarily implemented through the use of physical separation of the branch lines. In many cases, the initiating breaks do not whip or impinge on neighboring branch lines. In those cases where whip or impingement does occur, the impacted line is often designed such that failure will not occur. Whip and impingement by some lines are also prevented by pipe whip restraints. If the impact loads are excessive and it cannot be shown that failure would be acceptable (either by the criteria or engineering analysis), then protective barriers are installed to prevent the interaction.

3.6.2.3.2.4 Design and Verification of Adequacy of RCL Components and Supports. The methods described below are used in the Westinghouse design and verification of the adequacy of primary RCL components and supports. It is emphasized that these methods are used only to determine jet impingement loads on RCL components and supports.[(4)]

The design basis postulated pipe rupture locations are determined using the criteria given in paragraph 3.6.2.1. These design basis ruptures are used here as the rupture locations for consideration of jet impingement effects on primary equipment and supports.

A dynamic analysis is used to determine maximum piping displacements at each design basis rupture location. These maximum piping displacements are used to compute the effective rupture flow area at each location. This area and rupture orientation is then used to determine the jet flow pattern and to identify any primary components which are potential targets for jet impingement.

The jet thrust at the point of rupture is based on the fluid pressure and temperature conditions occurring during normal (100 percent) steady-state operating conditions of the plant. At the point of rupture, the jet force is equal and opposite to the jet thrust. The force of the jet is

[(4)] The jet impingement effects of a break in the RCL piping are not considered in the design verification of the RCL component supports (reactor vessel, pressurizer, reactor coolant pump and steam generator supports) in Units 1 and 2. These effects were eliminated by application of leak-before-break technology and their elimination was factored into the design verification calculations. The jet impingement effects of the most limiting branch line breaks, however, are factored into the Unit 1 and 2 RCL component support design verification.

conservatively assumed to be constant throughout the jet flow distance. The subcooled jet is calculated as discussed in reference 4.

If simplified static analysis is performed instead of a dynamic analysis, the above jet load (F_T) is multiplied by a dynamic load factor. For an equivalent static analysis of the target structure, the jet impingement force is multiplied by a dynamic load factor of 1.2 to 2.0, depending upon the time variance of the jet load. This factor assumes that the target can be represented as essentially a one-degree-of-freedom system, and the impingement force is conservatively applied as a step load.

3.6.2.3.3 Types of Pipe Whip Restraints

3.6.2.3.3.1 Pipe Whip Restraints. To satisfy varying requirements of available space, permissible pipe deflection, and equipment operability, the restraints are designed as a combination of an energy-absorbing element and a restraint structure suitable for the geometry required to pass the restraint load from the whipping pipe to the main building structure.

The restraint structure is typically a structural steel frame or truss and the energy-absorbing element is usually either stainless steel U-bars or energy-absorbing material as described below:

A. Stainless Steel U-Bar

This type consists of one or more U-shaped, upset- threaded rods of stainless steel looped around the pipe but not in contact with the pipe to allow unimpeded pipe motion during seismic and thermal movement of the pipe. At rupture, the pipe moves against the U-bars, which absorb the kinetic energy of pipe motion by yielding plastically. A typical example of a U-bar restraint is shown in figure 3.6.2-2.

B. Energy Absorbing Material

This type of restraint consists of a crushable, stainless steel, internally honeycomb-shaped element designed to yield plastically under impact of the whipping pipe. A design hot position gap is provided between the pipe and the energy-absorbing material to allow unimpeded pipe motion during seismic and thermal pipe movements. A typical example of an energy-absorbing material restraint is shown in figure 3.6.2-3.

3.6.2.3.4 Analytical Methods

3.6.2.3.4.1 Pipe Whip Restraints.

A. Location of Restraints

1. For purposes of determining pipe hinge length and thus locating the pipe whip restraints, the plastic moment of the pipe is determined in the following manner:

$M_p = 1.1\ z_p S_y$

where:

z_p = Plastic section modulus of pipe $D_m{}^2t$.

D_m = Mean piping diameter.

t = Wall thickness.

S_y = Yield stress at pipe operating temperature.

1.1 = 10-percent factor to account for strain hardening.

Pipe whip restraints are located as close to the axis of the reaction thrust force break as practicable. Pipe whip restraints are generally located so that a plastic hinge does not form in the pipe. If, due to physical limitations, pipe whip restraints are located so that a plastic hinge can form, the consequences of the whipping pipe and the jet impingement effect are further investigated. Lateral guides are provided where necessary to predict and control pipe motion.

2. Generally, restraints are designed and located with sufficient clearances between the pipe and the restraint such that they do not interact and cause additional piping stresses. A design hot position gap is provided that will allow maximum predicted thermal, seismic, and seismic anchor movement displacements to occur without interaction.

   Exception to this general criterion may occur when a pipe support and restraint are incorporated into the same structural steel frame, or when a zero design gap is required. In these cases the restraint is included in the piping analysis, if required.

3. In general, the restraints do not prevent the access required to conduct inservice inspection examination of piping welds. When the location of the restraint makes the piping welds inaccessible for inservice inspection, a portion of the restraint is made removable to provide accessibility.

B. Analysis and Design

Analysis and design of pipe whip restraints for postulated pipe break effects are in accordance with reference 4. Specifically, the following criteria are adopted in analysis and design:

1. Pipe whip restraints are designed based on energy absorption principles by considering the elastic-plastic, strain-hardening behavior of the materials used.
2. A rebound factor of 1.1 is applied to the jet thrust force.
3. Except in cases where calculations are performed to verify that a plastic hinge is formed, the energy absorbed by the ruptured pipe is conservatively assumed to be zero; i.e., the thrust force developed goes directly into moving the broken pipe and is not reduced by the force required to bend the pipe.
4. In elastic-plastic design, limits for strains are as follows:

   ε = Allowable strain used in design.

   a. Stainless Steel U-Bars

      $\varepsilon = 0.5\varepsilon_u$

where:

ε_u = ultimate uniform strain of stainless steel (strain at ultimate stress).

b. Energy-Absorbing Material

$\varepsilon = 0.8\varepsilon_u$

where:

ε_u = maximum crushable height at uniform crushable strength.

5. A dynamic increase factor is used for steel which is designed to remain elastic.

3.6.2.4 Protective Assembly Design Criteria

3.6.2.4.1 Jet Impingement Barriers and Shields

Barriers and shields, which may be of either steel or concrete construction, are provided to protect essential equipment including instrumentation from the effects of jet impingement resulting from postulated pipe breaks. Barriers differ from shields in that they may also accept the impact of whipping pipes. Barriers and shields include walls, floors, and structures specifically designed to provide protection from postulated pipe breaks. Barrier and shield design is based on the methods of reference 4, section 3.0, and the elastic-plastic methods for dynamic analysis included in reference 8. Design criteria and loading combinations are in accordance with subsections 3.8.3 and 3.8.4.

3.6.2.4.2 Auxiliary Guardpipes

The use of guardpipes has been minimized by plant arrangement and routing of high-energy piping. Where they are used, guardpipes are designed to withstand all dynamic and environmental effects of postulated breaks of the enclosed pipe. Auxiliary guardpipes are used only if inservice inspection requirements can be satisfied. Design criteria, loading combinations, and methods of analysis are similar to those for barriers and shields described in paragraph 3.6.2.4.1.

3.6.2.5 Material To Be Submitted for the Operating License Review

3.6.2.5.1 Piping Systems Other than RCL

Pipe break locations are obtained in accordance with the criteria of paragraph 3.6.2.1.

High-energy piping, with break locations identified, is provided in isometric drawings submitted by reference 13. Break types, i.e., circumferential or longitudinal, are also shown. The stress results utilized to determine the break types and locations are given, along with the associated stress nodes. High-energy pipe break effects analysis for a selected portion of the plant (auxiliary building level C, safety-related pump rooms on levels B and D) is discussed room-by-room in table 3F-1.

Moderate-energy piping crack locations are defined in paragraph 3.6.2.1.2.4. Evaluation of the effects of moderate-energy cracks is discussed in appendix 3F.

The augmented inservice inspection plan is discussed in section 6.6.

Pipe whip restraints are designed in accordance with paragraph 3.6.2.3. Pipe whip restraint location and orientation for each high-energy break are shown in reference 13. Barriers and shields are designed in accordance with the criteria of paragraph 3.6.2.4. Jet thrust and impingement forces were determined in accordance with reference 4. Thrust forces for each pipe whip restraint are presented in reference 13. These values are typically calculated without accounting for the resistance losses. If necessary, the thrust force will be reduced to account for the flow resistance losses.

3.6.2.5.2 Reactor Coolant Loop

A. Drawing AX6DD309 and table 3.6.2-3 identify the design basis break locations remaining and orientations for the RCLs, as explained in paragraph 3.6.2.1.1.A.1.

   The primary and secondary stress intensity ranges and the fatigue cumulative usage factors at the design break locations specified in section 3.6.2.1.1.A.1 are not tabulated since selection of these terminal end locations is independent of detailed stress and fatigue analyses.

B. The results of evaluating jet impingement loads associated with the branch line breaks, identified in 3.6.2.1.1.A.1 are provided by reference 13. As described in item C below, these loads are used to determine the adequacy of the primary equipment and supports.

C. Design loading combinations and applicable criteria for ASME Class 1 components and supports are provided in section 3.9. Pipe rupture loads include not only the jet thrust forces acting on the piping but also jet impingement loads on the primary equipment supports.

3.6.2.6 References

1. "Postulated Break and Leakage Locations in Fluid System Piping Outside Containment," NRC Branch Technical Position MEB 3-1, November 1975.
2. "Pipe Breaks for the LOCA Analysis of the Westinghouse Primary Coolant Loop," WCAP-8082-P-A (proprietary) and WCAP-8172-A (nonproprietary), January 1975.
3. ASME Section III, Subsection 3650, Summer 1979 Addendum to the 1977 Code Edition.
4. "Design for Pipe Break Effects," Bechtel Power Corporation, BN-TOP-2, Revision 2, May 1974.
5. Moody, F. J., Fluid Reaction and Impingement Loads, Paper Presented at the ASCE Specialty Conference, Chicago, December 1973.
6. "MULTIFLEX, A FORTRAN-IV Computer Program for Analyzing Thermal-Hydraulic-Structure System Dynamics," WCAP-8708 (proprietary), February 1976, and WCAP-8709 (nonproprietary), February 1976.

7. "Documentation of Selected Westinghouse Structural Analysis Computer Codes," WCAP-8252, Revision 1, May 1977.

8. Biggs, J. M., Introduction to Structural Dynamics, McGraw-Hill Book Company, New York, 1964.

9. Federal Register, 50 FR 5454, February 8, 1985.

10. "Technical Bases for Eliminating Large Primary Loop Pipe Rupture is the Structural Design Basis for Vogtle, Units 1 and 2," WCAP-10551, (Proprietary Class 2), May 1984.

11. Georgia Power Company letter to the Nuclear Regulatory Commission dated April 26, 1984.

12. Georgia Power Company letter to the Nuclear Regulatory Commission dated April 24, 1985.

13. Georgia Power Company letter to the Nuclear Regulatory Commission, ELV-01-388, March 1990.

TABLE 3.6.1-1 (SHEET 1 OF 2)

ESSENTIAL, HIGH-ENERGY, AND MODERATE-ENERGY SYSTEMS

System	Essential[a] Systems	High[b] Energy	Moderate Energy
Reactor coolant	0	0	
Nuclear service cooling water	0		0
Component cooling water	0		0
Safety injection	0	0	
Residual heat removal	0	0[c]	0
Containment spray	0		0
Chemical volume and control	0	0	0
Nuclear sampling		0	
Spent fuel cooling and purification	0		0
Auxiliary component cooling water			0
Main steam		0	0
Auxiliary feedwater	0	0	0
Condensate and main feedwater	0	0	
Auxiliary steam		0	
Steam generator blowdown	0	0	
Safety-related heating, ventilating, and air conditioning	0		0
Essential chilled water	0		0
Waste processing		0	

TABLE 3.6.1-1 (SHEET 2 OF 2)

System	Essential[a] Systems	High[b] Energy	Moderate Energy
Turbine-Generator		0	0
Auxiliary gas	0	0	
Diesel generator and related systems	0		0
Fire protection			0
Instrument and service air			0

a. Not all essential systems are required for all postulated piping failures; e.g., the containment spray system is essential for loss-of-coolant accident and main steam line break inside containment but is nonessential for piping failure outside containment. Not all portions of essential systems are required for postulated piping failure; e.g., the main steam system is only essential from the steam generator to the main steam isolation valves, including the safety and atmospheric steam relief valves.

b. Not all portions of high-energy systems contain high-energy fluid.

c. During the initial phase of cooldown, the residual heat removal system is a high-energy system. For interaction with the redundant train, the residual heat removal system is considered a dual-purpose, moderate-energy system. (See paragraph 3.6.1.1.G.)

TABLE 3.6.1-2 (SHEET 1 OF 2)

DESIGN COMPARISON TO POSITIONS OF NRC BRANCH TECHNICAL POSITIONS ASB 3-1

Branch Technical Position ASB 3-1		VEGP Design	
B.1	Plant Arrangement	B.1	Conforms. See paragraph 3.6.1.3.
	Protection of essential systems and components against postulated piping failures in high- or moderate-energy fluid systems that operate during normal plant conditions and that are located outside of containment should be provided	B.1.a	Conforms. See paragraph 3.6.1.3.2.1.A.
		B.1.a.(1)	Partial conformance as follows: The essential equipment located in the main bundle being uncovered. Main steamline breaks up to 1.0 ft^2 with steam generator tube bundle being uncovered are considered as discussed in paragraph 3.11.B.1.1. The essential equipment is designed to be protected from the jet impingement and pipe
		B.1.b	Conforms. See paragraphs 3.6.13.2.1B; 3.6.1.3.21C; 3.6.1.3.3; and 3.6.2.3
		B.1.c	
		B.1.c.1(a)	Conforms. As part of the design process, the restraint gap is verified large enough to accommodate thermal, seismic, and seismic anchor movements.
		B.1.c.1(b)	Partial conformance. See paragraph 3.6.2.3.3.1.A Additionally, final pipe whip restraint gap will be verified during hot-functional testing and thus will account for any differential settlement. Pipe relaxation is not specifically considered in the VEGP design.
		B.1.c.1(c)	See response to items (a) and (b) above.
		B.1.c.(2)	Conforms. Restraints which do not have adequate inservice inspection pipe weld space requirements are made removable.

TABLE 3.6.1-2 (SHEET 2 OF 2)

	Branch Technical Position ASB 3-1		VEGP Design
B.2	Design Features		
B.2.a	Essential systems and components should be designed to meet the seismic design requirements of Regulatory Guide 1.29.	B.2.a	Conforms, as described in sections 1.9 and 3.2.
B.2.b	Protective structures or compartments, fluid system piping restraints, and other protective measures.	B.2.b	Conforms. See subsections 3.8.3 and 3.8.4 for loading combinations. See paragraphs 3.6.1.1 and 3.6.1.3 for piping restraints and protection measures.
B.2.c	Fluid system piping in containment penetration areas should be designed to meet the break exclusion provisions contained in item B.1.b of BTP MEB 3-1.	B.2.c	Conforms. High-energy piping is designed as per B.1.b of MEB 3-1. Moderate-energy piping is designed as per B.2.B of MEB 3-1. For further information, refer to B.1.a.(1) above and paragraphs 3.6.2.1.1.D and 3.6.2.1.2.4.
B.2.d	Piping classification as required by NRC Regulatory Guide 1.26 should be maintained.	B.2.d.	Conforms. See paragraph 3.6.2.1.1.D.
B.3	Analyses and Effects of Postulated Piping Failures	B.3.a	Conforms. See paragraphs 3.6.1.1.D; 3.6.1.2; 3.6.1.3; and table 3.6.2-2.
		B.3.b.(1)	Conforms. See paragraph 3.6.1.1.E.
		B.3.b.(2)	Conforms. See paragraph 3.6.1.1.F.
		B.3.b.(3)	Conforms. Paragraph 3.6.1.1.G defines a train to include those systems which support its function.
		B.3.b.(4)	Conforms. See paragraph 3.6.1.1.H.
		B.3.c	Conforms. See paragraph 3.6.1.2 and section 6.4.
		B.3.d	Conforms. See paragraph 3.6.1.1.H and section 3.F.2.

TABLE 3.6.1-3 (SHEET 1 OF 3)

DESIGN COMPARISON TO NRC BRANCH TECHNICAL POSITION MEB 3-1[a]

	Branch Technical Position MEB.3-1		VEGP Design
B.1	High-Energy Fluid System Piping		
B.1.a	Fluid systems separated from essential systems and components.	B.1.a	Conforms. See paragraph 3.6.1.3.2.
B.1.b	Fluid system piping in containment penetration areas.	B.1.b	Conforms. See paragraph 3.6.2.1.1.D.
		B.1.b.(1)(a)-(c)	There is no Class 1 piping in containment penetration areas in the VEGP.
		B.1.b.(1)(d)	Conforms. See paragraphs 3.6.2.1.1.D and 3.6.2.1.1.B.
		B.1.b.(1)(e)	Conforms. For further discussion see paragraph 3.6.2.1.1.D.
		B.1.b.(2)	Conforms. See paragraph 3.6.2.1.1.D.
		B.1.b.(3)	Conforms. See paragraph 3.6.2.1.1.D. For guardpipes see paragraph 3.6.2.4.2.
		B.1.b.(4)	See conformance statement to ASB 3-1 position B.2.c.(1) and paragraph 3.6.2.1.1.D.
		B.1.b.(5)	High-energy containment flued head penetrations are integrally forged piped fittings. Pipe whip restraints do not require welding directly to the outer surface of the piping, except where such welds are 100-percent volumetrically examined in service and a review for local stresses is performed. The main steam and main feedwater lines outside the containment have an integrally forged pipe fitting as part of the five-way restraints.
		B.1.b.(6)	Conforms. See paragraph 3.6.2.4.2.
		B.1.b.(7)	Conforms. See paragraph 3.6.2.1.1.D.
B.1.c	Postulation of pipe rupture in areas other than containment penetration.	B.1.c	Conforms. See paragraph 3.6.2.1.1.
		B.1.c.(1)(a)-(d)	Partial conformance. If there are no intermediate locations where maximum stress ranges or cumulative usage factors exceed the threshold levels, no intermediate breaks are postulated provided the conditions of paragraph 3.6.2.1.1.A.2.C are satisfied. See paragraph 3.6.2.1.1.A. There are no postulated pipe breaks

TABLE 3.6.1-3 (SHEET 2 OF 3)

	Branch Technical Position MEB.3-1		VEGP Design
			in the reactor coolant loops of Units 1 and 2, nor are breaks postulated in the Class 1 branch lines of Unit 2 as discussed in Section 3.6.
		B.1.c.(2)(a)-(b)	Partial conformance. If there are no intermediate locations where maximum stress ranges or cumulative usage factors exceed the threshold levels, no intermediate breaks are postulated provided the conditions of paragraph 3.6.2.1.1.B.4 are satisfied. See paragraph 3.6.2.1.1.B.
		B.1.c.(3)	Partial conformance. See paragraph 3.6.2.1.1.C.
		B.1.c.(4)	Conforms. See paragraphs 3.6.2.1.1.B and 3.6.2.1.1.C. Nonnuclear high-energy pipes will either be restrained from impacting or affecting the separating structure or the separating structure will be designed for full effects.
		B.1.d.	Conforms. See paragraph 3.6.2.5.
		B.1.e.	Conforms. See paragraph 3.6.2.1.2.2.D. In the absence of mechanistic (terminal end or high stress) break locations. Breaks or critical cracks are postulated in ASME Section 3, Class 1, 2, and 3 and Nonsafety Class Piping, at locations that result in the most severe environmental consequences.
B.2	Moderate-Energy Fluid System Piping	B.2.a	Conforms. See paragraph 3.6.1.3 and appendix 3F.
		B.2.b	Conforms. See paragraph 3.6.2.1.2.4.
		B.2.c.(1)-(2)	Conforms. See paragraphs 3.6.2.1.2.4.
		B.2.d	Conforms. See paragraph 3.6.2.1.2.4.
		B.2.e	Conforms. See paragraph 3.6.1.1.A.
B.3	Type of Breaks and Leakage Cracks in Fluid System Piping	B.3.a.(1)	Conforms. See paragraph 3.6.2.1.2.2.
		B.3.a.(2)	Conforms. All high-energy Class 1, 2, and 3 piping systems are analyzed using industry approved computer programs in piping stress analysis. In the absence of stress analysis, nonnuclear class high-energy piping breaks are postulated at all welds, fittings, welded attachments, etc. See paragraph 3.6.2.1.1.
		B.3.a.(3)	Conforms. See paragraph 3.6.2.1.3.1.
		B.3.a.(4)	See paragraph 3.6.2.2.1.
		B.3.a.(5)	Conforms. See paragraph 3.6.1.1.J.
		B.3.b.(1)	Conforms. See paragraph 3.6.2.1.2.2.
		B.3.b.(2)	Per paragraph 3.6.2.1.2.2, only circumferential breaks are postulated at terminal ends, even if a longitudinal pipe weld is present at that point.

TABLE 3.6.1-3 (SHEET 3 OF 3)

Branch Technical Position MEB.3-1		VEGP Design
	B.3.b.(3)	Conforms. See paragraph 3.6.2.1.3.1.
	B.3.b.(4)	See paragraphs 3.6.2.2.1, 3.6.1.1.J, and 3.6.1.1.K.
	B.3.b.(5)	Conforms. See paragraphs 3.6.1.1.J and 3.6.2.1.3.1.
	B.3.c.(1)	Conforms. See paragraph 3.6.2.1.2.4.
	B.3.c.(2)	Conforms. See paragraph 3.6.2.1.3.2.
	B.3.c.(3)	Conforms. See paragraph 3.6.2.1.2.4.

a. This table summarizes conformance as to ASB 3-1 and MEB 3-1 which also covers the implementation of WCAP-8082-P-A and WCAP-8172-A.

TABLE 3.6.2-1

HIGH-ENERGY PIPE BREAK STRESS ANALYSIS RESULTS

For the high energy pipe break stress analysis results refer to the individual stress isometric drawings submitted in reference 13.

TABLE 3.6.2-2

HAZARD ANALYSIS RESULTS

Deleted. Refer to table 3F-1 and reference 13.

TABLE 3.6.2-3

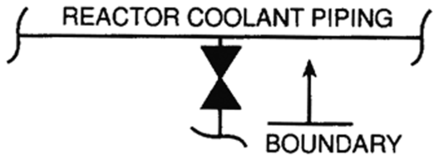
POSTULATED BREAK LOCATIONS FOR THE LOCA ANALYSIS
OF THE PRIMARY COOLANT LOOP(a)

Location of Postulated Rupture	Type	Break Opening Area	Area[b]
1. Residual heat removal (RHR) line/primary coolant loop	Guillotine (viewed from the RHR line)	Cross-sectional flow area of the RHR line	86.55 in.^{2}
2. Accumulator (ACC) line/primary coolant loop connection	Guillotine (viewed from the ACC line)	Cross-sectional flow area of the ACC line	60.10 in.^{2}

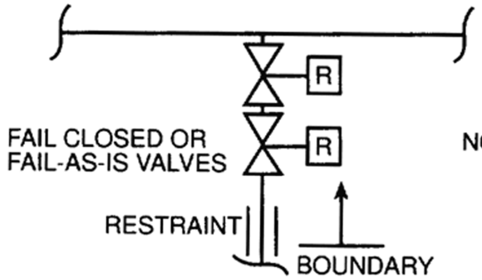
a. Refer to drawing AX6DD309 for location of postulated breaks in reactor coolant lines. This table applies to Unit 1 only. No breaks on Unit 2.

b. Less break opening area is used where justified by analysis, equipment, or consideration of physical restraints such as concrete walls or structural steel.

CASE I OUTGOING LINES WITH NORMALLY CLOSED VALVE

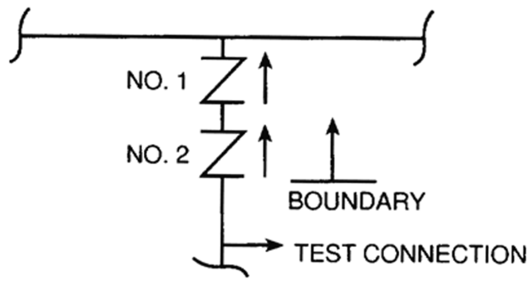


CASE II OUTGOING LINES WITH NORMALLY OPEN VALVES

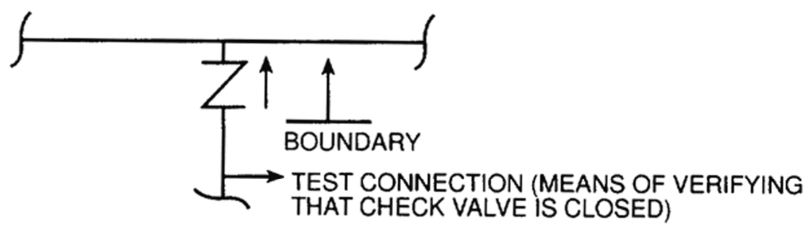


NOTE: THE REACTOR COOLANT PUMP NO. 1 SEAL IS ASSUMED TO BE EQUIVALENT TO FIRST VALVE

CASE III INCOMING LINES NORMALLY WITH FLOW



CASE IV INCOMING LINES NORMALLY WITHOUT FLOW



CASE V ALL INSTRUMENTATION TUBING AND INSTRUMENTS CONNECTED DIRECTLY TO THE REACTOR COOLANT SYSTEM IS CONSIDERED AS A BOUNDARY. HOWEVER, A BREAK WITHIN THIS BOUNDARY RESULTS IN A RELATIVELY SMALL FLOW WHICH CAN NORMALLY BE MADE UP WITH THE CHARGING SYSTEM.

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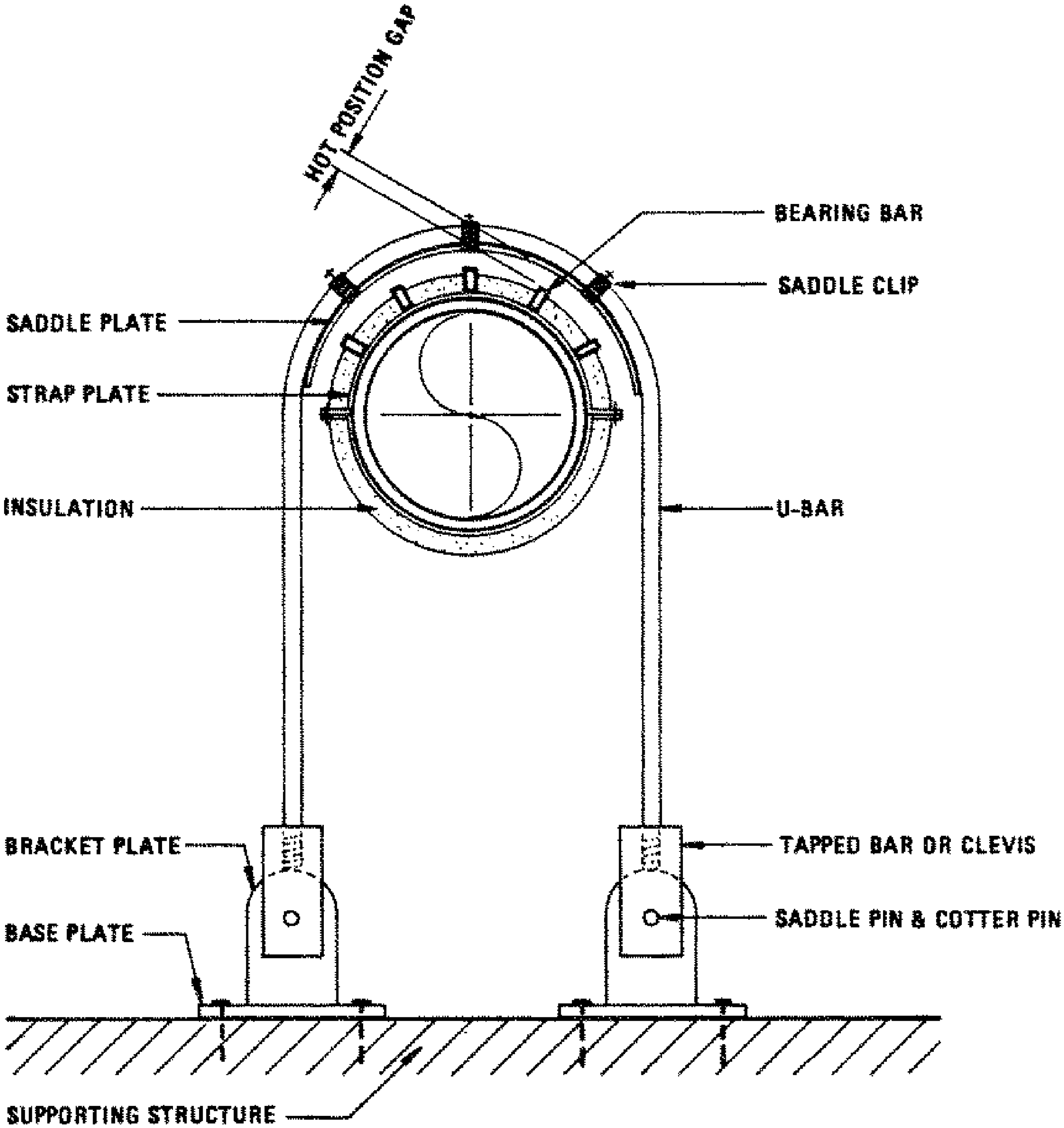
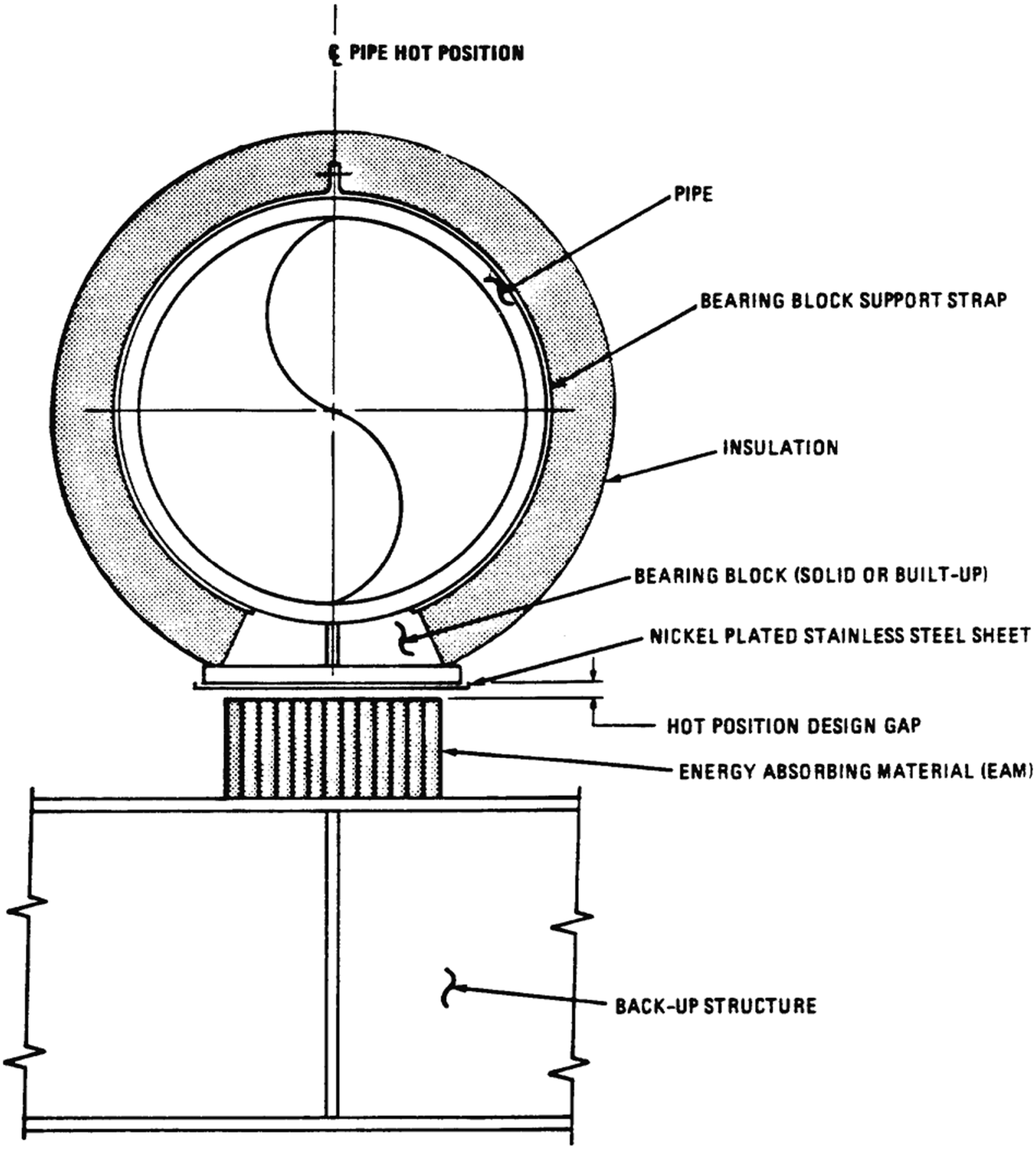
SOUTHERN COMPANY *Energy to Serve Your World®*	**VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2**	LOSS OF REACTOR COOLANT ACCIDENT BOUNDARY LIMITS
		FIGURE 3.6.2–1



REV 14 10/07

SOUTHERN COMPANY Energy to Serve Your World®	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	TYPICAL U–BAR RESTRAINT FIGURE 3.6.2–2



REV 14 10/07

SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	TYPICAL EAM RESTRAINT FIGURE 3.6.2–3

3.7.B SEISMIC DESIGN

All structures, systems, equipment, and components related to plant safety systems are required to have the ability to withstand potential earthquakes. Each structure, system, equipment, and component is placed in the applicable seismic category, depending on its function. A two-level system is used for the seismic classification of structures, systems, and equipment of the facility. A definition of the seismic classifications and a listing of structures, systems, and equipment are included in table 3.2.2-1.

Seismic loadings are characterized by the safe shutdown earthquake (SSE) and the operating basis earthquake (OBE). The SSE is defined as the maximum vibratory ground motion at the plant site that can be reasonably predicted from geologic and seismic evidence. The OBE is that earthquake which, considering the local geology and seismology, can be reasonably expected to occur during the plant life.

All Category 1 structures are designed for SSE and OBE conditions. Category 2 buildings are designed using the Uniform Building Code, 1976 edition. In addition, the radwaste transfer building, the radwaste transfer tunnel, the radwaste solidification building, the basemat of the alternate radwaste building, and the radwaste processing facility are designed to withstand the OBE event. Category 2 structures, systems, equipment, and components, whose failure could result in the loss of required safety functions of adjacent Category 1 structures, systems, equipment, or components shall be either separated by distance or barrier from the affected structure, system, equipment, or component or investigated for SSE loadings to ensure that their failure will not impair the safety-related functions of the adjacent Category 1 structures, systems, equipment, or components.

For Westinghouse-supplied items, refer to section 3.7.N.

3.7.B.1 SEISMIC INPUT

The seismic criteria for VEGP are developed by the Geology Department of the Hydro and Community Facilities Division of the Bechtel Corporation in San Francisco.

The plant site geologic and seismologic investigations are covered in section 2.5. Based on this data, the peak ground accelerations for SSE and OBE are established as 0.20 g and 0.12 g, respectively, as discussed in subsection 2.5.2.

3.7.B.1.1 Design Response Spectra

The VEGP site design response spectra are provided in figures 3.7.B.1-1 and 3.7.B.1-2 for the horizontal and vertical components of the SSE and in figures 3.7.B.1-3 and 3.7.B.1-4 for the horizontal and vertical components of the OBE. The design response spectra are in conformance with Regulatory Guide 1.60, Design Response Spectra for Seismic Design of Nuclear Power Plants.

3.7.B.1.2 Design Time-History

Synthetic earthquake acceleration time-histories are used as the basic input in the dynamic analysis of Category 1 structures under SSE and OBE conditions. The basis for the generation of the synthetic time-histories is discussed in section 2.5 of BC-TOP-4A.[1] Figures 3.7.B.1-5 and 3.7.B.1-6 show the synthetic acceleration time-history motions in the horizontal and vertical directions. Comparison between the free-field time-history response spectra, the design response spectra for both horizontal and vertical motions, and the frequencies at which the spectra values were calculated is provided in section 2.5 of BC-TOP-4A.[1]

In order to satisfy the input requirements of the FLUSH computer program that is used for the soil-structure interaction analyses of the deeply embedded structures, the following operations are performed on the original 4800-point time-histories digitized at 0.005 s to yield 2048-point time-histories digitized at 0.01 s. The time interval of the original 24 s time-histories is increased from 0.005 s to 0.01 s through the use of the computer program SHAKE, and the 20.48-s synthetic time-history motions are obtained by adopting the first 18 s of the time-histories followed by 2.48 s of a quiet zone. The differences between the response spectra derived from these motions and the response spectra obtained from the original time-histories are insignificant.

The synthetic time-history motions are scaled to 0.20 g and 0.12 g to obtain, respectively, the SSE design time-history and the OBE design time-history. The comparison of the response spectra obtained in the free field at the foundation level with the design response spectra is discussed in paragraph 3.7.B.2.4.1.

3.7.B.1.3 Critical Damping Values

Energy dissipation in a structural system is represented by equivalent viscous dampers. Evaluation of the damping coefficients is based on material, loading conditions, and type of connections used in the structural system. The damping values used in the dynamic analysis are those in table 3.7.B.1-1. The values are the same as those provided in Regulatory Guide 1.61, Damping Values for Seismic Design of Nuclear Power Plants, with the exception of damping values for cable trays and supports. The damping values for cable trays and supports are values based on test results[2] and were approved by the Nuclear Regulatory Commission in reference 3. The reports requested in reference 3 were provided to the commission in reference 4.[2] Damping values for heating, ventilation, and air-conditioning ducts and supports are those indicated in table 3.7.B.1-1 for welded or bolted steel structures, as applicable.

For Seismic analysis of ASME Boiler and Pressure Vessel Code, Section III, Division 1, Code Class 1, 2, and 3 piping systems, ASME Code Case N-411 damping values presented in figure 3.7.B.1-11 may be used in lieu of the values given in table 3.7.B.1-1, provided the following commitments are satisfied:

- Increased pipe deflections due to greater piping flexibility shall not violate project separation criteria.
- Criteria outlined in Regulatory Guide 1.61 shall not mix with the criteria of Code Case N-411 for a given piping analysis.
- As part of the integrated piping analysis/as-built reconciliation program, increased piping displacements and clearances shall be reviewed for acceptance.
- The N-411 values for seismic analysis apply to the primary loop piping systems and to other piping systems. These damping values, illustrated in figure 3.7.B.1-11, may be utilized only for piping systems analyzed by the response spectra method. Damping values, illustrated in table 3.7.B.1-1 shall be retained for piping analysis which utilizes the time-history integration method.

- With the exception of those stress calculations described in reference 9, Code Case N-411 damping values are not used in conjunction with multiple response spectrum methodology piping analysis.

The damping values presented in figure 3.7.B.1-11 were approved by the Nuclear Regulatory Commission in reference 8.

Consistent with Regulatory Guide 1.61, damping values higher than those listed in table 3.7.B.1-1 may be used if justified by test results.

For soil damping, strain-dependent damping values are used as given in figures 3.7.B.1-8, 3.7.B.1-9, and 3.7.B.1-10.

The damping values for compacted sand backfill shown in figure 3.7.B.1-8 are average values based on test results.(5) The clay marl bearing stratum is highly overconsolidated and has undrained shear strengths in excess of 10 k/ft^2. Because of this, the damping in this stratum would be somewhat lower than those for soft clays. The damping values shown in figure 3.7.B.1-9 are considered appropriate for the hard clay marl bearing stratum encountered at the site. The damping values for the lower sand stratum shown in figure 3.7.B.1-10 are based on the Seed and Idriss curve.(6)

3.7.B.1.4 Supporting Media for Seismic Category 1 Structures

The depth of bedrock below the plant site is approximately 950 ft, as defined in paragraph 2.5.1.2.1.3. The nominal finished grade level is el 220 ft. The explored depth at the site indicates an overburden which may be divided into the three distinct soil strata listed below:

- Upper sand stratum: sands and clayey sands, varying from loose to dense, to a depth of 75 to 90 ft.
- Clay bearing stratum: very hard, sandy, calcareous clay marl about 65 ft thick.
- Lower sand stratum: clean to silty, medium- to fine-grained dense sands below the marl to undetermined depth.

In the power block area containing the Category 1 structures, the upper sand stratum material and approximately the top 5 ft of clay marl bearing stratum are removed approximately to el 130 ft. Select engineered compacted sand backfill is placed on the top of the clay marl bearing stratum up to the design elevations of Category 1 structure foundations with the exception of the auxiliary building and the nuclear service cooling water (NSCW) towers, which are founded on the clay bearing stratum. Select compacted sand backfill is also placed on the sides of Category 1 structures up to grade elevation.

Values of low strain shear modulus (at shear strain less than or equal to 10^{-4} percent) of the compacted backfill are computed using the expression G = $(1000\ K_2\ (\sigma'_m)^{1/2}$ lb/ft^2), where G is the shear modulus in lb/ft^2; σ'_m is the mean principal effective stress in lb/ft^2; and K_2, the parameter reflecting primarily the effect of void ratio or relative density and the strain amplitude of the motions, is taken as 80.(5) The low strain shear moduli of the clay bearing stratum and the lower sand stratum are computed using the average measured shear wave velocities of 1700 ft/s and 1800 ft/s, respectively.(7) The unit weights of compacted backfill in moist and saturated conditions are 123 lb/ft^3 and 133 lb/ft^3, respectively. The saturated unit weight of clay bearing stratum and lower sand stratum is 115 lb/ft^3.

Table 3.7.B.1-2 includes foundation embedment depth, width of the structural foundation, and total structure height for Category 1 structures. Subsection 2.5.4 describes the soil properties, shear wave velocity, shear modulus, and density.

3.7.B.1.5 Standard Review Plan Evaluation

For deeply embedded Seismic Category 1 structures, the design ground motion (control motion) is applied at the finished grade level in the free field, instead of applied at the foundation levels of Category 1 structures in the free field.

Refer to paragraph 3.7.B.2.1 for an evaluation of the Standard Review Plan differences.

3.7.B.1.6 References

1. "Topical Report Seismic Analysis of Structures and Equipment for Nuclear Power Plants, Revision 3," BC-TOP-4A, November 1974.
2. "Cable Tray and Conduit Raceway Seismic Test Program, Release 4," Report 1053-21.1-4, ANCO Engineers, Inc., December 15, 1978.
3. NRC letter, Docket Nos. 50-424 and 50-425, dated February 12, 1982.
4. Georgia Power Company letter, Docket Nos. 50-424 and 50-425, to NRC, dated March 5, 1982.
5. Alvin W. Vogtle Nuclear Plant, Report on Dynamic Properties for Compacted Backfill, Bechtel Incorporated, Los Angeles, California, February 1978.
6. Seed, H.B., and Idriss, I.M., "Soil Moduli and Damping Factors for Dynamic Response Analysis," Earthquake Engineering Research Center, EERC 70-10, University of California, Berkeley, California, December 1970.
7. Alvin W. Vogtle Nuclear Plant, Report on Foundation Investigations, Volume 1, Bechtel Incorporated, San Francisco, California, July 1974.
8. NRC letter, Docket Nos. 50-424 and 50-425, to Georgia Power Company, dated March 18, 1985.
9. Georgia Power Company letter (GN-1257) Docket No. 50-424 and 50-425, to NRC, Dated December 22, 1986.

3.7.N SEISMIC DESIGN

This section describes the seismic design methods for nuclear steam supply system (NSSS) equipment.

In addition to the steady-state loads imposed on the system under normal operating conditions, the design of equipment and equipment supports requires consideration of abnormal loading conditions such as earthquakes. Seismic loadings are characterized by the safe shutdown earthquake (SSE) and operating basis earthquake (OBE). The SSE is defined as the maximum vibratory ground motion at the plant site that can reasonably be predicted from geologic and seismic evidence. The OBE is that earthquake which, considering the local geology and seismology, can be reasonably expected to occur during the plant life.

For the OBE loading condition, the NSSS is designed to be capable of continued safe operation. The design for the SSE is intended to ensure the following:

A. That the integrity of the reactor coolant pressure boundary is not compromised.

B. That the capability to shut down the reactor and maintain it in a safe condition is not compromised.

C. That the capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10 CFR 100 is not compromised.

The seismic qualification of safety-related instrumentation and electrical equipment is discussed in sections 3.10.N and 3.10.B. The safety class definitions and classification lists are given in section 3.2.

3.7.N.1 SEISMIC INPUT

3.7.N.1.1 Design Response Spectra

Refer to paragraph 3.7.B.1.1.

3.7.N.1.2 Design Time-History

The seismic analysis of the reactor coolant system utilizes nonlinear, three-dimensional, time-history dynamic analysis methods and a coupled building internals structure/reactor coolant loop model. The coupled model is subjected to three components of earthquake simultaneously at the basemat. The six time-history components, the north-south and east-west horizontal directions and the vertical direction, are statistically independent and are applied simultaneously for 10 s of OBE and SSE, which represents the most severe portion of the earthquake acceleration.

In order to perform the coupled building/loop analysis, it is necessary to construct a synthesized time-history motion. These time-histories are developed to conform to the three translational and three rotational response spectra from the soil-structure interaction analyses of the containment building described in section 3.7.B. The generation of these motions is accomplished by modifying existing earthquake records using spectral raising and suppressing techniques. With data from a real earthquake as input, spectral raising is accomplished by adding to the original time-history a function at the frequency of interest with a phase angle such that the response spectra value will be increased a desired amount. The time when the maximum vibration occurred will be the same. In this way, the characteristics of the required time-history will only be slightly altered. Spectral suppression is carried out by passing the time-history through a linearly damped oscillator connected in series to a second damper. This damping arrangement will reduce the response spectral value, locally, at the natural frequency of the oscillator to the desired amount. The repetitious application of the raising and suppressing techniques is used to arrive at a time-history motion whose response spectrum is sufficiently close to the design spectrum.

The statistical independence of the six synthetic acceleration time-histories is established by comparing statistical properties of the synthetic time-histories with properties derived from recorded earthquake accelerograms. In particular, the values of the normalized correlation coefficient at zero time delay and the average value of the coherence function over the seismic frequency range are calculated for the synthetic and real time-histories and shown to be comparable.

3.7.N.1.3 Critical Damping Values

The damping values given in table 3.7.N.1-1 are used in the systems analysis of Westinghouse equipment. These are consistent with the damping values recommended in Regulatory Guide 1.61 except in the case of the primary coolant loop system components and large piping (excluding reactor pressure vessel internals), for which the damping values of 2 and 4 percent are used as established in testing programs reported in reference 1.

As an alternative to Regulatory Guide 1.61, the damping values from ASME Code Case N-411 are used for piping system analysis with response spectra analysis techniques per reference 5. The Code Case N-411 frequency dependent damping values are illustrated in figure 3.7.N.1-1. No mixture of Regulatory Guide 1.61 criteria with the N-411 criteria is allowed in a given piping analysis. As part of the integrated piping analysis/as-built reconciliation program, GPC was assured that piping displacements and clearances are acceptable when N-411 criteria is applied. The damping values for control rod drive mechanisms (CRDMs) and the fuel assemblies of the NSSS, when used in seismic analysis, are in conformance with the values for welded and/or bolted steel structures (as appropriate) listed in Regulatory Guide 1.61. Consistent with Regulatory Guide 1.61, damping values higher than those listed in table 3.7.N.1-1 may be used if justified by test results.

Tests on fuel assembly bundles have justified conservative component damping values of 7 percent for OBE and 10 percent for SSE to be used in the fuel assembly component qualification. Documentation of the fuel assembly tests is provided in reference 2.

The damping values used in component analysis of CRDMs and their seismic supports were developed through a testing program performed by Westinghouse. The program consisted of transient vibration tests in which the CRDM was deflected a specified initial amount and suddenly released. A logarithmic decrement analysis of the decaying transient provides the effective damping of the assembly. Documentation of the CRDM tests is provided in references 3 and 4.

The test results indicated that the damping would be greater than 8 percent for both the OBE and the SSE based on a comparison between typical deflections during these seismic events to the initial deflections of the mechanisms in the test. Component damping values of 5 percent are, therefore, conservative for both OBE and SSE.

3.7.N.1.4 Supporting Media for Seismic Category 1 Structures

Refer to paragraph 3.7.B.1.4.

3.7.N.1.5 References

1. "Damping Values of Nuclear Plant Components," WCAP-7921-AR, May 1974.
2. Gesinki, T. L., and Chiang, D., "Safety Analysis of the 17x17 Fuel Assembly for Combined Seismic and Loss of Coolant Accident," WCAP-8236 (Proprietary), December 1973, and WCAP-8288 (Nonproprietary), January 1974.

3. Obermeyer, F. D., "Effective Structural Damping of the KEP L105 CRDM," WCAP-7427 (Proprietary), January 1970.

4. Obermeyer, F. D., WCAP-7427, Addendum 1 (Proprietary) December 1970.

5. "Piping Design Criteria for Vogtle, Units 1 and 2", NRC letter, T. M. Novak to D. O. Foster, March 18, 1985.

3.7.B.2 SEISMIC SYSTEM ANALYSIS

Category 1 structures, systems, and components are classified in accordance with Regulatory Guide 1.29. Seismic systems are defined as Category 1 structures considered in conjunction with foundation media in forming a soil-structure interaction model. All Category 1 structures not designated as seismic systems and all Category 1 systems, such as heating, ventilation, and air-conditioning systems, electrical cable tray, and piping are considered as seismic subsystems, and their analyses are described in subsection 3.7.B.3.

3.7.B.2.1 Seismic Analysis Methods

The acceptance criteria of Sections 3.7.1 and 3.7.2 of the Standard Review Plan (SRP) (7/81) require that the control motion be applied at the foundation levels of Category 1 structures in the free field.

In addition, the acceptance criteria of Section 3.7.2 require that modeling methods for implementing the soil structure interaction analysis include both the half space and finite boundaries approaches and that Category 1 structures, systems, and components be designed to accommodate responses obtained by one of the following:

- Envelope of results of the two methods.
- Results of one method with conservative design considerations of the effects from the use of the other method.
- Combination of the above with the provision of adequate conservatism in the design.

In the VEGP design, for shallowly embedded Category 1 structures, the control motion is applied at foundation levels of the structures in the free field, and soil-structure interaction analyses are performed using the impedance (half space) method. For deeply embedded Category 1 structures, the control motion is applied at the finished grade level in the free field, and soil-structure interaction analyses are performed using the finite element method.

VEGP design methods are based on the SRP (11/24/75) in effect in the years 1977 and 1978, during which period the VEGP design methods evolved from the discussions held during meetings with the Nuclear Regulatory Commission (NRC). The concerns expressed by the NRC staff in these meetings were addressed in Preliminary Safety Analysis Report (PSAR) Supplements 3 and 4 and in the

Georgia Power Company (GPC) letter to the NRC dated February 20, 1978, in which VEGP committed to multiply the envelope in-structure response spectra for the deeply embedded Category 1 structures by a scaling factor of 1.5, consistent with the SRP acceptance criteria stated above. (The scaling factor value of 1.5 was later incorporated through PSAR Supplement No. 5 of November 17, 1978.)

The NRC, in its letter to GPC dated March 27, 1978, accepted the seismic design methods proposed in PSAR Supplements 3 and 4 with the additional information provided in the GPC letter of February 20, 1978, on the scaling factor, subject to the completion of a confirmatory study and a sensitivity study.

The confirmatory study addressed the NRC staff's concerns on comparing the results of the two methods of soil-structure interaction analyses. The sensitivity study provided the justification for applying the deconvolved control motions at the foundation levels of deeply embedded Category 1 structures.

In the confirmatory study, the response spectra calculated from the finite element method of soil-structure interaction using the VEGP design procedure were compared with those obtained using the impedance (half space) method to confirm that the VEGP seismic design methods are founded on conservative design bases. The objective of the sensitivity study was to demonstrate that the deconvolution procedure in conjunction with the specification of the control motion (based on Regulatory Guide 1.60 response spectra) at the grade level in the free field, is applicable for the VEGP site.

The NRC also requested that the seismic analysis include consideration of torsional moment no less than that required by the Uniform Building Code (to account for the seismic wave propagation effects), in addition to the effects resulting from the eccentricity between the center of mass and center of rigidity at each level.

The reports on the confirmatory study and the sensitivity study together with the description of the methodology to account for torsion caused by the seismic wave propagation effects were submitted to the NRC in the GPC letter dated November 13, 1978. These reports are reproduced in section 3D.2.

For the confirmatory study, the containment building and the control building were analyzed to compute the in-structure response spectra using the VEGP design procedure for the finite element method of soil-structure interaction analysis of deeply embedded structures, applying the control motion at the finished grade level in the free field. This included the multiplication of the envelope in-structure response spectra by a scaling factor of 1.5. The calculated response spectra were compared with those obtained from the impedance (half space) method, wherein the control motion was applied at the foundation levels of the structures in the free field. The comparison showed that, in general, the response spectra from the VEGP design procedure and those from the impedance method exhibited similar characteristics in terms of accelerations and frequencies, despite the fact that there are differences in the methods of modeling and analysis. Additionally, a comparison of the VEGP design in-structure response spectra with the impedance method response spectra presented provided in the confirmatory study is provided in section 3D.3.

For the sensitivity study, a series of ground response studies were performed to determine the variation of ground motion with depth at the VEGP site. Recorded seismic rock motions were used to establish bedrock motions, which were then propagated upward through the site soils from the underlying bedrock. Recorded seismic motions on soil deposits with generally similar characteristics to those at the VEGP site were deconvolved through the soil profile. The computed responses from these two procedures were compared with each other and with the results of the seismic motions used in the VEGP design, namely the Regulatory Guide 1.60 design spectra applied at the grade level in free field and the motions at the foundation levels obtained by the deconvolution procedure.

The sensitivity study in its draft form was presented to the NRC staff on July 12, 1978, and was accepted by the NRC staff with comments. The staff's comments were incorporated into the report. The good agreement between the response spectra obtained from the analyses of the actual recorded earthquake motions and the response spectra obtained by the deconvolution of the Regulatory Guide 1.60 response spectra supports the use of the later spectra and motions obtained by its deconvolution for the VEGP design.

Therefore, the VEGP seismic analyses are based on the design methods presented in PSAR Supplements 3, 4, and 5 and are supported by the reports submitted to the NRC in the GPC letter dated November 13, 1978. The VEGP seismic design methods are discussed in detail in the following sections.

The seismic systems are analyzed for operating basis earthquake (OBE) and safe shutdown earthquake (SSE) conditions. Depending on whether the structure has shallow embedment or deep embedment, different methods are used for the seismic analysis. As shown in table 3.7.B.2-1, structures are classified as deeply embedded, shallowly embedded, or buried.

For shallowly embedded structures, foundation torsion, rocking, and translation effects are taken into consideration in the computation of equivalent soil spring constants and damping coefficients, as described in BC-TOP-4A.(1) A modal time-history analysis method is used for the analysis of these structures. The number of masses used, the number of modes considered, and the combination of modal responses are in accordance with BC-TOP-4A. For deeply embedded structures, foundation rocking and translation effects are accounted for through modeling the soil around the structures as finite elements. A complex response time-history analysis method is used for the analysis of deeply embedded structures. The number of masses used to represent the structure are as described in BC-TOP-4A. The buried structures essentially move with the ground, and the response of the structure is the same as the ground response. The hydrodynamic effects, if any, are modeled based on TID-7024.(2) Typical models of a shallowly embedded structure and deeply embedded structures are shown in figures 3.7.B.2-1 through 3.7.B.2-4.

3.7.B.2.2 Natural Frequencies and Response Loads

The seismic system analyses of the containment building and other major Category 1 structures are performed using a complex response time-history method. The final response spectra generated from this method is indicative of the frequency content of the soil-structure system. Natural frequencies, mode shapes, or modal responses are not obtained in this method as in the modal response spectra analysis method. Accelerations at the selected levels of major Seismic Category 1 structures are presented in table 3.7.B.2-2. These are indicative of response loads for major Seismic Category 1 structures. Selected response spectra determined by seismic analysis for major Seismic Category 1 structures are given in section 3D.1.

3.7.B.2.3 Procedure Used for Modeling

Shallowly embedded structures and deeply embedded structures, as listed in table 3.7.B.2-1, have different methods of modeling for the dynamic seismic analysis. For structures that have shallow embedment, the lumped parameter method is used to represent the soil-structure interaction. For structures that are deeply embedded, the finite element method is used to represent the soil-structure interaction. In both the methods, the buildings are modeled using beam elements and lumped masses.

3.7.B.2.4 Soil-Structure Interaction

The extent of structural embedment is listed in table 3.7.B.1-2. The depth of soil over rock and the soil stratum layering are described in paragraph 3.7.B.1.4.

3.7.B.2.4.1 Deeply Embedded Structures

The finite element method is used for structures having deep embedment to account for embedment effects and the effects of structure-to-structure interaction. The analytical model is provided with transmitting boundaries both on the left and right sides. The model consists of two types of elements: displacement-compatible isoparametric quadrilateral elements (solid elements) and linear bending elements (beam elements). Usage of transmitting boundaries, elements, and analytical techniques are described by Lysmer, et al.[5] The computer program FLUSH of the same reference is used to perform the analysis.

Soil properties such as shear moduli, Poisson's ratios, and densities for the various soil strata are established from the soils investigation and additional soil testing to establish the dynamic properties of compacted backfill as described in paragraph 3.7.B.1.4. The strain dependency of shear moduli and damping ratios for compacted sand backfill, clay marl bearing stratum, and the lower sand stratum are based on the standard curves proposed by Seed and Idriss[6] with appropriate modifications to account for the in situ soil conditions and backfill characteristics.

In the analysis, the strain-dependent shear moduli as shown in figures 3.7.B.2-5 through 3.7.B.2-7 are used.

In figure 3.7.B.2-5, the curve showing the variation of shear modulus with shear strain is shown for the compacted sand backfill. The curve is based on the average values obtained from test results.[7] The variation of shear modulus with shear strain for the clay marl bearing stratum is shown in figure 3.7.B.2-6. Because the marl is essentially a hard clay, the shear modulus will decrease with increasing shear strain but at a lesser rate than that applicable for soft clays. The shear modulus variation shown is therefore appropriate for the site condition and will be used in the analysis. The variation of shear modulus with shear strain for the lower sand stratum is shown in figure 3.7.B.2-7. This is based on the standard curve proposed by Seed and Idriss.

To account for the variation in soil properties, shear moduli with upper-bound values equal to 1.5 times the mean values and lower-bound values equal to the mean values divided by 1.5 are used in the analysis. The mean values of low strain shear moduli are computed as described in paragraph 3.7.B.1.4.

As discussed in paragraph 3.7.B.1.3, the damping values for the compacted sand backfill, the clay marl bearing stratum, and the lower sand stratum shown in figures 3.7.B.1-8 through 3.7.B.1-10 are used in the analysis.

In general, the soil properties are nonlinear in character. An iterative process is used to obtain equivalent linear properties which are strain dependent. The methods generally used for such an analysis are included in the computer program FLUSH.[5]

In the analyses for the vertical component of the earthquake, the soil properties for the layers below the water table are based on the iterated strain-dependent soil properties or a compression wave velocity of 5000 ft/s, whichever is greater. This is consistent with the assumption that, in saturated soils, the compression wave would travel with the compression wave velocity of the soil medium or the compression wave velocity of water, whichever is greater. The compression wave velocity of water has been assumed to be 5000 ft/s.

The generation of design time-history motions is described in paragraph 3.7.B.1.2. This ground motion is defined for the free field and applied at the finished grade level (el 220 ft 0 in.) of the site.

The time-history at the base of the idealized soil profile is obtained through deconvolution analysis of the design time-history specified at finished grade level, using appropriate soil properties. The time-history thus obtained is applied at the base of the soil-structure interaction system with appropriate soil properties for soil-structure interaction analysis. The resulting time-history responses are used to generate the in-structure response spectra at selected floor elevations. The analysis is performed with consideration given to the variation of soil parameters as indicated above using appropriate cutoff frequencies such that the acceleration profile in the free field is realistic. The envelope in-structure response spectra are developed by enveloping the response spectra obtained by considering the variation of soil properties. The envelope in-structure response spectra curves are multiplied by the scaling factor of 1.5, the basis for which is described in the following paragraph.

Response spectra corresponding to the free field time-history motions calculated at the elevations of Category 1 structural foundations are generated. Considering the variation of soil properties, envelope response spectra for each Category 1 foundation level are developed. The comparison of the envelope response spectra thus obtained in the free field at the foundation levels of deeply embedded Category 1 structures with 60 percent of the design response spectra is provided in figures 3.7.B.2-8 through 3.7.B.2-27. A scaling factor of 1.5 is selected so that when the envelope response spectra curves are multiplied by the scaling factor, the 60-percent design spectra curves are essentially enveloped.

The dynamic analysis performed using the computer program FLUSH is two dimensional, and any three-dimensional analysis is an approximation. The procedure for computing the three-dimensional response of the structures using a two-dimensional soil model is described below. This procedure combines a two-dimensional finite element representation of soil with a three-dimensional representation of structures.

First, a three-dimensional lumped mass model of the structure is created and expressed in the form of stiffness and mass matrices. A two-dimensional model of the soil with the structure removed is prepared and all nodes in contact with the structure (henceforth called common nodes) identified (figure 3.7.B.2-28).

The structure nodes associated with the common nodes have degrees of freedom only in the plane of the soil model in order that the FLUSH program can be executed. The reduction of degrees of freedom of the structure common nodes is accomplished through a mathematical transformation. There is no requirement that the degrees of freedom for the remaining structure nodes be reduced.

After the common degrees of freedom have been made compatible both in the structure and in the soil, then the total soil-structure system is assembled in global matrices and the solution is accomplished by FLUSH, as in a standard finite-element problem.

The response of the structure nodes that are not associated with the common nodes is in three orthogonal directions due to excitation in any one direction, and hence the codirectional responses due to both of the horizontal earthquakes and vertical earthquake can be obtained directly. This approach inherently accounts for the torsional effects in the structure.

In the soil finite element model, the side transmitting boundaries are located three elements away from the structures. This is consistent with the FLUSH program recommendations. The bottom boundary for the FLUSH model is taken so that it is at least at a depth one-half the model dimension of the basemat below that basemat level.

A total of six mathematical models are employed in the analysis of deeply embedded structures as shown in figure 3.7.B.2-2. The first is an east-west model which includes the auxiliary building. The second is also an east-west model which consists of the containment Unit 2, the fuel handling building, and the containment Unit 1. The effect of the diesel generator buildings on the response of the containment is accounted for by modeling their inertial properties with structural layers in the soil finite-element model. The third is an east-west model which includes the control building. The fourth is a north-south model which includes the auxiliary building, containment Unit 1, the control building, and the turbine building. Since the turbine building is a non-Category 1 structure, it is only necessary to consider its effect on adjacent Category 1 structures. Therefore, it is sufficient to model it as a structural layer in the soil finite-element model with proper inertial properties, without modeling the superstructure. The fifth is also a north-south model which includes the auxiliary building, the fuel handling building, the control building, and the turbine building. The sixth is a model which includes a nuclear service cooling tower. Considering the plant layout, it is assumed that there is no significant interaction between each of the nuclear service cooling water (NSCW) towers and the rest of the structures. Two typical FLUSH models are shown in figures 3.7.B.2-3 and 3.7.B.2-4.

The NSCW valve house is a single-story structure with insignificant equipment loads. The structural response of the valve house is dictated by the driving influence of the adjacent massive NSCW tower. Because of the relatively small size of the valve house and its close proximity to the tower, the response spectra of the tower at grade level is used as the response spectra of the valve house.

3.7.B.2.4.2 Shallowly Embedded Structures

For structures founded on the ground surface or having shallow embedment, the lumped parameter approach is used to represent the soil-structure interaction. The details of this method are described in BC-TOP-4A.[(1)] Strain-dependent soil properties are used in the computation of the impedances. The procedures by which soil layering effects are considered are discussed in appendix 3E. A typical model is shown in figure 3.7.B.2-1.

3.7.B.2.4.3 Buried Structures

Buried structures are surrounded by soil around their perimeters and essentially move with the ground. The response of the structure is the same as that of the ground. As an added conservatism, the seismic response spectra for the Category 1 tunnels are developed from the free field response spectra by multiplying them by a factor of 1.25.

3.7.B.2.4.4 Floor Flexibility

The impedance method (lumped parameter method) of soil-structure interaction analysis is employed to account for the effects of containment internal steel structure flexibility on the response spectra used for equipment qualification and to address, if significant, the coupling effects between the equipment and the steel structure. In addition, the impedance method is used to address the vertical floor flexibility effects in the other structures.

3.7.B.2.5 Development of Floor Response Spectra

Floor response spectra for Category 1 structures are developed using the time-history analysis. Both the horizontal and vertical floor response spectra are computed from the time-history motions at the various floors or other required locations. These motions are obtained from the time-history analysis of the structures due to each of the three orthogonal earthquake components under OBE and SSE events. The floor response spectra are computed at the frequencies given in table 3.7.B.2-3. These frequencies were selected using the suggested frequency intervals in Regulatory Guide 1.122. Since the natural frequencies are not computed in the FLUSH analyses and since the intervals between the selected frequencies are small, the natural frequencies of the system are not specifically included in this table of frequencies.

3.7.B.2.6 Three Components of Earthquake Motion

The three component earthquake effects are combined using the square root of the sum of the squares of the applicable maximum codirectional responses as described in section 4.3 of BC-TOP-4A[(1)] and is in conformance with Regulatory Guide 1.92.

3.7.B.2.7 Combination of Modal Responses

The combination of modal responses is performed as described in section 4.2 of BC-TOP-4A[(1)] and is in conformance with Regulatory Guide 1.92.

3.7.B.2.8 Interaction of Non-Category 1 Structures with Seismic Category 1 Structures

The equipment building, which forms part of the control building and fuel handling building, is designed to withstand the seismic loadings to the same criteria specified for Category 1 structures.

Other than Category 2 tunnels, the turbine building and radwaste transfer building are the only non-Category 1 structures adjacent to Category 1 structures. These are analyzed to demonstrate that under the SSE loadings they will not collapse on any Category 1 structure. In the FLUSH models, the turbine building basemat is modeled as a structural layer with proper inertial properties, and the rest of the turbine structure is modeled by a lumped mass. Shallowly embedded structures are not influenced by adjacent non-Category 1 structures.

3.7.B.2.9 Effects of Parameter Variations on Floor Response Spectra

The in-structure response spectra computed from the time-history acceleration response generally reflect two parameters:

- The amplification of the free field input produced by the soil-structure system.
- The frequency content associated with these amplification regions.

The effects of parameter variations on floor response spectra are accounted for by broadening the peaks associated with each structural frequency by +15 percent.

3.7.B.2.10 Use of Constant Vertical Static Factors

No constant vertical static factors are used for Category 1 structures. The same method of analysis is used for both vertical and horizontal directions.

3.7.B.2.11 Method Used To Account for Torsional Effects

The NRC requested[3] that all safety-related structures, systems, and components be designed to resist a static seismic torsional moment not less than that required by the Uniform Building Code. To accommodate this request, the following methodology is used. The methodology used to account for torsion caused by the seismic wave propagation effects is also described in the GPC letter[4] to the NRC; it is used both for deeply embedded and shallowly embedded structures.

3.7.B.2.11.1 Category 1 Structures

The seismic analyses of the structures are performed on the three-dimensional structure models that account for the eccentricities between the centers of mass and the centers of rigidity of the structures. The accelerations obtained from these analyses at all levels are first calculated. Then, in the design the actual eccentricity is increased by 5 percent of the maximum plan dimension at that level, and the design static seismic torsional moment is computed as the product of the augmented eccentricity and the story shear. This applies to the two orthogonal horizontal directions.

3.7.B.2.11.2 Category 1 Equipment, Systems, and Components

The intent of the additional torsional requirement is to account for the torsional motion imparted to the structure due to the effects of seismic wave propagation. Thus, this would affect only the horizontal in-structure response spectra used for equipment qualification. The procedure used to obtain the effect of this torsional ground motion is described below.

A three-dimensional lumped parameter model of the structure with soil springs is utilized to compute the torsional spectra. The structure model accounts for the eccentricities between the centers of mass and the centers of rigidity of the structure. The translational as well as the rotational stiffness and inertial characteristics are modeled. The foundation impedances consist of three translational (two horizontal and one vertical) and three rotational (two rocking and one torsional) springs and are based on the mean soil properties.

The model is analyzed for the design horizontal ground motion time-history conforming to the Regulatory Guide 1.60 horizontal response spectra applied in the free field at the foundation level of the structure. The base shear computed from this analysis, multiplied by 5 percent of the maximum plan dimension at the foundation level, yields the incremental static torsional moment (T_s) at that level.

Then a torsional ground motion time-history conforming to the Regulatory Guide 1.60 horizontal response spectra is applied again in the free field at the foundation level of the structure. The maximum dynamic torsional moment (T_d) at the base of the structure is computed from this dynamic analysis.

The magnitude of the torsional ground motion is adjusted so that T_d at the base of the structure resulting from the torsional ground motion analysis is equal to the T_s resulting from the 5-

percent eccentricity. The resulting time-history response from the torsional degree of freedom of the base node would then represent the torsional response of the basemat. Multiplying this by the distance along the north-south/east-west direction of the extreme point in the building to the lumped mass node would give the maximum possible additional east-west/north-south horizontal time-history response of the floor. From this, the additional horizontal in-structure response spectra can be computed.

The torsional responses of the nodes at different levels of the building from the torsional ground motion analysis are used with the respective extreme point distances along the north-south/east-west direction to compute the additional horizontal in-structure response spectra at these levels.

The amplification of the torsional response of the structure as a function of height tends to be smaller than the amplification of the horizontal response of the structure. Therefore, as an added conservatism, the torsional input ground motion is increased so that the ratio between the maximum torsional acceleration at a given node (caused by the torsional ground motion) to the maximum horizontal acceleration at the node (caused by the horizontal ground motion) is maintained the same as at the foundation level of the structure.

The computed additional horizontal in-structure response spectra to account for the torsional ground motion effects are added absolutely to the horizontal in-structure response spectra obtained using the methods described in paragraphs 3.7.B.2.4, 3.7.B.2.5, and 3.7.B.2.6, before the broadening of the peaks and smoothing of the curves are done. The peaks of the response spectra resulting from the addition of these two spectra are then broadened and the curves smoothed to arrive at the final design in-structure response spectra for the horizontal direction.

In the computation of the additional horizontal in-structure response spectra to be used for an equipment mounted on a specific location, the actual distance of this location from the lumped mass node may be used instead of the extreme point distance at that level.

The method applied to the NSCW tower, which contains a large water mass, is described below.

Once the magnitude of the torsional ground motion is established so that the Td at the base of the structure resulting from the torsional ground motion analysis is equal to the Ts resulting from the 5-percent eccentricity, the ratio of the horizontal acceleration at the extreme point in the basemat caused by the torsional ground motion to the maximum horizontal acceleration at the basemat caused by the horizontal ground motion is computed. The additional in-structure response spectra used to account for the torsional ground motion effects are computed by multiplying the horizontal in-structure response spectra developed using the methods described in paragraphs 3.7.B.2.4, 3.7.B.2.5, and 3.7.B.2.6 by this ratio.

3.7.B.2.12 Comparison of Responses

This section is not applicable to the VEGP, since only the time-history method is used in the seismic analysis.

3.7.B.2.13 Methods for Seismic Analysis of Dams

Since no dams are utilized directly or indirectly to provide water for the cooling system, this section is not applicable to this power plant.

3.7.B.2.14 Determination of Seismic Category 1 Structure Overturning Moments

The effects of overturning moments are evaluated by the methods described in section 4.4 of BC-TOP-4A.[1] This section includes a description of the methods used to compute foundation reactions and to account for vertical earthquake effects. The three components of the earthquake motion are taken into consideration in the overturning moments by use of the seismic accelerations.

3.7.B.2.15 Analysis Procedure for Damping

The analysis procedure used to account for damping in different elements of the model of a coupled system and the criteria used to account for composite damping are described in sections 3.2 and 3.3 of BC-TOP-4A.[1]

3.7.B.2.16 Standard Review Plan Evaluation

Differences are noted under Standard Review Plan 3.7.1. The Standard Review Plan states that modeling methods for implementing the soil-structure interaction analysis should include both the half-space and finite boundaries approaches. The VEGP soil-structure interaction analysis uses the finite element method for deeply embedded structures and the half-space method for shallowly embedded structures.

Refer to paragraph 3.7.B.2.1 for an evaluation of the Standard Review Plan differences.

3.7.B.2.17 References

1. "Topical Report Seismic Analysis of Structures and Equipment for Nuclear Power Plants," Bechtel Power Corporation, BC-TOP-4A, Revision 3, San Francisco, November 1974.
2. "Nuclear Reactors and Earthquakes," AEC Publication TID-7024, August 1963.
3. Nuclear Regulatory Commission Letter (Docket Nos. 50-424 and 50-425) Dated March 27, 1978, to Georgia Power Company.
4. Georgia Power Company Letter Dated November 13, 1978, to Nuclear Regulatory Commission.
5. Lysmer, J., et al., "Efficient Finite-Element Analysis of Seismic Structure-Soil-Structure Interaction," Earthquake Engineering Research Center, University of California, Report No. EERC 75-34, Berkeley, California, November 1975.
6. Seed, H. B., and Idriss, I. M., "Soil Moduli and Damping Factors for Dynamic Response Analysis," Earthquake Engineering Research Center, University of California, Report No. EERC 70-10, Berkeley, California, December 1970.
7. Alvin W. Vogtle Nuclear Plant, "Report on Dynamic Properties for Compacted Backfill," Bechtel Incorporated, Los Angeles, February 1978.

3.7.N.2 SEISMIC SYSTEM ANALYSIS

This section describes the methods of seismic analysis performed for safety-related components and systems within the Westinghouse scope.

3.7.N.2.1 Seismic Analysis Methods

Those components and systems that must remain functional in the event of a safe shutdown earthquake (SSE) (Seismic Category 1) are identified by applying the criteria of subsection 3.2.1.

In general, the dynamic analyses were performed using response spectrum analysis, integration of the uncoupled modal equations, direct integration of the coupled equations of motion, or nonlinear modal superposition. (See paragraph 3.7.N.2.1.5.)

3.7.N.2.1.1 Dynamic Analysis: Mathematical Model

The first step in any dynamic analysis is to model the structure or component, i.e., convert the real structure or component into a system of masses, springs, and dashpots suitable for mathematical analysis. The essence of this step is to select a model such that the displacements obtained will be a good representation of the motion of the structure or component. Stated differently, the true inertia forces should not be altered so as to appreciably affect the internal stresses in the structure or component. Some typical modeling techniques are presented in reference 1.

Equation of Motion

$$m_r \ddot{y}_r + \sum^{i} c_{ri} \dot{u}_i + \sum^{i} k_{ri} u_i = 0 \qquad (1)$$

Consider the multidegree-of-freedom system shown in figure 3.7.N.2-1. Making a force balance on each mass point r, the equations of motion can be written in the form:

where:

m_r = the value of the mass or mass moment of rotational inertia at mass point r.

$\ddot{y}_r$ = absolute translational or angular acceleration of mass point r.

c_{ri} = damping coefficient: external force or moment required at mass point r to produce a unit translational or angular velocity at mass point i, maintaining zero translational or angular velocity at all other mass points. Force or moment is positive in the direction of positive translational or angular velocity.

$\dot{u}_i$ = translational or angular velocity of mass point i relative to the base.

k_{ri} = stiffness coefficient: the external force (moment) required at mass point r to produce a unit deflection (rotation) at mass point i, maintaining zero displacement (rotation) at all other mass points. Force (moment) is positive in the direction of positive displacement (rotation).

u_i = displacement (rotation) of mass point i relative to the base.

Since:

$$\ddot{y}_r = \ddot{u}_r + \ddot{y}_s \qquad (2)$$

where:

$\ddot{y}_s$ = absolute translational (angular) acceleration of the base.

$\ddot{u}_r$ = translational (angular) acceleration of mass point r relative to the base.

Equation 1 can be written as:

$$m_r\ddot{u}_r + \sum^{i} c_{ri}\dot{u}_i + \sum^{i} K_{ri}u_i = -m_r\ddot{y}_s \qquad (3)$$

For a single-degree-of-freedom system with displacement u, mass m, damping c, and stiffness k, the corresponding equation of motion is:

$$m\ddot{u} + c\ddot{u} + ku = -m\ddot{y}_s \qquad (4)$$

3.7.N.2.1.2 Modal Analysis

Natural Frequencies and Mode Shapes. The first step in the modal analysis method is to establish the normal modes, which are determined by the eigen solution of equation 3. The right side and the damping term are set equal to zero for this purpose as illustrated in reference 2 (pages 83 through 111). Thus, equation 3 becomes:

$$m_r\,\ddot{u}_r + \sum^{i} K_{ri}u_i = 0 \qquad (5)$$

The equation given for each mass point r in equation 5 can be written as a system of equations in matrix form as:

$$(M)\,\{\ddot{\Delta}\} + (K)\,\{\Delta\} = 0 \qquad (6)$$

where:

(M)	=	mass and rotational inertia matrix.
$\{\Delta\}$	=	column matrix of the general displacement or rotation at each mass point relative to the base.
(K)	=	square stiffness matrix.
$\{\ddot{\Delta}\}$	=	column matrix of general translational and angular accelerations at each mass point relative to the base, $d^2\{\Delta\}/dt^2$.

Harmonic motion is assumed, and the $\{\Delta\}$ is expressed as:

$$\{\Delta\} = \{\delta\}\sin\omega t \qquad (7)$$

where:

$\{\delta\}$	=	column matrix of the spatial displacement or rotation at each mass point relative to the base.
ω	=	natural frequency of harmonic motion in radians per second.

The displacement function and its second derivative are substituted into equation 6 and yield:

$$(K)\{\delta\} = \omega^2(M)\{\delta\} \qquad (8)$$

The determinant $|(K) - \omega^2 (M)|$ is set equal to zero and is then solved for the natural frequencies. The associated mode shapes are then obtained from equation 8. This yields n natural frequencies and mode shapes where n equals the number of dynamic degrees of freedom of the system. The mode shapes are all orthogonal to each other and are sometimes referred to as normal mode vibrations. For a single-degree-of-freedom system, the stiffness matrix and mass matrix are single terms and the determinant $|(K) - \delta^2 (M)|$ when set equal to zero yields simply:

$$k - \omega^2 m = 0$$

or:

$$\omega = \sqrt{\frac{k}{m}} \tag{9}$$

where:

ω = natural angular frequency in radians per second.

The natural frequency in cycles per second is, therefore:

$$f = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \tag{10}$$

To find the mode shapes, the natural frequency corresponding to a particular mode, ω_n, can be substituted in equation 8.

Modal Equations. The response of a structure or component is always some combination of its normal modes. Accuracy can usually be obtained by using only the first few modes of vibration. In the normal mode method, the mode shapes are used as principal coordinates to reduce the equations of motion to a set of uncoupled differential equations that describe the motion of each mode n. These equations may be written as (reference 2, pages 116 through 125):

$$\ddot{A}_n + 2\omega_n p_n \dot{A}_n + \omega_n^2 A_n = \Gamma_n \ddot{y}_s \tag{11}$$

where the modal replacement or rotation, An, is related to the displacement or rotation of mass point r in mode n, urn by the equation:

$$U_{rn} = A_n \phi_{rn} \tag{12}$$

where:

ω_n = natural frequency of mode n in radians per second.
p_n = critical damping ratio of mode n.
Γ_n = modal participation factor of mode n given by:

$$\Gamma_n = \frac{\sum^n m_r \phi r'n}{\sum^n m_r \phi_{rn}^2} \tag{13}$$

where:

ϕ'_{rn} = value of the ϕ_{rn} in the direction of the earthquake.

The essence of the modal analysis lies in the fact that equation 11 is analogous to the equation of motion for a single-degree-of-freedom system that will be developed from equation 4. Dividing equation 4 by m gives:

$$\ddot{u} + \frac{c}{m}\,\dot{u} + \frac{k}{m}\,u = -\,\ddot{y}_S \tag{14}$$

The critical damping ratio of the single-degree-of-freedom system, p, is defined by the equation:

$$p = \frac{c}{c_C} \tag{15}$$

where the critical damping coefficient is given by the expression:

$$c_c = 2\,m\omega \tag{16}$$

Substituting equation 16 into equation 15 and solving for c/m gives:

$$\frac{c}{m} = 2\omega p \tag{17}$$

Substituting this expression and the expression for k/m given by equation 9 into equation 14 gives:

$$\ddot{u} + 2\omega p\dot{u} + \omega^2 u = -\,\ddot{y}_S \tag{18}$$

Note the similarity of equations 11 and 18. Thus, each mode may be analyzed as though it were a single-degree-of-freedom system and all modes were independent of each other. By this method a fraction of critical damping, i.e., c/c, may be assigned to each mode, and it is not necessary to identify or evaluate individual damping coefficients, i.e., c. However, assigning only a single damping ratio to each mode is not appropriate for a slightly damped structure supported by a massive, moderately damped structure. There are several methods which can be used to incorporate damping in a structural system.

One method is to develop and analyze separate mathematical models for both structures using their respective damping values. The massive, moderately damped support structure is analyzed first. The calculated response at the support points for the slightly damped structures is used as a forcing function for the subsequent detailed analysis. A second method is to inspect the mode shapes to determine which modes correspond to the slightly damped structure and then use the damping associated with the structure having predominant motion. A third method is to use the Rayleigh damping method based on computed modal energy distribution. In yet another method, the damping value for a given mode is derived from the calculation of the composite modal damping which is based on the distribution of energy in the structure for that mode.

3.7.N.2.1.3 Integration of Modal Equations

This method can be separated into the following two basic parts:

A. Integration procedure for the uncoupled modal equation 11 to obtain the modal displacement and accelerations as a function of time.

B. Use of these modal displacements and accelerations to obtain the total displacements, accelerations, forces, and stresses.

Integration Procedure. Integration of these uncoupled modal equations is done by step-by-step numerical integration. The step-by-step numerical integration procedure consists of selecting a suitable time interval, Δt, and calculating modal acceleration, A_n, modal velocity, A_n, and modal displacement, A_n, at discrete time stations Δt apart, starting at t = 0 and continuing through the range of interest for a given time-history of base acceleration.

Total Displacements, Accelerations, Forces, and Stresses. From the modal displacements and accelerations, the total displacements, accelerations, forces, and stresses can be determined as follows:

A. Displacement of mass point r in mode n as a function of time is given by equation 12 as:

$$u_{rn} = A_n \phi_{rn}$$

with the corresponding acceleration of mass point r in mode n as:

$$u_{rn} = \ddot{A}_n \phi_{rn}$$

B. The displacement and acceleration values obtained for the various modes are superimposed algebraically to give the total displacement and acceleration at each time interval.

C. The total acceleration at each time interval is multiplied by the mass to give an equivalent static force. Stresses are calculated by applying these forces to the model or from the displacements at each time interval.

3.7.N.2.1.4 Integration of Coupled Equations of Motion

The dynamic transient analysis is a time-history solution of the response of a given structure to known forces and/or displacement forcing functions. The structure may include linear or nonlinear elements, gaps, interfaces, plastic elements, and viscous and Coulomb dampers. Nodal displacements, nodal forces, pressure, and/or temperatures may be considered as forcing functions. Nodal displacements and elemental stresses for the complete structure are calculated as functions of time.

The basic equations for the dynamic analysis are as follows:

$$(M)\ \{\ddot{x}\} + (C)\ \{\dot{x}\} + (K)\ \{x\} = \{F(t)\} \qquad (22)$$

where the terms are as defined earlier; {F(t)} may include the effects of applied displacements, forces, pressures, temperatures, or nonlinear effects such as plasticity and dynamic elements with gaps. Options of translational accelerations input to a structural system and the inclusion of static deformation and/or preload may be considered in the nonlinear dynamic transient analysis. The option of translational input such as uniform base motion to a structural system is considered by introducing an inertia force term of -(M){z} to the right side of the basic equation 22, i.e.,

$$(M)\ \{\ddot{x}\} + (C)\ \{\dot{x}\} + (K)\{x\} = \{F\} - (M)\{\ddot{z}\} \qquad (23)$$

The vector $\{\ddot{z}\}$ is defined by its components $\ddot{z}_i$ where: i refers to each degree of freedom of the system; $\ddot{z}_i$ is equal to a_1, a_2, or a_3 if the ith degree of freedom is aligned with the direction of the system translational acceleration a_1, a_2, or a_3, respectively; and $\ddot{z}_i$ is equal to 0 if the ith degree of freedom is not aligned with any direction of the system transitional acceleration. Typical application of this option is a structural system subjected to a seismic excitation of a given ground acceleration record. The displacement {x} obtained from the solution of equation 23 is the displacement relative to the ground.

The option of the inclusion of initial static deformation or preload in a nonlinear transient dynamic structural analysis is considered by solving the static problem prior to the dynamic analysis. At each state of integration in transient analysis, the portion of internal forces due to static deformation is always balanced by the portion of the forces which are statically applied. Hence, only the portion of the forces that deviate from the static loads will produce dynamic effects. The output of this analysis is the total result due to static- and dynamic-applied loads.

3.7.N.2.1.5 Nonlinear Modal Superposition

In the nonlinear modal superposition method, the nonlinearities are presented as pseudoforces. The mass and stiffness matrices are calculated only once; the corresponding mode shapes and natural frequencies are associated with the linear system, simulating initial states of the undamped structure with an external force acting on it. This state of the structure is hereafter referred to as the reference state. During the time-history analysis, as the nonlinear behavior comes into action, the true frequencies and mode shapes change. The effect of the variation of the true frequencies and mode shapes from the original ones is represented by pseudoforces on the right side of the equation of motion.

The generalized equation of motion for a nonlinear structure is:

$$[M]\{\ddot{X}\} = (C_{n1})\{\dot{X}\} + (K_{n1})\ \{X\} = \{F\} \tag{24}$$

where:

$[M]$	=	mass matrix.
$[C_{n1}]$	=	nonlinear damping matrix, dependent upon velocity and displacement.
$[K_{n1}\}$	=	nonlinear stiffness matrix, dependent upon displacement.
$\{X\}, \{\dot{X}\}$	=	displacement, velocity, acceleration and
$\{\ddot{X}\}, \{F\}$		applied-force vector

Let

$$\begin{aligned} [C_{n1}] &= [C] + [\overline{C}] \\ [K_{n1}] &= [K] + [\overline{K}] \end{aligned} \tag{25}$$

where [C] and [K] are the damping and stiffness matrices representing the reference state of structure; [C] and [K] are the damping and stiffness matrices, dependent on the velocity and displacement. Substitution of equations 25 into equation 24 gives:

$$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + (K)\{X\} = \{F\} - \{F_{nl}\} \tag{26}$$

where the pseudoforce vector is defined by

$$\{F_{nl}\} = [\overline{C}]\ \{\dot{X}\} + [\overline{K}]\ \{X\} \tag{27}$$

The homogeneous, undamped equation of motion representing the references state of the structure is:

$$[M]\{\ddot{X}\} + (K)\{X\} = \{0\} \tag{28}$$

Let (ω) and (ϕ) be the natural frequency and normalized mode shape matrix. The following transformation

$$\{X\} = (\phi)\{q\} \tag{29}$$

is substituted in equation 26, resulting in the following uncoupled nodal equations:

$$\{\ddot{q}\} + 2\zeta_j\omega_j\{\dot{q}\} + (\omega j^2)\{q\} = \{Q\} - \{Q_{nl}\} \tag{30}$$

where:

ζ_j = percentage of the critical damping for the 7th mode.

$\{Q\}$ = $(Q)^T \{F\}$ = generalized applied – force vector.

$\{Q_{nl}\}^I$ = $(Q)^T \{F_{nl}\}^I$ = generalized pseudoforce vector.

Arrays $\{q\}$, $\{\dot{q}\}$, and $\{\ddot{q}\}$ are the modal displacement, velocity, and acceleration vectors, respectively. The generalized pseudoforce vector is a function of displacement and velocity. For a given time step, it can be approximated by a Taylor series.

For a given time step, modal equations of motion are integrated analytically. Then the displacement and velocities of the nodes associates with nonlinear elements are calculated. This information is used to calculate the generalized pseudoforce vector and its time derivatives. Then the modal equations are integrated for the next time step.

3.7.N.2.2 Natural Frequencies and Response Loads

Not applicable.

3.7.N.2.3 Procedures Used for Modeling

Procedures used for modeling are discussed in paragraph 3.7.N.2.1.1.

3.7.N.2.4 Soil-Structure Interaction

Refer to paragraph 3.7.B.2.4.

3.7.N.2.5 Development of Floor Response Spectra

Refer to paragraph 3.7.B.2.5.

3.7.N.2.6 Three Components of Earthquake Motion

The seismic design of the piping and equipment includes the effect of the seismic response of the supports, equipment, structures, and components. The system and equipment response is determined using three earthquake components, two horizontal and one vertical. The seismic input described in subsection 3.7.N.1 is the basis for generating these three input components.

In computing the system and equipment response by the response spectrum modal analysis, the methods of 3.7.N.2.7 are used to combine all significant modal responses to obtain the combined unidirectional responses.

The combined total response is then calculated using the square root of the sum of the squares (SRSS) formula applied to the resultant codirectional responses. For instance, for each item of interest, such as displacement, force, stresses, etc., the total response is obtained by applying the SRSS method. The mathematical expression for this method (with R as the item of interest) is:

$$R_C = \left(\sum_{T=1}^{3} R_T^2 \right)^{1/2} \tag{31}$$

where

$$R_T = \left(\sum_{i=1}^{N} R_{Ti}^2 \right)^{1/2} \tag{32}$$

and

R_c	=	total combined response at a point.
R_T	=	value of combined response of direction T.
R_{Ti}	=	value of response for direction T, mode i.
N	=	total number of modes considered.

Westinghouse employs methods of combining modal responses that have been accepted by the Nuclear Regulatory Commission as an alternative to Regulatory Guide 1.92 (section 1.9).

The subscripts can be reversed without changing the results of the combination.

For systems having modes with closely spaced frequencies, R_T in equation 31 is determined by equation 33 in paragraph 3.7.N.2.7.

The system and equipment response can also be determined using time-history analyses.

When a time-history analysis is performed, the two horizontal and the vertical time-history components are applied simultaneously.

3.7.N.2.7 <u>Combination of Modal Responses</u>

The total codirectional seismic response is obtained by combining the individual modal responses utilizing the SRSS method. For systems having modes with closely spaced frequencies, this method is modified to include the possible effect of these modes. The groups of closely spaced modes are chosen such that the difference between the frequency of the first mode and the last mode in the group does not exceed 10 percent of the lower frequency. Groups are formed starting from the lowest frequency and working toward successively higher frequencies. No one frequency is in more than one group. The combined total response for systems that have such closely spaced modal frequencies is obtained by adding to the SRSS of all modes the product of the responses of the modes in each group of closely spaced modes and a coupling factor ε. This can be represented mathematically as:

$$R_T^2 = \sum_{i=1}^{N} R_i^2 + 2\sum_{j=1}^{S}\ \sum_{K=M_j}^{N_j-1}\ \sum_{\ell=K+1}^{N_j} R_K R_\ell \varepsilon K_\ell \tag{33}$$

where:

R_T	=	total unidirectional response.
R_i	=	absolute value of response of mode i.
N	=	total number of modes considered.
S	=	number of groups of closely spaced modes.
M_j	=	lowest modal number associated with group j of closely spaced modes.
N	=	highest modal number associated with group j of closely spaced modes.
εK_ℓ	=	coupling factor with

$$\varepsilon K\ell = \left\{1 + \frac{(\omega'_K - \omega'_\ell)^2}{(\beta'_K\ \omega_K + \beta_\ell \omega_\ell)}\right\}^{-1} \quad \text{and} \tag{34}$$

$$\omega'_K = \omega_K\,(1 - (\beta'_K)^2)^{1/2} \tag{35}$$

$$\beta'_K = \beta_K + \frac{2}{\omega_K\ t_d} \tag{36}$$

where:

ω_K	=	frequency of closely spaced mode K.
β_K	=	fraction of critical damping in closely spaced mode K.
t_d	=	duration of the earthquake.

3.7.N.2.8 Interaction of Non-Category 1 Structures with Seismic Category 1 Structures

Refer to paragraph 3.7.B.2.8.

3.7.N.2.9 Effects of Parameter Variations on Floor Response Spectra

Refer to paragraph 3.7.B.2.9.

3.7.N.2.10 Use of Constant Vertical Static Factors

Constant vertical static factors are not used as the vertical floor response load for the seismic design of safety classed systems and components within Westinghouse's scope of responsibility. All such systems and components are analyzed in the vertical direction.

3.7.N.2.11 Methods Used to Account for Torsional Effects

In the coupled building internals structure/reactor coolant loop model, the torsional effects are accounted for by including the torsional component due to earthquake motions along with the three translational and two rocking components as described in paragraphs 3.7.N.1.2 and 3.7.B.2.11.

3.7.N.2.12 Comparison of Responses

Not applicable.

3.7.N.2.13 Methods for Seismic Analysis of Dams

Refer to paragraph 3.7.B.2.13.

3.7.N.2.14 Determination of Seismic Category 1 Structure Overturning Moments

Refer to paragraph 3.7.B.2.14.

3.7.N.2.15 Analysis Procedure for Damping

Procedures for damping are discussed in paragraph 3.7.N.1.3.

3.7.N.2.16 Standard Review Plan Evaluation

Closely spaced modes should be combined in accordance with procedures stated in Regulatory Guide 1.92. The VEGP uses the "pi epsilon" method for closely spaced mode combinations.

Westinghouse combines closely spaced modes using the "epsilon" method (section 3.7). The Westinghouse method for combining closely spaced modes represents an alternative to Regulatory Guide 1.92. This method has been accepted by the Nuclear Regulatory Commission's Structural Engineering and Mechanical Engineering Branch on specific plant dockets. Most recently, the Westinghouse position on combining closely spaced modes has been accepted on other dockets.

3.7.N.2.17 References

A. Lin, C. W., How to Lump the Masses - A Guide to the Piping Seismic Analysis, ASME Paper 74-NE-7 Presented at the Pressure Vessels and Piping Conference, Miami, June 1974.

B. Biggs, J. M., Introduction to Structural Dynamics, McGraw- Hill, New York, 1974.

3.7.B.3 SEISMIC SUBSYSTEM ANALYSIS

Seismic subsystems are those systems whose models lack soil/structure interaction relationships. Therefore, all Category 1 structures not designated as seismic systems and all Category 1 systems and equipment are considered seismic subsystems.

3.7.B.3.1 Seismic Analysis Methods

Methods used for seismic analysis of seismic subsystems are described in this paragraph.

3.7.B.3.1.1 Category 1 Structures

3.7.B.3.1.1.1 Containment Building. The shell and basemat internal forces and moments produced by the operating basis earthquake (OBE) and safe shutdown earthquake (SSE) are determined by statically applying, at different elevations, the acceleration values obtained from the dynamic time-history analysis described in subsection 3.7.B.2. The containment building finite element model used for the static analysis is described in subsection 3.8.1.

3.7.B.3.1.1.2 Containment Internal Structure. The concrete internal structure internal forces and moments produced by the OBE and SSE are determined by statically applying, at different elevations, the acceleration values obtained from the dynamic time-history analysis described in subsection 3.7.B.2. The containment internal structure finite element model used for the static analysis is described in subsection 3.8.3.

The structural steel access platform framing system outside the secondary shield wall is modeled as subsystem supported at the containment basemat and the secondary shield walls. The frequencies and mode shapes for this subsystem were used in a response spectrum analysis using an envelope spectra of the concrete internals structure obtained from the time-history analysis described in subsection 3.7.B.2.

3.7.B.3.1.1.3 Auxiliary Building. Forces and moments produced by the OBE and SSE in shear walls, slabs, and basemat are determined by statically applying acceleration values obtained from the dynamic time-history analysis described in subsection 3.7.B.2, simultaneously, at appropriate slab elevations. Further discussion on the methods used in the analysis is provided in subsection 3.8.4.

3.7.B.3.1.1.1 Fuel Handling Building. Forces and moments provided by the OBE and SSE in shear walls, slabs, and basemat are determined by two methods. Elevations 220 ft 0 in. and below are analyzed using a finite element model. Elevations 220 ft 0 in. and above are analyzed using hand calculations to distribute the seismic loads. For both methods of analysis, forces and moments are determined by statically applying acceleration values obtained from the dynamic time-history analysis described in subsection 3.7.B.2, simultaneously, at appropriate slab elevations.

3.7.B.3.1.1.5 Control Building. The forces and moments produced by the OBE and SSE are computed using the procedure described in paragraph 3.7.B.3.1.1.3.

3.7.B.3.1.1.6 Other Category 1 Structures. This paragraph describes the seismic analysis methods used for the following structures:

- Nuclear service cooling water (NSCW) towers and valve houses.
- Refueling, reactor makeup, and condensate water storage tanks.

- Diesel generator buildings.
- Diesel fuel oil storage tank pumphouses.
- Auxiliary feedwater pumphouses.
- All tunnels.

All structures are analyzed using the OBE and SSE acceleration values obtained from the dynamic time-history analysis described in subsection 3.7.B.2. All structures are analyzed using equivalent statically applied accelerations with the exception of the NSCW valve houses. The NSCW valve houses are analyzed using the response spectrum method. Finite element computer models are used for the analysis of the NSCW towers and valve houses. Classical shell solutions are used in the analysis of the water storage tanks. Conventional hand analysis methods are used to distribute the forces in the other structures.

3.7.B.3.1.2 Category 1 Systems

For the analysis of electrical cable trays and tray supports and heating, ventilation, and air conditioning (HVAC) ducts and duct supports, the modal response spectrum analysis method or the equivalent static load method is used. This method is described in paragraph 3.7.B.3.5.

For the analysis of piping systems, the floor response spectra of the specific level of the appropriate building are used as the seismic input. The generation of response spectra and the damping values used are described in subsection 3.7.B.1. The methods used for analyzing piping systems are described in sections 2.0, 4.0, and Appendix D of BP-TOP-1.[2]

3.7.B.3.1.3 Category 1 Subsystems and Components

Either modal response spectrum analysis method or equivalent static load method is used for the analyses of Category 1 sub- systems and components. The criteria for the number of masses used, the number of modes considered, and the combination of modal responses are in accordance with BC-TOP-4A.[1] The equivalent static load method is described in paragraph 3.7.B.3.5.

3.7.B.3.2 <u>Determination of Number of Earthquake Cycles</u>

The procedures used to determine the number of earthquake cycles for piping during seismic events are discussed in section 6.2 of BP-TOP-1.[2] Structures and equipment designed on the basis of analysis are not fatigue controlled since most stress and strain reversals occur only a small number of times. The allowable stresses used in the design of these structures are not affected by 50 maximum stress cycles that could be caused by the OBEs. The number of earthquake cycles simulated in the equipment qualified by testing is discussed in sections 3.9 and 3.10.

3.7.B.3.3 <u>Procedure Used for Modeling</u>

Sections 2.0 and 3.0 of BP-TOP-1 discuss the techniques and procedures used to model Seismic Category 1 piping other than the buried type.[2]

3.7.B.3.4 Basis for Selection of Frequencies

Piping system frequencies are calculated and dynamic inter- actions with support structures are accounted for in accordance with section 2 of BP-TOP-1.[2]

3.7.B.3.5 Use of Equivalent Static Load Method of Analysis

The equivalent static load method involves equivalent horizontal and vertical static forces applied at the center of gravity of various masses. The equivalent force at a mass location is computed as the product of the mass and the seismic acceleration value applicable to that mass location.

The magnitude of the seismic acceleration is established on the basis of the dynamic response characteristics of the component. Components which can be adequately characterized as a single-degree-of-freedom system are designed for accelerations associated with their natural frequency. Seismic acceleration values used for design of multidegree-of-freedom systems, which may be in the resonance region of the amplified response spectra curves, are the peak acceleration values multiplied by a factor of 1.5 unless a lower factor is justified. In lieu of using the peak acceleration value, the actual frequency may be calculated and the corresponding acceleration value may be used. In this case, the calculated frequency must be higher than that frequency related to the peak acceleration; otherwise, the peak acceleration value is used in design. For systems and components which have fundamental frequencies of 33 Hz or greater, the zero period acceleration is taken as the seismic acceleration value.

The above equivalent static load method of analysis is used for design of platforms, electrical cable trays and supports, conduits and supports, HVAC ducts and supports, simple piping systems, and other substructures.

The equivalent static load method of analysis can also be used for design of complex piping systems, with significant responses at several vibrational frequencies. In this case, the static load factor of 1.7 shall be applied to the peak accelerations of the applicable floor response spectra. Use of this method for complex piping systems is limited to small-bore (2 in. or less in diameter), non-NSSS piping.

In lieu of the equivalent static load method, the method stated in BP-TOP-1, section 2.3.2 and Appendix D (2) may be used for piping.

3.7.B.3.6 Three Components of Earthquake Motion

Section 5.1 of BP-TOP-1 provides the criteria used to combine the results of horizontal and vertical seismic responses for piping systems.[2]

For the structures, systems, and equipment qualified by analysis, the three component earthquake effects are combined using the square root of the sum of the squares method, as described in paragraph 3.7.B.2.6, or an equivalent method yielding essentially the same results. For equipment qualified by testing, the three component earthquake effects are considered using the guidelines provided in Institute of Electrical and Electronics Engineers Standard 344-1975.[3]

3.7.B.3.7 Combination of Modal Responses

Sections 5.1 and 5.2 of BP-TOP-1 describe the criteria used for Category 1 piping systems.[2]

The combination of modal responses for other structural systems is performed as described in section 4.2 of BC-TOP-4A[1] and is in conformance with Regulatory Guide 1.92.

3.7.B.3.8 Analytical Procedures for Piping Systems

Section 2.0 of BP-TOP-1 describes the design criteria and the analytical techniques applicable to piping systems. Section 3.3 of BP-TOP-1 discusses selection of response spectra, including those situations where the piping system spans a large difference in elevation or the piping system goes between two different structures. In addition to the seismic spectrum selection techniques identified in section 3.3 of BP-TOP-1, multiple response spectrum methodology may also be used. With the exception of those stress calculations described in reference 4, this methodology is not used in conjunction with ASME Code Case N-411 Damping values. Section 4.0 of BP-TOP-1 also discusses relative displacements between piping and support points.[2]

3.7.B.3.9 Multiply-Supported Equipment and Components with Distinct Inputs

Section 4.0 of BP-TOP-1 discusses the methods used for piping systems.[2]

3.7.B.3.10 Use of Constant Vertical Static Factors

A constant seismic vertical load factor is not used for the seismic design of Seismic Category 1 structures, systems, components, and equipment. Use of equivalent static load method of analysis is described in paragraph 3.7.B.3.5.

3.7.B.3.11 Torsional Effects of Eccentric Masses

The significant torsional effects of valves and other eccentric masses are taken into account in the seismic piping analysis by the techniques discussed in section 3.2 of BP-TOP-1.[2]

3.7.B.3.12 Buried Seismic Category 1 Piping Systems and Tunnels

Section 6.0 of BC-TOP-4A discusses the techniques used to calculate the stresses from seismic loadings for buried seismic piping.[1] The buried Seismic Category 1 piping is designed to remain functional when subjected to seismic loads. This is accomplished by limiting the calculated stresses in the pipe material under all loading combinations, including earthquake, as discussed below. The sum of the stresses produced by internal and/or external pressure and those produced by seismic forces shall not exceed 2.4 (S) for the SSE or 1.2 (S) for the OBE. "S " indicates the allowable stresses prescribed as S in tables I-7-1, I-7-2, and I-7-3 of Appendix I of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section III. Where American National Standards Institute Standard B31.1.0 is used, the allowable stresses are indicated in tables A-1, A-2, and A-3 of Appendix A of that code.

The methods of analysis for Category 1 buried tunnels are discussed in subsection 3.7.B.2.

Buried electrical duct banks containing Class 1E electrical circuits are designed to meet Seismic Category 1 requirements. Engineering analysis is based on the buried structure analysis procedures outlined in Chapter 6 of BC-TOP-4A.[1] Duct banks are provided with a seismic separation at structure, manhole, and intersection interfaces. The duct banks are placed at depths consistent with protection requirements for missile penetration and large surface surcharge loadings.

3.7.B.3.13 Interaction of Other Piping with Seismic Category 1 Piping

Nonsafety-related piping systems which are classified as Seismic Category 1 are analyzed as described in paragraph 3.7.B.3.8. The results of the analysis are evaluated using the ASME Code, Section III, Class 2 and 3 equations with appropriate load combinations as described in subsection 3.9.B.3.

Section 3.4 of BP-TOP-1 describes the techniques used to consider the interaction of Seismic Category 1 piping with non-Seismic Category 1 piping.[2]

3.7.B.3.14 Seismic Analyses for Reactor Internals

Refer to 3.7.N.3.14.

3.7.B.3.15 Analysis Procedure for Damping

Section 2.4 of BP-TOP-1 describes the procedure used to account for damping of Category 1 piping systems.[2] The analysis procedures used for structures are described in paragraph 3.7.B.2.15.

3.7.B.3.16 References

1. "Seismic Analyses of Structures and Equipment for Nuclear Power Plants," Bechtel Power Corporation, BC-TOP-4A, Revision 3, San Francisco, November 1974.
2. "Seismic Analysis of Piping Systems," Bechtel Power Corporation, BP-TOP-1, Revision 3, San Francisco, January 1976.
3. "Recommended Practices for Seismic Qualification for Class 1E Equipment for Nuclear Power Generating Stations," Institute of Electrical and Electronics Engineers (IEEE) Standard 344-1975.
4. Georgia Power Company Letter (GN-1257) Docket No 50.424 and 50-425, To NRC, dated December 22, 1986.

3.7.N.3 SEISMIC SUBSYSTEM ANALYSIS

This section describes the seismic analysis performed on subsystems within the Westinghouse scope of responsibility.

3.7.N.3.1 Seismic Analysis Methods

Seismic analysis methods for subsystems within the Westinghouse scope of responsibility are given in paragraphs 3.7.N.2.1 and 3.7.N.3.5.

3.7.N.3.2 Determination of Number of Earthquake Cycles

The operating basis earthquake (OBE) is conservatively assumed to occur five times over the life of the plant. A time-history study has been conducted to arrive at a realistic number of maximum stress cycles per OBE occurrence for all Westinghouse systems and components.

This evaluation considered both the equipment and its supporting building structure as single-degree-of-freedom systems, which tend to produce a more uniform and unattenuated response than a complex interacting system. The natural frequencies for the building equipment are conservatively chosen to coincide.

As a result of this study, 10 maximum stress cycles for flexible equipment (natural frequencies less than 33 Hz) and 5 maximum stress cycles for rigid equipment (natural frequencies greater than 33 Hz) for each OBE occurrence are used for fatigue evaluation of Westinghouse systems and components.

3.7.N.3.3 Procedure Used for Modeling

Refer to paragraph 3.7.N.2.1 for modeling procedures for subsystems in the Westinghouse scope of responsibility.

3.7.N.3.4 Basis for Selection of Frequencies

In the analysis of Class 1 branch lines attached to the reactor coolant loop (including the surge line), the frequencies of these lines are controlled to avoid the peak building frequencies and the lowest fundamental frequencies of the primary equipment, if necessary, to maintain equipment and support loads within allowable limits.

There are no specific design criteria that attempt to cause the fundamental frequencies of nuclear steam supply system (NSSS) equipment to be different from the forcing frequencies of the supporting structures. The effect of the equipment fundamental frequencies relative to the supporting structure forcing frequencies is, however, considered in the analysis of the NSSS equipment.

3.7.N.3.5 Use of Equivalent Static Load Method of Analysis

The static load equivalent or static analysis method involves the multiplication of the total weight of the equipment or component member by the specified seismic acceleration coefficient. The magnitude of the seismic acceleration coefficient is established on the basis of the expected dynamic response characteristics of the component. Components that can be adequately characterized as single-degree-of-freedom systems are considered to have a modal participation factor of 1. Seismic acceleration coefficients for multidegree-of-freedom systems that may be in the resonance region of the amplified response spectra curves are increased by 50 percent to account conservatively for the increased modal participation.

3.7.N.3.6 Three Components of Earthquake Motion

Methods used to account for three components of earthquake motion for subsystems in the Westinghouse scope of responsibility are given in paragraph 3.7.N.2.6.

3.7.N.3.7 Combination of Modal Responses

Methods used to combine modal responses for subsystems in the Westinghouse scope of responsibility are given in paragraph 3.7.N.2.7.

3.7.N.3.8 Analytical Procedures for Piping

The Class 1 piping systems are analyzed according to the rules of the American Society of Mechanical Engineers (ASME) Code, Section III. When response spectrum methods are used to evaluate piping systems supported at different elevations, the following procedures are used. The effect of differential seismic movement of piping supports is included in the piping analysis according to the rules of the ASME Code, Section III. According to ASME definitions, these displacements cause secondary stresses in the piping system.

In the response spectrum dynamic analysis for evaluation of piping systems supported at different elevations, spectra that envelop the floor response spectra corresponding to the applicable support locations are used. Westinghouse does not have in its scope of analysis any piping systems interconnected between buildings.

3.7.N.3.9 Multiple Supported Equipment Components with Distinct Inputs

When response spectrum methods are used to evaluate reactor coolant system (RCS) primary components interconnected between floors, the procedure outlined in the following paragraphs is used. The primary components of the RCS are supported at no more than two floor elevations.

A. A dynamic response spectrum analysis is first made assuming there is no relative displacement between support points. The response spectra used in this analysis is the most severe floor response spectra.

B. Secondly, the effect of differential seismic movement of components interconnected between floors is considered statically in the detailed component analysis. Per ASME Code rules, the stress caused by differential seismic motion is clearly secondary for piping (NB-3650) and component supports (NF-3231). For components, the differential motion is evaluated as a free end displacement, per ASME III NB-3213.19.

C. The results of these two steps, the dynamic inertia analysis and the static differential motion analysis, are combined absolutely with due consideration for the ASME classification of the stresses.

3.7.N.3.10 Use of Constant Vertical Static Factors

Constant vertical load factors are not used as the vertical floor response load for the seismic design of safety-related components and equipment within the Westinghouse scope of responsibility.

3.7.N.3.11 Torsional Effects of Eccentric Masses

The effect of eccentric masses, such as valves and valve operators, is considered in the seismic piping analyses. These eccentric masses are modeled in the system analysis, and the torsional effects caused by them are evaluated and included in the total system response. The total response must meet the limits of the criteria applicable to the safety class of the piping.

3.7.N.3.12 Buried Seismic Category 1 Piping Systems and Tunnels

Refer to paragraph 3.7.B.3.12.

3.7.N.3.13 Interaction of Other Piping with Seismic Category 1 Piping

Refer to paragraph 3.7.B.3.13.

3.7.N.3.14 Seismic Analyses for Reactor Internals

Fuel assembly component stresses induced by horizontal seismic disturbances are analyzed through the use of finite element computer modeling.

The time-histories were developed from the plant specific response spectra for the OBE and the safe shutdown earthquake at the required elevations. The reactor internals and the fuel assemblies are modeled as spring and lumped-mass systems or beam elements. The seismic response of the fuel assemblies is analyzed to determine design adequacy. A detailed discussion of the analyses performed for typical fuel assemblies is contained in reference 1.

Fuel assembly lateral structural damping obtained experimentally is also presented in reference 1 (figure B-4). The data indicate that the damping values exceeded 10 percent of the critically damped value at fuel assembly displacements greater than 0.11 in. Although the distribution of fuel assembly amplitudes decreases as one approaches the center of the core, the amplitude for the minimum displacement fuel assembly is well above 0.11 in. for the SSE.

Fuel assembly displacement time-history for the SSE seismic input is illustrated in reference 1 (figure 2-3). The fuel assembly amplitude resulting from the time-history response is used to determine the various fuel assembly component stresses.

The control rod drive mechanisms (CRDMs) are seismically analyzed to confirm that system stresses under the combined loading conditions, as described in subsection 3.9.N.1, do not exceed allowable levels as defined by ASME Code, Section III, for upset and faulted conditions. The CRDM is mathematically modeled as a system of lumped and distributed masses. The model is analyzed under appropriate seismic excitation, and the resultant seismic bending moments along the length of the CRDM are calculated. The corresponding stresses are then combined with the stresses from the other loadings required, and the combination is shown to meet the requirements of the ASME Code, Section III.

3.7.N.3.15 Analysis Procedure for Damping

Analysis procedures for damping for subsystems in the Westinghouse scope of responsibility are given in paragraph 3.7.N.1.3.

3.7.N.3.16 References

1. "Safety Analysis of the 8-Grid 17x17 Fuel Assembly for Combined Seismic and Loss of Coolant Accidents," WCAP-8236, Addendum 1 (Proprietary), March 1974, and WCAP-8288, Addendum 1 (Nonproprietary), April 1974.

3.7.N.3.17 Bibliography

Thomas, T. H., et al., "Nuclear Reactors and Earthquakes," U.S. Atomic Energy Commission, TID-7024, Washington, D.C., August 1963.

3.7.4 SEISMIC INSTRUMENTATION

3.7.4.1 Comparison with Regulatory Guide 1.12

The seismic monitoring system for VEGP consists of digital time history recorders that have remote accelerometers connected to them. Each recorder receives the acceleration signal from one triaxial accelerometer model FBA-3. The recorder stores the data on a Personal Computer Memory Card International Association (PCMCIA) card. The data can be retrieved automatically or manually. Each recorder has its own clock and the time is recorded.

The central controller unit, which consists of a computer, keyboard display, and printer, retrieves the data from the recorders, analyses it, and compares the result with the site response spectra for each location of the accelerometers. A printed report is provided showing the time-history, the peak acceleration value recorded at each location of the accelerometers, the response spectra, and the operating basis earthquake (OBE) exceedance. The recorders and the central controller are connected to an alarm panel. The alarm panel provides dry contact closures for different alarms to be sent to the control room. These alarms are:

- Event detected.
- OBE exceedance.
- Loss of dc.
- Loss of ac.
- System Health.

The system is in full compliance with the requirements of the Regulatory Guide 1.12, Rev.2, March 1997 with the following exception - the required maintenance and testing periodicities are determined using Performance Based Analysis rather than those specified in ANSI N18.5/ANS 2.2. The recording, analysis, and alarm system is housed in a seismically-braced cabinet, furnished for these instruments and devices necessary for system testing, annunciating, calibration, and control. This panel is located in the control room.

Since both units share common buildings and the expected seismic response is the same for both containments, only one complete set of seismic instrumentation is provided for the site in conformance with American National Standards Institute Standard N18.5, Section 4.4. Additional seismic instrumentation is installed to better evaluate the effect of an earthquake on building structures.

3.7.4.2 Location and Description of Instrumentation

The following instrumentation and associated equipment are used to measure plant response to earthquake motion:

- Seven triaxial strong motion accelerometers (SMA) Model FBA-3 connected to seven digital recorders. The new system monitors each accelerometer and when the acceleration level exceeds a preset threshold, it triggers and starts recording the event using a user-set pre-event and post-event memory. The total recording time for each digital recorder is 30 minutes.
- One cabinet to house the alarm panel, the digital recorders, the central controller, and the Uninterruptible Power Supply (UPS). The new central controller will provide the functions of detecting an event, retrieving the data, computing the response spectra for a given damping value, comparing the computed response spectra with the corresponding site response spectra, declaring an OBE exceedance alarm based on the OBE exceedance criteria, detecting a system health problem, and setting an alarm. All alarms are set through the alarm panel.

The UPS provides power for the central controller. Each digital recorder has its own battery and charger. In case of a loss of ac, the digital recorders will continue to operate for another 30 h, and the central controller will operate for 1 h. (See figure 3.7.4-1.) The seismic instrumentation is normally powered from the non-Class 1E, uninterruptible 120-V-ac power supply. However, the power supply for the supplemental SMA is powered by a non-class 1E 120-V-ac power supply. The system also has a self-contained power supply with a 30-min capacity for those cases where normal power is lost. Inservice surveillance for those instruments required to meet the recommendations of Regulatory Guide 1.12 is addressed in the Technical Requirements Manual. The remainder of the instrumentation is addressed in plant procedures.

3.7.4.2.1 Strong Motion Accelerometers

Each accelerometer provides an analog signal (voltage) directly proportional with the acceleration at the measuring points. This signal is recorded in digital Format (18-bit resolution) and analyzed, and the analysis results are printed. Each sensor unit contains three accelerometers mounted in a mutually orthogonal array. Accelerometers have their principal axes oriented identically, with one horizontal axis parallel to the major horizontal axis assumed in the seismic analysis.

One SMA is located in the field at approximately 225 ft from the containment. A second SMA is located in the Unit 1 containment tendon gallery such that it measures the input vibratory motion on the basemat. A third SMA is located on the Unit 1 containment building operating floor. A fourth SMA is located on el 197 ft 6 in. in Unit 1. A fifth SMA is located in the auxiliary building floor on level 1. A sixth SMA is installed on the basemat of the auxiliary building. The seventh SMA is on the slab floor of the diesel generator building.

Each sensor is used by the new system as a potential system trigger.

An eighth SMA is located in the ‘free field’ at approximately 2000 ft from containment. Independent to the primary seismic monitoring system, the supplemental ‘free field’ SMA is equipped with a self-contained seismic trigger and 18-bit digital recorder. The software provided with the central controller may be used to analyze the data provided by this stand-alone recorder and generate an analysis report similar to the one generated by the seismic monitoring system.

3.7.4.2.2 Response Spectrum Analyzer

Following the detection of an event, the central controller will send the event alarm, retrieve the data, analyze the data, and compare the result with the site OBE response spectrum for each accelerometer location. The OBE exceedance alarm will be set based on this comparison. For this configuration, it will take between 10 and 30 minutes to have the data analyzed and the report printed automatically.

3.7.4.2.3 System Control Panel

The cabinet in the control room houses the recorders, the alarm panel, and the central controller. It will provide a digital time-history, record, and complete analysis automatically in conjunction with the accelerometers. All required alarms will be set based on selected criteria.

3.7.4.3 Control Room Operator Notification

Exceedance of the preset values at the location of the accelerometers will provide a dry contact closure through the alarm panel. This can be used to activate corresponding alarms in the control room in accordance with the requirements of the Regulatory Guide 1.12, rev.2. These initial setpoints are based on experience in existing plants and may be changed once significant plant operating data have been obtained, which indicate that a different setpoint would provide a better SMA system operation.

The peak acceleration level at each location of the accelerometers, on each axis is available immediately and automatically after data analysis for each axis (in less than 30 minutes).

Response spectra for each axis of each accelerometer is available automatically after data analysis (in less than 30 minutes). For the free field, the response spectra can be computed and be available in the same format, depending on how fast the data is retrieved from the stand-alone recorder. The comparison with the OBE or safe shutdown earthquake (SSE) site spectra is done automatically, and the OBE exceedance is indicated based on the selected criteria. The system provides acceleration response spectra, velocity response spectra, and cumulative absolute velocity (CAV) computation automatically after the event (maximum 30 minutes). As mentioned before, separate alarms are sent to the control room if the threshold value is exceeded at the location of an accelerometer.

3.7.4.4 Comparison of Measured and Predicted Responses

The plan for utilization of the seismic data includes both the function of the operator and engineering to evaluate the effects of an earthquake on the plant. A detailed description of the data flow is provided in applicable plant procedures.

Initial determination of the earthquake level is performed automatically immediately after the earthquake by comparing the measured response spectra from the containment tendon gallery (and all other accelerometer locations) with the OBE and SSE response spectra for the corresponding location. If the measured spectra exceed the OBE response spectra, the plant will be shut down and a detailed analysis of the earthquake motion will be undertaken.

After an earthquake, the data from the seismic recorders and recording instruments are reviewed. The data from these instruments are analyzed automatically and a printed report generated by the central controller to obtain the seismic accelerations experienced at the location of major Category 1 structures and equipment. The measured responses spectra from the SMAs, at the location of each Seismic Category 1 structure and system are compared automatically with those used in the design to determine whether the OBE has been exceeded.

TABLE 3.7.B.1-1

DAMPING VALUES FOR FIXED BASE STRUCTURES AND COMPONENTS[a]

	Percent of Critical Damping Per Mode	
Structure of Component	OBE	SSE
Equipment and large-diameter piping systems (pipe diameter in excess of 12 in.)[b]	2	3
Small-diameter piping systems (pipe diameter equal to or less than 12 in.)[b]	1	2
Welded steel structures	2	4
Bolted steel structures	4	7
Prestressed concrete structures	2	5
Reinforced concrete structures	4	7
Electrical cable trays and supports	(See figure 3.7.B.1-7.)	

a. Damping values for foundation material, used in foundation-structure interaction analysis, are not included in this table.

b. In lieu of these values, for ASME Boiler and Pressure Vessel Code, Section III, Division 1, Code Class 1, 2, and 3 piping systems, the damping values provided in figure 3.7.1-11 may be used per Code Case N-411.

TABLE 3.7.B.1-2

EMBEDMENT DEPTHS OF CATEGORY 1 STRUCTURES[c]

Structure	Foundation[a] Embedment Depth (ft)	Least Foundation Width (ft)	Structure[b] Height (ft)
Containment building	61	154	243
Auxiliary building	111	129	179
Control building	47	148[d]	140
Fuel handling building	66	76	134
NSCW towers	89	100	136
Diesel generator building	9	92	71
Condensate storage tanks	4	63	60
Refueling water storage tank	3	62	66
Reactor makeup water tank	2	51	46
Auxiliary feedwater pumphouse	7	40	31
NSCW valve house	20	20	50

a. Distance from bottom of foundation to grade level.

b. Distance from bottom of foundation to highest point of structure.

c. Buried structures are not included in this table.

d. Typical width for most parts of the foundation.

TABLE 3.7.N.1-1

DAMPING VALUES USED FOR SEISMIC SYSTEMS ANALYSIS

Item	Upset Conditions (OBE)	Faulted Condition (SSE, DBA)[a]
Primary coolant loop system components and large piping[b][c]	2	4
Small piping [c]	1	2
Welded steel structures	2	4
Bolted and/or riveted steel structures	4	7
Reinforced concrete structures	4	7

a. Design basis accident.

b. Applicable to 12-in. or larger diameter piping including pressurizer surge line piping.

c. As an alternative, ASME Code Case N-411 is used with response spectra analysis techniques. See figure 3.7.N.1-1.

TABLE 3.7.B.2-1

CLASSIFICATION OF SEISMIC SYSTEMS

Deeply embedded structures

- Containment building and containment internal structures
- Auxiliary building
- Control building[a]
- Fuel handling building[a]
- NSCW towers
- NSCW tower valve house

Shallowly embedded structures

- Diesel generator building
- Condensate storage tanks
- Refueling water storage tank
- Reactor makeup water tank
- Auxiliary feedwater pumphouse

Buried structures

- Category 1 tunnels
- Diesel fuel oil storage tank pumphouse
- Category 1 buried piping

a. Includes those portions of the equipment building supported by this structure.

TABLE 3.7.B.2-2

ACCELERATIONS OF MAJOR SEISMIC CATEGORY 1 STRUCTURES

Node Elevation	SSE Acceleration (g) Horizontal	SSE Acceleration (g) Vertical	OBE Acceleration (g) Horizontal	OBE Acceleration (g) Vertical
Containment internals				
169 ft 0 in. (basemat)	0.21	0.38	0.14	0.23
261 ft 0 in.	0.50	0.50	0.35	0.35
Containment shell				
169 ft 0 in. (basemat)	0.21	0.38	0.14	0.23
258 ft 0 in.	0.31	0.41	0.22	0.27
361 ft 0 in. (spring line)	0.45	0.43	0.30	0.29
Auxiliary building				
119 ft 3 in. (basemat)	0.20	0.28	0.12	0.18
220 ft 0 in.	0.28	0.30	0.18	0.20
288 ft 3 in. (roof)	0.38	0.35	0.24	0.22
Control building				
180 ft 0 in. (basemat)	0.26	0.40	0.17	0.24
220 ft 0 in.	0.29	0.41	0.19	0.25
280 ft 0 in. (roof)	0.73	0.88	0.53	0.69
Fuel handling building				
160 ft 0 in. (basemat)	0.24	0.38	0.16	0.24
220 ft 0 in.	0.39	0.42	0.25	0.29
288 ft 2 in. (roof)	0.60	0.48	0.41	0.33
Diesel generator building				
219 ft 0 in. (basemat)	0.26	0.30	0.16	0.19
274 ft 0 in. (roof)	0.34	0.32	0.21	0.19

TABLE 3.7.B.2-3

THE FREQUENCIES FOR FLOOR RESPONSE SPECTRA CALCULATIONS (Hz)

0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3
2.4	2.5	2.6	2.7	2.8	2.9	3.0	3.15	3.3	3.45	3.6
3.8	4.0	4.2	4.4	4.6	4.8	5.0	5.25	5.5	5.75	6.0
6.25	6.5	6.75	7.0	7.25	7.5	7.75	8.0	8.5	9.0	9.5
10	10.5	11	11.5	12	12.5	13	13.5	14	14.5	15
16	17	18	20	22	25	28	31	34		

TABLE 3.7.4-1 (SHEET 1 OF 2)

SEISMIC MONITORING INSTRUMENTATION REQUIREMENTS

	Triaxial Time-History Accelerograph			Triaxial Response Spectrum Recorder			Triaxial Peak Accelerograph			Triaxial Seismic Switch			Seismic Trigger		
Instrumentation Location	RG 1.12 Req.	SRP Req.	VEGP	RG 1.12 Req.	SRP Req.	VEGP	RG 1.12 Req.	SRP Req.	VEGP	RG 1.12 Req.	SRP Req.	VEGP	RG 1.12 Req.	SRP Req.	VEGP
I. Field															
Requirement - Location															
1. Field – 225 ft from structure	1	1[a]	1(1)[g][e][h]												
2. Free field – Approximately2000 ft from structure			1(2)[g][e]												
II. Inside Containment															
Requirement - Location															
1. Basemat - Tendon gallery	1	1[a]	1[g](2)[d]	1[a]	1[a]	0[d]									
2. Structure - Operating floor el 220 ft	1	1	1[g](3)												
3. Structure - Bioshield wall el 202 ft			1[g](4)[d]	1	1	0[d]									
III. Outside Containment															
Requirement - Location															
1. Seismic Cat. 1 piping support or floor - Aux. bldg. floor el 220 ft			1[d](6)[g]	C	C	0[d]									
2. Independent Seismic Cat. 1 foundation struct. - Aux. bldg. slab			1[d](7)[g]	C	C	0[d]									
3. Diesel generator bldg. el 220 ft	0	0	1[g]												

Note: A response spectrum analyzer is located in the control room.

TABLE 3.7.4-1 (SHEET 2 OF 2)

---

a. Sensor shall have control room readout.

b. Audible and visual alarm in the control room.

c. Denotes one of two locations.

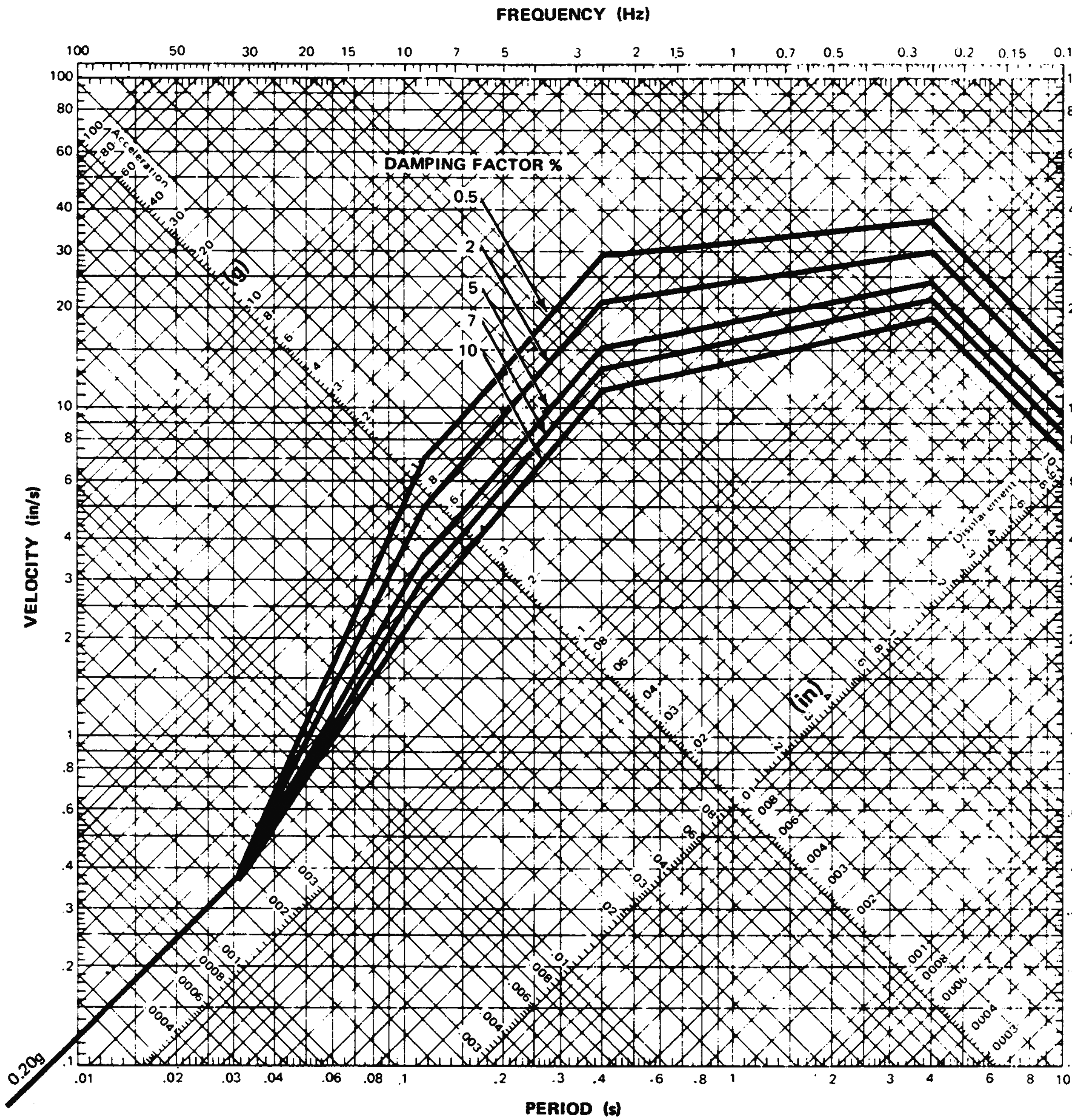
d. Response-spectrum analyzer preferred instead of response-spectrum recorder.

e. Parenthetical numbers show instrument locations in figure 3.7.4-1.

f. Starts time-history accelerograph.

g. Absolute acceleration data from each accelerometer are recorded and manually fed into a playback response spectrum analyzer to provide control room acceleration data and response spectrum for each sensor.

h. When utilizing data from this SMA, potential effects due to soil/structure interaction with adjacent structures will be considered.



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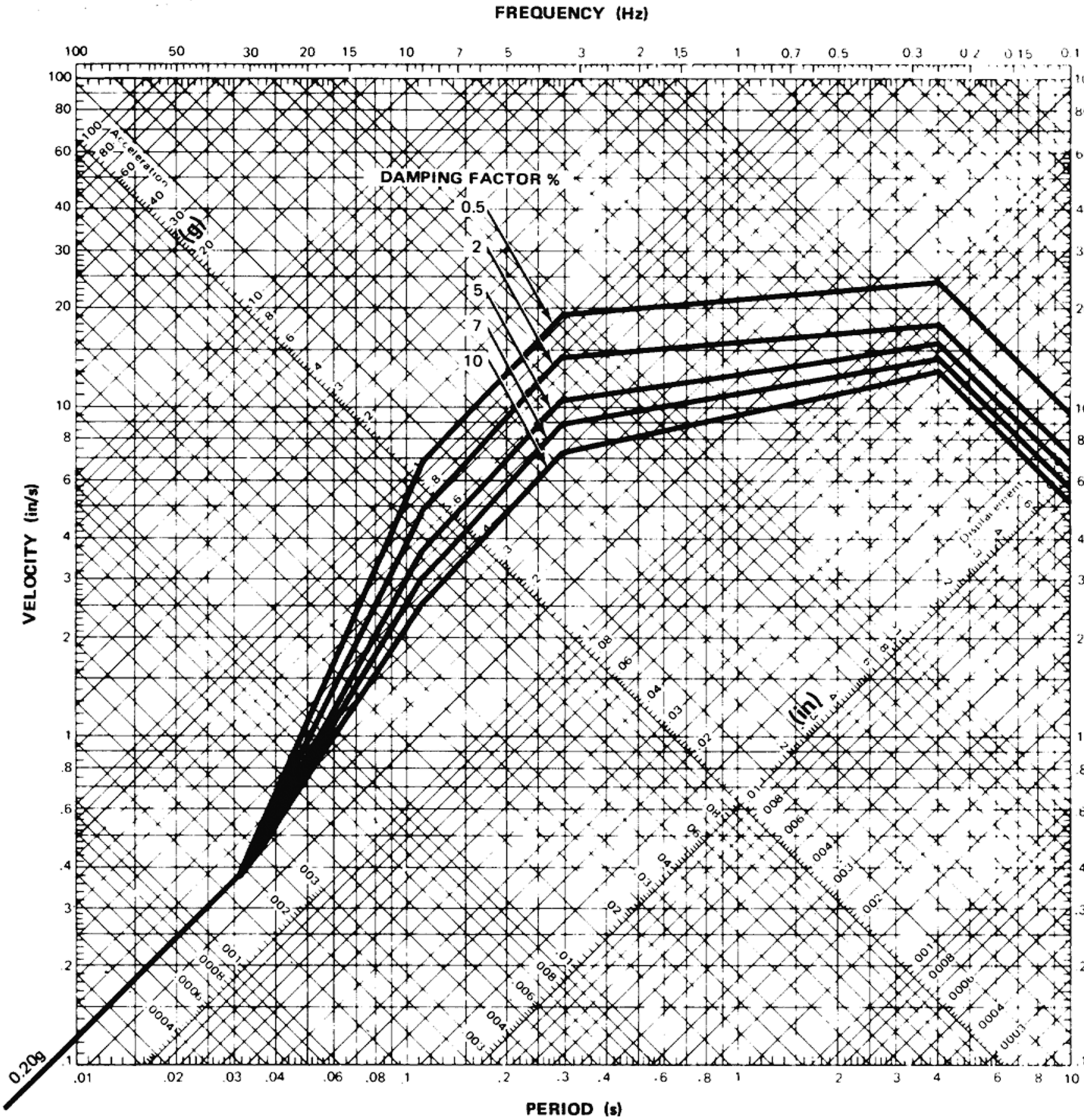
SOUTHERN COMPANY
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

SAFE SHUTDOWN EARTHQUAKE
HORIZONTAL RESPONSE SPECTRA

FIGURE 3.7.B.1–1



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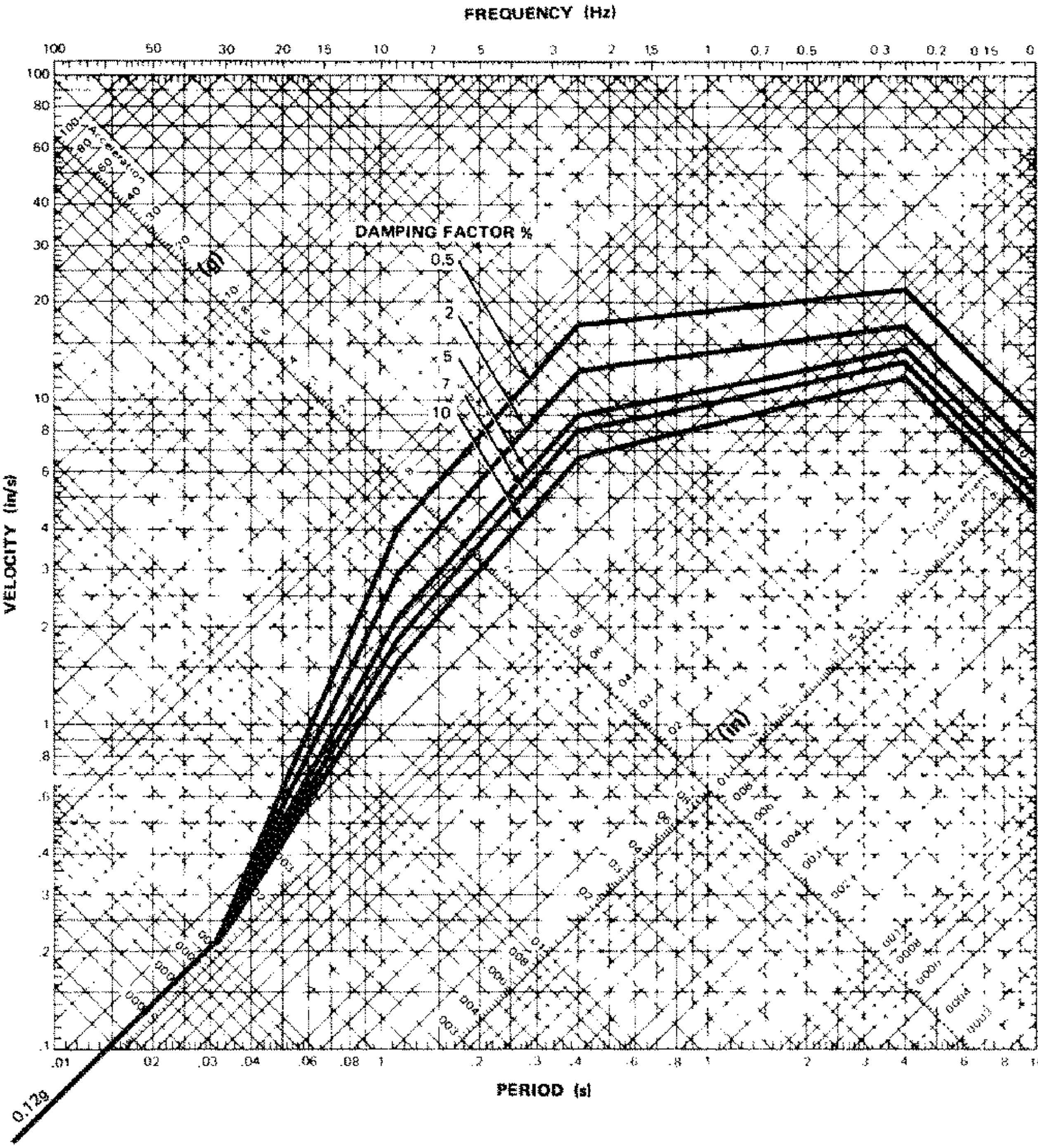
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

SAFE SHUTDOWN EARTHQUAKE
VERTICAL RESPONSE SPECTRA

FIGURE 3.7.B.1–2



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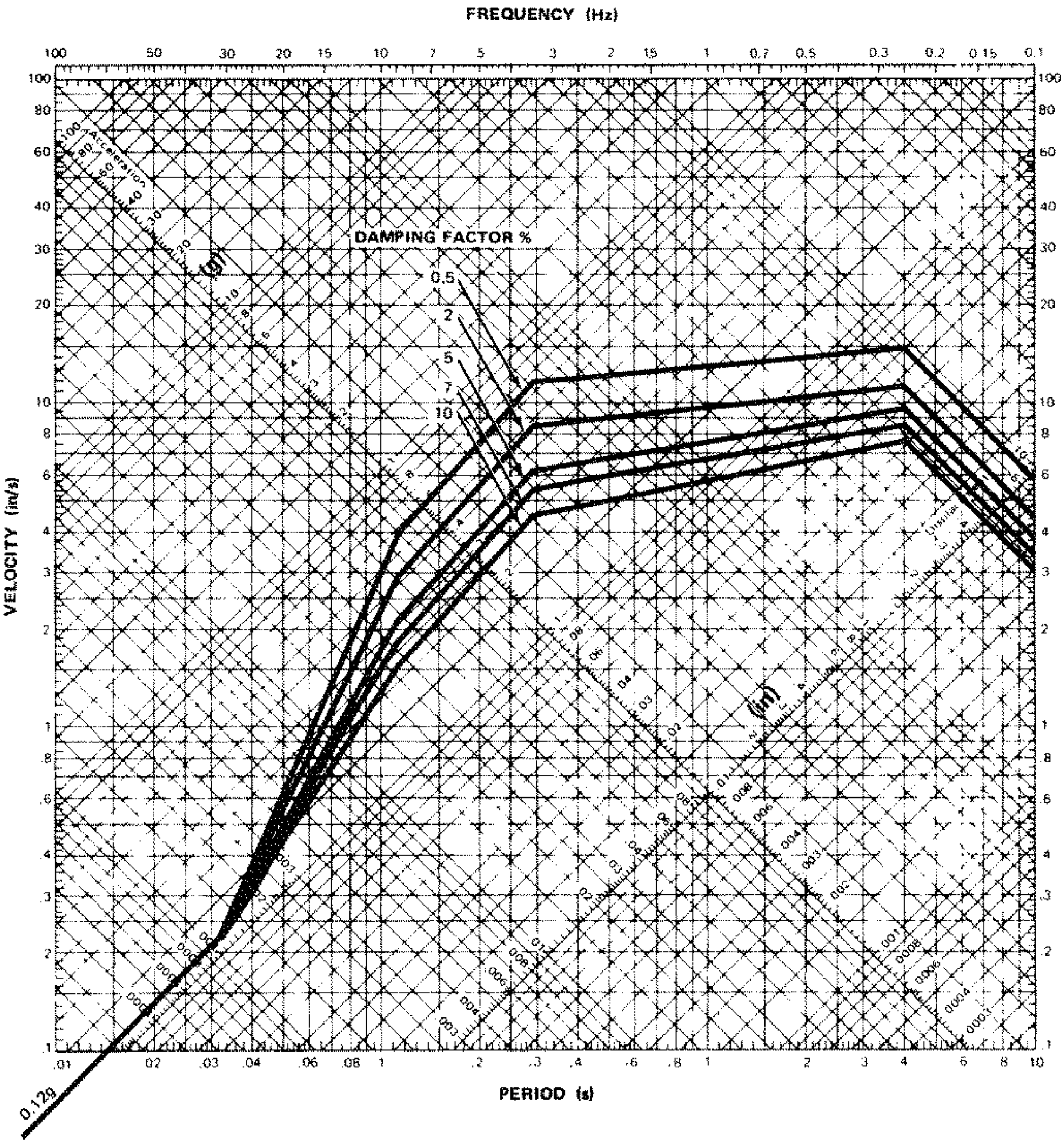
VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

OPERATING BASIS EARTHQUAKE
HORIZONTAL RESPONSE SPECTRA

FIGURE 3.7.B.1–3



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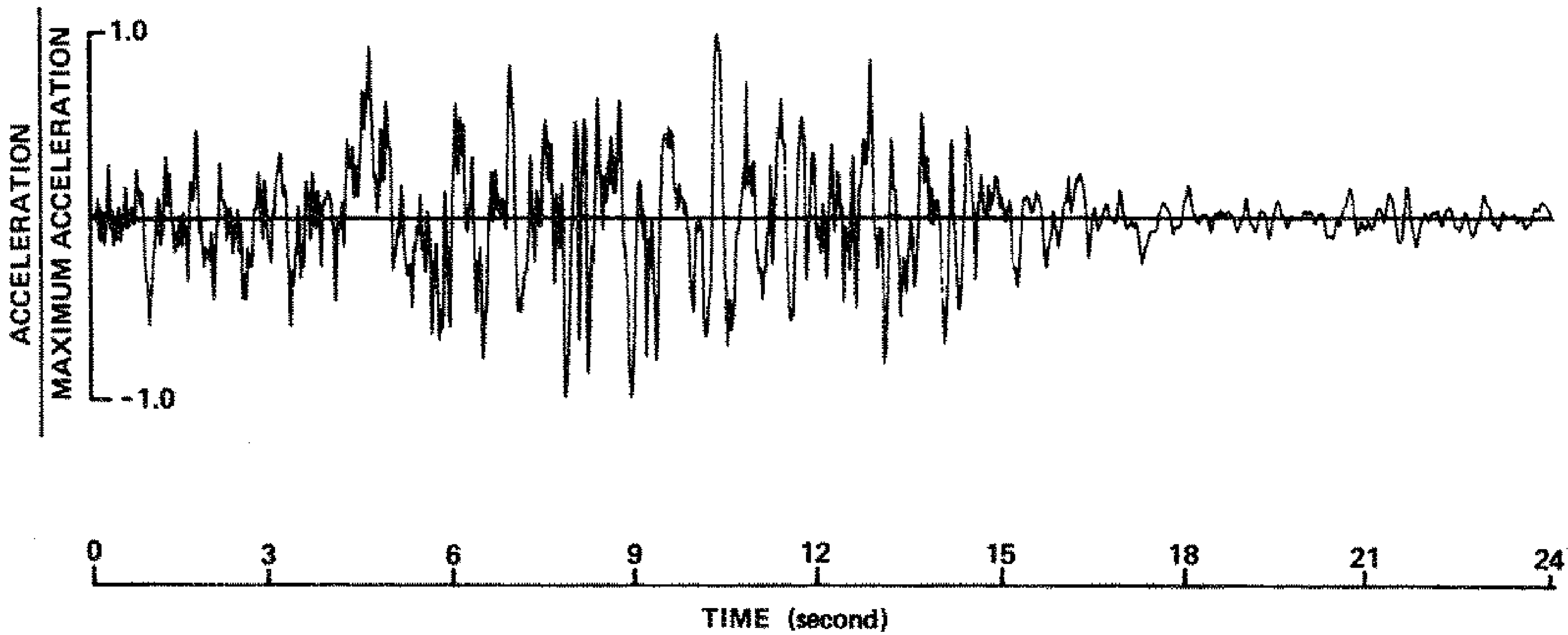
VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

OPERATING BASIS EARTHQUAKE
VERTICAL RESPONSE SPECTRA

FIGURE 3.7.B.1–4



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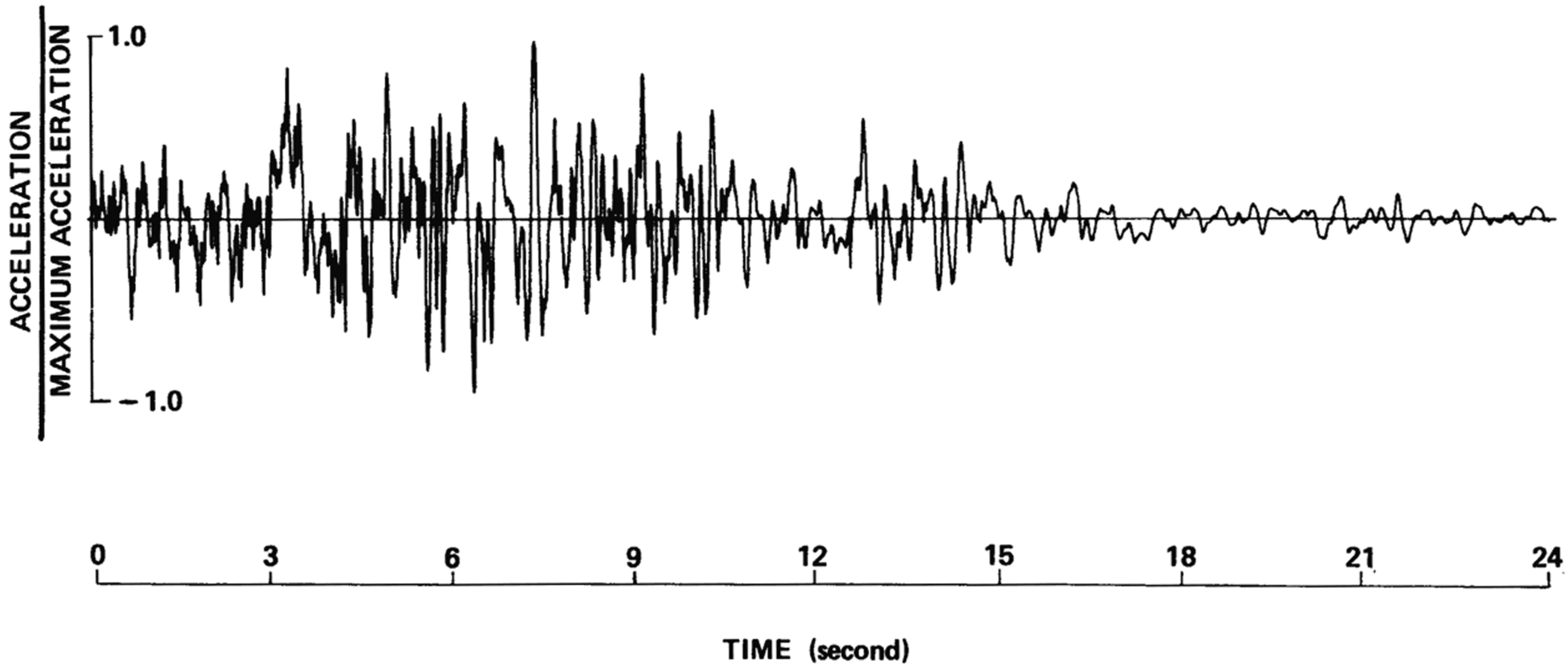
VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

SYNTHETIC TIME HISTORY
HORIZONTAL DIRECTION

FIGURE 3.7.B.1–5



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VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

SOUTHERN COMPANY
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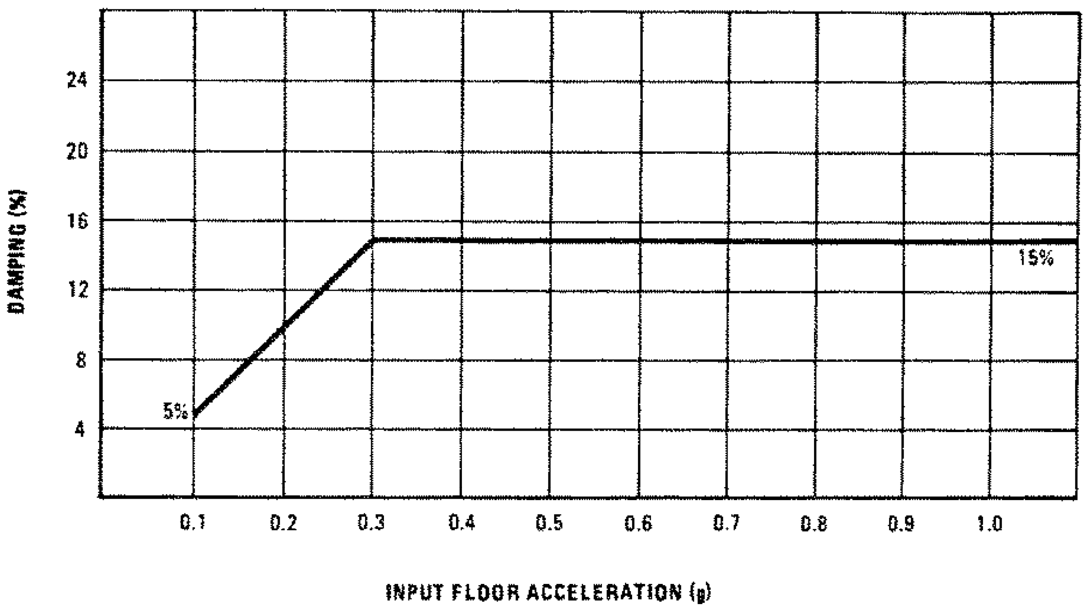
SYNTHETIC TIME HISTORY
VERTICAL DIRECTION

FIGURE 3.7.B.1–6



NOTE:

THE CURVE SHOWN IS APPLICABLE FOR 50% TO FULLY LOADED TRAYS. FOR UNLOADED TRAYS, DAMPING VALUES OF 2% FOR OBE AND 4% FOR SSE ARE APPLICABLE.

FOR TRAYS LOADED LESS THAN 50%, LINEAR INTERPOLATION IS USED TO DETERMINE THE ASSOCIATED DESIGN DAMPING VALUES.

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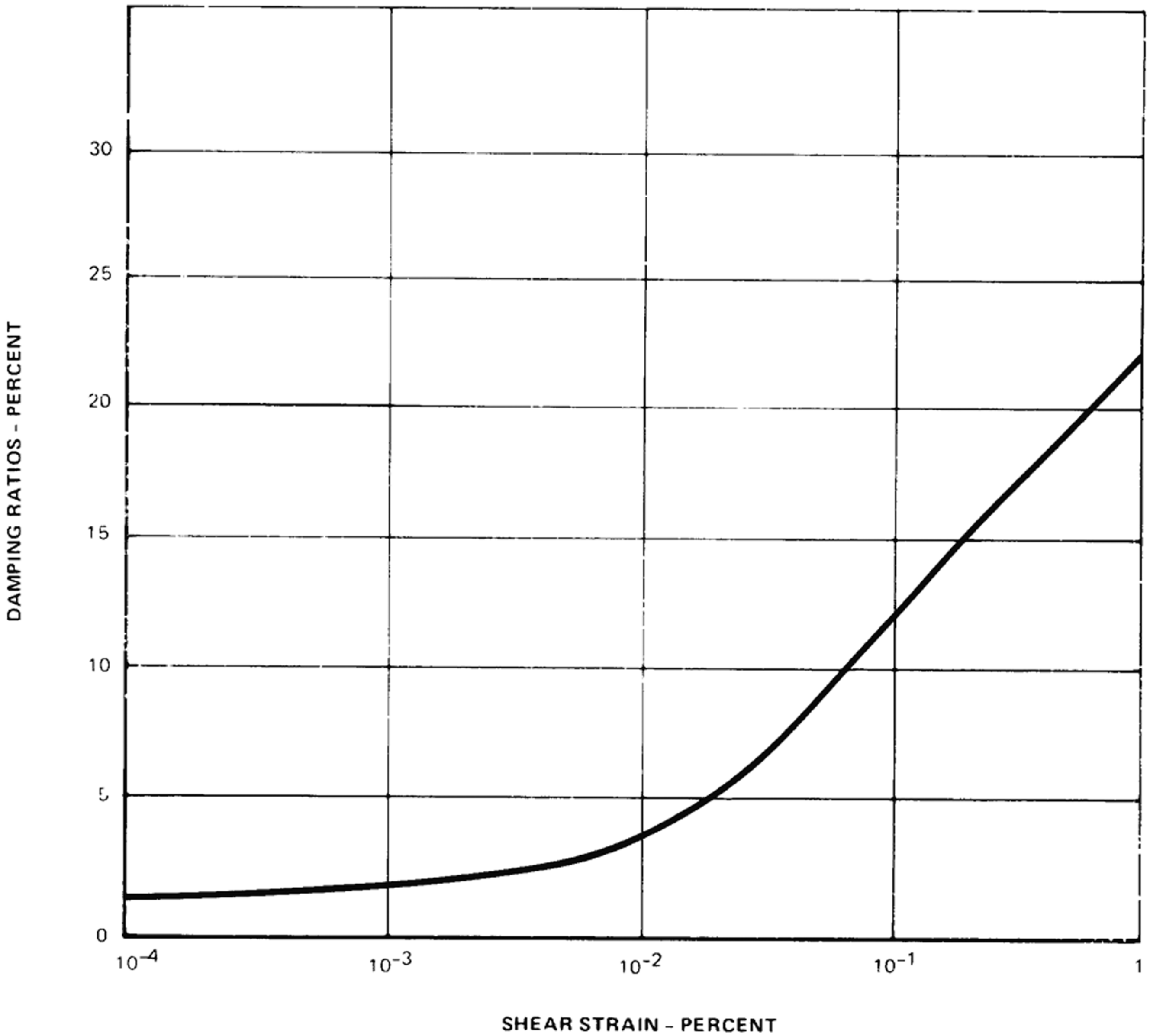
SOUTHERN COMPANY Energy to Serve Your World®	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	DAMPING VALUES FOR CABLE TRAYS AND SUPPORTS FIGURE 3.7.B.1–7



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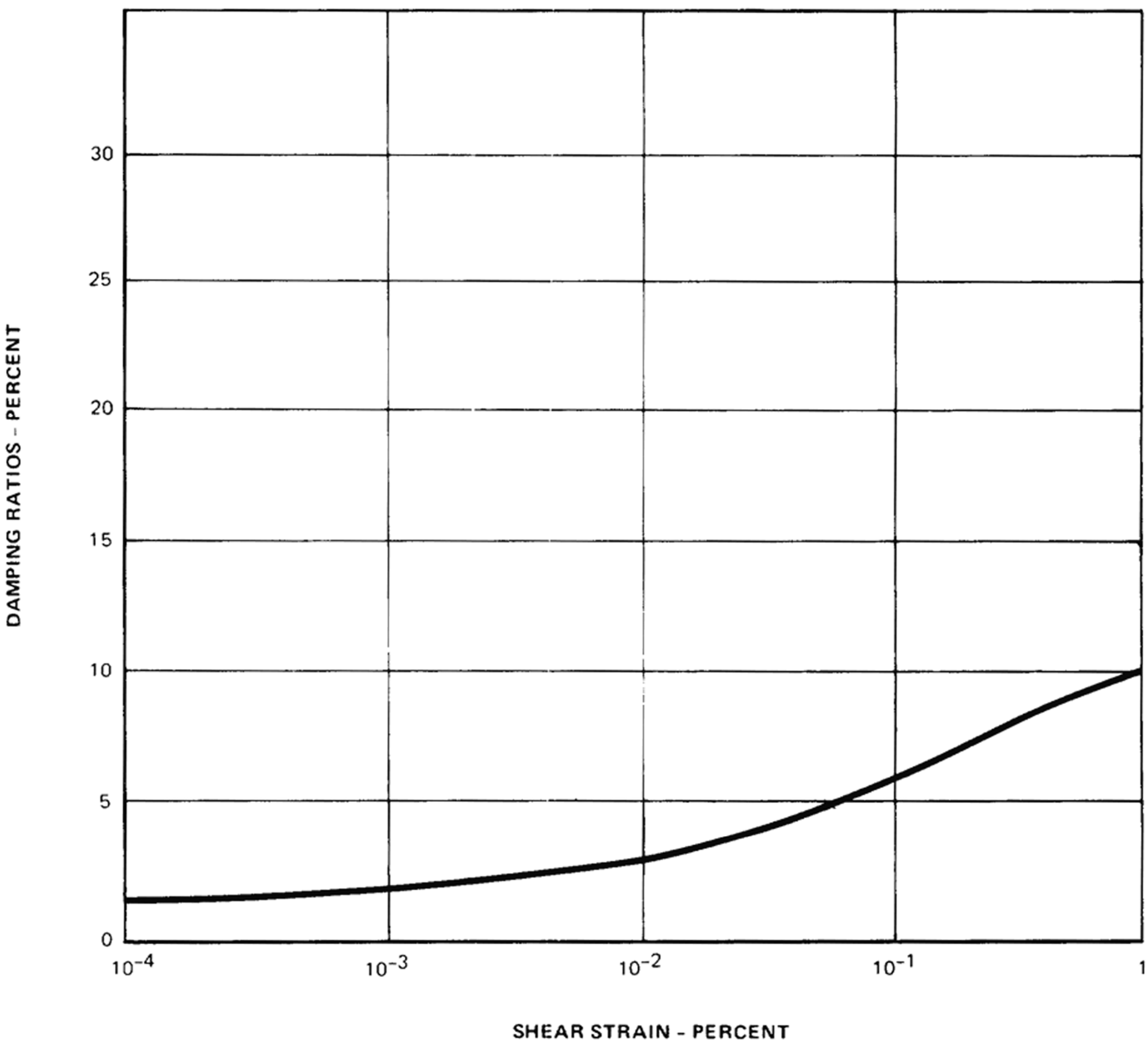
SOUTHERN COMPANY
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

DAMPING RATIOS VS SHEAR STRAIN
FOR COMPACTED SAND BACKFILL

FIGURE 3.7.B.1–8



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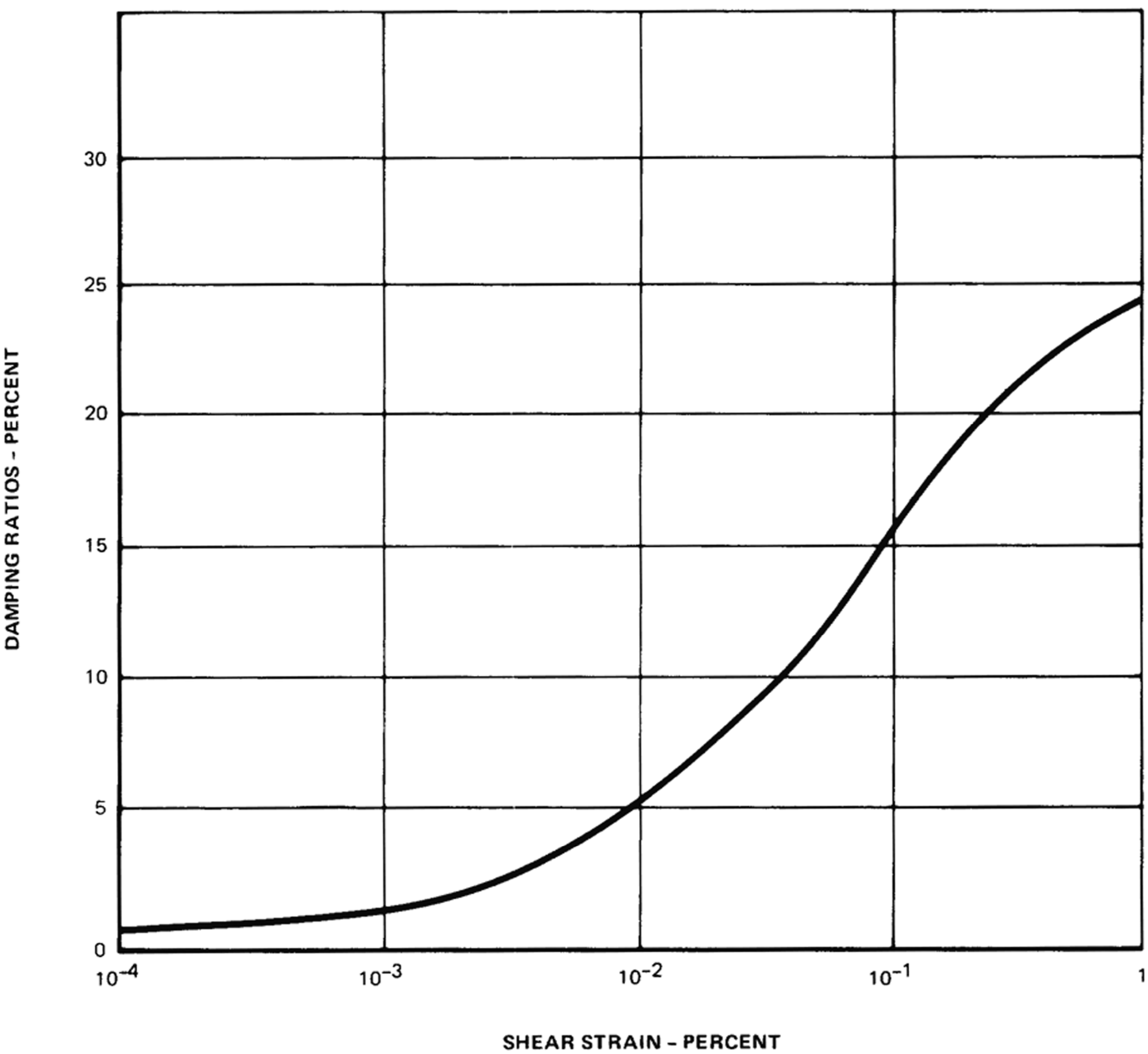
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

DAMPING RATIOS VS SHEAR STRAIN FOR CLAY MARL BEARING STRATUM

FIGURE 3.7.B.1–9



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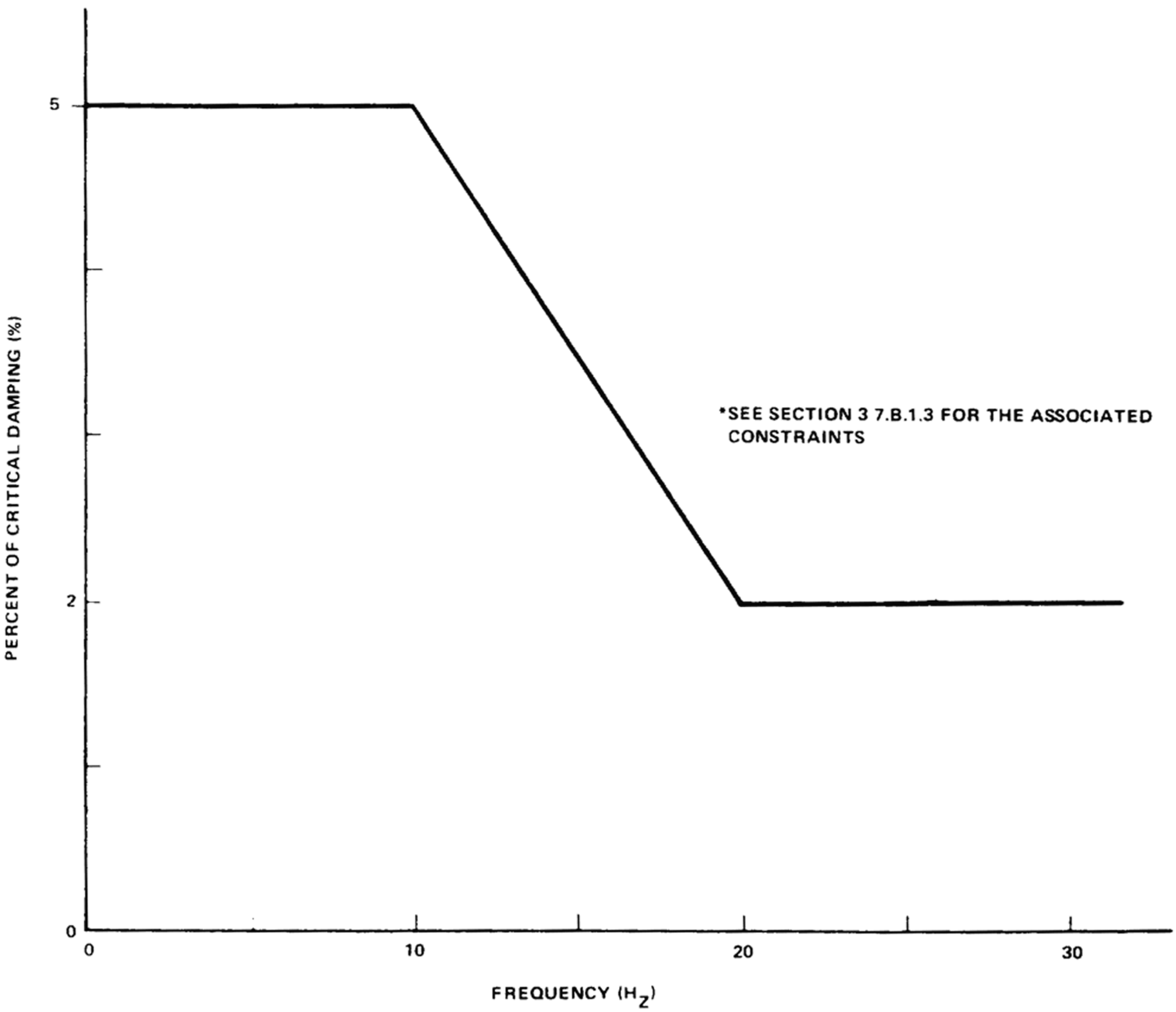
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

DAMPING RATIOS VS SHEAR STRAIN FOR LOWER SAND STRATUM

FIGURE 3.7.B.1–10



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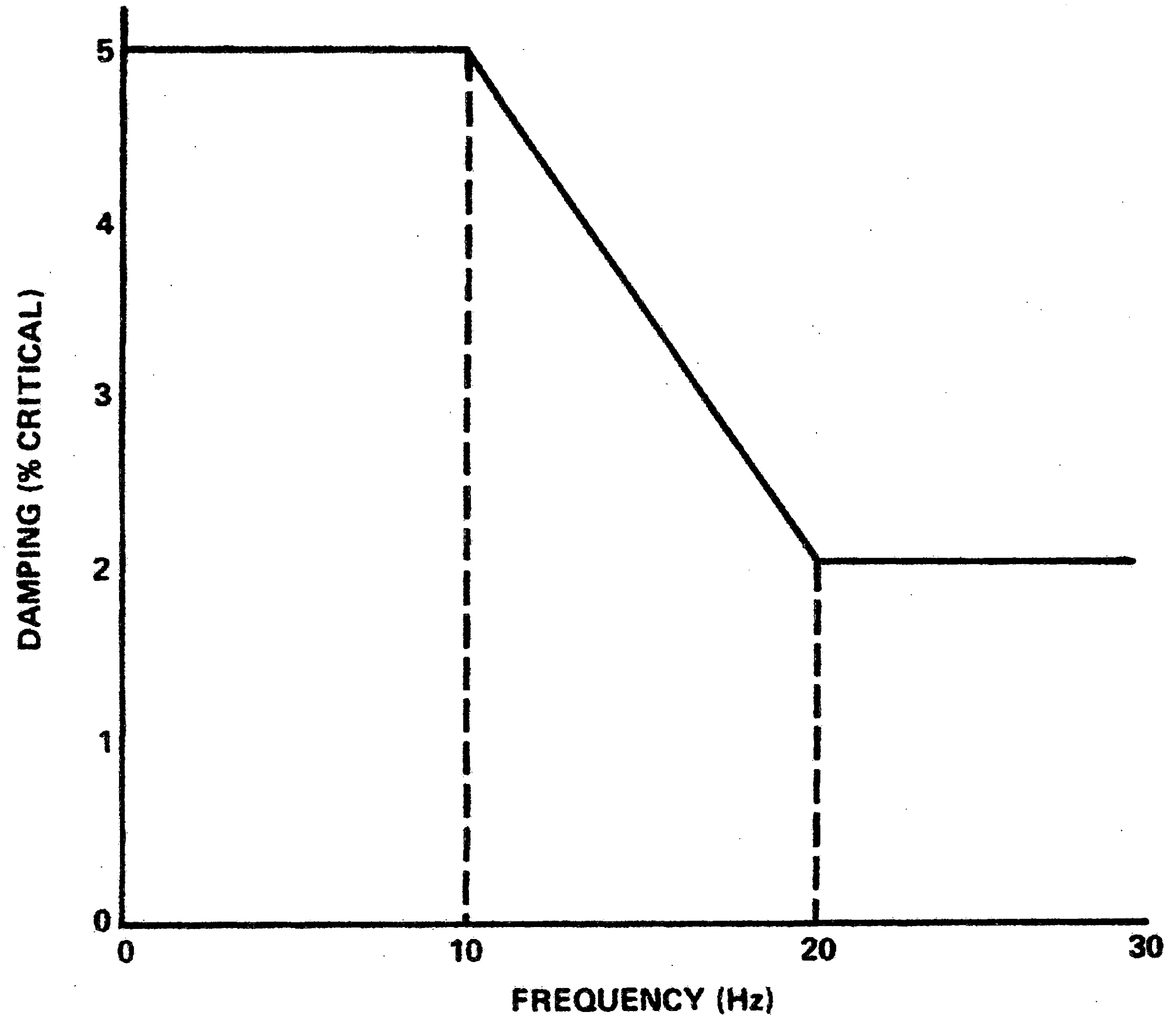
SOUTHERN COMPANY *Energy to Serve Your World®* VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	ALTERNATE DAMPING VALUES FOR ASME B&PV CODE, SECTION III, DIVISION 1, CODE CLASS 1, 2 AND 3 PIPING SYSTEMS
	FIGURE 3.7.B.1–11



(APPLICABLE TO BOTH OBE & SSE, INDEPENDENT OF PIPE DIAMETER)

*These damping values are used only for the piping systems analyzed by response spectra analysis techniques and the as-built piping displacements and clearances are acceptable.

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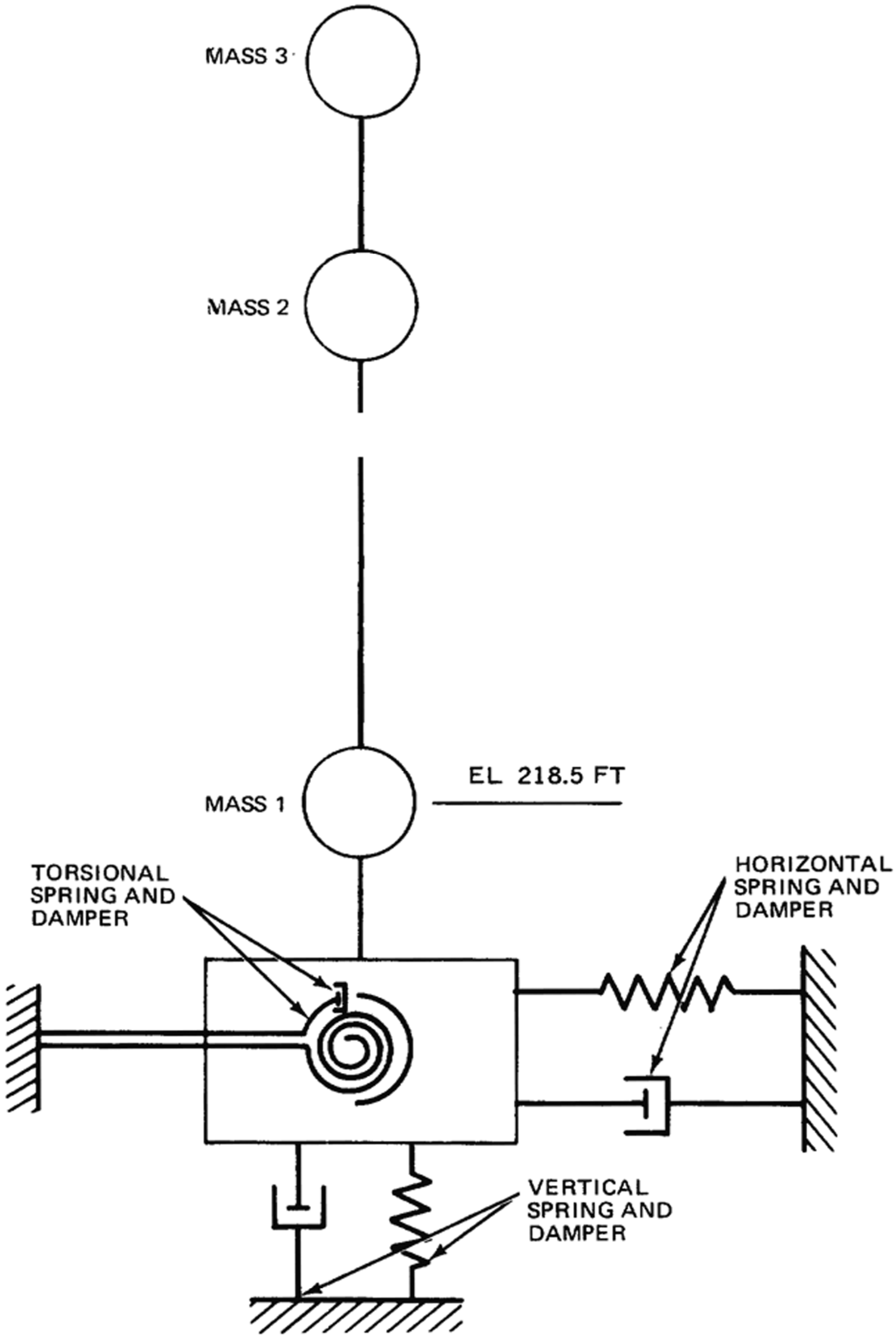
SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	ALTERNATE DAMPING VALUES USED FOR SEISMIC ANALYSIS OF PIPING FOR WESTINGHOUSE ANALYZED PIPING SYSTEMS* FIGURE 3.7.N.1–1



THE MODEL SHOWN IS TWO DIMENSIONAL FOR CLARITY. THE ACTUAL MODEL USED IS THREE DIMENSIONAL, WITH 6 (3 TRANSLATIONAL AND 3 ROTATIONAL) DEGREES OF FREEDOM PER NODE.

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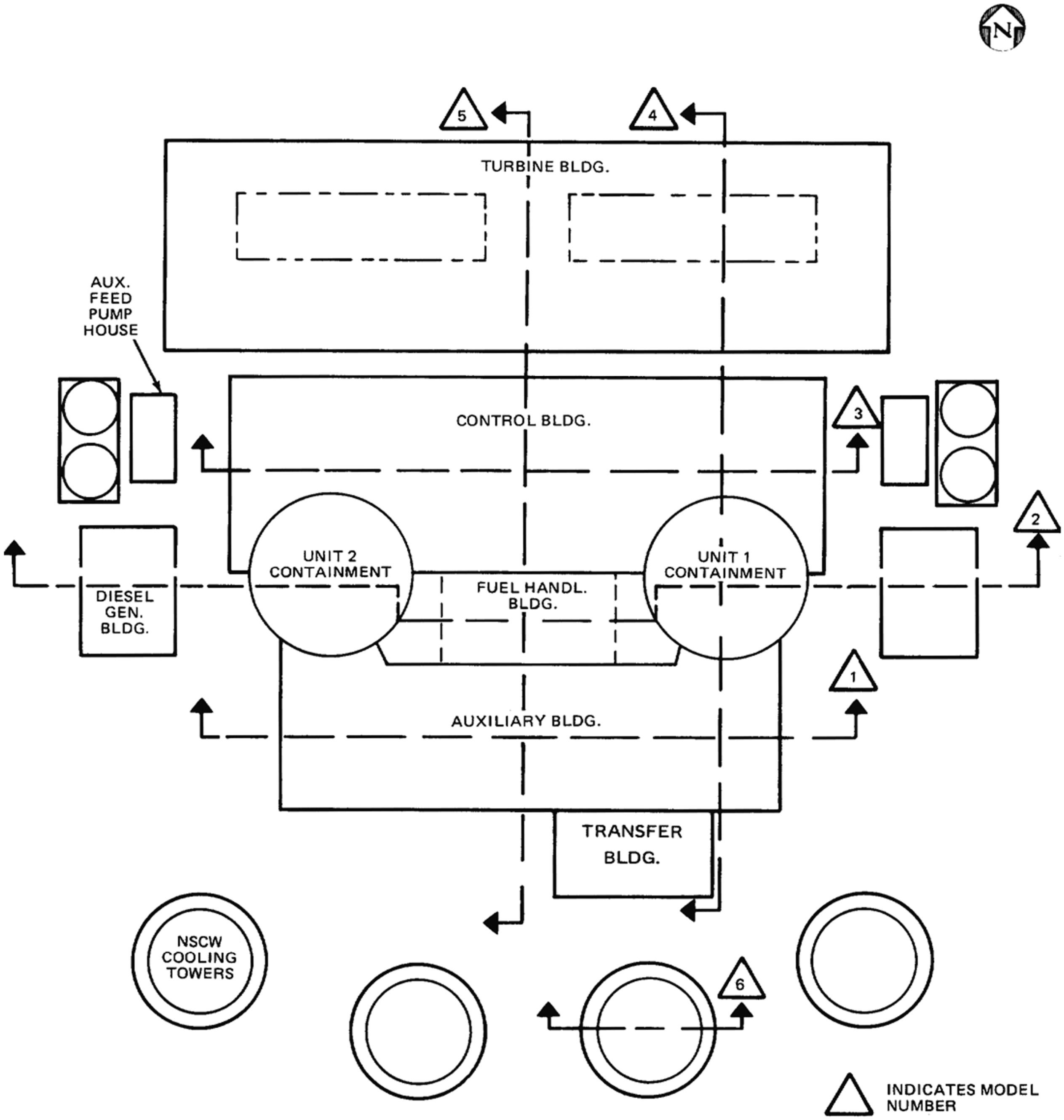
SOUTHERN COMPANY Energy to Serve Your World®	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	LUMPED PARAMETER MODEL OF DIESEL GENERATOR BUILDING
		FIGURE 3.7.B.2–1



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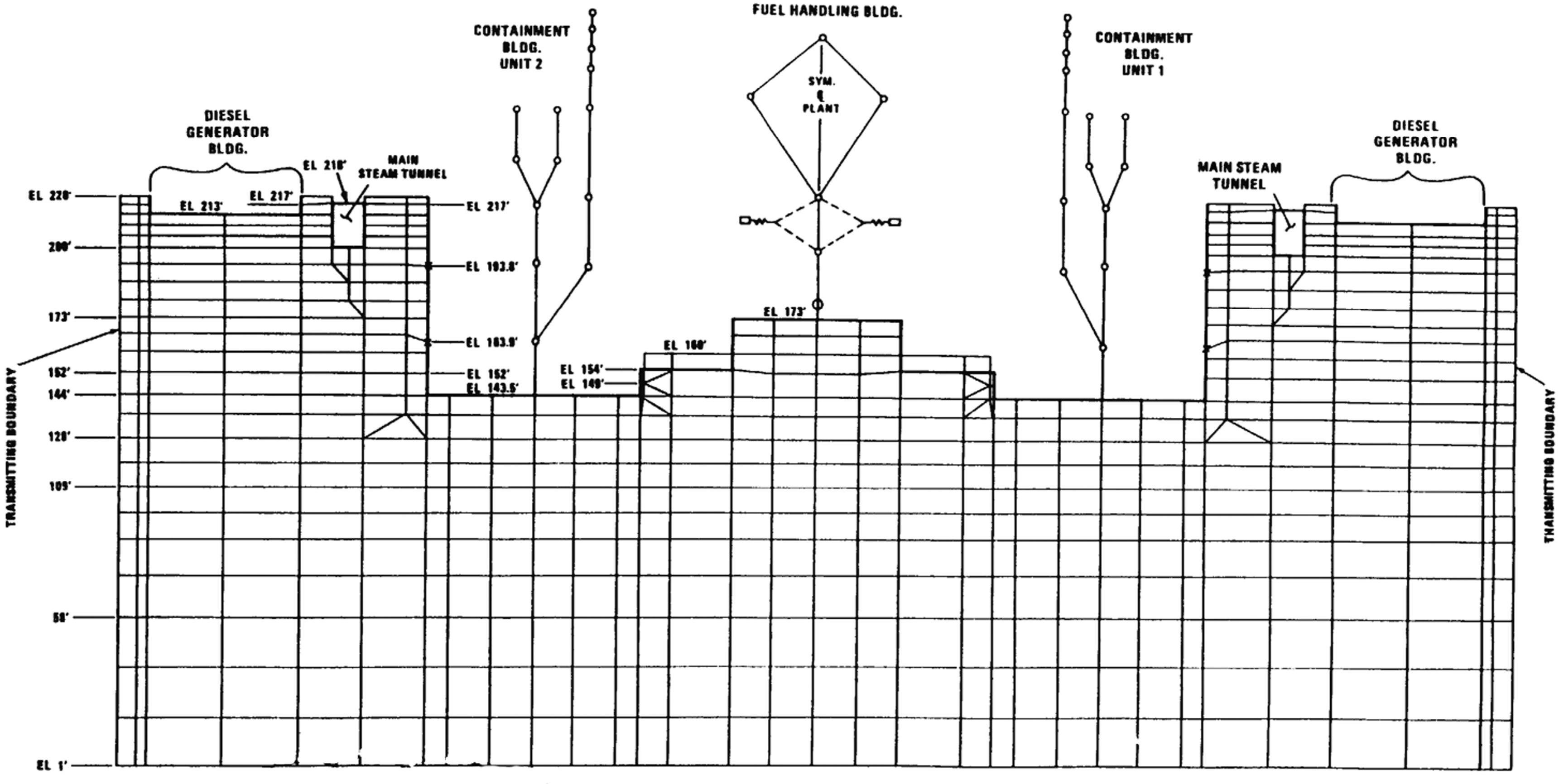
SOUTHERN COMPANY *Energy to Serve Your World®*	**VOGTLE** **ELECTRIC GENERATING PLANT** **UNIT 1 AND UNIT 2**	POWER BLOCK PLAN VIEW SHOWING SECTIONS FOR FINITE ELEMENT SOIL–STRUCTURE INTERACTION FLUSH MODELS FIGURE 3.7.B.2–2



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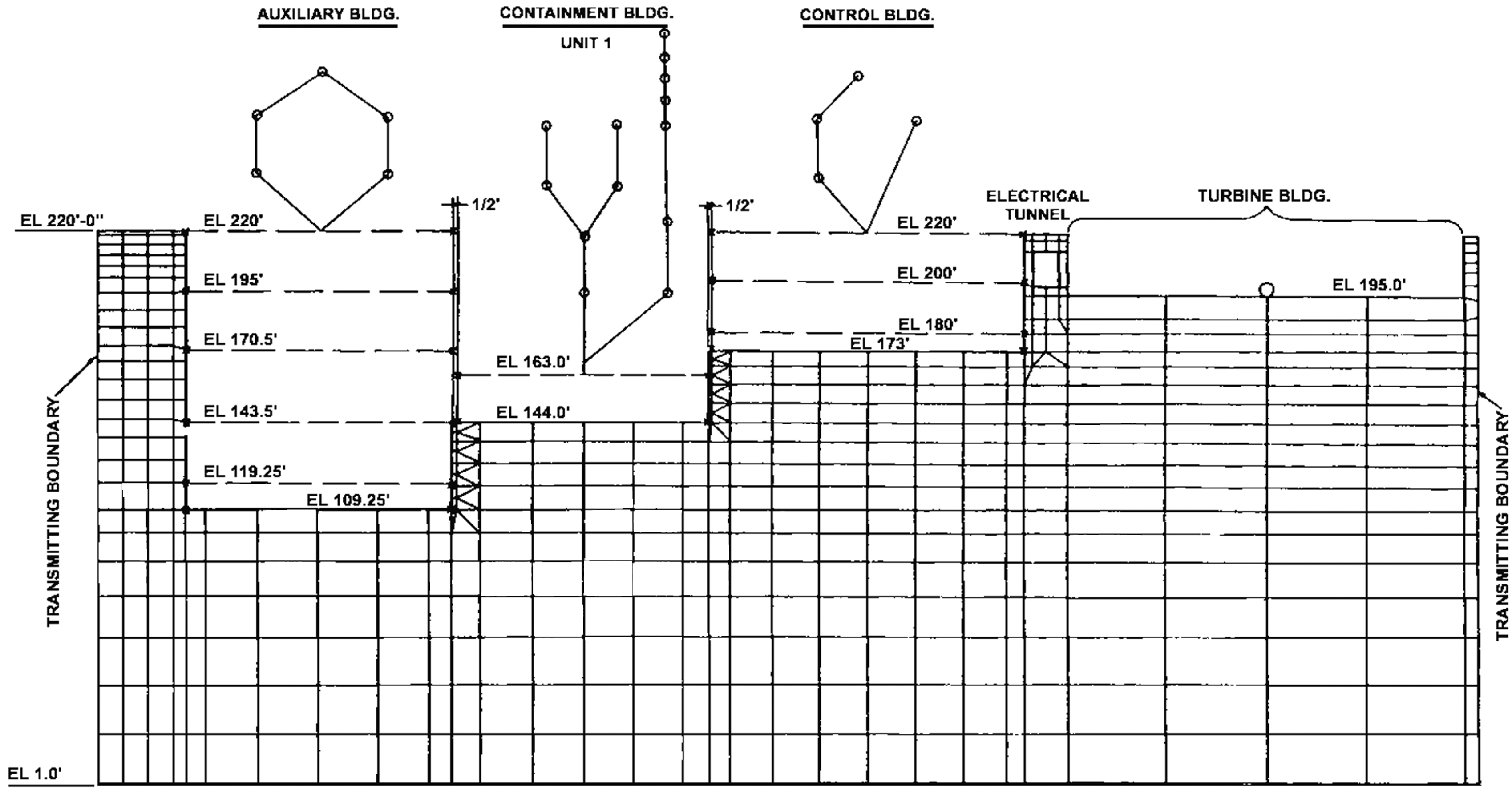
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

FLUSH MODEL SECTION 2
OF FIGURE 3.7.B.2–2

FIGURE 3.7.B.2–3



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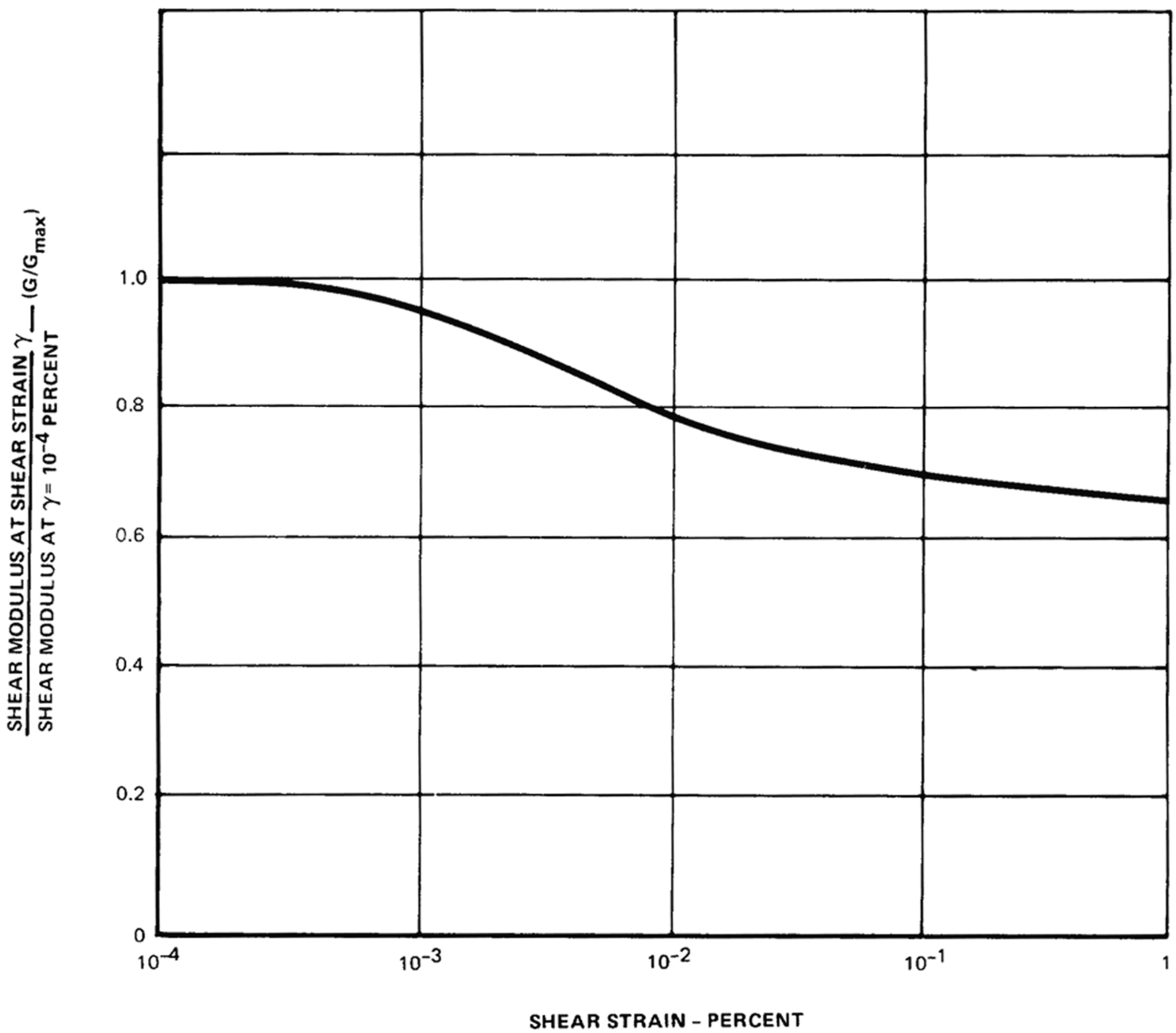
VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

FLUSH MODEL ALONG SECTION 4
OF FIGURE 3.7.B.2–2

FIGURE 3.7.B.2–4



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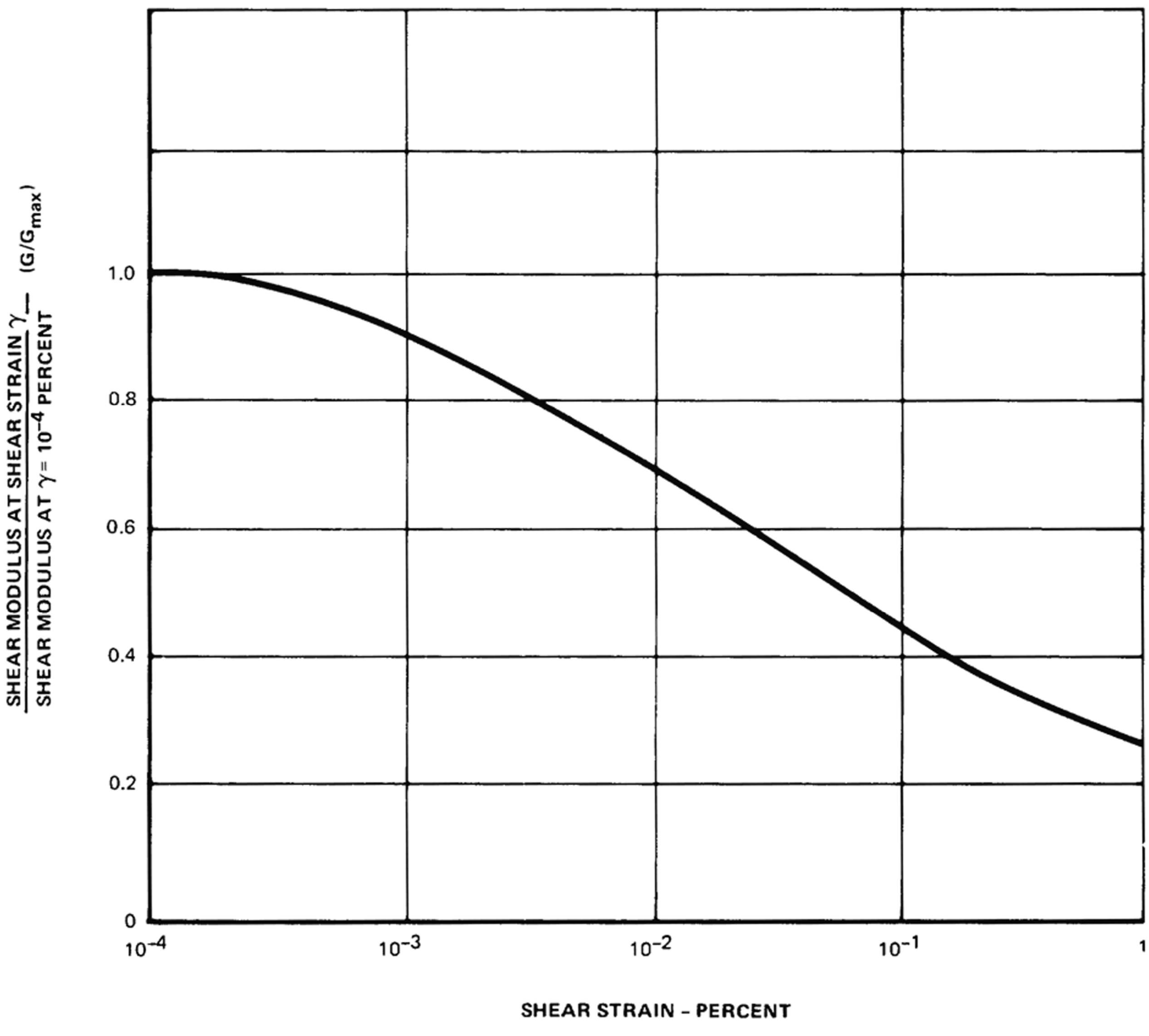
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

G/G_{MAX} VS SHEAR STRAIN FOR COMPACTED SAND BACKFILL

FIGURE 3.7.B.2–5



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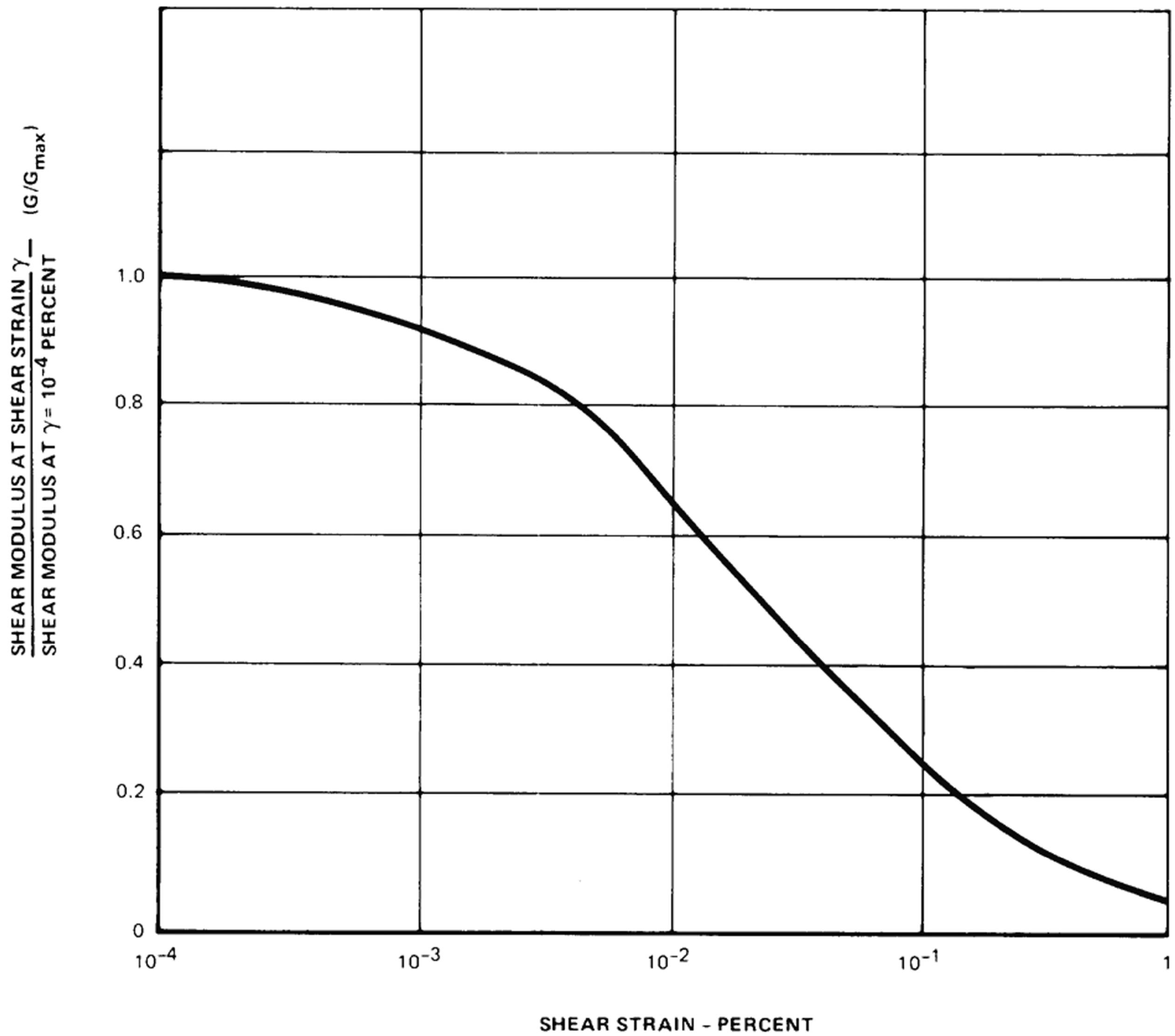
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

G/G_{MAX} VS SHEAR STRAIN FOR SATURATED CLAY MARL–BEARING STRATUM

FIGURE 3.7.B.2–6



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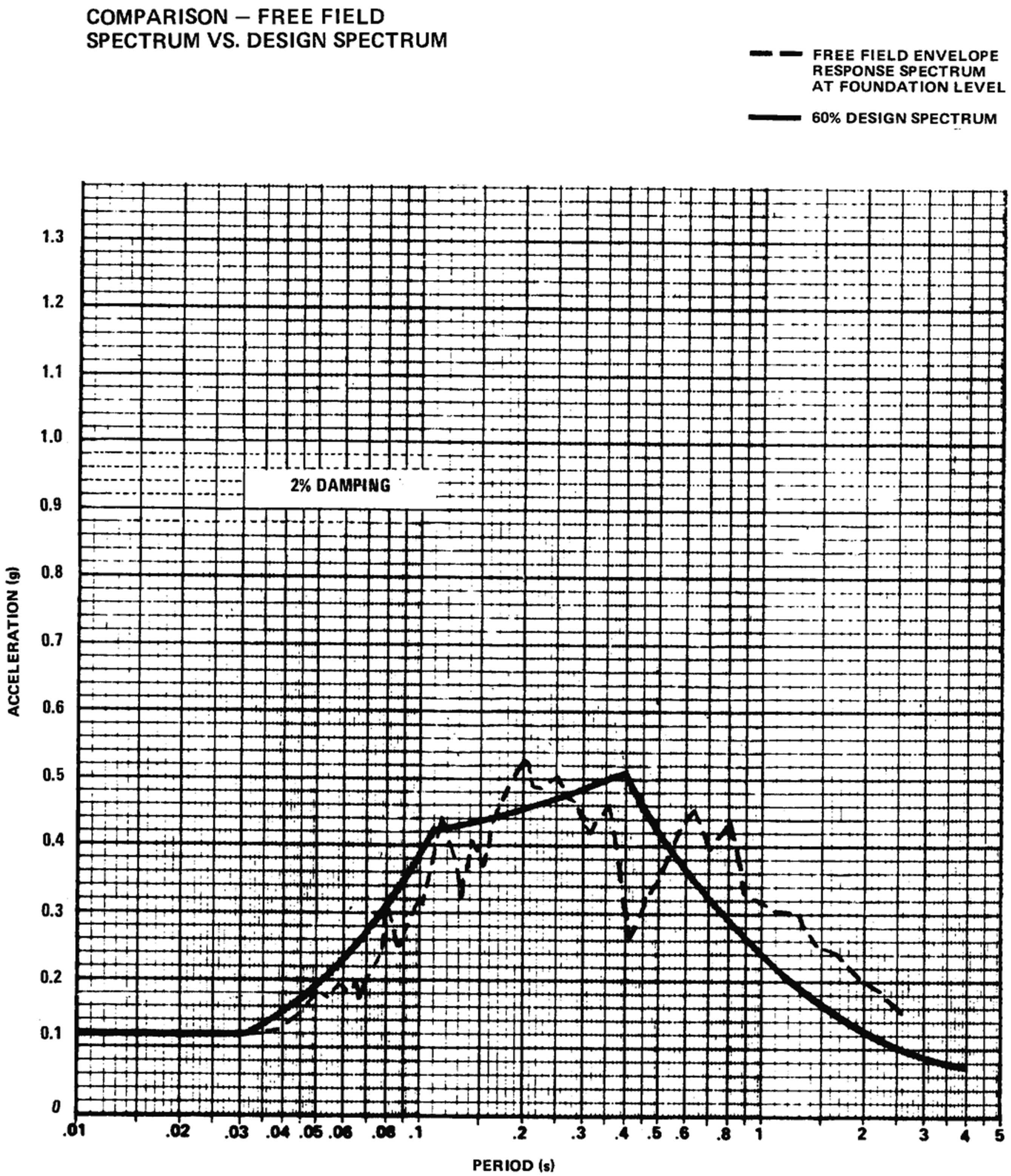
SOUTHERN COMPANY *Energy to Serve Your World®* VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	G/G_{MAX} VS SHEAR STRAIN FOR LOWER SAND STRATUM
	FIGURE 3.7.B.2–7



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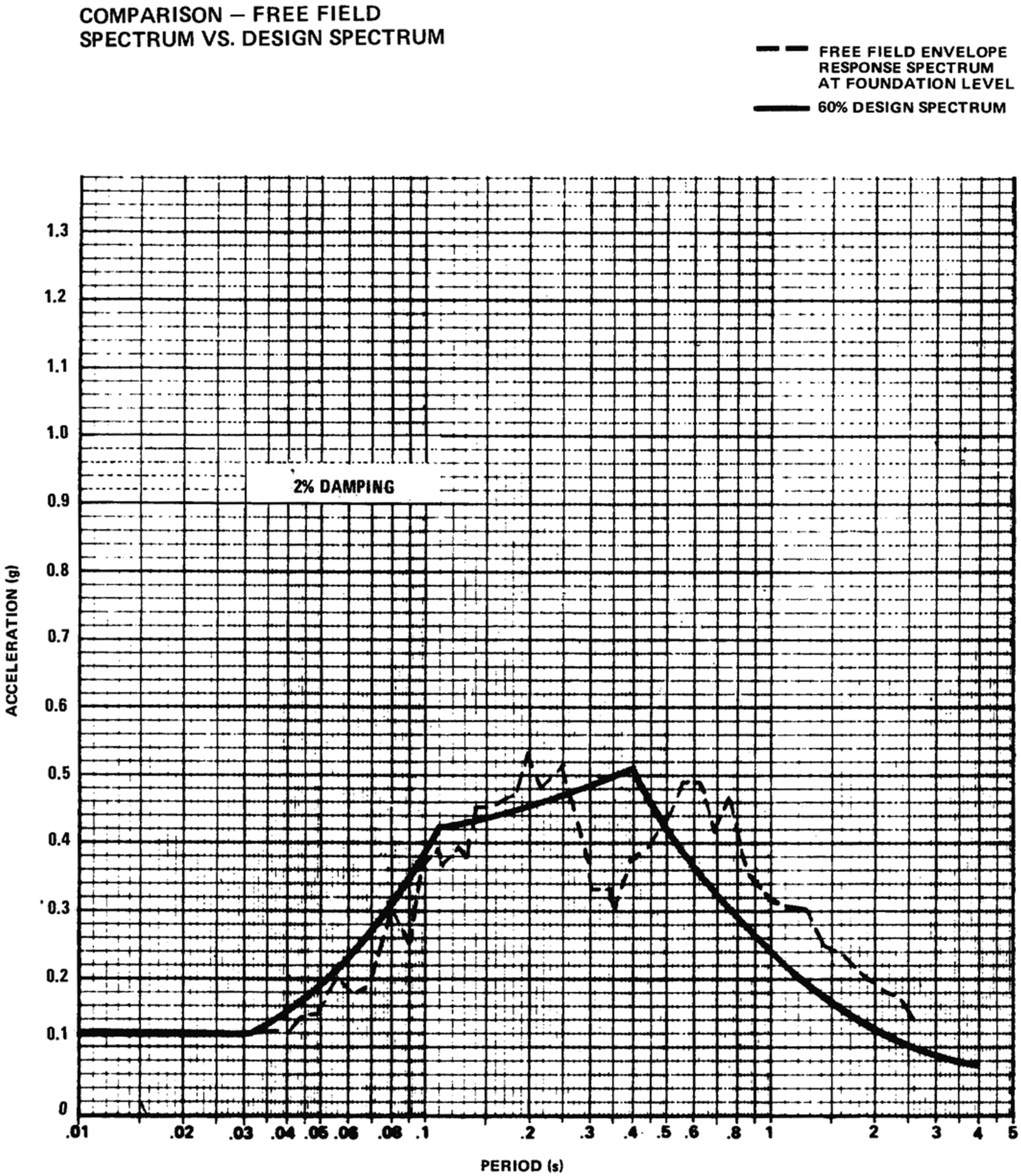
SOUTHERN COMPANY *Energy to Serve Your World®* VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	AUXILIARY BUILDING el 109 ft 0 in., SSE HORIZONTAL
	FIGURE 3.7.B.2–8



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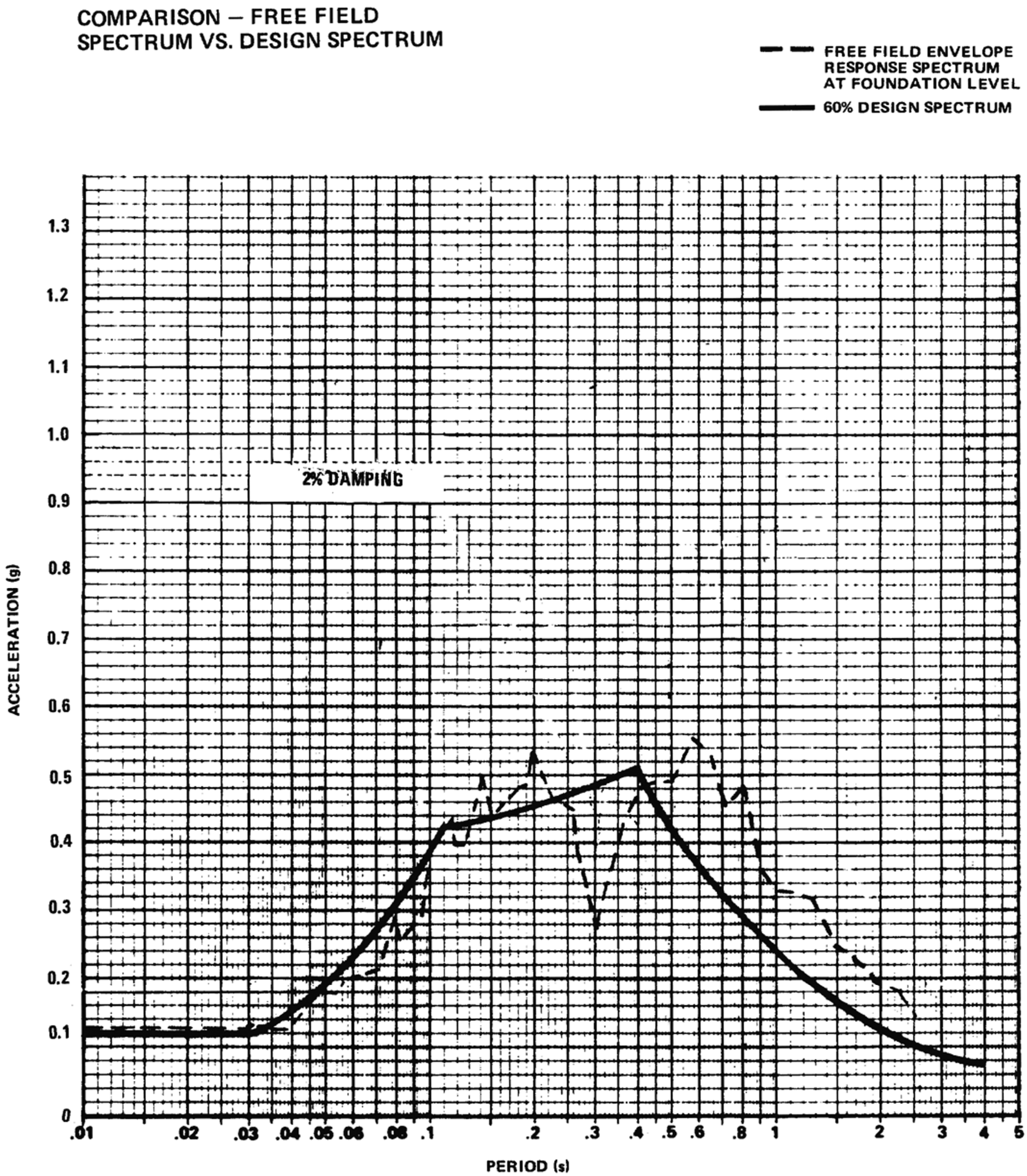
SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	NUCLEAR SERVICE COOLING WATER TOWER el 128 ft 0 in., SSE HORIZONTAL FIGURE 3.7.B.2–9



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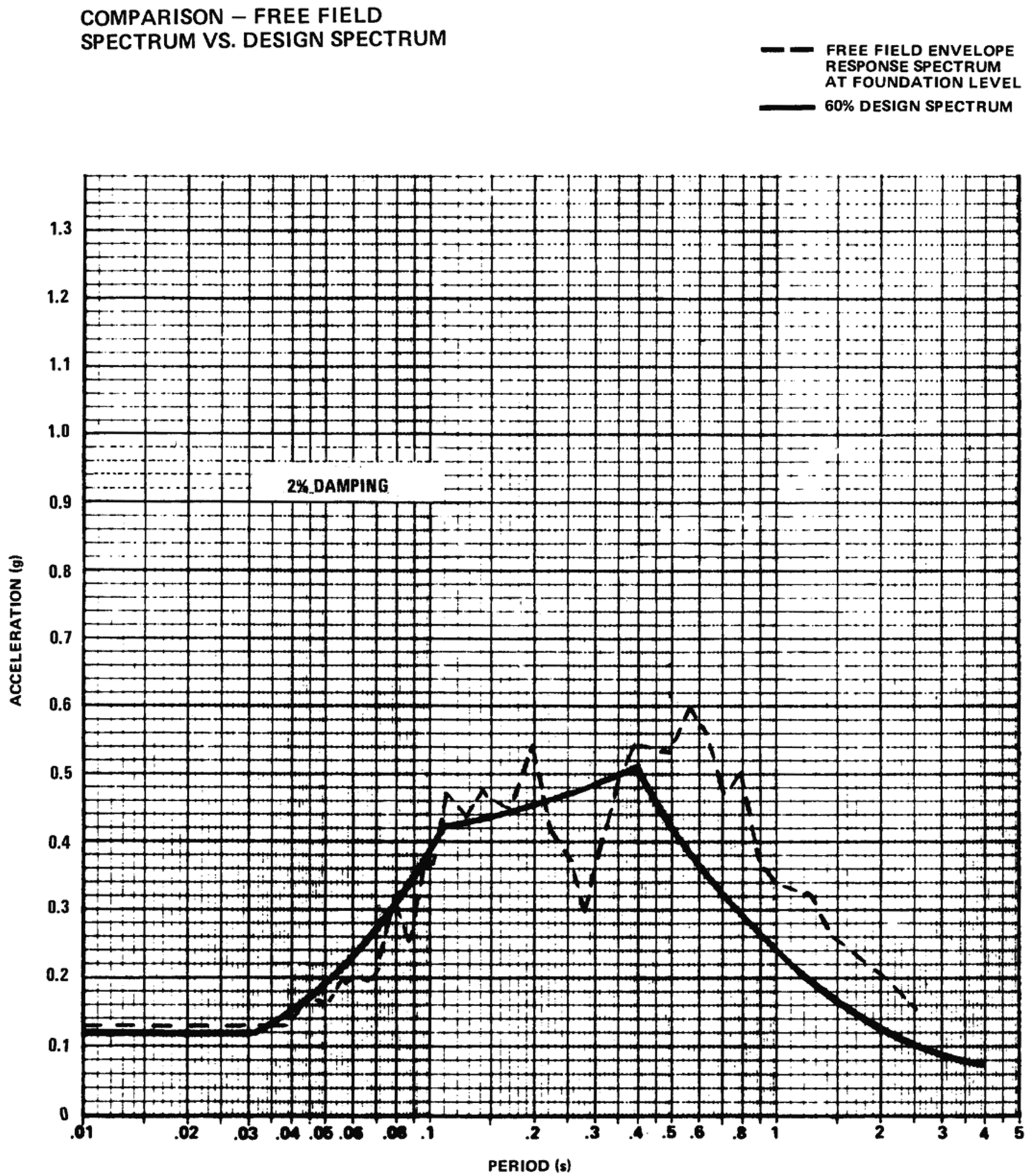
SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	CONTAINMENT BUILDING el 144 ft 0 in., SSE HORIZONTAL
		FIGURE 3.7.B.2–10



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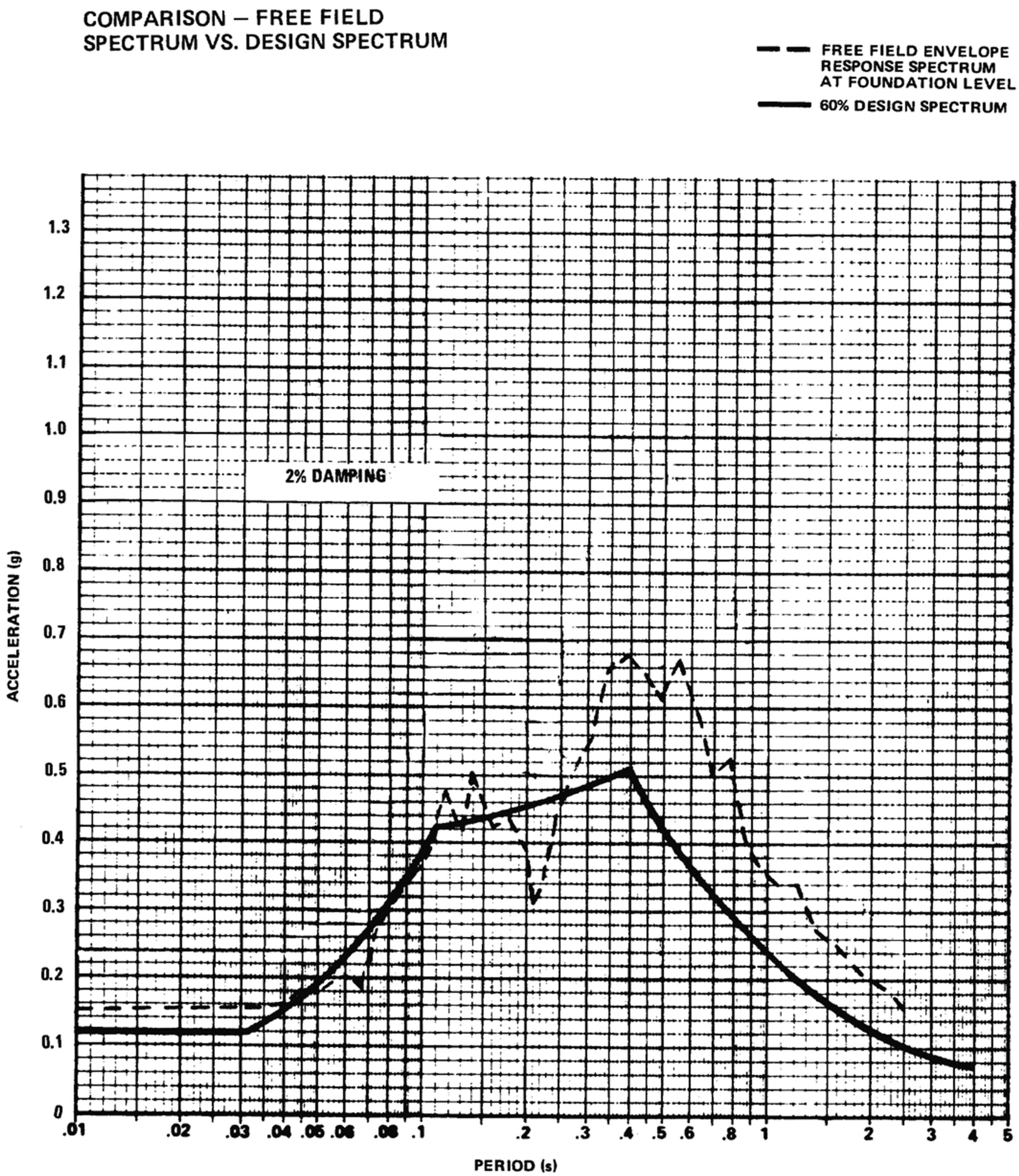
SOUTHERN COMPANY *Energy to Serve Your World®* VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	FUEL HANDLING BUILDING el 152 ft 0 in., SSE HORIZONTAL
	FIGURE 3.7.B.2–11



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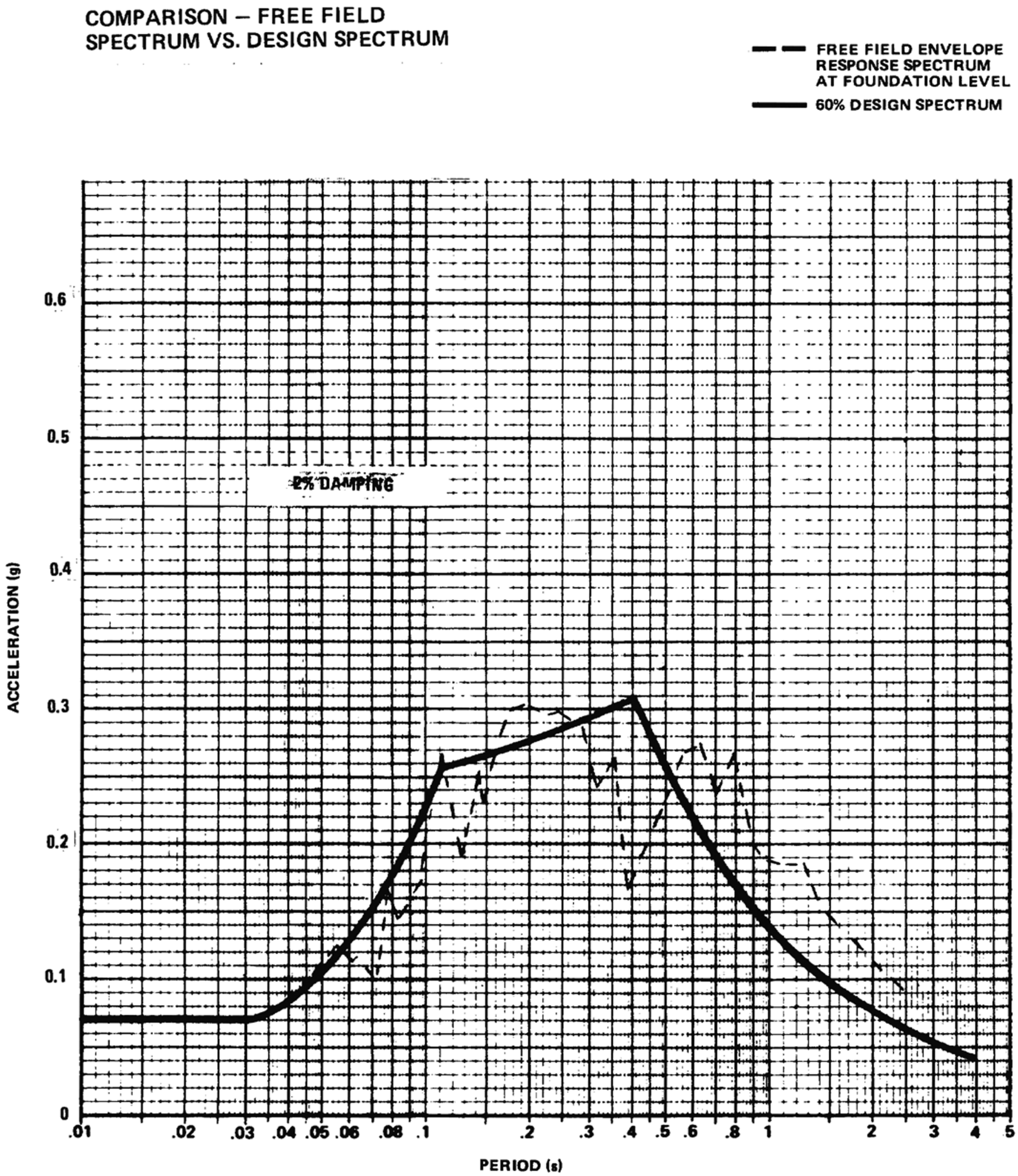
SOUTHERN COMPANY *Energy to Serve Your World®*	**VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2**	CONTROL BUILDING el 173 ft 0 in., SSE HORIZONTAL FIGURE 3.7.B.2–12



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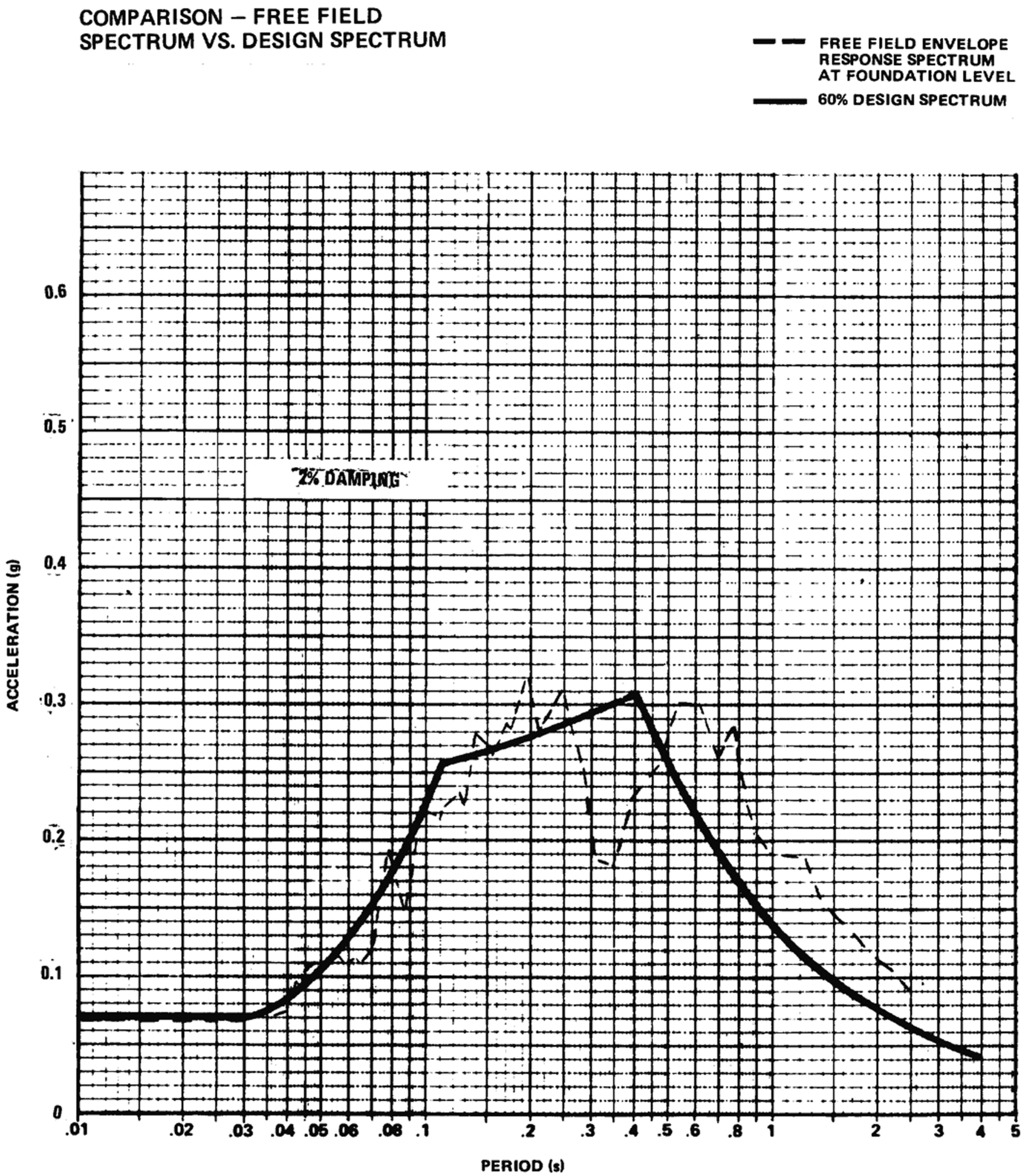
SOUTHERN COMPANY *Energy to Serve Your World*® VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	AUXILIARY BUILDING el 109 ft 0 in., OBE HORIZONTAL FIGURE 3.7.B.2–13



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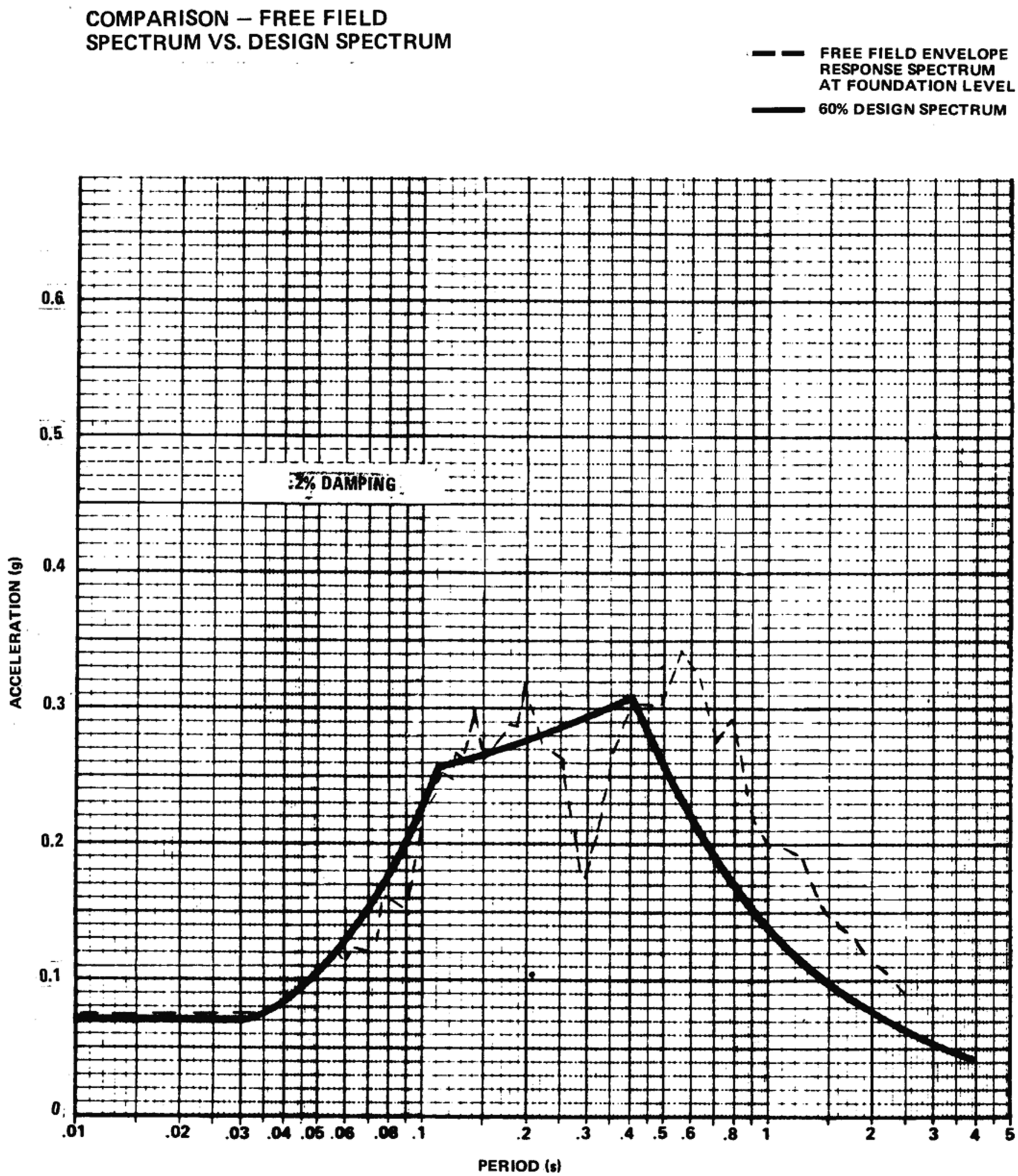
SOUTHERN COMPANY Energy to Serve Your World® VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	NUCLEAR SERVICE COOLING WATER TOWER el 128 ft 0 in., OBE HORIZONTAL
	FIGURE 3.7.B.2–14



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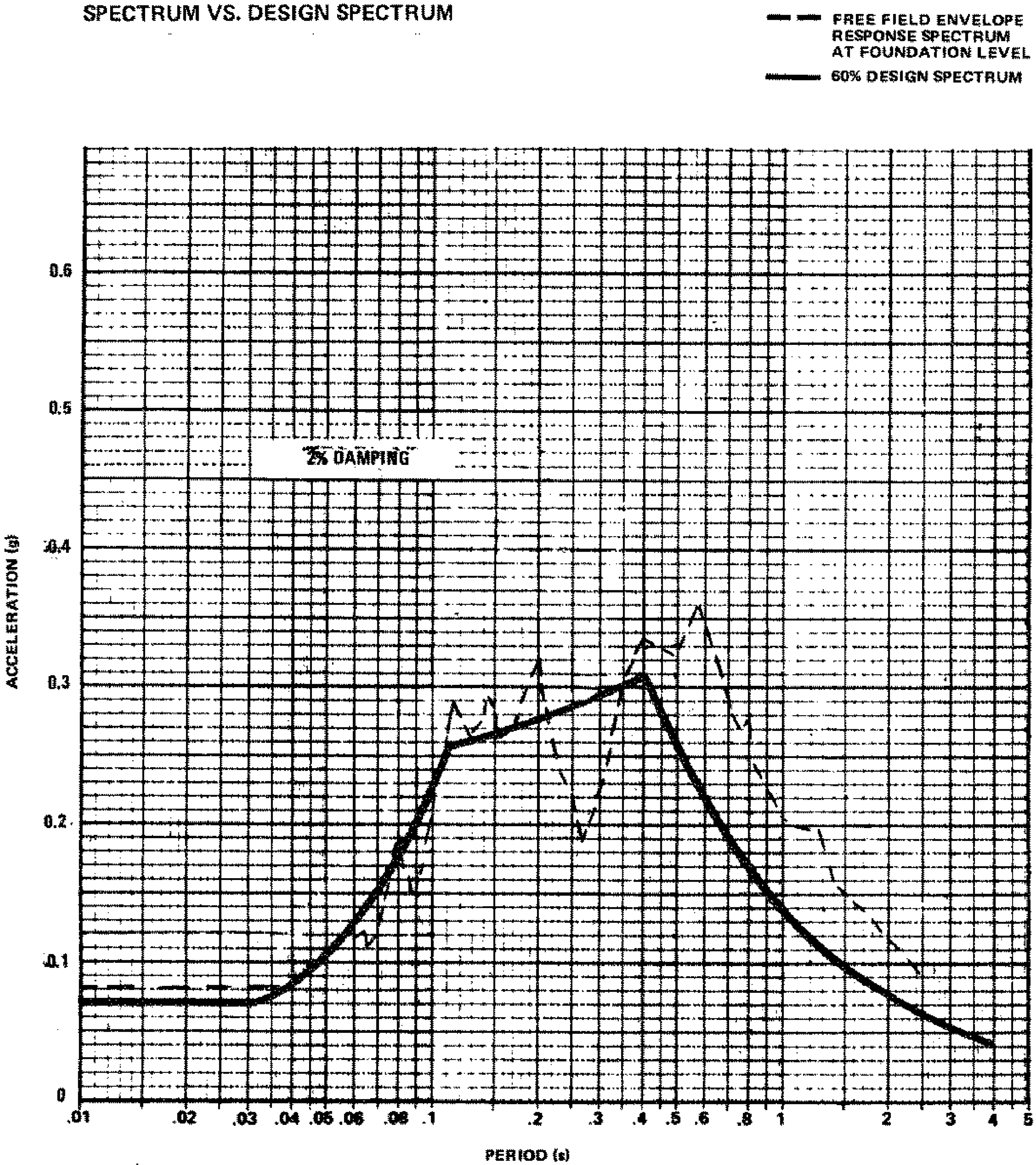
SOUTHERN COMPANY Energy to Serve Your World® — VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	CONTAINMENT BUILDING el 144 ft 0 in., OBE HORIZONTAL
	FIGURE 3.7.B.2–15

COMPARISON – FREE FIELD
SPECTRUM VS. DESIGN SPECTRUM



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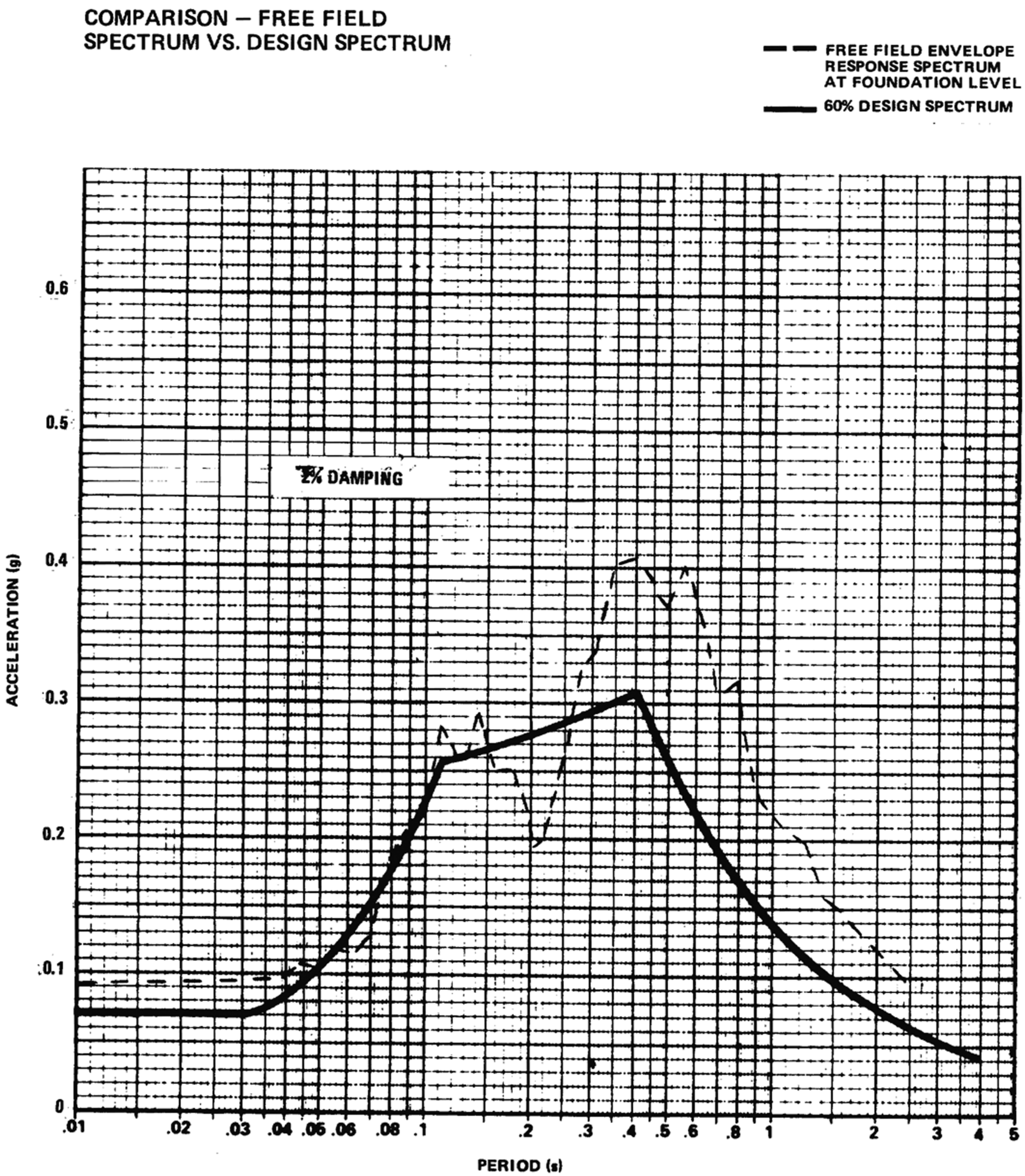
SOUTHERN COMPANY Energy to Serve Your World® VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	FUEL HANDLING BUILDING el 152 ft 0 in., OBE HORIZONTAL FIGURE 3.7.B.2–16



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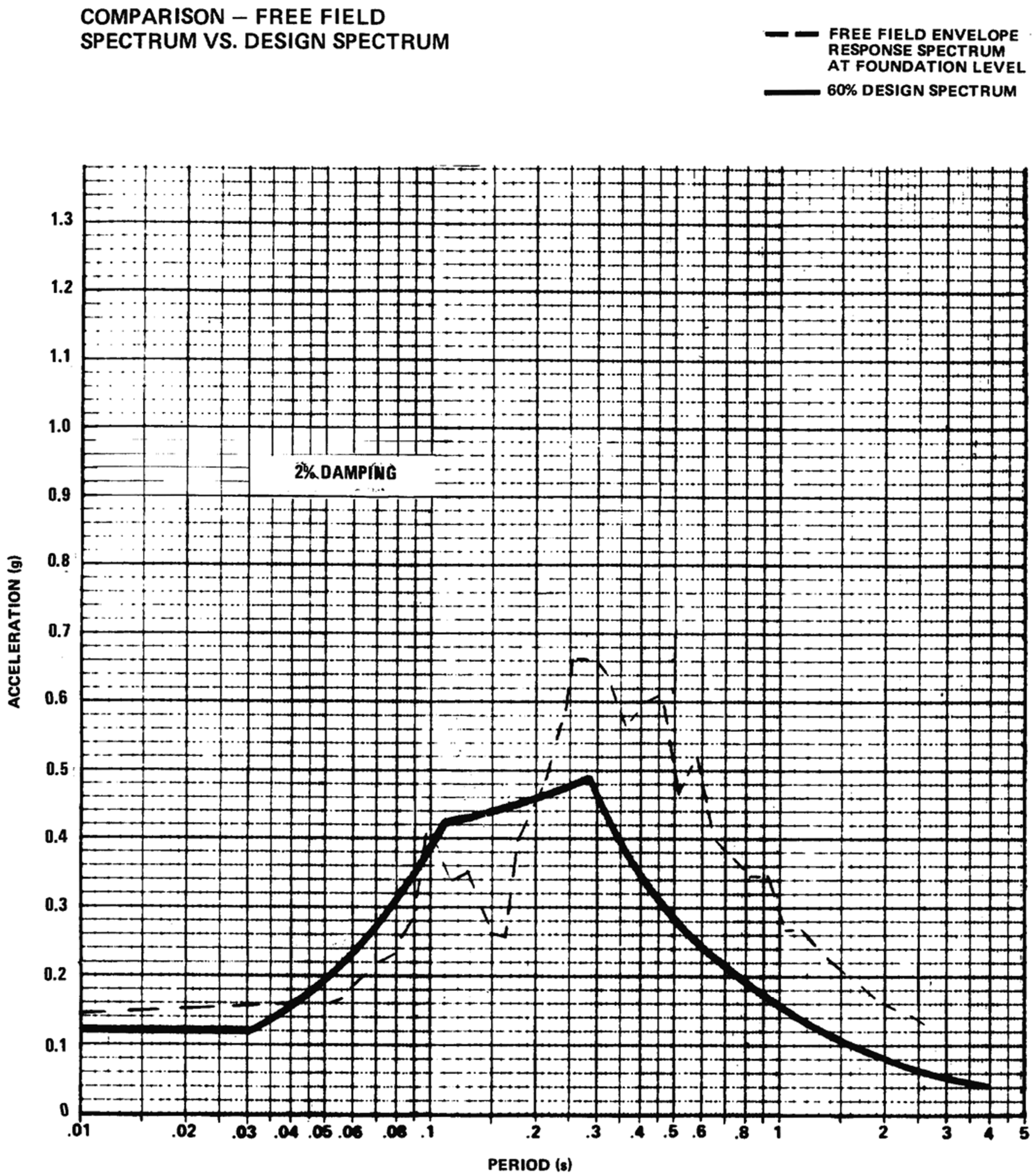
SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	CONTROL BUILDING el 173 ft 0 in., OBE HORIZONTAL FIGURE 3.7.B.2–17



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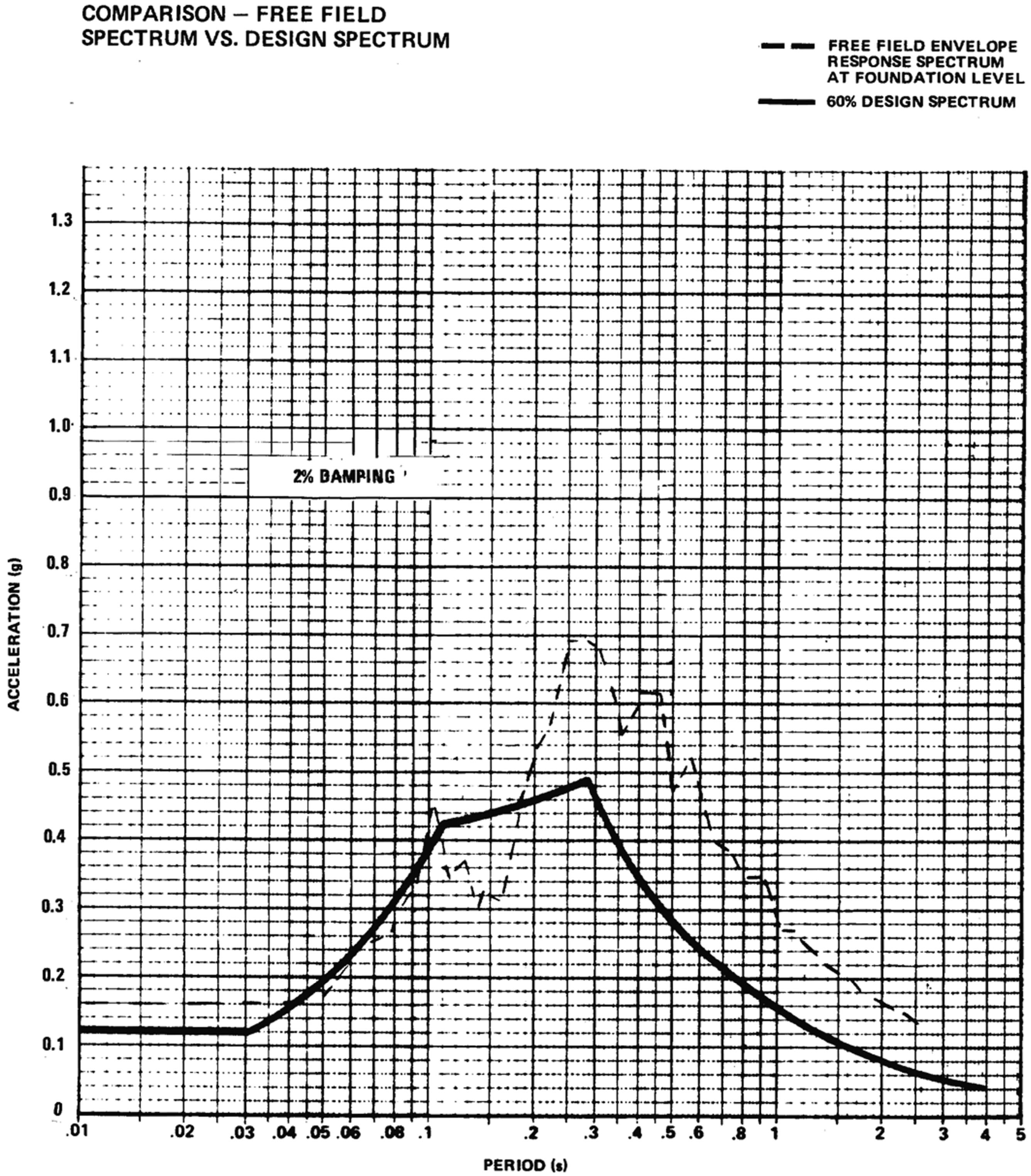
SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	AUXILIARY BUILDING el 109 ft 0 in., SSE VERTICAL FIGURE 3.7.B.2–18



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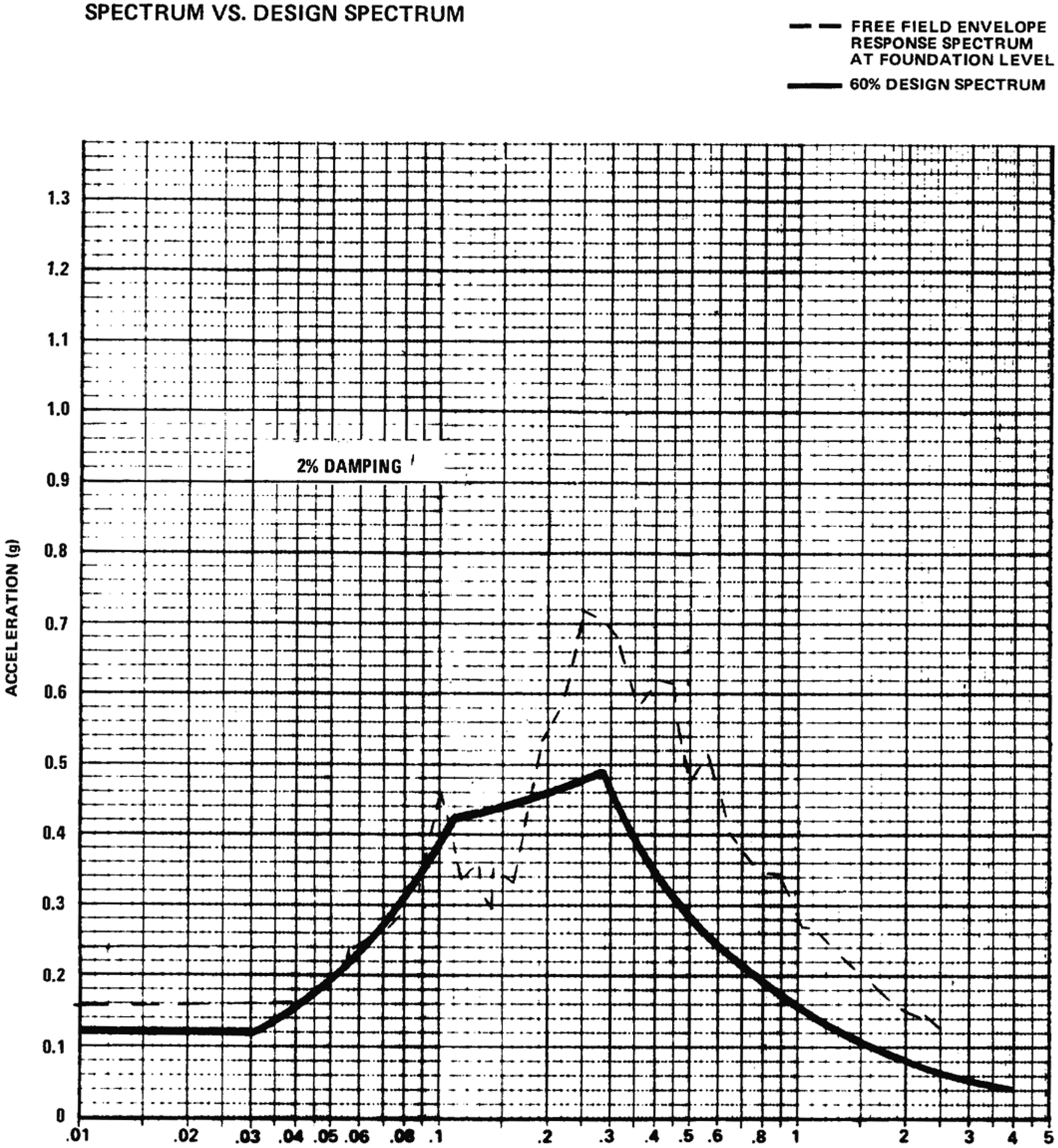
SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	NUCLEAR SERVICE COOLING WATER TOWER el 128 ft 0 in., SSE VERTICAL FIGURE 3.7.B.2–19

COMPARISON – FREE FIELD
SPECTRUM VS. DESIGN SPECTRUM



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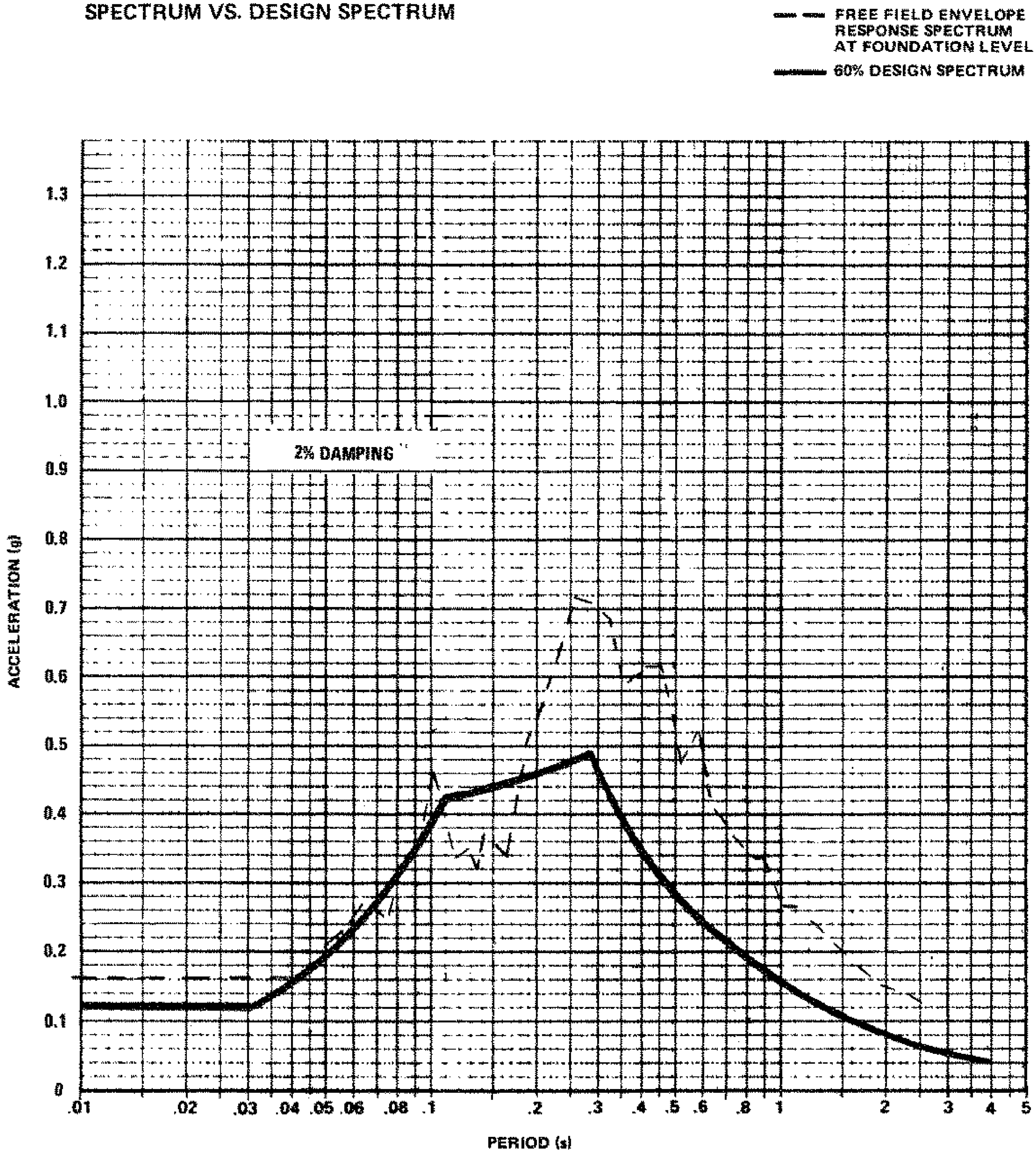
SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	CONTAINMENT BUILDING el 144 ft 0 in., SSE VERTICAL
		FIGURE 3.7.B.2–20

COMPARISON – FREE FIELD
SPECTRUM VS. DESIGN SPECTRUM



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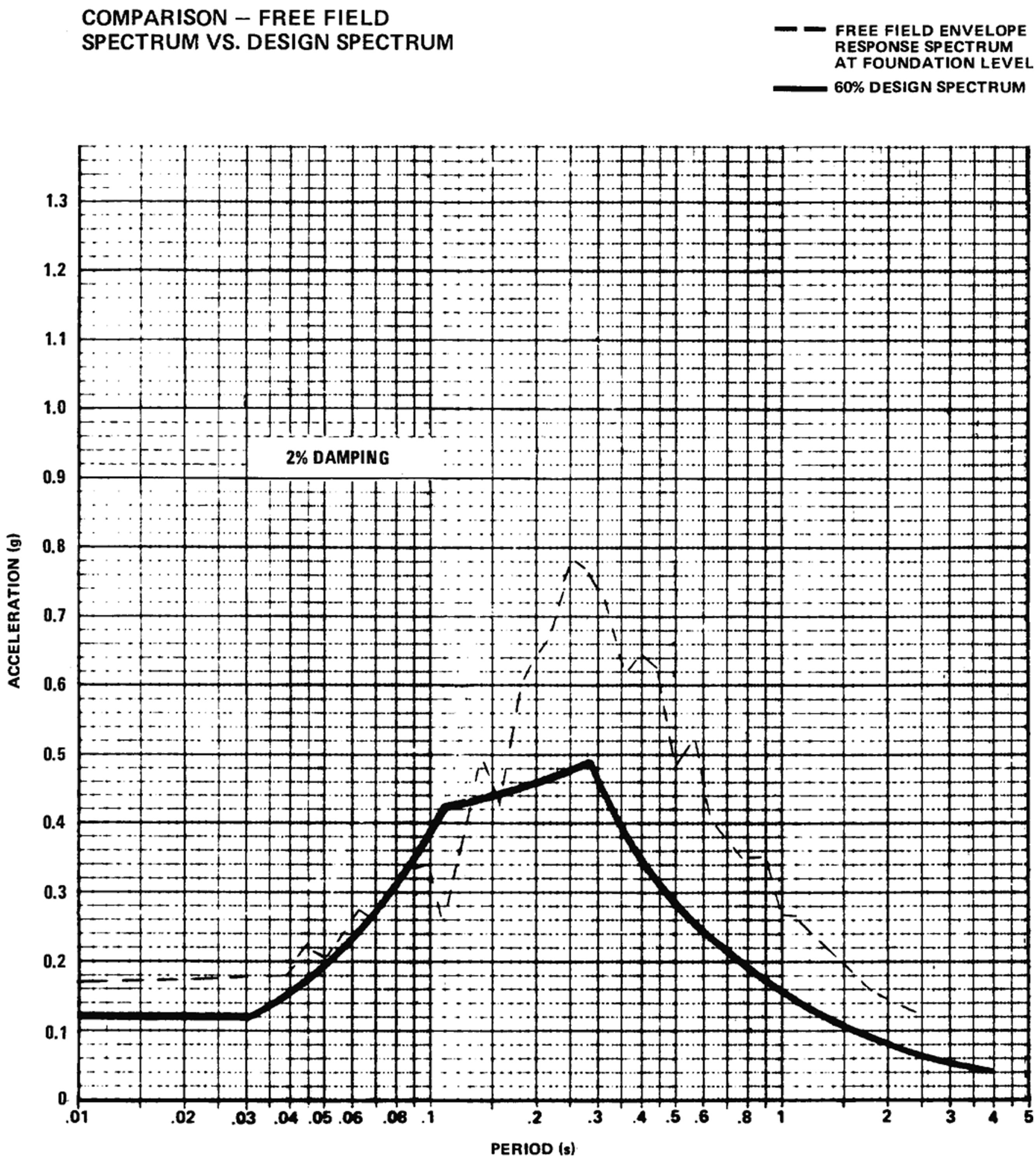
SOUTHERN COMPANY
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

FUEL HANDLING BUILDING
el 152 ft 0 in., SSE VERTICAL

FIGURE 3.7.B.2–21



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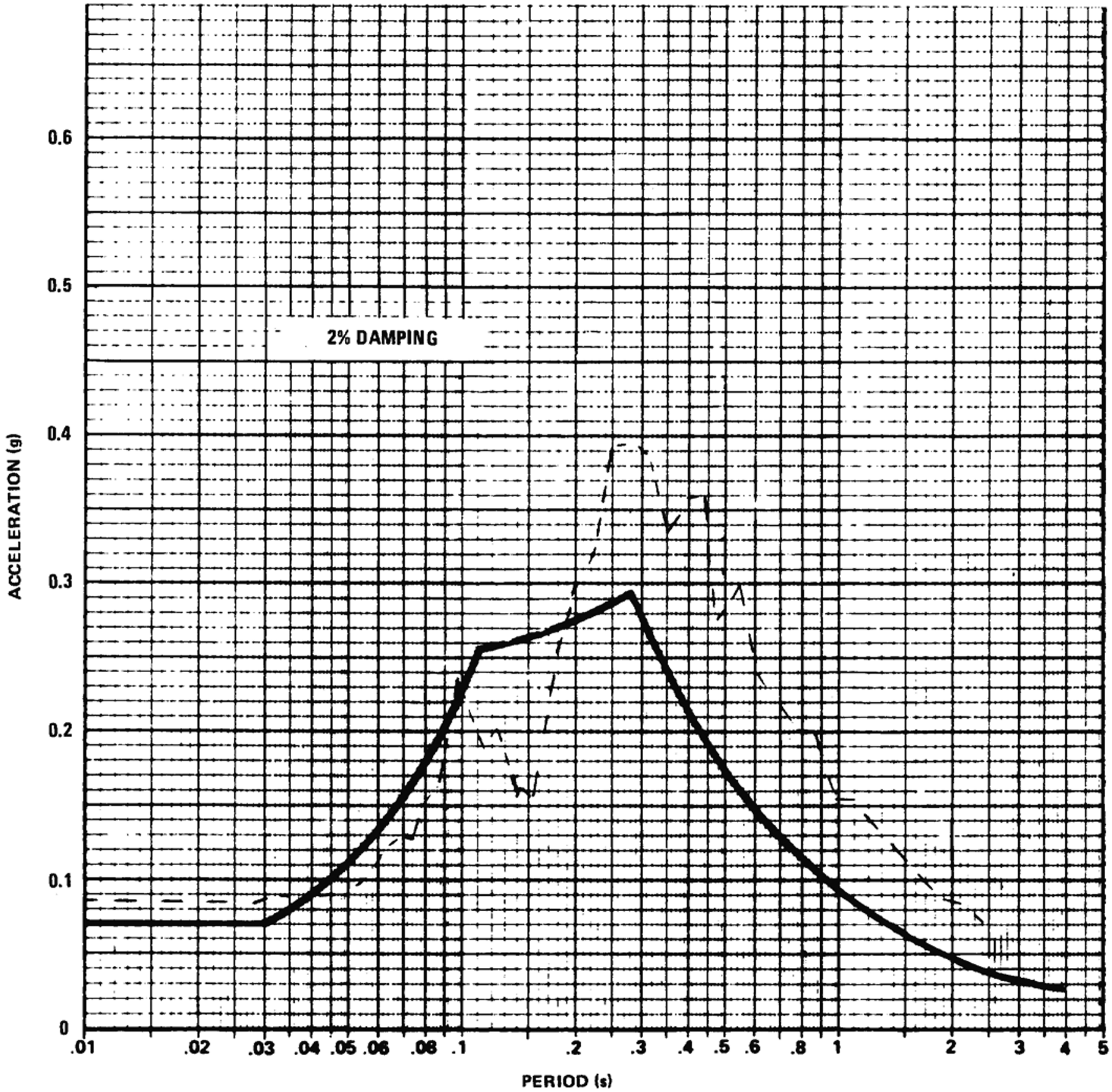
SOUTHERN COMPANY *Energy to Serve Your World®* VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	CONTROL BUILDING el 173 ft 0 in., SSE VERTICAL
	FIGURE 3.7.B.2–22

COMPARISON – FREE FIELD
SPECTRUM VS. DESIGN SPECTRUM

FREE FIELD ENVELOPE RESPONSE SPECTRUM AT FOUNDATION LEVEL
60% DESIGN SPECTRUM



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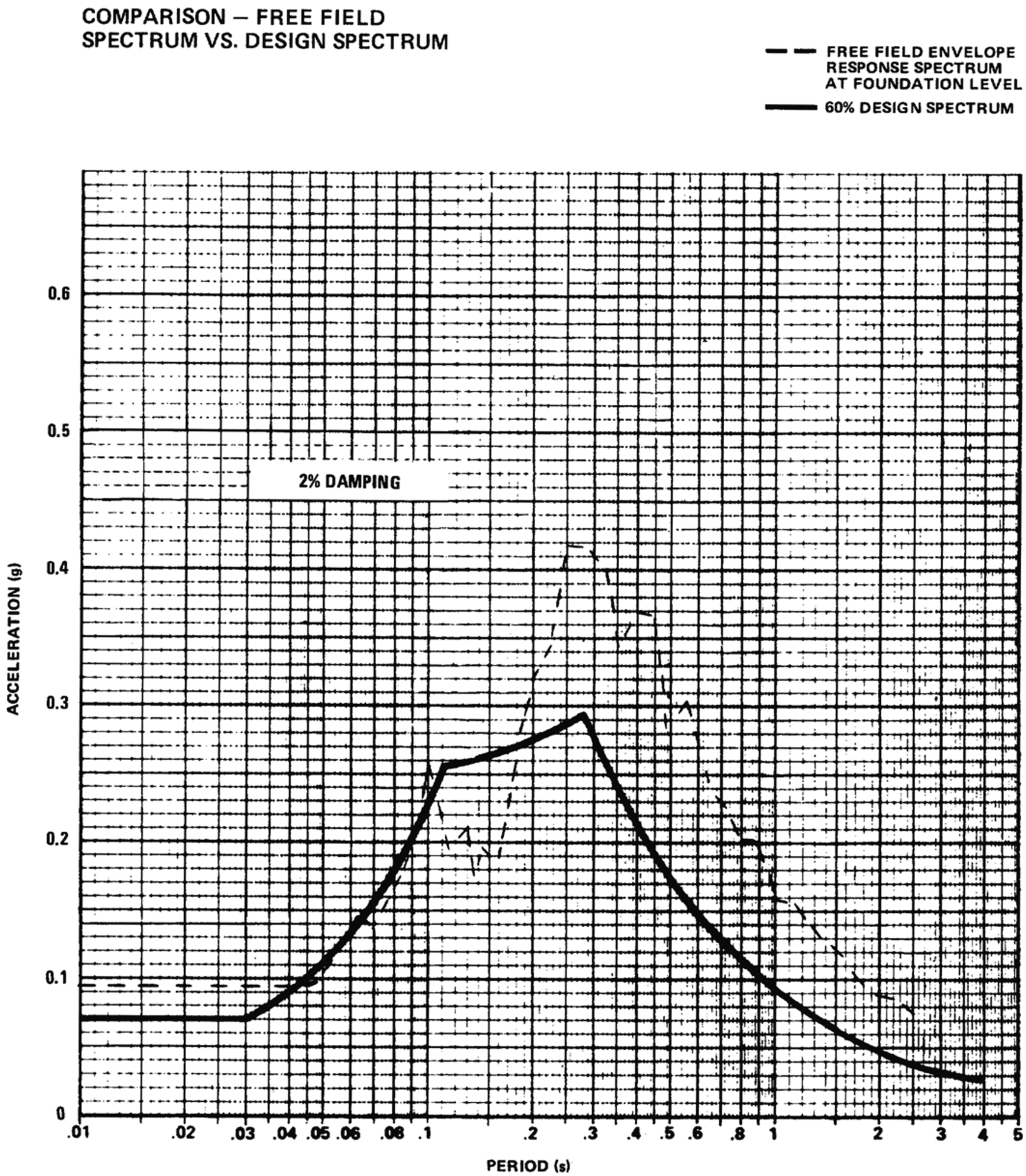
SOUTHERN COMPANY
Energy to Serve Your World®

VOGTLE
ELECTRIC GENERATING PLANT
UNIT 1 AND UNIT 2

AUXILIARY BUILDING
el 109 ft 0 in., OBE VERTICAL

FIGURE 3.7.B.2–23



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SOUTHERN COMPANY *Energy to Serve Your World®*	VOGTLE ELECTRIC GENERATING PLANT UNIT 1 AND UNIT 2	NUCLEAR SERVICE COOLING WATER TOWER el 128 ft 0 in., OBE VERTICAL FIGURE 3.7.B.2–24