GULF STATES UTILITIES COMPANY

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June 22, 1984
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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 is Gulf States Utilities Company's (GSU) partial response to the Nuclear Regulatory Commission's (NRC) Safety Evaluation Report (SER) confirmatory item No. (3) identified in Section 2.5.5.2 regarding the factor of safety against failure for all slopes adjacent to the Unit 1 structures. During a telephone conference on June 15, 1984, further clarification was provided to GSU by the NRC's Structural and Geotechnical Engineering Branch (SGEB) reviewer, Mr. Banad Jagannath. The information requested is addressed herein. An additional response addressing the factor of safety against sliding for the service water tunnel (G) that leads to Unit 2 excavation area is scheduled for submittal the week of July 2, 1984.

Sincerely,

J. E. Booker

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

JEB/~~RJK~~ eu/je

Attachment

B001
/1

Where:

h_x = 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 (values provided in Reference 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 determined 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-yr 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 INSERT A event of seepage, the groundwater level would not be significantly affected.

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 A

If no berm were in place, a 25-year rainfall event would result in a maximum ponding level of about 68.7 ft msl, based on no seepage.

2.5.5.1 Plant Area Slopes

2.5.5.1.1 Slope Characteristics

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.

2.5.5.1.2 Design Criteria and Analyses

The maximum groundwater level within the cut slopes around the plant site, coincident with the SSE, has been conservatively estimated at el 68.7 ft msl (Section 2.4). 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.

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

an assumed soil displacement such that no 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.

2.5.5.2 Unit 2 Excavation Slopes

2.5.5.2.1 Slope Characteristics

The slopes and berm location in the Unit 2 excavation are shown in plan on Fig. 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 backfill soils are described in Section 2.5.4.5, Appendix 2M, and Fig. 2.5-74. Information for the natural soils is located as described in Section 2.5.5.1.1.

The soil stratigraphy for the cut slopes surrounding the Unit 2 excavation is shown on Figs. 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. INSERT B Consequently, the Unit 2 area slopes were analyzed for the following combinations of seismic events and ponded water levels:

Static	Ponded water to el 80 ft msl
OBE	Ponded water to el 73 ft msl
SSE	Ponded water to el 68.7ft msl

X

For the stability analyses, the level of groundwater is conservatively assumed to equal the level of ponded water. In reality, this does not occur. Since the normal groundwater level is 8 ft or more below the bottom of the Unit 2 area, ponded water tends to infiltrate vertically until a mound of groundwater is formed on the 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

Insert B

As described in Section 2.4, a berm is used to control the ponded water levels in the Unit 2 excavation. However, analysis of the 25-yr rainfall event + SSE without the berm in place indicates that a ponded water le'·l of el. 68.7 ft msl would result in the Unit 2 excavation. For the 1/2 PMP + OBE and for the 1/2 PMP followed by the PMP (which is considered without a seismic event) the berm is required in order to maintain a ponded water level below the design basis ponding level (el. 80 ft msl) in the excavation. Should an SSE result in disruption of the berm, the berm will be restored.

slopes. The analyses 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, which are constructed of backfill and are reasonably homogeneous, and 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 cut slopes.

In the case of the cut slopes, either the massive failure of the west slope or a localized failure of the slope face could impact the safe operation of Unit 1. A massive failure of the west slope would impact the safety of Unit 1 if West Creek was breached and its flow diverted into the excavation. Similarly, if a localized slope failure breached the berm, the drainage characteristics of the site would be altered, creating the possibility of excessive ponding in the excavation. Therefore these events were analyzed to show that:

1) A massive failure of the west slope does not occur under static, OBE, or SSE conditions and

2) The berm is not breached under static or OBE conditions.

For analyses on cut slopes the effects of both the berm and live loads due to traffic on adjacent roadways are included. The geometry analyzed is shown on Figures 2.5-72b and 2.5-72d.

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[(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 blowcounts exceeded the Seed & Idriss criteria for the OBE. For the SSE, all but six of the blowcounts exceeded the Seed and 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 would not occur during an OBE and is highly unlikely during an SSE.

The impact of liquefaction of localized inclusions of loose sand was evaluated in the SSE stability calculations by varying the strength parameters for soils between elevation 40 and 59 ft msl. Cut slopes were analyzed using angle of internal friction values of 10 deg, 20 deg , and 35 deg for soil in that zone which corresponds to

approximately 75 percent, 50 percent, and 0 percent liquefaction, respectively. Fig 2.5-72b shows the geometry that was used to model the west slope.

Since all blowcounts exceeded the Seed & Idriss criteria for the OBE, no liquefaction was postulated in the OBE analysis.

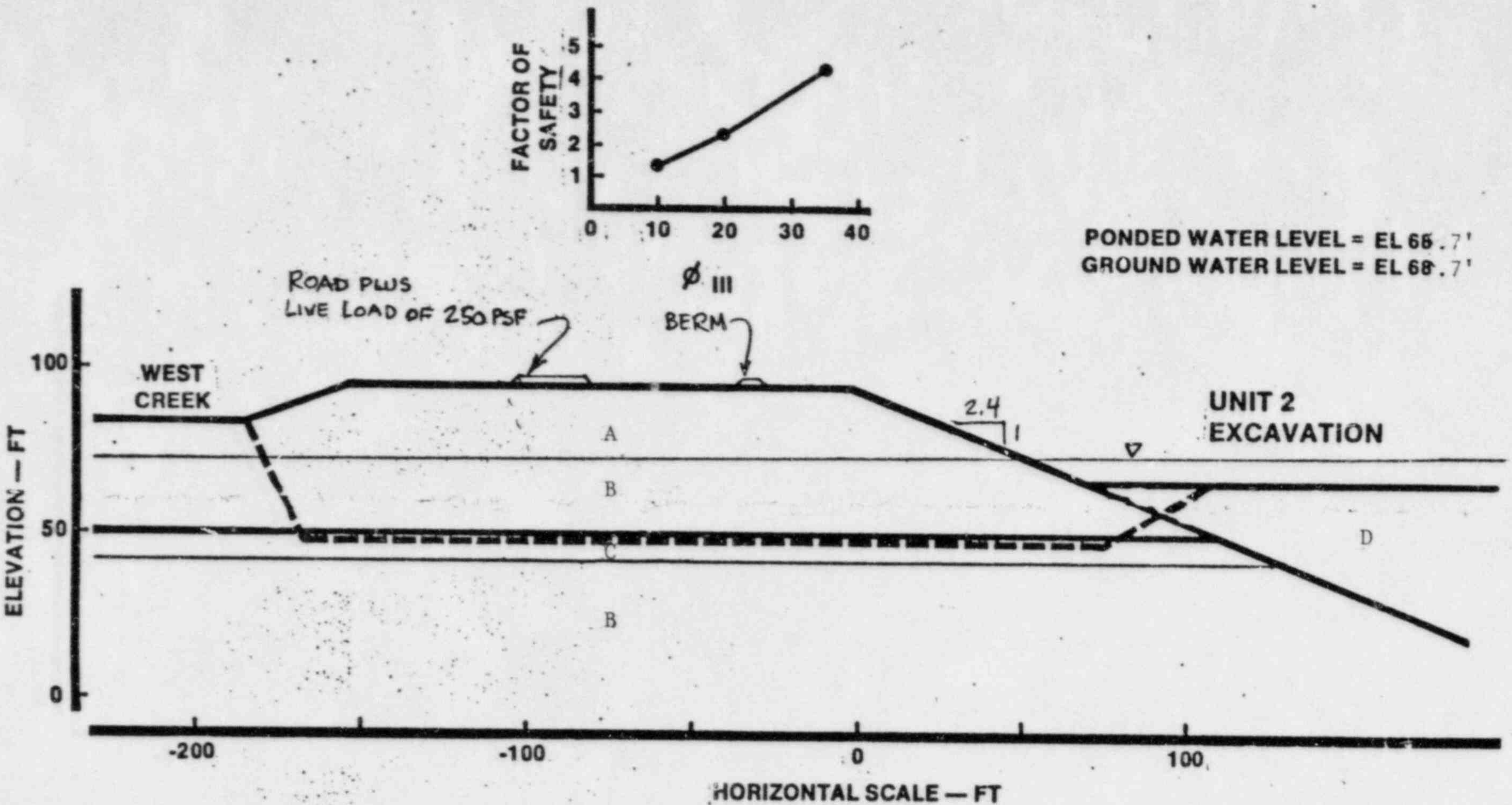
Since the berm is not required during a 25-yr storm + SSE, the berm is located so that it is stable during the OBE. The berm location based on the OBE analysis is then used in subsequent analyses of slope stability.

The local stability of the face of the cut slopes under the OBE loading was evaluated using the simplified Bishop Method. The geometry analysed and typical results of the computerized calculation are shown in Figure 25-72d. Also shown in that figure, is the factor of safety against an infinite slope failure under all loading conditions. It is concluded that the face of cut slopes is stable under the OBE and static conditions. Under SSE loading, analysis shows in stability at the face of the slope which may breach the berm. However, as stated in Section 2.5.5.2.1 safe operation of Unit 1 would not be affected.

The slopes located east of Unit 2 were analyzed using the simplified Bishop Method. Since these slopes 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 Fig. 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 1.12 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 deg. Laboratory testing of backfill samples (Appendix 2M - Triaxial S Tests) indicate that for backfill compacted to 89 percent relative density a friction angle of 42 deg can be justified. Using a friction angle of 42 deg in the infinite slope analysis results in a factor of safety of approximately 1.4. Failure surfaces large enough to breach the berm have factors of safety in excess of 3. It is concluded that the backfill slopes east of the Unit 2 excavation are stable.

The massive stability of the west slope was evaluated using the Morgenstern-Price Method. In this case, the critical factor of safety corresponds to the SSE loading; therefore, results are presented only for the SSE case.

Given the adequate factors of safety for these two conditions. it is concluded that the safe operation of Unit 1 is not impacted by the slopes associated with the Unit 2 excavation. The results of the Morgenstern-Price analysis for SSE loading are summarized on Fig. 2.5-72b. The analysis shows that for the extremely unlikely event of liquefaction of 75 percent of the soils at elevation 50 ft msl (corresponding to ϕIII=10 deg) the factor of safety against sliding is approximately 1.3. Given the conservatism of the analysis that results in a factor of safety of 1.3, it is concluded that the cut slopes of the Unit 2 area are stable.



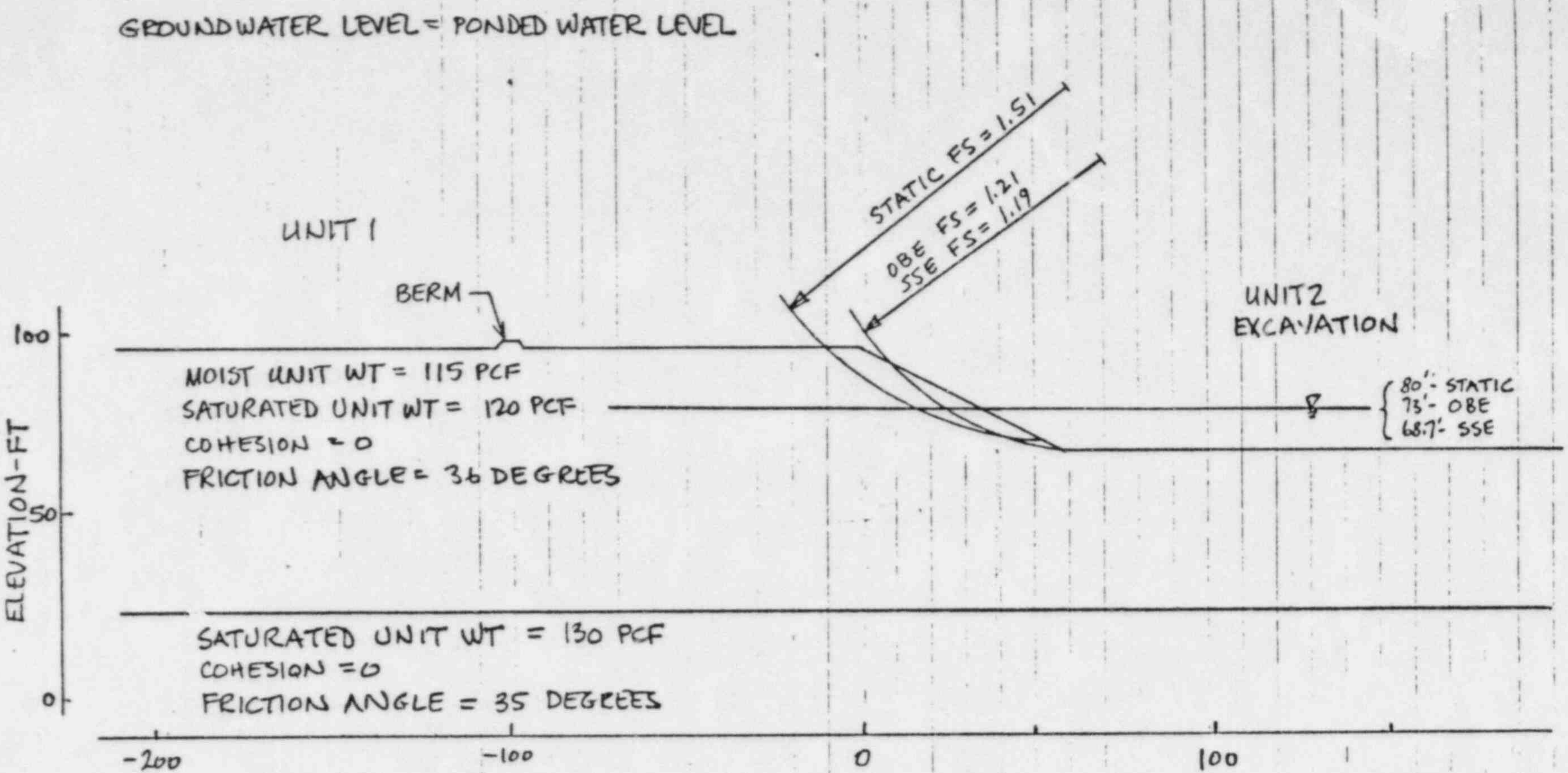
SOIL PARAMETERS

TYPE	UNIT WEIGHT PCF	COHESION	FRICTION ANGLE DEGREES
A	115.	0	32
B	125.	0	32
C	135	0	VARIES
D	135	0	32

FIGURE 2.5-72b

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

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT



INFINITE SLOPE FACTORS OF SAFETY

CASE	$\phi = 36°$	$\phi = 42°$
STATIC	1.45	1.80
OBE	1.28	1.59
SSE	1.12	1.39

FIGURE 2.5-72C

SUMMARY OF SLOPE STABILITY ANALYSIS
UNIT 2 EXCAVATION BACKFILL SLOPES
SIMPLIFIED BISHOP - ALL LOADINGS

RIVER BEND STATION
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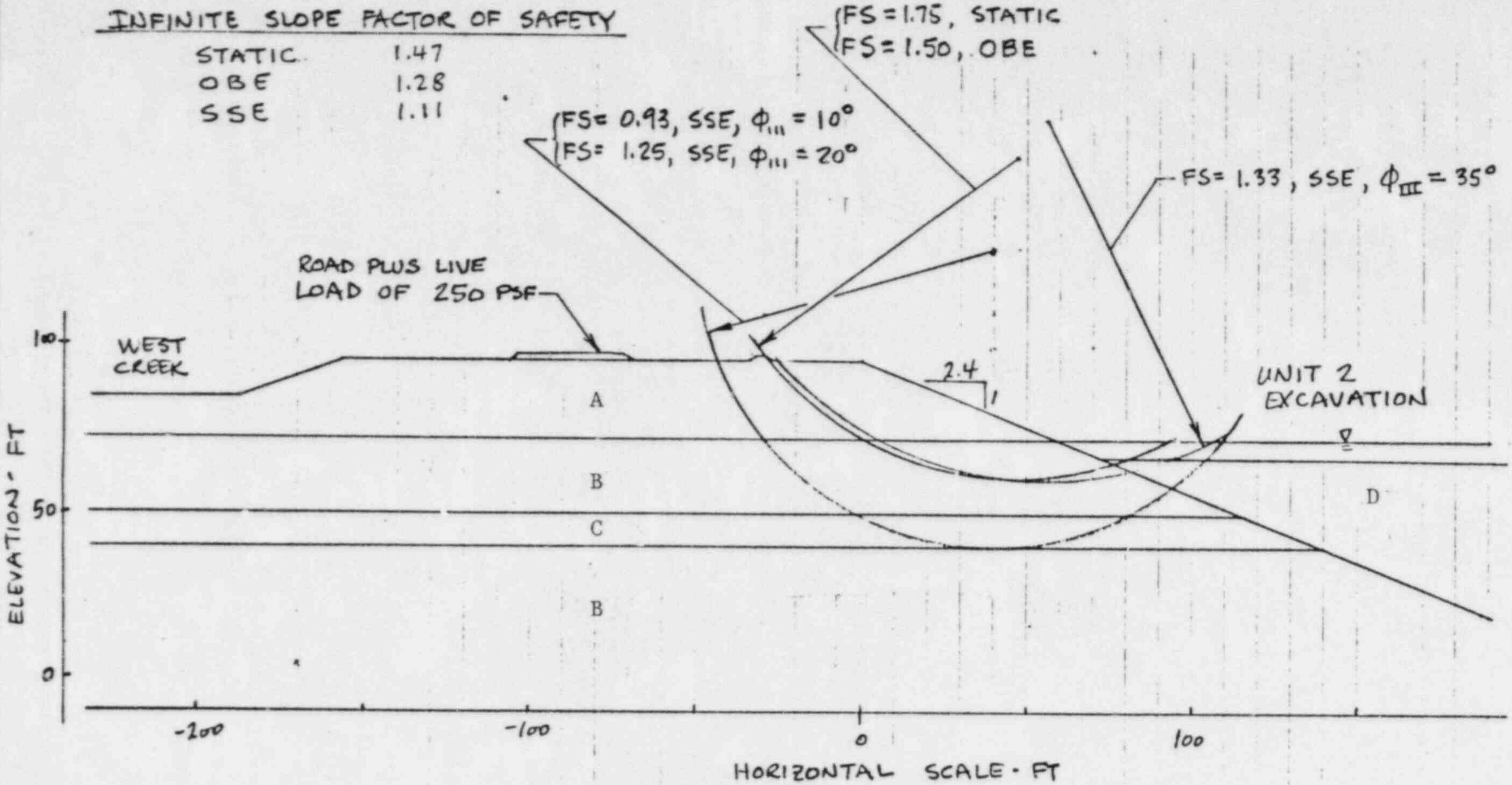


SOIL PARAMETERS

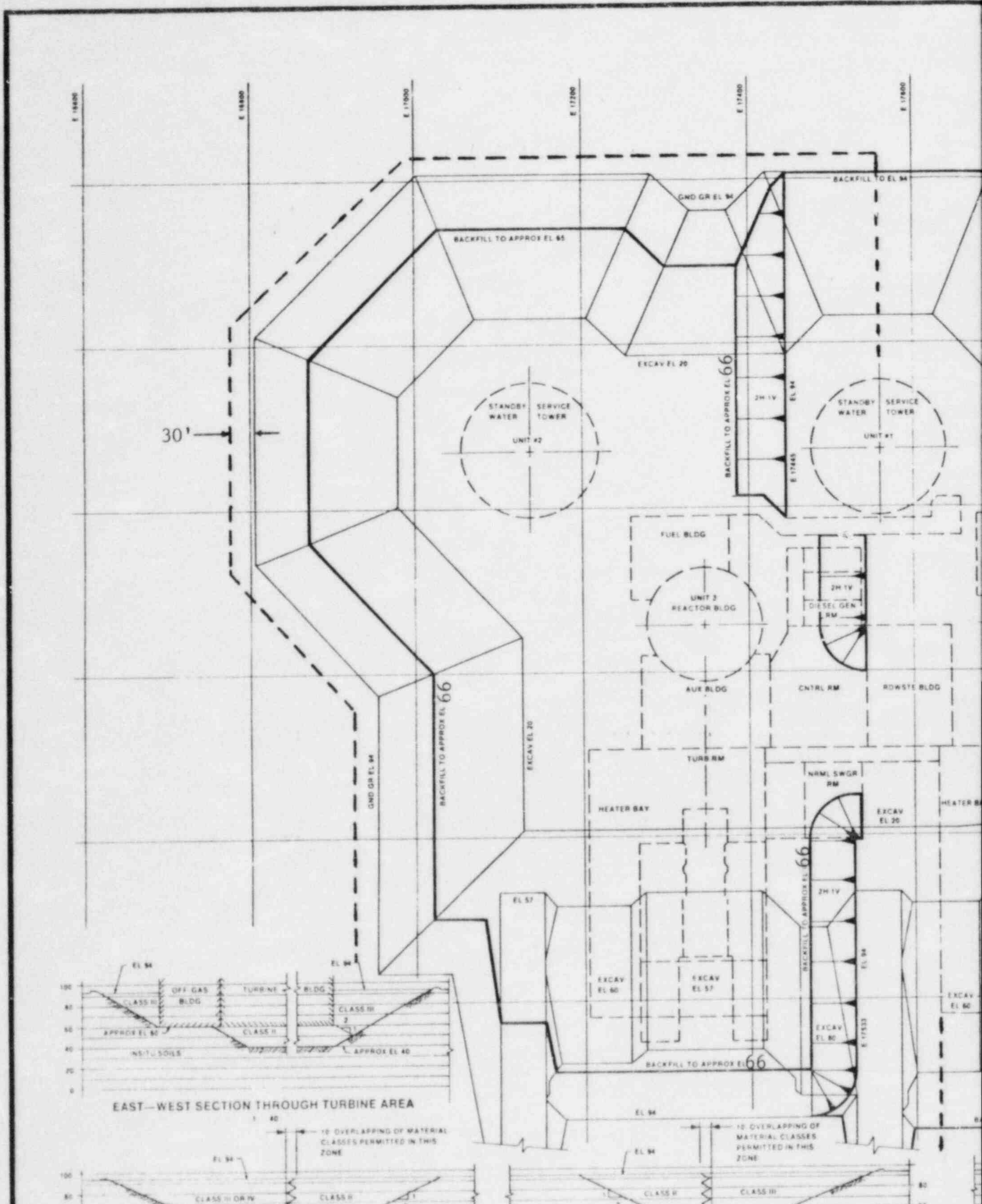
TYPE	UNIT WEIGHT PCF	COHESION	FRICTION ANGLE DEGREES
A	115.	0	32.
B	125.	0	32.
C	135.	0	VARIES FOR SSE OTHERWISE 35.
D	135.	0	32.

FIGURE 2.5-72d

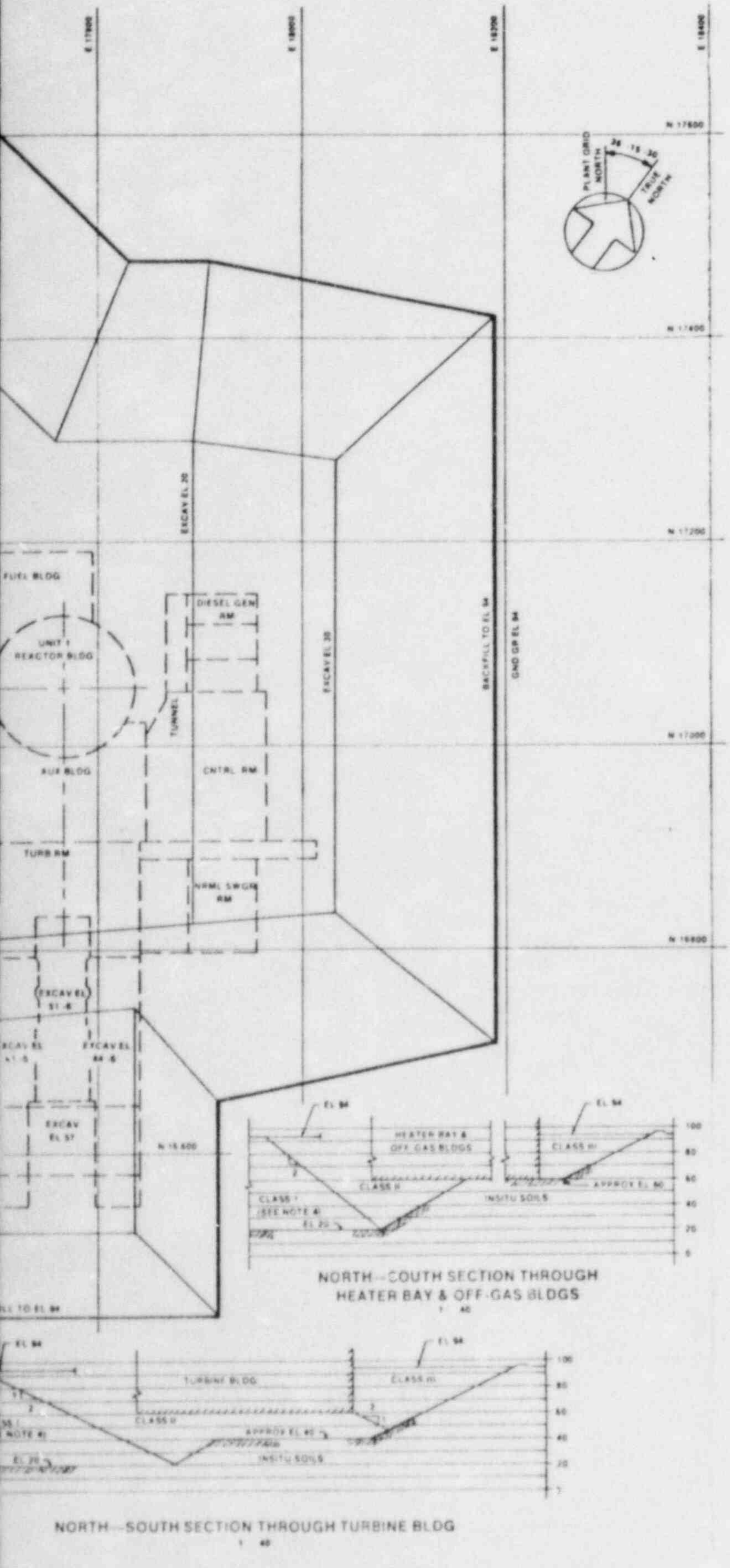
SUMMARY OF SLOPE STABILITY ANALYSIS
UNIT 2 EXCAVATION CUT SLOPES
SIMPLIFIED BISHOP - ALL LOADINGS

RIVER BEND STATION
FSAR

ATTACHME



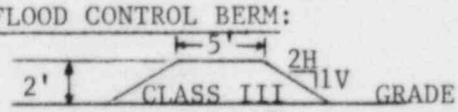
30'
BACKFILL TO APPROX EL 65
GND GR EL 94
BACKFILL TO EL 94
EXCAV EL 20
BACKFILL TO APPROX EL 66
STANDBY WATER
SERVICE TOWER
UNIT #2
UNIT #1
FUEL BLDG
UNIT 2 REACTOR BLDG
DIESEL GEN RM
AUX BLDG
CNTRL RM
RDWSTE BLDG
TURB RM
HEATER BAY
NRML SWGR RM
GND GR EL 94
EXCAV EL 20
EXCAV EL 60
EXCAV EL 57
EL 57
EL 94
EAST—WEST SECTION THROUGH TURBINE AREA
OFF GAS BLDG
TURBINE BLDG
CLASS III
CLASS II
APPROX EL 60
APPROX EL 40
INSITU SOILS
10' OVERLAPPING OF MATERIAL CLASSES PERMITTED IN THIS ZONE
CLASS III OR IV
CLASS I (SEE NOTE 4)
EL 20
EL 30
TYPICAL NORTH & WEST BACKFILL SECTION
TYPICAL EAST BACKFILL SECTION



APERTURE CARD

NOTES
1. SCALE 1 : 50 U.O.S.
2. UNIT 2 SIMILAR TO UNIT 1 EXCEPT AS SHOWN
3. DASH LINE INDICATES BUILDING FOUNDATION OUTLINE
4. CLASS I FILL IS QA CAT I FILL. ALL OTHER FILL IS NON CAT I FILL.
5. ---- INDICATES FLOOD CONTROL BERM CENTERLINE

FLOOD CONTROL BERM:



Also Available On Aperture Card

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FIGURE 2.5-72a

PLANT EXCAVATION BACKFILL PLAN WITH BERM

RIVER BEND STATION
FINAL SAFETY ANALYSIS REPORT