GULF STATES UTILITIES COMPANY
POST OFFICE BOX 2951 · BEAUMONT, TEXAS 77704
AREA CODE 713 838-6631

April 10, 1984
RBG- 17,552
File No. G9.5,
G9.8.6.2

Mr. Harold R. Denton, Director
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Dear Mr. Denton:

River Bend Station - Unit 1
Docket No. 50-458

Enclosed are the results of Gulf States Utilities Company (GSU) studies regarding the effect on plant structures and slopes of ponding in the Unit 2 excavation. These studies respond to open items identified in the Draft Safety Evaluation Report (DSER) by the Environmental and Hydrological Engineering Branch (EHEB) and the Structural and Geotechnical Engineering Branch (SGEB). The analyses indicate, as described in the revised Final Safety Analysis Report (FSAR) text, ponding in the excavation area will not affect the structural integrity of adjacent plant structures nor will maximum ground water levels affect any other River Bend Station (RBS) structures if a berm is constructed around the excavation area. Hence, GSU will place a berm around the Unit 2 excavation to prevent runoff flow into the area and maintain plant structural integrity. Enclosure 1 addresses the methodology, analysis, and results of the ponding study, including the addition of a berm. Enclosure 2 addresses the stability of plant slopes. The revisions indicated in this transmittal will be incorporated in a future FSAR amendment.

Sincerely,

J. E. Booker

J. E. Booker
Manager-Engineering
Nuclear Fuels & Licensing
River Bend Nuclear Group

JWL
JEB/WJR/JWL/je

Enclosure

B001
1/1

ENCLOSURE 1

11
2.4.2.3 Effects of Local Intense Precipitation

INSERT 1

An analysis of plant drainage was performed to determine whether safety-related equipment could be flooded during an occurrence of the probable maximum precipitation (PMP). The following discussion pertains to flooding in the immediate
11 plant area. Flooding of local streams, in combination with severe seismic events, is discussed in Section 2.4.3.

Safety-related equipment at the River Bend Station is located in buildings protected from floodwater entry or situated at a minimum elevation of 98 ft msl. The design basis flood levels of buildings housing safety-related equipment are presented in Table 3.4-1. Finish grade at the edge of plant buildings is about 95 ft msl. The elevation of the road surrounding the building varies from 94 to 100 ft msl. Grassed areas between buildings and roads are at 93 to 94 ft msl. Railroad spurs in the plant area have a top-of-rail elevation of 95 ft msl.

INSERT 2

Fig. 2.4-6 shows the immediate plant area and drainage
5 patterns. The area that could produce runoff accumulation near plant buildings is outlined. Normal plant area drainage is effected by directing runoff into the storm drain system, drainage ditches, and culverts. All local runoff is conveyed to West Creek or East Creek. There are no onsite areas which could produce ponding of runoff to an elevation greater than 96 ft msl.

INSERT 3

During an extreme meteorological event such as the PMP, a portion of storm runoff in the eastern area of the site would drain to the storm drain system. For a rainfall intensity greater than the storm drain capacity (5.5 in/hr), water would pond to 94 ft msl, then overflow a 1,000-ft long southeast portion of the ring road to the low area west of the cooling towers and east of the plant. Water ponded in this area would overtop a 1,000-ft long section of the cooling tower access road between 93 and 94 ft msl and flow directly to East Creek. It is unlikely that runoff from the eastern area of the site would top the railroad track and flow to the Unit 2 excavation.

INSERT 4

INSERT 5

A portion of the PMP runoff in the construction parking area, immediately north of the plant, would drain to West Creek via culvert or overland flow. The remainder of the area would drain to a paved ditch along the east, south, and west embankments. A single 24-in-diameter culvert currently provides conveyance of runoff from this ditch to West Creek (see Figure 2.4-6). The culvert is located beneath the intersection of the plant ring road and the West Plant

INSERT 6

Insert 1 for Page 2.4-12

2.4.2.3.1 Effects on Safety-Related Equipment

Insert 2 for Page 2.4-12

plant buildings

Insert 3 for Page 2.4-12

Overland runoff can enter West Creek by overtopping a railroad spur at 95 feet msl; it can enter East Creek by overtopping the Cooling Tower Access Road at 93.3 feet msl.

Insert 4 for Page 2.4-12

enter the storm drain system and flow to East Creek.

Insert 5 for Page 2.4-12

northwest corner of the

Insert 6 for Page 2.4-12

parking

Access Road, at the northwest corner of the immediate plant area.

INSERT 1

PMP runoff in the area of the Unit 2 excavation would be partially intercepted by the storm drain system (flowing to West Creek). and partially drain to the excavation or enter the excavation as direct rainfall.

INSERT 2

In the event the Unit 2 excavation is filled to grade (approximately 94 ft msl), storm runoff could pond to a maximum elevation of approximately 95 ft msl in the Unit 2 area before it would overflow the surrounding railroad track and enter West or East Creeks. There are no postulated flood conditions that would inhibit these runoff paths. 11

INSERT 3

As discussed in Section 2.4.3, there is no flooding condition on West Creek which produces a water elevation above the top of the Fabriform channel, and water does not enter the plant area from any postulated condition of stream flooding.

Insert 1 for Page 2.4-12a

If this culvert became clogged, runoff would overflow the ditch and move west to West Creek and south to the excavation and Unit 1.

Insert 2 for Page 2.4-12a

back

Insert 3 for Page 2.4-12a

As discussed in Section 2.4.3,

THIS PAGE INTENTIONALLY BLANK

11

Precipitation that falls on the roofs of onsite buildings is collected in gutters along the roof edge and discharged via downspouts to the plant yard adjacent to buildings. Overflow from the roof gutters spills directly onto the plant yard. All building roofs are sloped, and no potential exists for significant ponding of rainfall on the roofs. No parapet walls exist on plant buildings which would encourage rainfall ponding. Safety-related equipment is not jeopardized by roof drainage during even the most severe postulated rainfall event. 11

It is clear from a review of plant drainage that runoff from the PMP could not pond above 98 ft msl and jeopardize plant safety-related equipment. Ponding above 95 ft msl would produce flow into the Unit 2 excavation or into West or East INSERT 1 Creeks. The analysis of site drainage will now focus on ponding in the Unit 2 excavation, and the potential INSERT 2 exceedence of the design groundwater level of 70 ft msl (see Section 2.4.13.5). 11

DELETE AND REPLACE WITH INSERT 3

An analysis of storm runoff ponding in the Unit 2 excavation, and the potential impacts on the design basis groundwater level, is currently in progress.

2.4.3 Probable Maximum Flood (PMF) on Rivers and Streams

2.4.3.1 Probable Maximum Precipitation (PMP)

The PMF analysis for the Mississippi River did not involve a PMP determination (Section 2.4.3.4). The following discussion pertains to precipitation in local drainage

Insert 1 for Page 2.4-13

by overtopping roads or railroad spurs.

Insert 2 for Page 2.4-13

basis

Insert 3 for Page 2.4-13

2.4.2.3.2 Effects on Design Basis Groundwater Level

Plant buildings are designed to withstand the pressure and buoyancy effects of a 70 ft msl groundwater level. This is about 13 ft above normal groundwater. The radwaste building and tunnels adjacent to the excavation have also been shown to be able to withstand the pressure and buoyancy effects from a ponding level in the excavation of 80 ft msl (see Section 2.5). The following discussion presents an estimate of ponding in the Unit 2 excavation, and indicates that the design basis levels for excavation ponding and groundwater will not be exceeded during the most severe postulated meteorology.

The Unit 2 excavation, shown on Fig 2.4-6, has an average bottom elevation of 66 ft msl, and the bottom area is about 500,000 sq ft. The area at grade 94 ft msl is about 700,000 sq ft. A berm around the Unit 2 excavation prevents surface runoff in the plant area from entering the excavation. Surface runoff exits the plant area through the storm drainage system, by overtopping the Cooling Tower Access Road at 93.3 ft msl and flowing to East Creek, or by overtopping a railroad spur at 95 ft msl and flowing to West Creek. It was assumed that the berm will be set back from the excavation, and an 800,000 sq ft area was used to compute the volume of direct rainfall entering the excavation during a storm.

For the analysis of excavation ponding, it was assumed that a 72 hr PMP storm is preceded by a 72 hr 1/2 PMP storm. Per Regulatory Guide 1.59, Rev. 2, it was assumed that 72 hours separated the two storms. Additionally, it was conservatively assumed that 2 ft of ponding (to 68 ft msl) exists in the excavation prior to the 1/2 PMP antecedent storm. Two analyses of ponding were performed: Case 1 assumed no seepage from the excavation during the 9 day storm period, which allowed for a computation of the maximum ponding level in the excavation; Case 2 assumed seepage from the excavation would occur, which would allow for a computation of the maximum groundwater level under plant buildings.

Table 2.4-36 presents the PMP values for the 72 hr storm, which were taken from Ref. 11 and 63, and arranged in a conservative sequence as discussed in Section 2.4.3. Assuming no seepage from the excavation, the maximum volume of water collected in the excavation would be about 6,584,000 cu ft, including PMP, 1/2 PMP, and 2 ft of antecedent ponding. This corresponds to a maximum water level of 78.1 ft msl, about 2 ft below the design basis level for the radwaste building and tunnels adjoining the excavation. Since no seepage was assumed for this case, the normal groundwater levels would remain unchanged and would not impact plant buildings.

For the case of seepage from the excavation, Ref. 68 provided the methodology for analysis. This methodology was developed to estimate the rise of piezometric levels in an aquifer, and the rate of seepage, caused by a change of water level in a canal. The approach applies to the excavation, where the ponding level undergoes change due to incident rainfall and seepage. The volume of seepage per foot of canal length, or in our case per foot of excavation perimeter, is expressed as

$$V = \frac{K(h_1^2 - h_1^2)(t_2^{\frac{1}{2}} - t_1^{\frac{1}{2}})}{(\pi v)^{\frac{1}{2}}}$$

where V = seepage volume, cu ft per foot of excavation perimeter (3000 ft)
K = horizontal permeability, cfs per foot
t_1, t_2 = time period of interest, sec
h_1^2 = vertical distance from bottom of aquifer to ponded level in excavation, ft
h_2 = vertical distance from bottom of aquifer to normal groundwater level, ft

And $$v = \frac{K(h_3)}{S}$$

where S = specific yield of aquifer
h_3 = weighted mean depth of saturation, ft

Normal groundwater is at 57 ft msl, and the bottom of the Terrace Aquifer is at about -40 ft msl, giving a value for h_2 of 97 ft. Horizontal permeability values were drawn from Section 2.5. The values for the Terrace Sands range from 1700 to 2700 gpd per sq ft, or an average value of 2200. For the backfill in the Unit 1 area and beneath the excavation, the values range from 550 to 870 gpd per ft, or an average value of 710. To determine a weighted average value of horizontal permeability, the backfill value was applied over the depth of backfill, from 66 ft msl to 20 ft msl,

while the Terrace Sands value was applied over the 20 ft msl to -40 ft msl range. The weighted average value is 1553 gpd per sq ft, or 0.0024 cfs per sq ft.

It is acknowledged that seepage from the north, south, and west excavation slopes would be through the Terrace Sands almost exclusively, and it would be appropriate to use the higher permeability value associated with the Terrace Sands rather than a weighted average to calculate seepage through these slopes. However, the area of concern is the east slope, where seepage would act directly to increase groundwater levels beneath plant buildings. The excavation and Unit 1 area consist of backfill down to 20 ft msl. By applying a weighted average horizontal permeability value for all excavation slopes based on the soil conditions for the east excavation slope, the calculation of seepage is simplified, and a conservative estimate of ponding levels in the excavation and seepage profiles beneath plant buildings is obtained.

Specific yield of the aquifer, which is essentially equal to effective porosity, was determined by a method similar to that for horizontal permeability. Effective porosity varies from 0.20 to 0.28 in the Terrace Sands and from 0.29 to 0.31 in the backfill. By computing average values for each soil type, and deriving a weighted average over the soil column from 66 ft msl to -40 ft msl, a value of 0.27 was obtained.

The weighted mean depth of saturation in the area surrounding the excavation was approximated by assuming the seepage wedge is triangular in cross section, with the seepage profile decreasing from the ponded level in the excavation to the normal groundwater level at some distance into the embankment. The mean height of the seepage wedge is thus one-third of the distance between normal groundwater level and ponding level, and the mean depth of saturation is the depth of the aquifer (97 ft) plus the mean height of the seepage wedge. It is understood that the seepage profile is somewhat concave in shape, but the triangular wedge method provides a good approximation to the mean depth of saturation. Also, the seepage method is not particularly sensitive to this variable.

The seepage methodology from Ref. 68 was developed for the case of an instantaneous rise in water level in a canal, followed by an indefinite maintenance of that level. This has been modified to apply to the case of fluctuating excavation ponding level through the use of a time step approach, where rainfall inflow, seepage outflow, and ponding storage in the excavation are evaluated for an average ponding depth during a specific time period. The procedure is as follows: the inflow to the excavation from incident rainfall is calculated; an average ponding depth is assumed for the storm time period; the storage in the excavation is calculated; the seepage volume from the excavation is calculated; a comparison is made between inflow,

and storage plus seepage. If this comparison is close, than the proper value of average ponding depth has been assumed. If not, a new ponding depth is assumed and the procedure is repeated.

Storage in the excavation is computed by determining the average ponded area over the depth of ponding in the excavation, and multiplying by the depth of ponding. The bottom excavation is about 500,000 sq ft, the top area about 700,000 sq ft, and a linear increase in surface area was assumed over the 28 ft depth.

The seepage profile in the east embankment of the excavation (under plant buildings) is determined from the Ref. 68 formula

$$h_4^2 = h_2^2 + (h_1^2 - h_2^2)\mathrm{erfc}(x/2(vt)^{\frac{1}{2}})$$

where h_4 = vertical distance from bottom of aquifer to seepage profile at distance x from the excavation, ft

x = horizontal distance into the excavation embankment, ft.

erfc = complementary error function. These values are provided in Ref. 68.

t = time from beginning of seepage, sec

h_1, h_2, v = as previously defined

Table 2.4-37 shows the fluctuation in ponding level during the storm period. It is noted that the ponded level would drop below the bottom of the excavation between the 1/2 PMP and PMP storms. Therefore, a ponded level of 68 ft msl was again assumed prior to the beginning of the PMP storm, for conservatism. Table 2.4-38 provides the seepage profiles for peak periods of ponding. Approximately 100 ft east of the excavation, the maximum groundwater level would be about 68.3 ft msl. This is less than the design basis groundwater level of 70 ft msl in the plant area. At further distances from the excavation, the groundwater level would decline to 57 ft msl, normal groundwater.

PMP Summary

It has been demonstrated that with a berm around the Unit 2 excavation, the maximum ponding level in the excavation and the maximum groundwater level beneath plant buildings would be below the design basis level.

Combined Events in Plant Area

It has been determined (see Section 2.5.5.2) that an occurrence of the OBE or SSE in the plant area would cause only minor

sloughing of excavation slopes. The maximum level of ponding in the excavation due to a 1/2 PMP event associated with an OBE would be about 69.6 ft msl, assuming no seepage from the excavation during the 72 hr event. Based on the analysis for the PMP, groundwater beneath plant buildings would not exceed 70 ft msl in the event of seepage.

The 24 hr - 25 year rainfall associated with an SSE would be 9.1 in. Assuming no seepage from the excavation, the maximum ponding level would be about 67.2 ft msl. In the event of seepage, the groundwater level would not be significantly affected.

60. Robertson, J.B. A Method to Describe the Flow of Radioactive Ions in Groundwater. Scandia Labs, Report SCCR-70-6139, December 1970.

61. Pinder, G.F. Groundwater Contamination, Part A: Mass
1 Transport. BSCES-ASCE Geotechnical Lecture Series for 1981, Groundwater Hydrology.

5 62. Henderson, F.M. Open Channel Flow, MacMillan Co., New York, 1966.

63. U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration. Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian. Hydrometeorological Report No. 52, Washington, DC, 1982.

64. U.S. Dept. of Agriculture, Soil Conservation Service. Urban Hydrology for Small Watersheds, Technical Release
11 Number 55, Washington, DC, 1975.

65. Chow, V.T. Hydrologic Determination of Waterway Areas for the Design of Drainage Structures in Small Drainage Basins. University of Illinois Bulletin, Vol. 59, Urbana, IL, March 1962.

66. U.S. Dept. of the Interior, Geological Survey, and Louisiana Dept. of Transportation. Floods in Louisiana, Magnitude and Frequency. Third Edition, Baton Rouge, 1976.

67. U.S. Army Corps of Engineers. HEC-2 Water Surface Profiles, Users Manual. Hydrologic Engineering Center, Davis, CA, August 1979.

68. Marino, M.A. and Luthin, J.N. Seepage and Groundwater, Elsevier Scientific Publishing Co., New York, New York, 1982.

TABLE 2.4 - 36

PMP AND ½PMP RAINFALL IN THE PLANT AREA

Time (hr)	Accumulated PMP (in)	Accumulated ½PMP (in)
6	3.1	1.55
12	9.8	4.9
13	11.7	5.85
14	13.9	6.95
15	17.7	8.85
16	37.1	18.55
17	39.8	19.9
18	41.8	20.9
24	47.1	23.35
48	51.8	25.9
72	55.7	27.85

TABLE 2.4 - 37

PONDING FLUCTUATION IN UNIT 2 EXCAVATION

Time (hr)	Incremental Rainfall (in)	Incremental Inflow to Excavation (1) (ft^3)	Water Level in Excavation (2) (ft msl)
0-6	1.55	103,333	68.0(3)
6-12	3.35	223,333	68.0(3)
12-13	0.95	63,333	68.0(3)
13-14	1.10	73,333	68.0(3)
14-15	1.90	126,667	68.1
15-16	9.70	646,666	69.2
16-17	1.35	90,000	69.2
17-18	1.00	66,667	69.2
18-24	2.65	176,667	68.7
24-48	2.35	156,667	66.9
48-72	1.95	130,000	(4)
72-144	0	0	(4)
0-6	3.1	206,666	68.0(2,3)
6-12	6.7	446,666	68.0(3)
12-13	1.9	126,666	68.1
13-14	2.2	146,666	68.2
14-15	3.8	253,334	68.3
15-16	19.4	1,293,332	70.9
16-17	2.7	180,000	71.1
17-18	2.0	133,334	71.2
18-24	5.3	353,334	71.0
24-48	4.7	313,334	69.0
48-72	3.9	260,000	Less than 69.0

NOTES:
(1) For an area within the berm of 800,000 sq. ft.
(2) An initial water level in the excavation of 68 ft. msl is conservatively assumed.
(3) Seepage at the initial ponding level of 68 ft. msl exceeds inflow to excavation; no change in ponding level conservatively assumed.
(4) Water level recedes below bottom of excavation at 66 ft msl.

TABLE 2.4 - 38
GROUNDWATER PROFILES IN PLANT AREA DURING PMP STORM

Time (hr)	Ponding Level in Excavation (ft msl)	Distance from East Slope of Excavation (ft)	Groundwater Level (ft msl)
16-17	71.1	50	69.5
		100	67.9
		300	62.4
17-18	71.2	50	69.6
		100	68.1
		300	62.6
18-24	71.0	50	69.7
		100	68.3
		300	63.5
24-48	69.0	50	68.2
		100	67.4
		300	64.0

INSERT 1 — INSERT 2

The most critical postulated flood condition, as identified in Section 2.4, results in standing water 7 in deep above the average yard grade of 94 ft - 6 in. The dynamic effect resulting from splashing 7 in-high flood waters is considered negligible. Therefore, the hydrodynamic loads due to floods are not considered in designing the Seismic Category I structures.

Insert 1 for Page 3.4-3

2.4.2.3

Insert 2 for Page 3.4-3

8

ENCLOSURE 2

QUESTION 241.3

Provide stability analyses for the slopes (referred to in Question 241.2) caused by the Unit 2 excavation adjacent to the Unit 1 structures.

RESPONSE

The response to this request is provided in revised Sections 2.5.4.11 and 2.5.5.

2.5.4.11 Design Criteria

INSERT 1
DELETE AND REPLACE WITH INSERT 2 AS A NEW PARAGRAPH
INSERT 3
INSERT 4
INSERT 5

The major plant buildings were analyzed to assess their sliding and overturning stability during the SSE and OBE. For this analysis, a structure is assumed to be driven by the seismic response of the structure and dynamic soil and water pressures. Resistance is assumed to be provided by base friction in the case of sliding and by the dead weight of the structure in the case of overturning. Since many of the structures will have a shake space adjacent to them (for seismic isolation from other structures), neither passive soil pressure nor soil-wall friction are relied upon for resistance in this stability analysis. The compacted sand backfill was modeled with a friction angle of 36 deg and no cohesion. Test results on the backfill indicate this friction angle to be conservative (refer to Fig. 2.5-74 and to Report on Engineering Characteristics of Granular Fill[77]). The base friction angle for concrete poured on compacted fill was taken as 90 percent of the soil friction angle. This is based on the laboratory test results of Potyondy[84]. For the sliding analysis, the base shear resistance is based on the effective stress during the seismic event.

The seismic responses of the structures are the results of the dynamic analyses described in Section 3.7.2. The seismic structural analyses were made for the SSE and OBE cases for soil shear moduli of 12, 18, and 24 ksi. The dynamic analyses provide the axial forces, shear forces, moments, and the three components of acceleration at the foundation level. From these data, the forces and moments acting at the base of the foundations were computed. The critical sliding or overturning situation for a given structure is then based on the least favorable direction of the earthquake in combination with the least favorable soil shear modulus.

For the stability analysis, the soil- and water-driving pressures were computed as shown on Fig. 2.5-79. Note that the increased K_0 due to compaction was included. Dynamic water pressures were evaluated according to Westergaard[85]. The dynamic soil pressures are calculated as suggested by Seed and Whitman[86].

3 The static load per foot of wall expression (Fig. 2.5-79) is derived by computing the area of the leftmost pressure distribution:

Insert 1 for Page 2.5-124

The analysis included the effects of the Unit 2 excavation and ponded water levels that result from the accumulation of runoff in the Unit 2 excavation as discussed in Section 2.4. Although the groundwater level would be slightly affected by ponding, the stability analyses conservatively consider the ground water level equal to the ponded water level to simplify the analyses.

Insert 2 for Page 2.5-124

For the sliding and overturning analyses

Insert 3 for Page 2.5-124

and wall friction where appropriate

Insert 4 for Page 2.5-124

is not

Insert 5 for Page 2.5-124

The friction angle for backfill against formed concrete is taken as 50 percent of the soil friction angle.

structural response and the seismic soil and water pressures.

Section 3.8.5 specifies that, for sliding and overturning, the minimum required factors of safety are 1.1 for SSE and 1.5 for OBE. The results of the sliding and overturning analysis are presented in Table 2.5-16, which is a listing of the calculated factors of safety. Note that even with the conservative loading conditions and soil properties used in the analysis, all factors of safety for overturning are
1.9 above ~~2.6~~ and all those for sliding are above ~~1.3~~. All 2 1.5
major structures have adequate sliding and overturning stability for OBE and SSE loading.

The stability of the major structures against flotation was evaluated by comparing maximum buoyant pressure during PMF with total average distributed dead load for a given structure. Table 2.5-17 lists both of these quantities and the ratio of the two. The lowest factor of safety against
flotation is ~~on the order of 5~~, well above the minimum
2.6 acceptable of 1.1 which is set forth in Section 3.8.5. Hence, flotation is not a realistic possibility for the plant structures, even under flood conditions.

2.5.4.12 Techniques to Improve Subsurface Conditions

The only techniques used to improve subsurface conditions were the excavation and backfill beneath all Seismic Category I structures (Section 2.5.4.5). In addition, the surface of the excavation was thoroughly compacted with the same vibratory equipment planned for the fill before any backfill was placed.

2.5.4.13 Subsurface Instrumentation

The instrumentation program is intended to measure the magnitude and distribution of vertical soil movements caused by unloading of the foundation soils during excavation and by settlement or reconsolidation of these soils during and subsequent to placement of the structural backfill and foundation loads. The locations of instruments have been chosen to measure both the vertical and horizontal distribution of soil movements, permitting construction of profiles of vertical movements.

The information obtained from this program is used to assess the changes in the subsoils caused by excavation and backfilling, the effects of these changes on the structural foundations, and the long term time-dependent behavior of the foundations.

2.5.4.14 Construction Notes

Problems associated with compaction control for the Seismic Category I backfill are as described in detail in Section 2.5.4.5. There were no other significant problems during construction.

2.5.5 Stability of Slopes

2.5.5.1 Slope Characteristics

The only slopes in the site area are the cut slopes around the immediate plant site (from plant grade to the surrounding original ground surface) and those along Grants Bayou and its seasonal tributaries.

INSERT 1

INSERT 2

Insert 1 for Page 2.5-129b

the cut slopes in the Unit 2 excavation area, the backfill slopes between Unit 1 and the Unit 2 excavation,

Insert 2 for Page 2.5-129b

Section 2.5.5.1 describes slopes in the site area excluding those slopes in the immediate vicinity of the Unit 2 excavation. Section 2.5.5.2 describes the slopes that result from the cancellation of construction on Unit 2.

INSERT 1

The permanent slopes adjacent to the plant are shown in plan on Fig. 2.5-91 and in cross section on Fig. 2.5-92. The slopes along Grants Bayou and its tributaries are shown by the contours on Fig. 2.5-91 and by the survey cross sections on Fig. 2.5-93.

The relationship of the original topography to finish grade and to plant structure locations is shown on Fig. 2.5-91 and 2.5-92, cross sections X-X' and Y-Y'.

Both field and laboratory information is available for the soils composing the plant area slopes. Fig. 2.5-23 and 2.5-24 are boring location plans. Test boring information is included on Fig. 2.5-53 through 2.5-65 and in Appendix 2H. Table 2.5-8 is a summary of Atterberg limits testing for the loessal deposits and the Port Hickey top-stratum silts and clays. Sieve analysis results are shown on Fig. 2.5-50 through 2.5-52.

The soil stratigraphy for the cut slopes surrounding the site is shown on Fig. 2.5-28 and 2.5-29. The soil stratigraphy and properties are described in Section 2.5.4.2.

1.2

2.5.5.2 Design Criteria and Analyses

The maximum groundwater level within the cut slopes around the plant site, coincident with the Design Basis Flood level, has been conservatively estimated at el 70 ft msl (Section 2.4.13.5). The cut slopes have an overall maximum height of 30 ft, which occurs only to the north of the plant site. The slopes are cut at a slope of three horizontal to one vertical. This corresponds to a slope of 18 deg with the horizontal. The overall distance between the top of the slope to the north and the nearest Seismic Category I structure is 480 ft. The cut slope west of the plant is 900 ft from the nearest Seismic Category I structure.

SSE

68

1

The stability of the slope to the north of the plant was evaluated both for static conditions and under the SSE loading. For the dynamic analysis, it was conservatively assumed that a 20-ft thick layer of fine sand and clayey sand below the water table might liquefy. The analysis showed that even if this should happen, the shearing resistance of the soil above the liquefied zone would be sufficient to preclude major ground movement, and thus prevent a lateral movement of the soil mass into the plant area. Liquefaction, if it were to occur, could cause settlement and cracking within the slope and sand boils. However, this would not present any hazard to the plant.

Insert 1 for Page 2.5-130

2.5.5.1 Plant Area Slopes
2.5.5.1.1 Slope Characteristics

The static and dynamic analyses for the slope to the north of the plant was based on computer-assisted simplified Bishop and Morgenstern-Price methods (Lease II). Since an extensive liquefied zone was postulated for the dynamic case, and since the simplified Bishop Method is more 1
applicable to circular-type failure geometries which are typical for the static case, the Morgenstern-Price method was used to model the sliding-block or wedge-type geometry.

The slope geometry, including the failure surface, is presented in Fig. 2.5-94 together with the soil parameters used in the analyses. The soil unit weights and friction angles shown on Fig. 2.5-94 are based on the field and lab testing described in Section 2.5.4.2. The resulting factors of safety are 1.3 for SSE loading with the conservative assumption of a liquefied layer and greater than 5 for the static case. In summary, even considering SSE loading and assuming liquefaction, the permanent slopes remain stable.

Analyses were also done for the temporary excavated construction slopes using the stability number method. This provided a conservative stability estimate for these 1
relatively homogeneous, slightly cohesive slopes. The analyses indicated that the slopes had more than adequate stability. No excavation slope failures occurred during construction. These excavation slopes are shown on Fig. 2.5-72.

Stability analyses are currently underway for the slopes caused by the Unit 2 excavation adjacent to the Unit 1 structures. The analysis will include evaluation of the factor of safety against failure for all slopes adjacent to 11
Unit 1 structures as well as evaluation of the factor of safety against sliding for the service water tunnel which leads to Unit 2. These excavation slopes are shown on Fig. 2.5-72A.

It is anticipated that these analyses will be completed in the first quarter of 1984 and will show adequate factors of safety. However, should it be necessary, stability of the slopes and/or tunnel can be enhanced with the addition of backfill. This section will be revised to include the results of these analyses once they are completed.

Cross sections of the valley of Grants Bayou are presented in Fig. 2.5-93. It can be seen that except for the small channel of the present stream in the bottom of the valley, the slopes are very shallow, generally flatter than 20:1. The stream bed in the reach of the bayou adjacent to the plant site is above el +70 ft msl, indicating that the

groundwater flow is generally from the stream to the terrace aquifer.

It was conservatively assumed that the small channel in the bottom of the valley would be filled by landslides, either due to static instability or due to SSE liquefaction of any low density layers of fine sand and clayey fine sand below the water table. This is is a conservative assumption since a large portion of the soil mass of the slopes is above the water table and the overall valley slopes are very flat; hence, flow slides are not probable. In the event of a recurrence of the maximum historic earthquake, as discussed in Section 2.5.1.2.8, liquefaction would not take place and hence blockage of the valley by landslides would not occur. Nevertheless, for design purposes a partial blockage of the Grants Bayou valley was assumed. This blockage is based on an assumed soil displacement such that nc slopes would remain shallower than 20:1. The blockage condition is shown in Fig. 2.5-94. A similar blockage was assumed for West Creek. These assumed blockages would have no effect on the plant except as it relates to the site flooding discussed in Section 2.4.4.

INSERT

2.5.5.3 Logs of Borings

Soil investigations, borings, and testing are described in Sections 2.5.4.2 and 2.5.4.3.

Insert for Page 2.5-131a

2.5.5.2. Unit 2 Excavation Slopes

2.5.5.2.1 Slope Characteristics

The slopes in the Unit 2 excavation are shown in plan on Figure 2.5-72a. The slopes located to the north, west, and south of the Unit 2 area are cut slopes in natural soils. The slopes located east of the Unit 2 area are constructed of backfill.

Laboratory test data for the backfilled soils are described in Section 2.5.4.5, Appendix 2M, and Figure 2.5-74. Information for the natural soil is located in preceding Section 2.5.5.1.1.

The soil stratigraphy for the cut slopes surrounding the Unit 2 excavation is shown on Figures 2.5-28 and 2.5-29. As developed in Section 2.4, water accumulates in the Unit 2 excavation as the result of heavy precipitation. Consequently, the Unit 2 area slopes were analyzed for the following combinations of seismic events and ponded water levels:

Static	Ponded water to elev 80 ft msl
OBE	Ponded water to elev 73 ft msl
SSE	Ponded water to elev 68 ft msl

For the stability analyses, the level of groundwater is assumed conservatively to equal the level of ponded water. In reality, this does not occur. Since the normal groundwater level is 8 feet or more below the bottom of the Unit 2 area, ponded water tends to infiltrate vertically until a mound of groundwater is formed on top of the normal water table. The height of the groundwater mound attenuates with distance from the excavation and hence is always less than the ponded height. For slope stability analysis it is conservative to assume that the groundwater level equals the level of ponding for two reasons. First, the real unbalanced hydrostatic force acts toward the slope tending to resist sliding. Second, seepage into the slope creates seepage forces that also tend to stabilize the slopes. The analysis of the Unit 2 slopes conservatively does not consider these stabilizing influences.

2.5.5.2.2 Design Criteria and Analyses

The static and dynamic analyses for slopes in the Unit 2 area were based on computer - assisted simplified Bishop and Morgenstern-Price methods (LEASE II). The simplified Bishop Method was used to analyze the slopes east of Unit 2 since they are constructed of backfill and are reasonably homogeneous in the face of the west excavation slope. The Morgenstern-Price method was used to model the sliding-block geometry which is appropriate where a weak layer can be postulated, as for the north, south, and west slopes.

In the case of the cut slopes only the massive failure of the west slope could impact the safe operation of Unit 1. To impact the operation of Unit 1, this slope would have to fail in such a way as to breach West Creek.

In order to evaluate stability of the cut slopes, it was necessary to evaluate the potential for liquefaction of the natural soils. This was accomplished by comparing the results of standard penetration testing to acceptance criteria established using the method of Seed and Idriss (Reference 75) previously described in Section 2.5.4.8. Seventeen borings (209 through 214 and 218 through 230) are located between West Creek and the west slope of the Unit 2 excavation. In these borings between the elevations 30 and 65 ft msl, approximately 120 standard penetration (blowcount) tests were performed. All but six of the blowcounts exceeded the Seed & Idriss criteria. Of those six, two were associated with cohesive material and therefore are not indicative of liquefaction. It was concluded that liquefaction of a continuous soil layer is not likely.

The impact of liquefaction of localized inclusions of loose sand was evaluated in the stability calculations by varying the strength parameters for soils below elevation 50 ft msl. Cut slopes were analyzed using angle of internal friction values of 10 ,20 , and 35 for soil below elevation 50 ft msl, which correspond to approximately 75%, 50% and 0% liquefaction, respectively. Figure 2.5-72b shows the geometry that was used to model the west slope.

The results of the stability calculations are discussed below. In all cases, the critical factor of safety corresponds to the SSE loading, therefore results are presented only for the SSE case. The Morgenstern-Price Method was used to analyze the wedge failure mass. The results of the Morgenstern-Price analysis for SSE loading are summarized on Figure 2.5-72b. The analysis shows that for the extremely unlikely event of liquefaction of 75% of the soils at elevation 50 ft msl (corresponding to = 10 degrees) the factor of safety against sliding is approximately 1.3. The simplified Bishop Method was used to evaluate the local stability of the face of the slope. The results of the simplified Bishop analysis indicate that minor sloughing of soil at the face of the lope may occur during an SSE event. This minor instability would have no impact on the integrity of the West Creek fabriform channel or Unit 1. Given the conservatism of the analysis that results in the factor of safety of 1.3, it is concluded that the north, south, and west slopes of the Unit 2 area are stable.

The slopes located east of Unit 2 were analyzed using the simplified Bishop Method. Since these slopes in the areas of safety related structures are constructed of engineered backfill as described in Section 2.5.4.5.3, no liquefaction was postulated. The geometry analyzed and typical results of the

computerized calculation are shown on Figure 2.5-72c. In general, for trial circles at the same center, the factor of safety decreases with decreasing radius, indicating that the critical factor of safety corresponds to an infinite slope type failure. Analysis of the infinite slope problem under SSE loading gives a factor of safety of 1.15 which corresponds to minor surficial sloughing at the face of the slope. This analysis is based on a friction angle of the backfill material equal to 36 degrees. Laboratory testing of backfill samples (Appendix 2M - Triaxial S Tests) indicate that for backfill compacted to 89% relative density a friction angle of 42 degrees can be justified. Using a friction angle of 42 degrees in the infinite slope analysis results in a factor of safety of approximately 1.4. It is concluded that the backfill slopes east of the Unit 2 excavation are stable.

TABLE 2.5-16

SLIDING AND OVERTURNING FACTORS OF SAFETY
FOR MAJOR STRUCTURES

	Factor of Safety			
	OBE		SSE	
Structure	Sliding	Overturning	Sliding	Overturning
Diesel Generator Building	$\boxed{2.3}$ 2.6	$\boxed{5.7}$ 6.5	$\boxed{1.3}$ 1.6	$\boxed{2.7}$ 3.6
Control Building	2.3	$\boxed{7.6}$ 6.0	1.6	$\boxed{4.0}$ 3.7
Fuel Building	$\boxed{2.3}$ 2.9	$\boxed{5.5}$ 3.8	$\boxed{1.3}$ 1.7	$\boxed{2.6}$ 2.0
Turbine Building	$\boxed{3.5}$ 4.2	$\boxed{24.5}$ 23.7	-	-
Reactor Building	$\boxed{3.8}$ 5.1	6.5	$\boxed{2.2}$ 2.9	$\boxed{3.7}$ 3.8
Auxiliary Building	$\boxed{3.4}$ 3.3	$\boxed{6.5}$ 4.5	$\boxed{1.8}$ 1.6	$\boxed{3.3}$ 2.4
Standby Service Water Tower	$\boxed{2.2}$ 2.7	$\boxed{7.7}$ 7.4	$\boxed{1.5}$ 1.8	$\boxed{4.8}$ 4.7
Service Water Tunnel	2.3	2.6	1.5	1.9

2

TABLE 2.5-17

STATIC DESIGN CRITERIA OF MAJOR STRUCTURES

	Shield Building and Containment	Radwaste Building	Auxiliary Building	Fuel Building	Control Building	Diesel Generator Building	Standby Service Water Pump House and Tower Basin	Turbine Building	Turbine Pedestal
Approximate Foundation Dimensions (ft)	150 dia	156x105	154x117	124x105	148x78	95x70	162 dia	320x$\boxed{135}$ 215	180x40
Maximum Buoyant Pressure (psf)	$\boxed{620}$ 936	$\boxed{620}$ 1248	$\boxed{620}$ 936	$\boxed{370}$ 998	$\boxed{310}$ 936	$\boxed{310}$ 936	$\boxed{620}$ 1310	$\boxed{620}$ 1248	$\boxed{1,150}$ 1778
Total Average Distributed Dead Load (ksf)	8	6	$\boxed{7}$ 5	6	$\boxed{4}$ 3	$\boxed{3}$ 4	3	$\boxed{3.5}$ 3	5.5
Factor of Safety Against Flotation	$\boxed{13}$ 6.2	$\boxed{10}$ 4.9	$\boxed{11}$ 5.3	$\boxed{16}$ 6.3	$\boxed{13}$ 2.8	$\boxed{10}$ 4.6	$\boxed{5}$ 4.9	$\boxed{7}$ 2.6	$\boxed{5}$ 3.2
Total Ultimate Bearing Capacity (ksf)	350	310	330	310	280	150	360	380	290
Factor of Safety Total Ultimate Bearing Capacity/Total Uniformly Distributed Dead Load	44	52	$\boxed{47}$ 66	52	70	$\boxed{50}$ 37	119	84	53
Foundation Elevation (ft above msl)	60	60	$\boxed{60}$ 65	64	65	65	$\boxed{60}$ 59	60	51.5
Net Average Distributed Dead Load (ksf)	2.8	1.4	2.4	1.4	-0.6	1.3	-2.1	-0.5	-0.3
Net Allowable Bearing Capacity (ksf)	12	12	12	12	12	12	12	10	10

TABLE 2.5-17 (Cont)

NOTES: All foundations are mat foundations.

Soil unit weights in pcf:
wet = 115
submerged = 60

Groundwater at el +57.0

80.0

Maximum flood level at el +$\boxed{70.0}$ for maximum bouyant force

Finish grade at el +95.0

For foundation with groundwater elevation equal to or less than B below foundation level, use reduction factor for γwet = 0.5 (1+D/B) γwet, where D equals depth to groundwater level below bottom of mat.

For net uniform distributed dead load, average ground surface elevation prior to excavation was taken conservatively as el +105.

Formulas:

Total Ultimate Bearing Capacity (Terzaghi & Peck Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York, NY, 1968.)

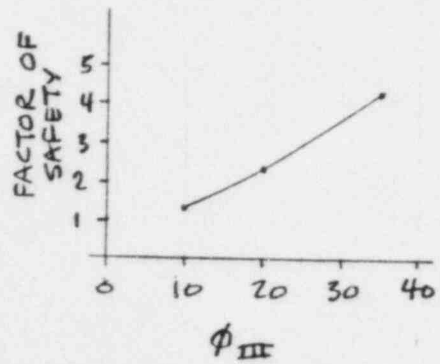
Circular mat: $q_{dr}=1.2cN_c+\gamma D_f N_q+0.6\gamma r N_\gamma$

Rectangular mat: $q_{db}=1.2cN_c+\gamma D_f N_q+0.4\gamma B N_\gamma$

Factor of Safety Against Flotation

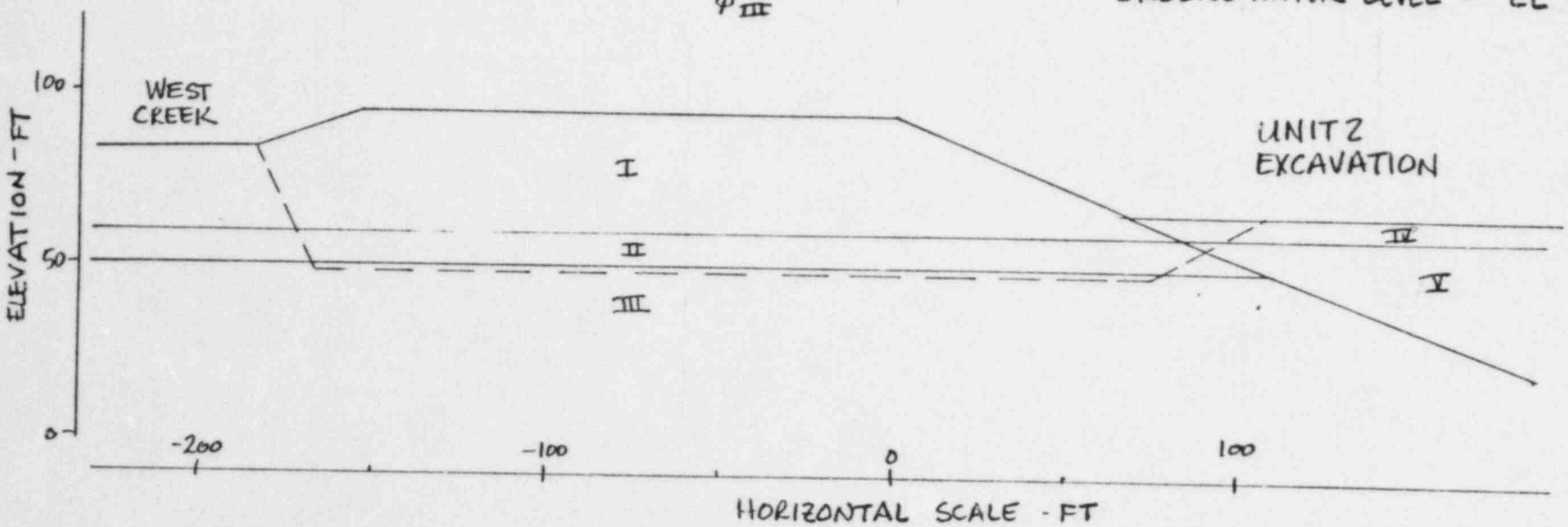
$$\text{F.S.} = \frac{\text{Total average distributed dead load}}{\text{Maximum bouyant force}}$$

C=0 $N_c=0$
$\phi=36^\circ$ $N_q=53$ $N_\gamma=52$



PONDED WATER LEVEL = EL 68'

GROUND WATER LEVEL = EL 68'



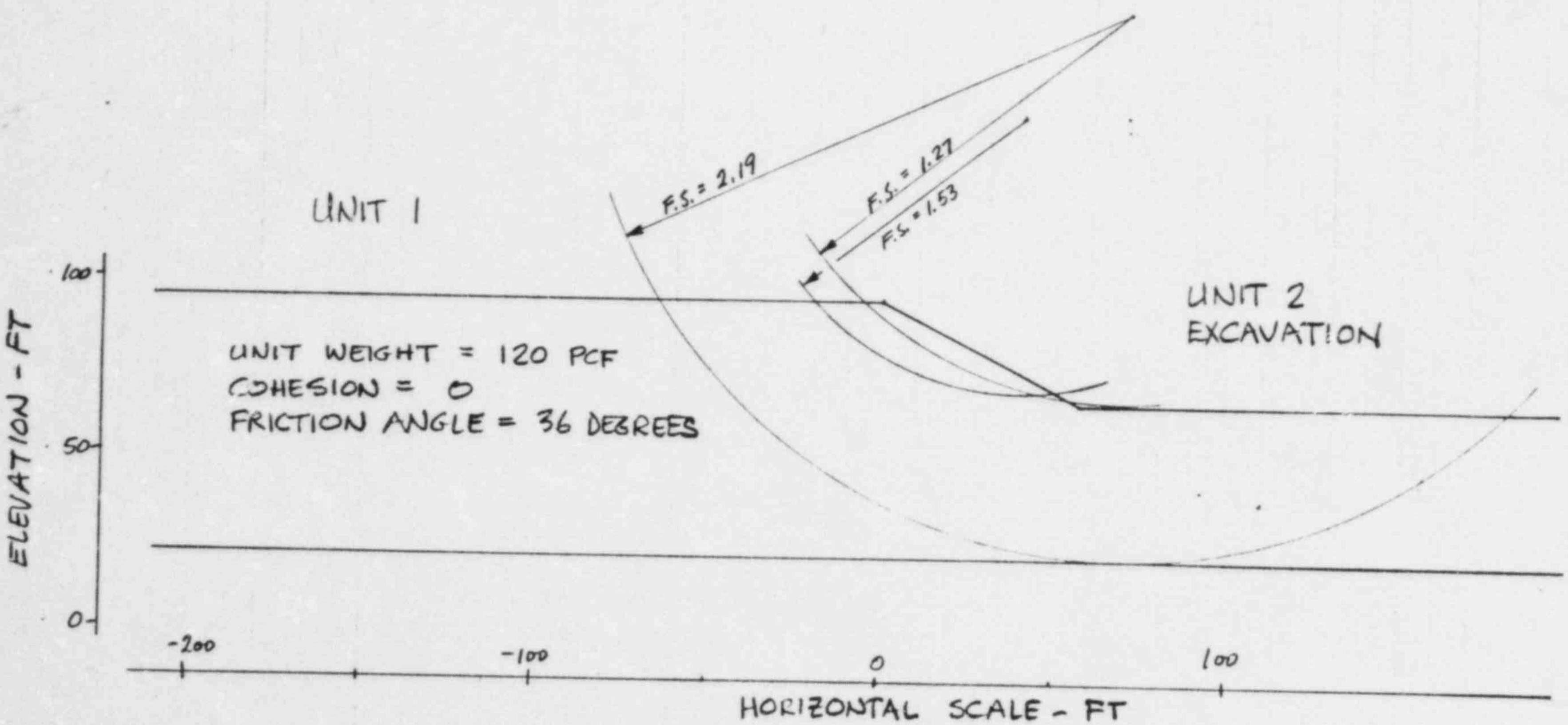
SOIL PARAMETERS

TYPE	UNIT WEIGHT PCF	COHESION	FRICTION ANGLE DEGREES
I	115.	0	32
II	125.	0	32
III	130.	0	VARIES
IV	120.	0	36
V	130.	0	36

FIGURE 2.5-72b

SUMMARY OF SLOPE STABILITY ANALYSIS - UNIT 2 EXCAVATION CUT SLOPES SSE LOADING

RIVER BEND STATION FSAR



PONDED WATER LEVEL = EL 68'

GROUNDWATER LEVEL = EL 68'

FIGURE 2.5-72C
SUMMARY OF SLOPE STABILITY ANALYSIS-UNIT 2 EXCAVATION BACKFILL SLOPES SSE LOADING
RIVER BEND STATION FSAR

TABLE 3.4-1

STRUCTURES, PENETRATIONS, AND ACCESS OPENINGS
DESIGNED FOR FLOOD PROTECTION

Description	Reactor Building	Standby Service Water Tower Basin	Control Building	Auxiliary Building	Diesel Generator Building	Fuel Building
Groundwater Level -						
Normal	57-0	57-0	57-0	57-0	57-0	57-0
Design Basis Flood Level (DBFL)(see Note 5)	70-0	70-0	70-0	70-0	70-0	70-0
Top of Base Mat	70-0	64-6	70-0	70-0	70-0	70-0
Average Plant Grade	94-6	94-6	94-6	94-6	94-6	94-6
DBFL - Maximum Postulated Flood Level	95-$\boxed{1}$ 2	95-$\boxed{1}$ 2	95-$\boxed{1}$ 2	95-$\boxed{1}$ 2	95-$\boxed{1}$ 2	95-$\boxed{1}$ 2
Exterior access openings located below DBFL - Subject to flood potential	Shield Bldg. Equip. Hatch	-	-	-	-	Fuel Bldg. Truck Door
Penetrations - Subject to flood potential (see Note 4)	Pipe Penet. @ 96-0	Pipe Penet. @ 90-6 and 95-6	-	Pipe Penet. @ 95-0	-	-
Electric Duct Bank Penetrations - Subject to flood potential (see Note 3)	-	-	-	-	Top of Duct El 72'-9 1/4"	Top of Duct El 71'-6 1/4" & El 91'-9 1/4"

NOTES: 1. All dimensions are msl elevations in feet and inches.
2. The tunnels housing Seismic Category I systems and components are accessible only from the adjoining buildings. Pipe penetrations through the tunnel walls, if any, are provided using watertight seals designed to withstand the flood loads.
3. As the electrical ducts penetrate the Seismic Category I structures, they are sealed with waterstops to prevent any adverse effect from flooding.
4. Pipe penetration elevations are penetration centerlines. 2
5. A discussion of the effect on plant buildings of runoff ponding in the Unit 2 excavation is included in Section 2.5.