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SEISMIC STABILITY EVALUATION OF ALBEN BARKLEY LOCK AND DAM PROJECT

Volume 5

STABILITY EVALUATION OF GEOTECHNICAL STRUCTURES

by

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13. ABSTRACT *(Maximum 200 words)*

This report describes the procedures used to interpret the data obtained from field investigations to determine the liquefaction potential and post-earthquake strengths of the foundation soils at the Alben Barkley Lock and Dam Project, Kentucky. The basid components of the report include the interpretation of field data to characterize the site and evaluate the foundation stratigraphy, dynamic response analyses of two representative embankment sections, the analysis and interpretation of Cone and Standard Penetration Test data to determine the cyclic strengths for the evaluation of the liquefaction potential and the post-earthquake strengths were input to the seismic stability analysis documented in Volume 5 of this series of reports.

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PREFACE

The US Army Engineer Waterways Experiment Station (WES) was authorized to conduct this study by the US Army Engineer District, Nashville (ORN), by Intra-Army Order for Reimbursable Services Nos. 77-31 and 77-112. This report is Volume 5 of a 5-volume set which documents the seismic stability evaluation of Alben Barkley Dam and Lake Project. The 5 volumes are as follows:

Volume 1: Summary Report

Volume 2: Geological and Seismological Evaluation

Volume 3: Field and Laboratory Investigations

Volume 4: Liquefaction Susceptibility Evaluation and Post-Earthquake Strength Determination

Volume 5: Stability Evaluation of Geotechnical Structures

The work in this volume is a joint endeavor between ORN and WES. Mr. Paul F. Bluhm, of the Geotechnical Branch (CE-ORNED-G) at ORN, coordinated the contributions from ORN. Mssrs. Ronald E. Wahl of Soil and Rock Mechanics Division, Richard S. Olsen, and Dr. M. E. Hynes of the Earthquake Engineering and Geophysics Division (EEGD), Geotechnical Laboratory (GL), WES, coordinated the work by WES. The preliminary stages of this project were directed by Dr. William F. Marcuson, III, who was Principal Investigator from 1976 to 1979. From 1979 to 1988, Dr. M. E. Hynes-Griffin was Principal Investigator. Mr. Wahl was Principal Investigator from 1988 to project completion. Significant engineering support was provided by Mr. Donald E. Yule of EEGD. Additionally, Ms. Charlotte Caples, Mr. Daniel Habeeb, and Mr. Melvin Seid provided valuable assistance in the preparation of this report.

Overall direction at WES was provided by Dr. A. G. Franklin, Chief, EEGD, and Dr. Marcuson, Chief, GL.

Overall direction at ORN was provided by Mr. James E. Paris, Chief, Soils and Embankment Design Section, Mr. Marvin D. Simmons, Chief, Geology Section, and Mr. Frank B. Couch, Jr., Chief, Geotechnical Branch. Mr. Rick Connor is Chief, Engineering Division. LTC Stephen M. Sheppard is District Commander of ORN. Technical Advisors to the project were the late Professor H. B. Seed (University of California, Berkeley), Professors Alberto Nieto (University of Illinois, Champaign-Urbana) and L. Timothy Long (Georgia Institute of Technology), and Dr. Gonzalo Castro (Geotechnical Engineers, Inc.).

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acre-feet	1,233.489	cubic metres
cubic feet	0.02831685	cubic metres
cubic yards	0.7645549	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
feet per mile	0.1893935	metres per kilometer
inches	2.54	centimetres
kips (force) per square foot	47.88026	kilopascals
miles (US statute)	1.609347	kilometres
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
square miles	2.589998	square kilometres
yards	0.9144	metres

SEISMIC STABILITY EVALUATION OF ALBEN BARKLEY LOCK AND DAM PROJECT

STABILITY EVALUATION OF GEOTECHNICAL STRUCTURES

PART 1: INTRODUCTION

Background

1. This report is Volume 5 of a five volume set that documents the investigations and results of a seismic stability evaluation of the Alben Barkley Lock and Dam Project, located on the Cumberland River, approximately 25 miles upstream of Paducah, Kentucky. This seismic safety evaluation was performed as a cooperative effort between the US Army Engineer Waterways Experiment Station (WES) and the US Army Engineer District, Nashville (ORN), and in accordance with Engineering Regulation 1110-2-1806.

2. Construction of the Barkley Project began in 1957 and was completed in 1966. As a key unit in the comprehensive plan of development of the Cumberland River, the multi-purpose Barkley Project provides flood control, hydroelectric power, navigation, and recreation. The reservoir is contained by a concrete gravity section flanked by earth embankment dams. The concrete gravity dam, powerhouse and lock system is 109 feet tall at maximum section. The embankment dams are founded on an alluvial deposit with a maximum thickness of approximately 120 feet. The alluvial deposit is underlain by Mississippian limestone. The alluvium, a complex layering of clays, silts, sands, and gravels, is the focus of concern in the seismic safety assessment due to the possibility of liquefaction of these sediments during an earthquake. The dam supports a railroad track system which traverses most of the dam crest. A canal, large enough for barge traffic, connects Barkley and Kentucky Lakes about 2.5 miles upstream from the dam. At the maximum flood control pool, elevation 375 feet, the reservoir stores 2,082,000 acre-feet, with 13 feet of freeboard (minimum crest elevation 388 feet). For normal operation, the pool elevation varies from 354 to 359 feet, and stored volume varies from 610,000 to 869,000 acre-feet, respectively. A pool elevation of 360 feet was used for the seismic stability evaluation. A location map and plan of the project are shown in Figure 1.

3. A summary of the major elements for this project are contained in Volume 1 (Wahl and Bluhm, 1992). Detailed information for each of these major elements are contained in four additional volumes. The geological and seismological investigations for the project are documented in Volume 2 (Krinitzsky, 1986) of this report series. The most severe seismic threat was determined to be an earthquake of body-wave magnitude, m_b, of 7.5, at a distance of about 118 km, in the New Madrid source zone. The earthquake motions estimated to occur at Barkley from an earthquake occurring in this source zone are a horizontal peak acceleration of 0.24 g, a peak velocity of approximately 35 cm/second, and a duration above 0.05 g of approximately 60 seconds.

4. Volume 3 (Olsen, et al. 1989) of this report series describes the results of the field and laboratory investigations which provided the information to estimate the response of the dam and foundation to earthquake ground motions, to measure the resistance to liquefaction of the soils in the alluvial foundation and to provide sufficient stratigraphic detail so that the areal extent of possible problem zones could be estimated.

5. The dynamic site response analysis in which the earthquake-induced shear stresses in the foundation alluvium are computed and the investigations made to determine the extent of liquefaction expected in the alluvial foundation are documented in Volume 4 (Wahl, et al, 1992). In addition, the post-earthquake strengths of the materials which were input to the post-earthquake slope stability analysis, were also reported and discussed in Volume 4.

6. This volume evaluates the post-earthquake slope stability of the dam and is based on the results of the liquefaction and post-earthquake strength investigations which were reported in Volume 4. Two sections of the dam were evaluated, one representing the main embankment and a second that cuts through the switchyard area and exits into the tailrace channel. The results of the field and laboratory investigations, the extent of liquefaction in the foundation and the residual strengths of the materials determined in Volumes 3 and 4 are used as input to the slope stability analysis. The final deformations and configurations of the dam were estimated from the results of the liquefaction, stability analyses, and comparisons to case histories.

PART II: EMBANKMENT SECTIONS ANALYZED

Main Embankment Section

7. The main part of the right embankment, from station 44+00L to the right abutment, is a homogeneous, rolled-earth, compacted impervious fill with a horizontal downstream drainage blanket. Figure 2 shows a detailed section of the main embankment. The upstream slopes are 1 vertical to 2.5 horizontal from the upstream toe of the dam to elevation 380 feet, and 1 vertical to 2 horizontal from elevation 380 feet to the top of the dam, elevation 388 feet. The crest of the dam, which supports a railroad track is 38 feet wide. The downstream slopes are 1 vertical to 2 horizontal from the dam crest to elevation 375 feet, and 1 vertical to 4.5 from this elevation to the downstream toe of the slope. A two-foot thick drainage blanket extends from 20 feet downstream of the dam's centerline to a rock toe drainage ditch. A ten-foot wide, five foot deep key trench, with 1 vertical to 1 horizontal side slopes keys the dam to the foundation. As was described in Volume 3 of this series of reports, the alluvial foundation is divided into three Units. Unit 1 consists of a medium stiff clay which extends from the ground surface to elevation 325 feet, a thickness of 15 to 20 feet. Underlying this to elevation 300 feet is Unit 2 where the materials susceptible to liquefaction are present. These materials consists of a highly stratified sequence of clays, silts, and sands as well as mixtures of silty sands and sandy clays. Denser sands are present below elevation 300 feet and make up Unit 3.

Switchyard Section

8. A switchyard and access roads are located downstream of the centerline of the dam from the powerhouse to station 44+00L. A typical section through the switchyard is shown on Figure 3. On the upstream side of a typical section is a 1 vertical to 30 horizontal random fill berm which starts at the ground surface and extends to elevation 350 feet. From this point the slope is 1 vertical to 2.5 horizontal to elevation 381 feet and 1 vertical to 2 horizontal from this elevation to the dam's crest. The top elevation of the dam varies from 394.5 feet at the power house to elevation 389.2 feet at station 44+00L, and the crest is 24 feet wide. On the downstream side, the slope is 1 vertical to 2.5 horizontal from the crest to a 20-foot wide access road which leads to the top

of the dam. From here the slopes are 1 vertical to 1.75 horizontal to the switchyard which has a surface elevation of 366 feet. The switchyard is 275 feet wide and meets the existing ground at elevation 345 feet with a 1 vertical to 2.5 horizontal slope. An inclined drain, which starts at the centerline at elevation 370 feet and is 9 feet wide (horizontal measurement) was added to control seepage in this area. It has a slope of 1 vertical to 1.5 horizontal and connects to a horizontal drainage blanket. As described previously in Volumes 1 through 4, the alluvial foundation is divided into three units, with Unit 2 being the zone most susceptible to liquefaction. Extensive exploration in this area shows that Unit 1 extends from the surface to elevation 320 feet, Unit 2 from elevation 320 to 305 feet, and Unit 3, which is subdivided into three zones, A, B and C, from elevations 305 feet to the top of rock. Unit 3A is located between elevations 305 and 295 feet and consists of dense sands and gravels interbedded with thin layers of clay. Between elevation 295 and 288 feet is Unit 3B which consists of a soft clay layer that appears to be continuous across the site. This clay layer is interbedded with thin layers of sand. Below elevation 288 feet are denser sands which make up Unit 3C. This part of the embankment is also keyed to the foundation with a trench that is about 23 feet in depth. A sheetpile cutoff was driven through the natural alluvium to rock and a grout curtain was constructed from stations 33+81L to 38+52L. Retaining walls were built upstream and downstream of the powerhouse, parallel to the direction of flow, to protect the embankment dam and its alluvial foundation from erosion. Figures 4 and 5 show sections of the sheetpile cutoff, grout curtain and retaining walls.

PART III: PRE-EARTHQUAKE CONDITIONS

Sections Analyzed

9. Stability analyses were performed on two sections of the dam, one for the main embankment and the other through the switchyard. A plan view showing the location of these sections is shown in Figure 6 and typical cross sections through the main embankment and switchyard are shown on Figures 2 and 3, respectively. Circular failures were assumed for the section through the main embankment. Geometry of the dam and foundation dictated that circular failures be assumed because the free field beyond the toe of the dam extends for a large distance and Unit 2 does not daylight as it does in the switchyard area. Critical wedge failure surfaces approximate a circular surface and therefore were not used. In the switchyard section a wedge type failure is assumed to occur, exiting into the tailrace channel. This section is curved section in plan, cutting through the embankment and curving toward the tailrace channel.

Material Properties

Embankment Material Properties

10. The material properties for the embankment and switchyard were determined from the results of tests reported for construction record samples (Reference 8) and from recent laboratory and in-situ tests performed on samples from borings made for the seismic analysis (see Volume 3). From field densities measured during the construction of the dam, the average moist and saturated unit weights were calculated to be 126 and 128 pounds per cubic foot, respectively. Strength test results were also reported for the samples taken for construction records and performed for samples from recent borings. Figures 7 and 8 show strength envelopes estimated from results of the triaxial tests performed on the embankment materials. The values were selected to represent the embankment strengths prior to the earthquake. Table 1 summarizes these parameters. The shear strength parameters for the random fill represent conservative values as described in the original Design Memorandum 3C.

Foundation Material Properties

11. In the stability analysis, the soil parameters for the three units of the foundation were determined from reported results of tests performed on samples obtained prior to construction of the dam and also from tests on samples

from the recent borings made in connection with the seismic analysis. Table 1 summarizes these parameters which represent the estimated strengths prior to the postulated design earthquake. Figures 9 and 10 give the strength envelopes estimated from results of the triaxial tests for Unit 1 (clay) of the foundation. Unit 2 , which is dominated by soft clays interbedded with thin layers of sand, is subdivided into these two materials (clay and sand), and the strength of each material is given in Table 1. Figures 11 and 12 give the strength envelopes estimated from the results of the triaxial tests performed on the soft clays and Figure 13 give the strength envelopes estimated from the results of the direct shear tests performed on the sands. No tests were performed on the dense sands and gravels of Units 3A and 3C on the right bank. However, tests were performed on samples taken in this zone on the left bank (Reference 10) and evaluations made in Design Memorandum 3C indicate that these materials were similar to Unit 3A and 3C. Therefore, these strength values were used and no additional tests were made. Tests were not performed on the soft clays in Units 3B and their strengths were assumed to be the same as those of the clays in Unit 2 because the Cone Penetration Test (CPT) values of both units were similar.

Piezometric and Pool Levels

12. The piezometric and normal pool levels are discussed in detail in Volume 3 of this series of reports and are briefly discussed in this Volume. For the two sections analyzed the upstream pool was assumed to be at elevation 360 feet. For the main part of the dam, a straight upper piezometric line was assumed to pass from the pool elevation through the dam to the drainage blanket and a ground water elevation of 345 feet was assumed beyond the downstream toe. In the switchyard section, the upper piezometric line of seepage was assumed to pass from the pool elevation, through the dam to the inclined drain, then down to the tailwater elevation of 305 feet. The corresponding piezometric lines for the two sections analyzed are shown in Figures 2 and 3.

PART IV: POST-EARTHQUAKE CONDITIONS

Main Embankment Cross Section

Embankment and Unit 1 Post-earthquake Strengths

13. No laboratory cyclic strength tests were performed on samples from the embankment or Unit 1 of the foundation. However, work by Ellis and Hartman (1967) and Thiers and Seed (1968) shows that a strength loss of between 10 and 20% can be expected for clayey materials whose peak cyclic strain is about half of its failure strain in a static test. Therefore, the assumption was made these materials would experience a 20% reduction in their strengths after the earthquake motions had ceased. Table 1 gives the reduced strength values.

Foundation Unit 2 Post Earthquake Strengths

14. General: As was discussed in Volume 4, the sand components (fine sand, silty sand and sandy silt) of Unit 2 of the foundation are the materials most susceptible to liquefaction, high strains, and severe strength loss. In the stratigraphy analysis (Volume 3) it was conservatively concluded that the sand components were continuous. The clay component in Unit 2 was determined to be non-liquefiable.

15. Strength of Liquefied Zone: The liquefaction analysis from Volume 4 indicates that liquefaction would occur both in the free field and under the dam, although the analysis indicates that liquefaction will not occur under the slopes of the embankment as shown in Figure 14. As discussed in Volume 4, the N_{1eff} (fines corrected blowcount used to determine the undrained residual strength of a liquefied soil) of this zone is 17.5 which corresponds to an estimated residual strength of 700 psf.

16. Strength of Non-Liquefied Zone: Although the materials in this zone are predicted not to undergo liquefaction, they will have some strength reduction due to the generation of earthquake induced excess pore pressure, ru, which is defined as the ratio of the excess pore pressure, u_e, to the effective overburden pressure, σ_v' ($r_u = u_e/\sigma_v'$). The liquefaction analysis indicated that the factors of safety were generally close to 1.1 in this zone. At a factor of safety of 1.1, excess pore pressures are expected to be greater than 50%. Therefore, it was assumed that in this zone a residual strength of 700 psf would also be used.

Foundation Unit 3 Post-Earthquake Strength

17. General: Liquefaction is not expected to occur under the main embankment in Unit 3, however liquefaction is expected to occur beyond the toes of the embankment between elevation 295 and 285 feet. Below elevation 285 feet liquefaction is not expected to occur. Under the main embankment the excess pore pressures in Unit 3 will result in a decrease in strength although not as severe as that in Unit 2. Figure 14 shows the location of the zones of liquefaction and the estimated boundaries between the zones of liquefaction and non-liquefaction.

18. Strength of Liquefied Zone: In Unit 3, the N_{1eff} is 25 blows/ft. As recommended in Volume 4, an estimated residual strength of 800 psf was assigned to Unit 3. The residual strength of Unit 3 is significantly higher than the residual strength of Unit 2.

19. Strength of Non-Liquefied Zone: The liquefaction analysis indicated that the factor of safety was 1.25 or greater except for small isolated zones. At a factor of safety of 1.25, excess pore pressures are expected to be about 30%. Therefore, an estimate of 50 percent was conservatively used for the excess pore pressures in the stability analysis.

Switchyard Cross Section

Embankment and Unit 1 Post-Earthquake Strength

20. The post earthquake strengths used for the main embankment and Unit 1 cross section were assumed to be the same for the switchyard section as described in Paragraph 13.

Foundation Unit 2 Material Properties

21. General: It was determined from the liquefaction analysis that liquefaction would occur in the free field and under portions of the switchyard area but not underneath the dam although underneath the dam excess pore pressures will also result in a decrease in strength. Figure 15 shows the locations of the zones of liquefaction and the estimated boundaries between the zones of liquefaction and non-liquefaction.

22. Strength of Liquefied Zone: The materials in the liquefied zone have an average N_{1eff} of 15.5 which corresponds to an estimated residual strength of 450 psf as discussed in Volume 4.

23. Strength of Non-Liquefied Zone: As established by criteria discussed in Volume 4, the strength of the non-liquefied zone will be controlled by the clays, and a strength to effective overburden pressure (c/p) ratio of 0.31 was assumed. Using the same rationale as in paragraph 13, this was reduced by 20%, for a c/p = 0.25.

Foundation Unit 3 Material Properties

24. General: As was discussed in Paragraph 8, in the switchyard area this unit has been subdivided into three smaller units, A, B and C. As in Unit 2, the liquefaction analysis also determined that liquefaction would occur in the free field and under some areas of the switchyard in Unit 3A and only in the free field in Unit 3C. Figure 15 shows the location of the zones of liquefaction and the estimated boundaries between the zones of liquefaction and non-liquefaction.

25. Strength of Liquefied Zone: In Units 3A and C, the N_{1eff} is 25.5 which, corresponds to a residual strength of 800 psf, significantly higher than that in Unit 2.

26. Strength of Non-Liquefied Zone: As was the case for Unit 2, the materials in Units 3A and C are predicted not to undergo liquefaction, although they will have some strength reduction due to generation of excess pore pressures. In Unit 3A the pore pressure ratios are expected to reach 35 percent beneath the center of the dam. In Unit 3C, the pore pressures ratios vary from 20 percent beneath the dam to 50% in the switchyard area. However, a value of 50 percent was used for the entire zone.

27. Strength of Unit 3B Clay: The clays in Unit 3B are expected to undergo large strains. No strength tests were performed on this material because none were sampled as they were not thought to be of any concern in the liquefaction analysis. As indicated in Volume 4 the results of the CPT program indicate that these materials behave like a normally consolidated clay. This would correspond to a strength to overburden ratio (c/p) of 0.31. Using the same rationale as in Paragraph 13, this was reduced by 20%, for a c/p value of 0.25.

PART V: STABILITY ANALYSIS

Method of Analysis

28. The stability analysis was performed using the computer program UTEXAS2. This program has four methods of analysis, Spencer's, simplified Bishop's, modified Swedish, and Lowe and Karafiath's from which to select. Spencer's method was used in this study as it satisfies complete static equilibrium for each slice and it also has the capability of computing factors of safety for both circular and planar surfaces. It was assumed that the embankment had reached a steady state seepage condition when the earthquake occurs which corresponds to a consolidated undrained condition for laboratory analysis.

Pre-Earthquake Embankment Stability

29. For comparison purposes, the stability of both the main embankment and the switchyard section were evaluated using the soil parameters listed in Table 1 to arrive at the final minimum failure surfaces as discussed in the following paragraphs. The results are shown in Table 2.

Post-Earthquake Embankment Stability

30. The problems of predicting or estimating deformations of an embankment following liquefaction of the foundation are difficult and not well defined. Predicting deformations which occur due to liquefaction and the effects of both static and inertial forces acting on an embankment are problems that are probably beyond the current state-of-the-art in geotechnical engineering. Analogy and empiricism were used to determine the residual strengths in the foundation. Reasonable assumptions regarding the strain levels required for liquefaction were used to estimate the deformations in the embankment cross-sections. However, deformations in this case were estimated by analogy to observed embankment and foundation deformations reported by Seed, Lee, Idriss and Makdisi (1975) and Seed (1987).

Conditions and Assumptions of Analysis

31. The stability and deformations of the embankment for Barkley Dam were therefore evaluated based on the following conditions.

32. Liquefaction (defined as a condition where the pore pressure ratio, ru = 100 percent) of the foundation occurs near the end of the earthquake. This is assumed to occur when the computed factor of safety against liquefaction is close to one and the results of the analysis given in Volume 4 shows the zones where it will occur. Only static stresses will be acting on the embankment and deformations can be estimated for this condition.

33. The entire critical zone defined by the liquefaction analysis is assumed to have liquefied (see Figures 14 and 15 for location of liquefied zones). This is conservative, as explorations and the downstream river bank exposure indicates that this zone is dominated by soft clays, interbedded with thin layers of sand, which were assumed to be continuous. Liquefaction was also assumed to have occurred in the free field beyond the switchyard area between elevation 305 feet and 295 feet and below elevation 288 feet in the switchyard area.

34. For the main embankment section, circular failure surfaces were assumed. Assuming that the entire identified foundation zone has liquefied is conservative as the failure circles must pass through the soft clays.

35. In the switchyard section, it was assumed that a continuous sand layer can exist at any elevation interval in the liquefied zone. A wedge type of analysis was used in this area and a failure plane was assumed to occur through the embankment and along the sand layer exiting into the tailrace canal.

Procedure

36. Evaluating the stability of the embankment under the above conditions can be complex. Accordingly, Seed (1987) proposed the procedure in the following paragraphs for evaluating the stability of structures after liquefaction has occurred in the foundation.

37. Assume first that the full residual strength of the liquefied soil is mobilized. If the computed factor of safety is less than or close to 1.0, then sliding and unacceptably large deformations are expected. For Barkley Dam this would be failure of the dam and loss of the reservoir.

38. If, in the condition described in Paragraph 37, the safety factors against sliding with full residual strength are greater than 1.0 and failure of the dam does not occur, then assume that the strength in the liquefied zone is zero. If, using zero strength in the liquefied zone, the factor of safety from

a stability analysis is significantly greater than one (a factor of safety of 1.2 is considered "significantly greater" for Bar'.ley for this case), then the stability of the embankment is controlled by the nonliquefied soil and the deformations of the embankment will be small (i.e. less than 5 to 6 feet).

39. If, in the condition described in Paragraph 38, the factor of safety is not significantly greater than one (i.e. 1.2), then the residual strength required to be mobilized to produce a stable condition (a stable condition is defined as a condition having a factor of safety of 1.2) should be computed. If the residual strength estimated from empirical or laboratory methods is less than the residual strength required, then large scale deformations will occur and it is not possible tc accurately predict the final configuration of the embankment. If, however, the estimated residual strength is sufficient to produce a stable condition, then the shear strain which would have to develop in the liquefied soil in order to mobilize this resistance could be estimated. Knowing this strain, the potential deformation of the embankment could be evaluated. This can then be compared to the available freeboard of 28 feet.

Results of the Analysis

40. Based on the assumptions and procedure outlined in Paragraphs 31 through 39, slope stability analyses were performed on the two typical sections. Table 3 gives the strengths of the materials used.

41. Main Embankment: When a zero strength was assumed in the critical zone as defined by the liquefaction analysis, the resulting minimum factors of safety for the upstream and downstream slopes were both 0.7 with the minimum circles tangent to a plane at elevation 300 feet (see Figure 16). Using the full residual strength of 700 psf for the liquefied soil produced factors of safety of 1.3 for both the upstream and downstream slopes (see Figure 17). (By comparison, the pre-earthquake safety factors for both the upstream and downstream failure surfaces were 3.2). Thus, according to Paragraph 37, sliding and large scale deformations are not expected to occur however, both the upstream and downstream portion of the main dam are expected to undergo large strains. Seed has estimated that the strains required to mobilize the full residual strength are about 25% (See Seed's letter dated February 3, 1986 in Appendix A of Report 1).

42. <u>Switchyard Section</u>: Because liquefaction can occur in Units 2, 3A and 3C, stability analyses were performed on failure planes at elevations of 305, 295 and 288 feet (the search routine on the computer program found that the critical failure planes for this units corresponded to the base elevations of the these Units). As outlined in Paragraph 37, the full residual strength was used for the liquefied zones and the minimum factors of safety were determined for the failure planes at the three elevation intervals. The minimum failure planes are shown in Figures 18-23.

43. Using the full residual strength of 450 psf, the minimum failure plane occurs at elevation 305 feet with factors of safety of 1.6 and 1.8 for the downstream and upstream slopes, respectively. Therefore, large scale movements and deformations are not anticipated in this area. However, using zero strength in the critical zone for failure plane elevations of 305 feet will result in factors of safety less than one for both the upstream and downstream conditions (0.8 and 0.7, respectively). Analyses were then performed assuming that the dam has strained and the residual strength required to produce a stable condition (factor of safety = 1.2) was determined. For the minimum failure planes at this elevation, a residual strength of about 200 psf is needed to produce a stable condition for the upstream slope and 250 psf for the downstream slope, which is less than the estimated maximum residual strength of 450 psf of the soil. The pre-earthquake safety factors for the upstream and downstream surfaces are 4.4 and 5.6, respectively. Figures 18-19 shows the location of the failure surfaces and Table 2 summarizes the results.

44. Using the full residual strength of 1200 psf, the minimum failure surface at elevation 295 feet produced factors of safety of 3.8 and 2.6 for the upstream and downstream slopes, respectively, indicating that large scale movement and deformations are not anticipated. Using zero strengths in the liquefied zones produced factors of safety of 0.5 and 0.6 for the respective failure surfaces. For the minimum upstream and downstream failure planes, a residual strength of 200 and 250 psf, respectively, is needed for a stable condition. The pre-earthquake safety factors for the upstream and downstream surfaces are 4.4 and 7.2, respectively. The minimum failure surfaces are shown in Figures 20 and 21 and Table 2 summarizes the results.

45. Using the full residual strength of 1200 psf, the minimum failure surface at elevation 288 feet produced factors of safety of 2.7 and 3.3 for the upstream and downstream slopes, respectively, indicating that large scale movement and deformations are not anticipated. Using zero strengths in the liquefied zones produced factors of safety of 0.5 and 2.5 for the respective failure surfaces. Because of the large non-liquefied zone under the switchyard, it contributed a large portion of the strength along the minimum failure plane and a high factor of safety. On the upstream side, the entire zone at this elevation was considered to have liquefied and with zero strength produced a safety factor less than one. For the minimum upstream failure plane, a residual strength of 200 psf is needed for a stable condition. The pre-earthquake safety factors for the upstream and downstream surfaces are both 5.2. The minimum failure surfaces are shown in Figures 22-23 and Table 3 summarizes the results.

46. Estimated Deformations: As mentioned previously, strains of 20 to 25 percent are required to mobilize the full residual strength. Since the thickness of the liquefied zone of Unit 2 along the main embankment is 25 feet and the zone consists of 20 percent sand, then 2 to 3 feet of horizontal movement can be expected. In the switchyard area where the primary zone of liquefaction is 15 feet thick and contains 20 percent sand, the expected horizontal deformations will be about 1 to 2 feet.

47. For both sections, the expected vertical deformations should be about of about the same order of magnitude or smaller as those for the horizontal component. The vertical movements can be attributed to other failure mechanisms activated by the earthquake such as bearing capacity and settlement.

Conclusions

48. Stability analyses were performed on two sections of the dam, one representing the main portion of the embankment and the second through the switchyard area, exiting into the tailrace channel. These stability analyses were based on procedures suggested by Seed (1987).

The results of this analysis indicate that wide scale deformations or slope failure which would result in loss of the reservoir are not expected. Deformations on the order of 2 to 3 feet can be expected on the slopes of the main portion of the dam (reference February 1986 letter from Dr. Seed, Appendix A). In the switchyard area deformations of about 1 to 2 feet can be expected, but loss of the reservoir will not occur. These estimated deformations are relatively small in light of the fact that a freeboard of 28 feet is expected to be available at the time of the earthquake.

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12. Wright, Stephen G. and Edris, Earl V. Jr. 1987. "User's Guide: UTEXAS2 Slope Stability Package," US Army Engineer Waterways Experiment Station, Instructional Report GL-87-1, Volume 1, Vicksburg, MS.

TABLE 1

PRE-EARTHQUAKE SOIL PARAMETERS

SOIL TYPE	UNIT WEIGHTS (PCF) MOIST	UNIT WEIGHTS (PCF) SAT	R STRENGTHS C (PSF)	R STRENGTHS PHI	S STRENGTHS C (PSF)	S STRENGTHS PHI
EMBANKMENT AND SWITCHYARD	126	128	1000	22	0	26.5
RANDOM FILL*	126	128	400	8.5	0	14.
UNIT 1 - CLAY	115	125	1200	15	600	22.
UNIT 2 - CLAYS	122	126	700	14	0	31.
SANDS	122	126	----	----	0	31.
UNIT 3A - DENSE SANDS AND GRAVELS*	126	128	200	35	300	35.
UNIT 3B - CLAYS	122	126	700	14	0	31.
UNIT 3C - DENSE SANDS AND GRAVELS*	126	128	200	35	300	35.

* INDICATES ESTIMATED VALUES FROM DESIGN MEMORANDUM - 3C.

TABLE 2

SUMMARY OF STABILITY ANALYSES

	Factors of Safety						Residual Strength Required (psf)	
	Preearthquake		Full Residual		Zero Strength			
	U/S	D/S	U/S	D/S	U/S	D/S	D/S	U/S
Main Embankment	3.2	3.2	1.3	1.3	0.7	0.7	---	---
Switchyard El 305	4.4	5.6	1.8	1.6	0.8	0.7	200	200
El 295	4.4	7.2	3.8	2.6	0.5	0.6	200	250
El 288	5.2	5.2	2.9	3.3	0.5	2.5	200	---

Note: Columns 2-7 are the factors of safety for the conditions given. The last two columns represent the residual strength in psf required to produce a factor of safety of 1.2 or greater.

TABLE 3

PARAMETERS USED IN POST EARTHQUAKE STABILITY ANALYSIS

SOIL TYPE	UNIT WEIGHTS (PCF) MOIST	SAT	STRENGTH C (PSF)	PHI	EXCESS PORE PRESSURE
EMBANKMENT AND SWITCHYARD	126	128	800	18	
RANDOM FILL	126	128	320	6	
UNIT 1 - CLAY	115	125	960	12	
UNIT 2 - LIQUEFIED ZONE RESIDUAL STRENGTH					
SWITCHYARD AREA	122	126	450	0	
MAIN EMBANKMENT AREA	122	126	700	0	
UNIT 2 - NON-LIQUEFIED ZONE	122	126	0.25P		
UNIT 3A - LIQUEFIED ZONE RESIDUAL STRENGTH	126	128	800	0	
UNIT 3A - NON-LIQUEFIED ZONE	126	12[illegible]	0	31	35%
UNIT 3B - CLAY	122	126	0.25P		
UNIT 3C - LIQUEFIED ZONE RESIDUAL STRENGTH	126	128	800	0	
UNIT 3C - NON-LIQUEFIED ZONE	126	128	0	35	50%

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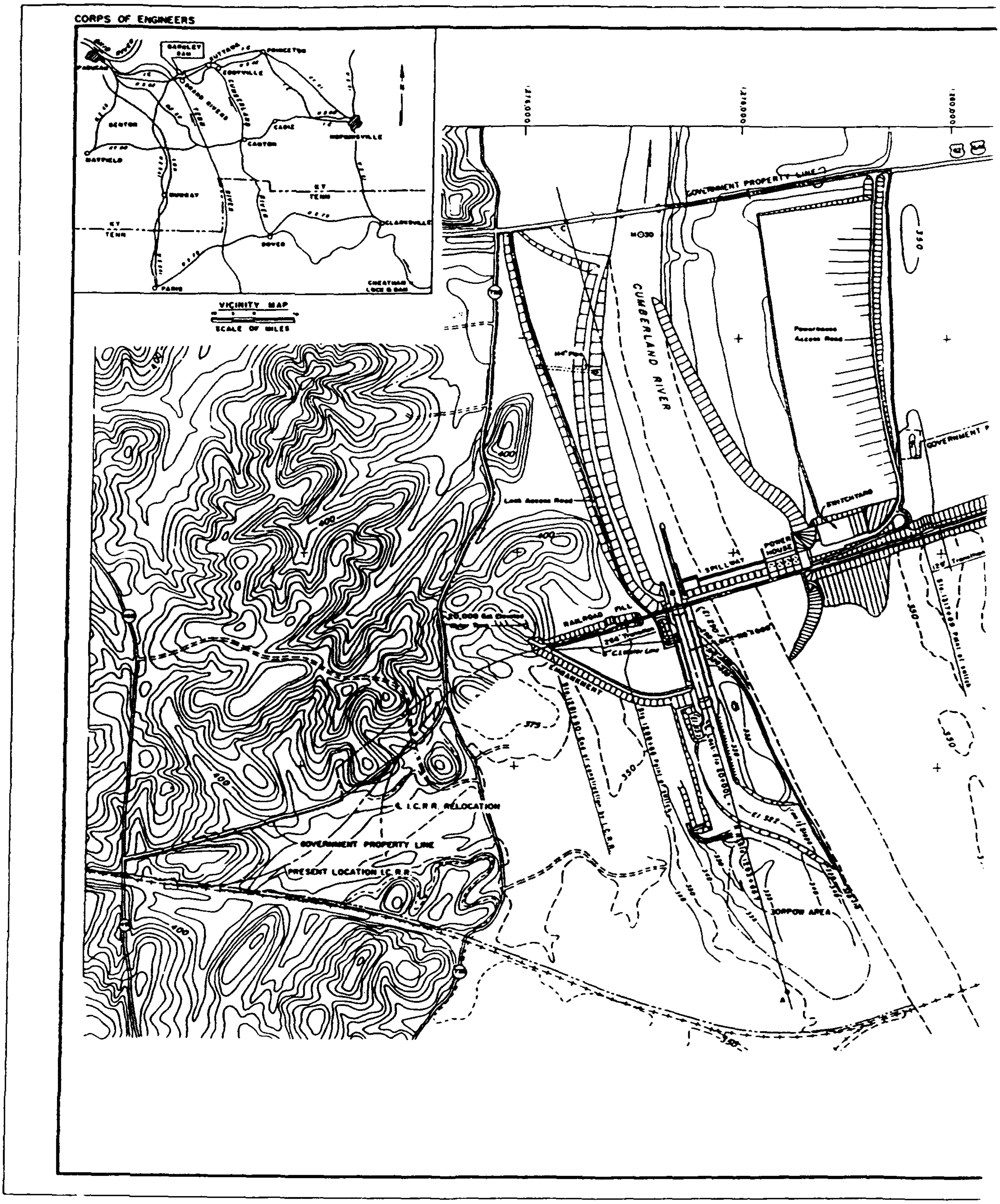
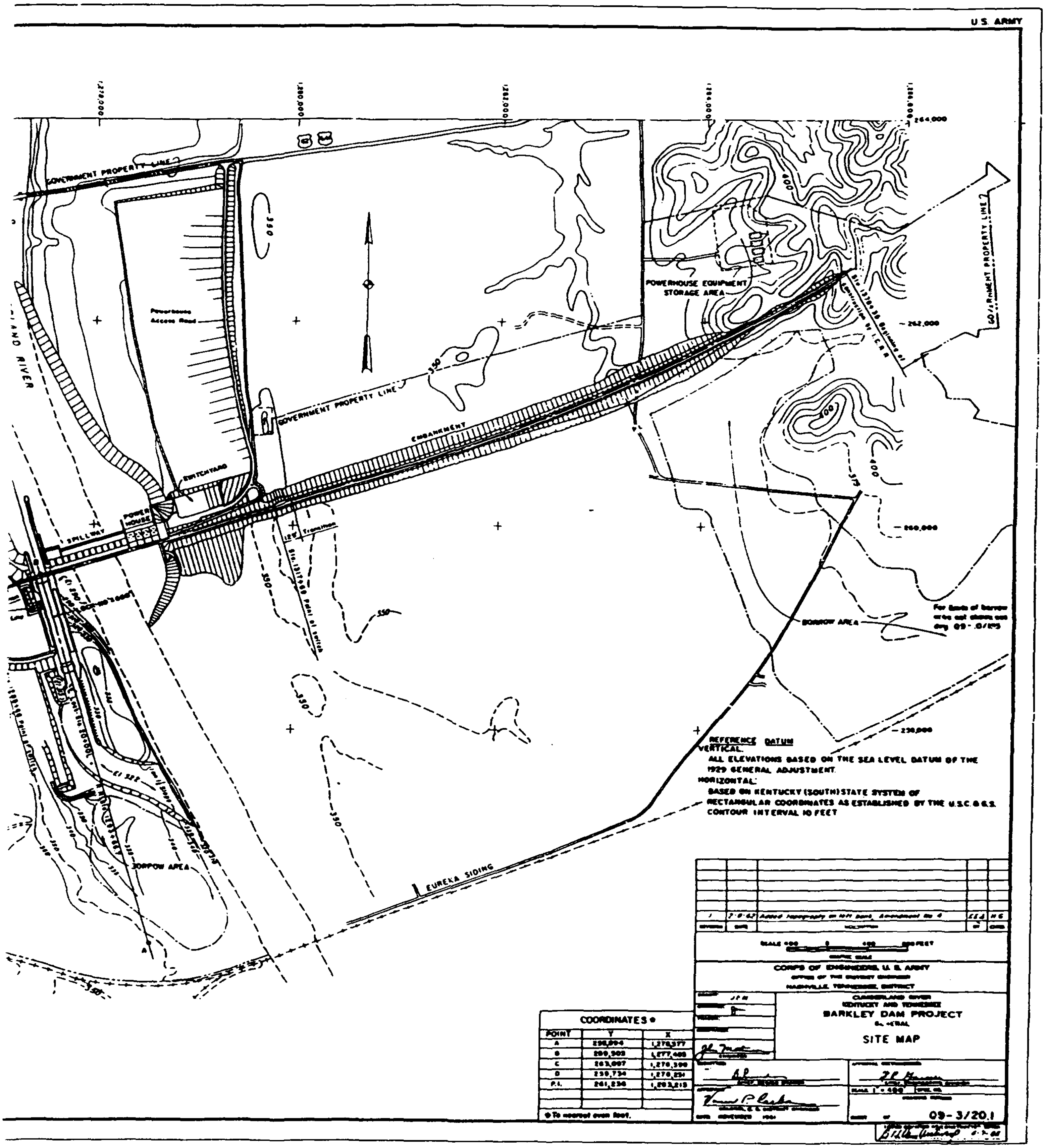


Figure 1. Plan view of the Barkley Lake



ı view of the Barkley Lake and Dam Project.

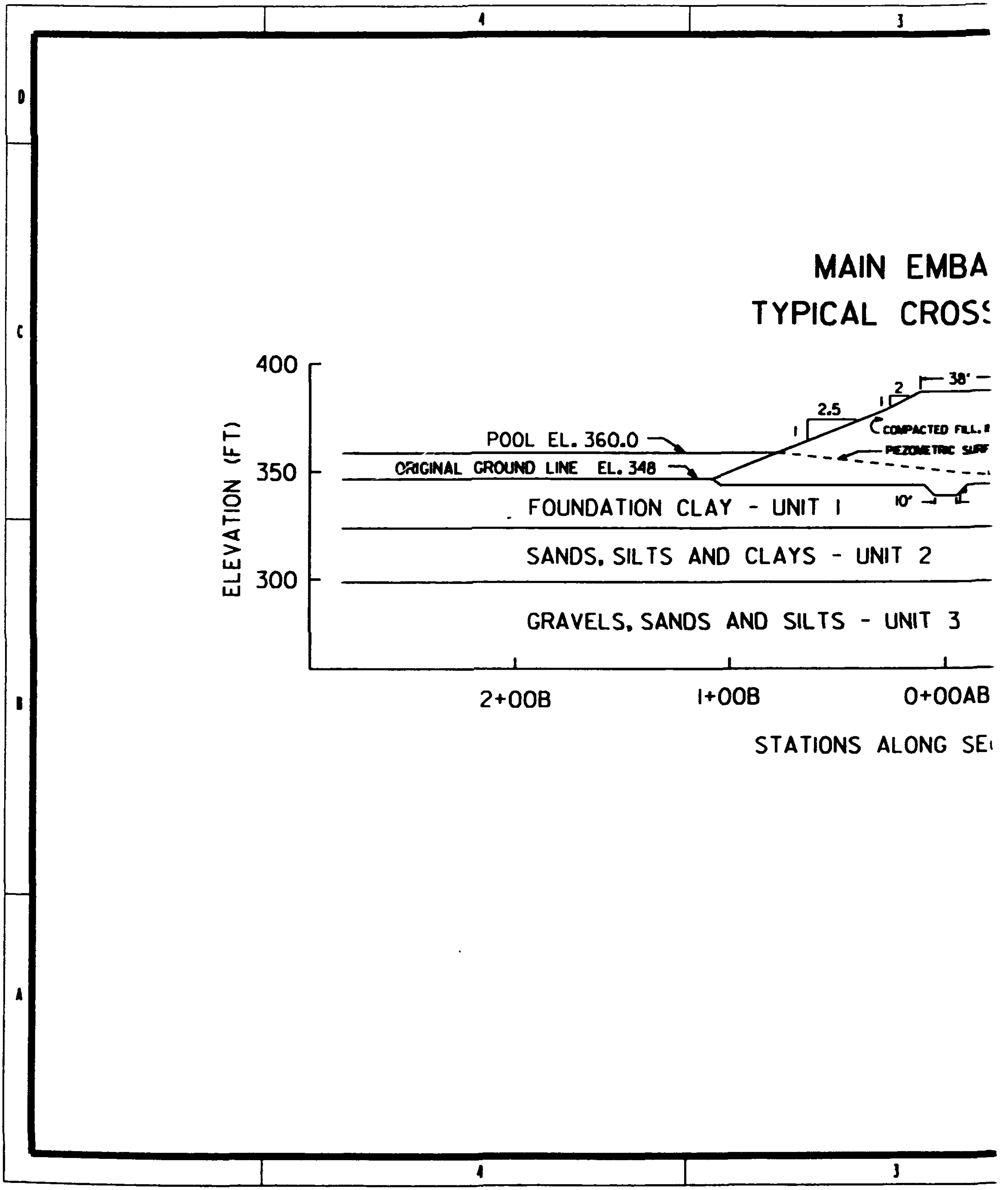
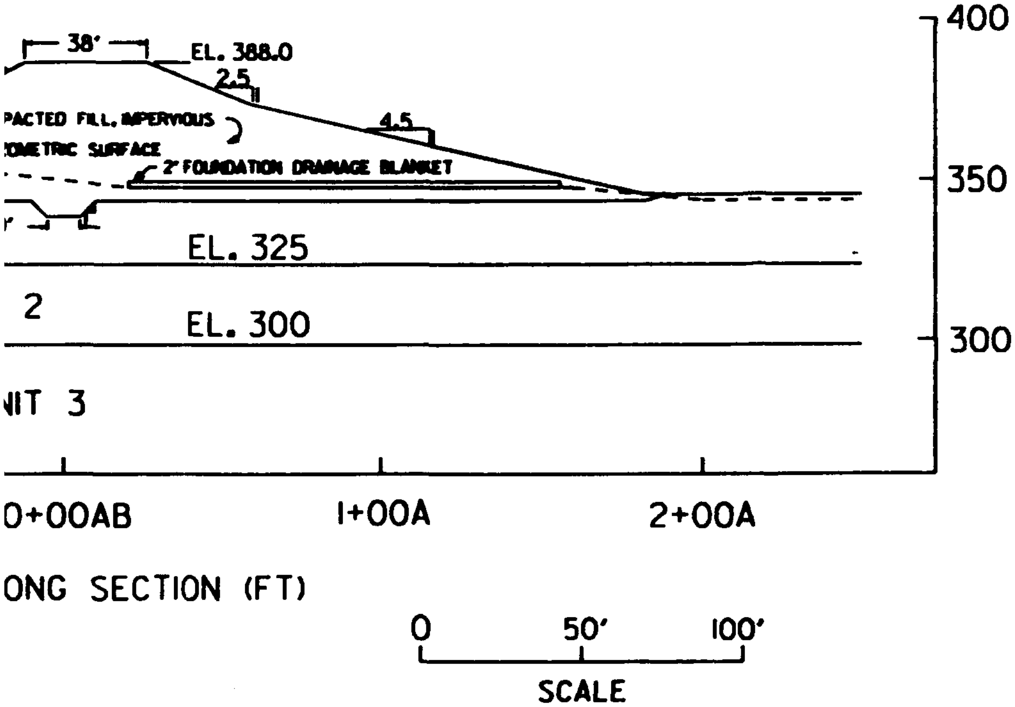


Figure 2. Main embankment - t

EMBANKMENT
CROSS SECTION



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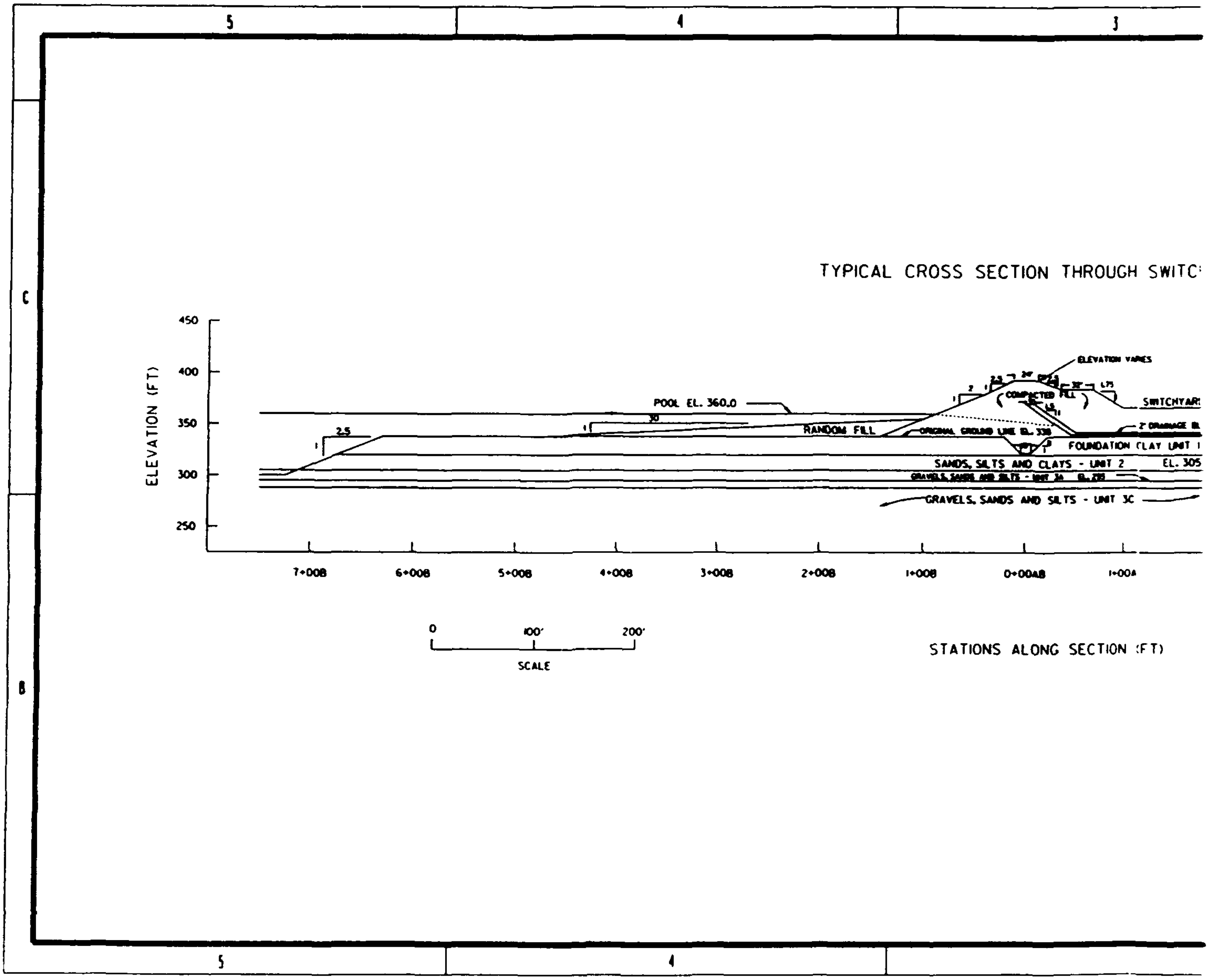
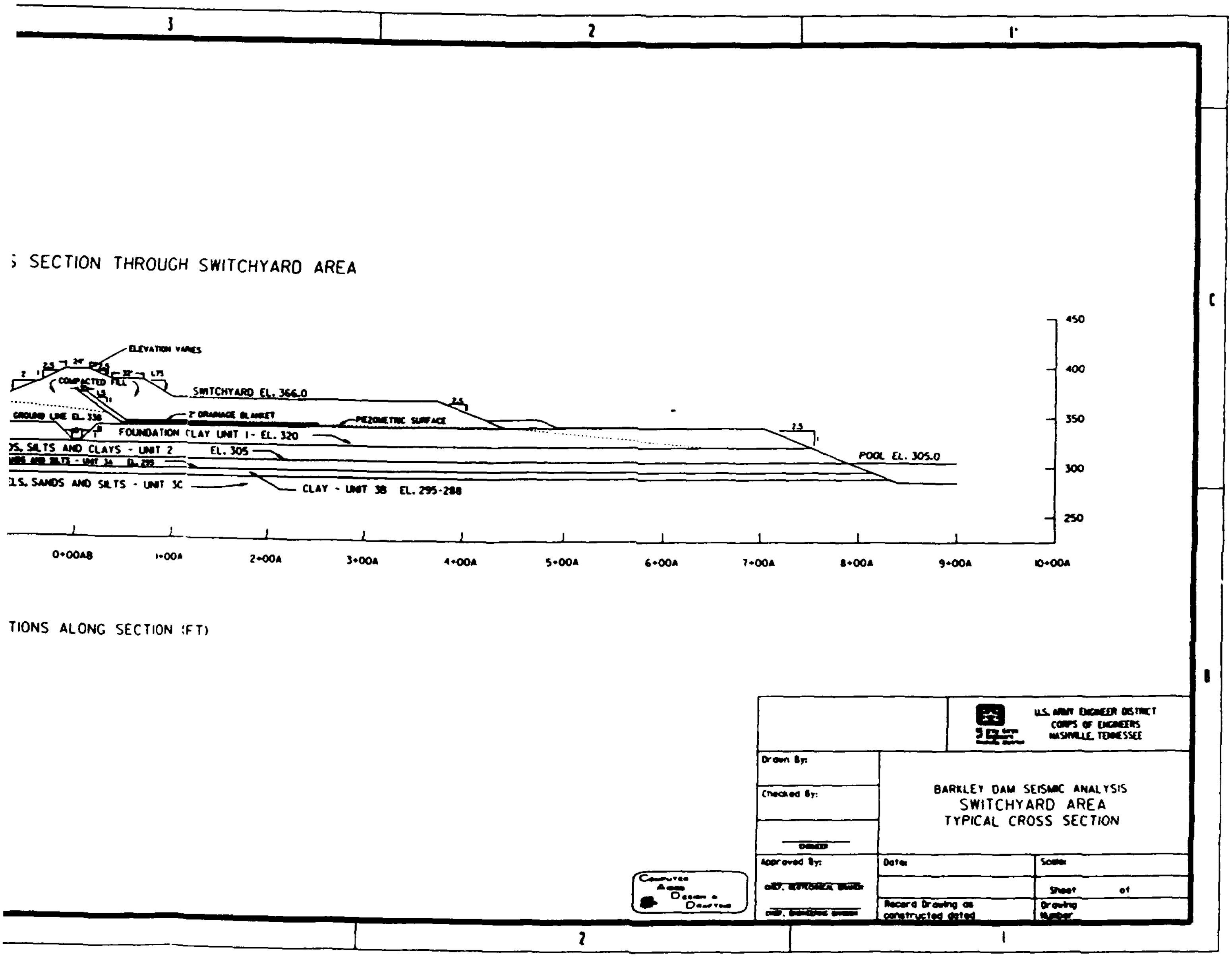


Figure 3. Typical cross sec



ypical cross section in switchyard area.

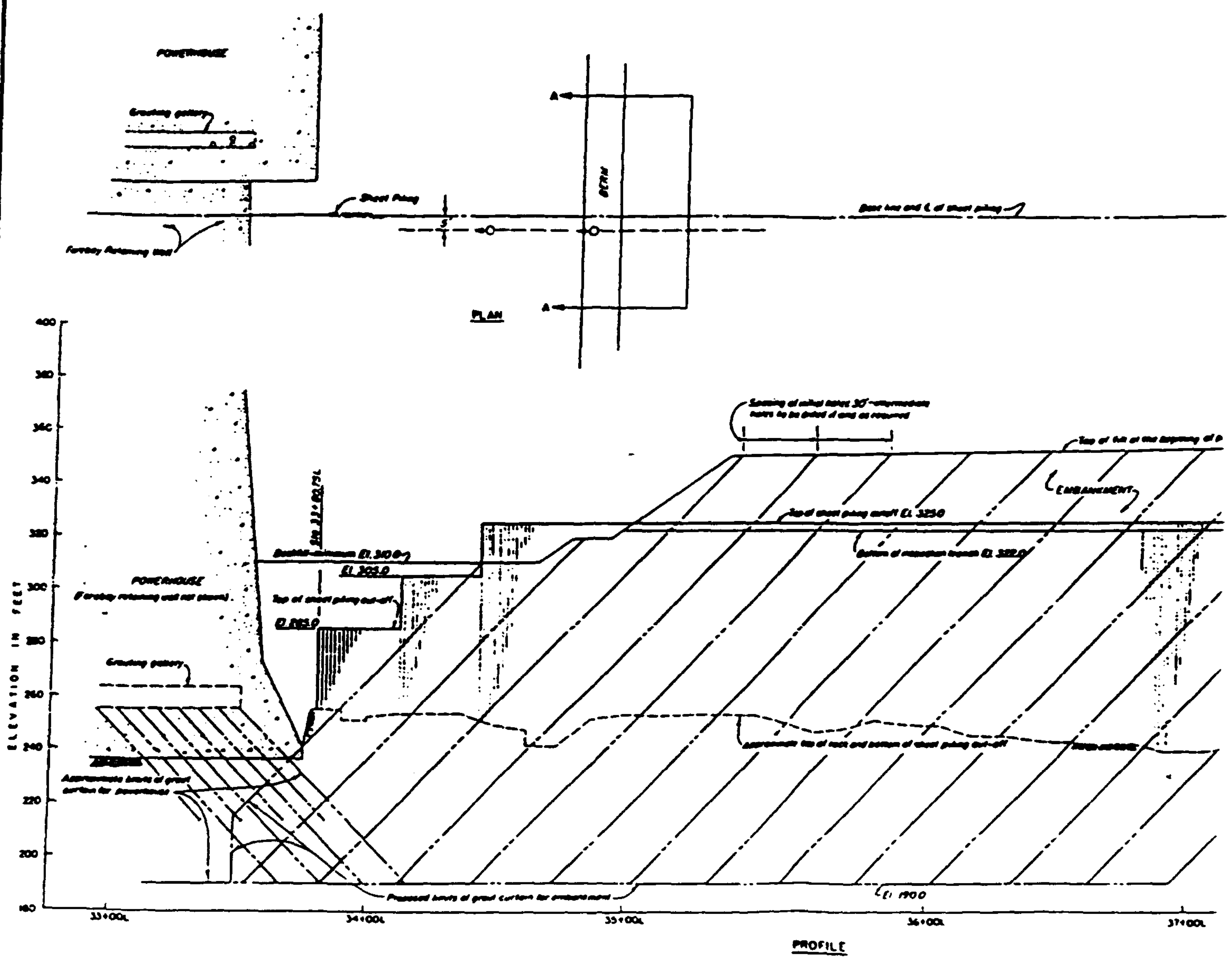
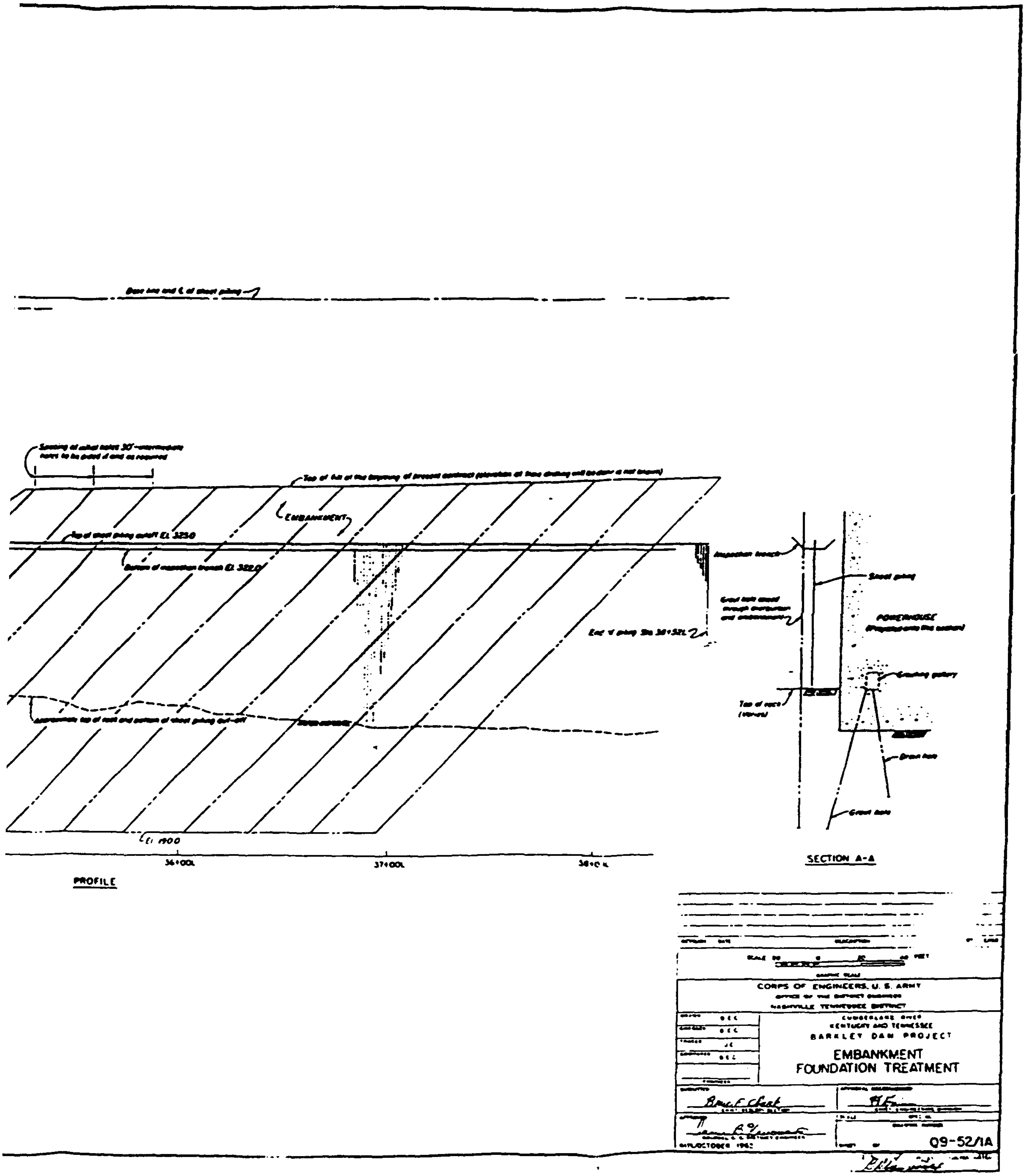


Figure 4. Details of grout curtain and sheetpile wal switchyard section.



grout curtain and sheetpile wall in the switchyard section.

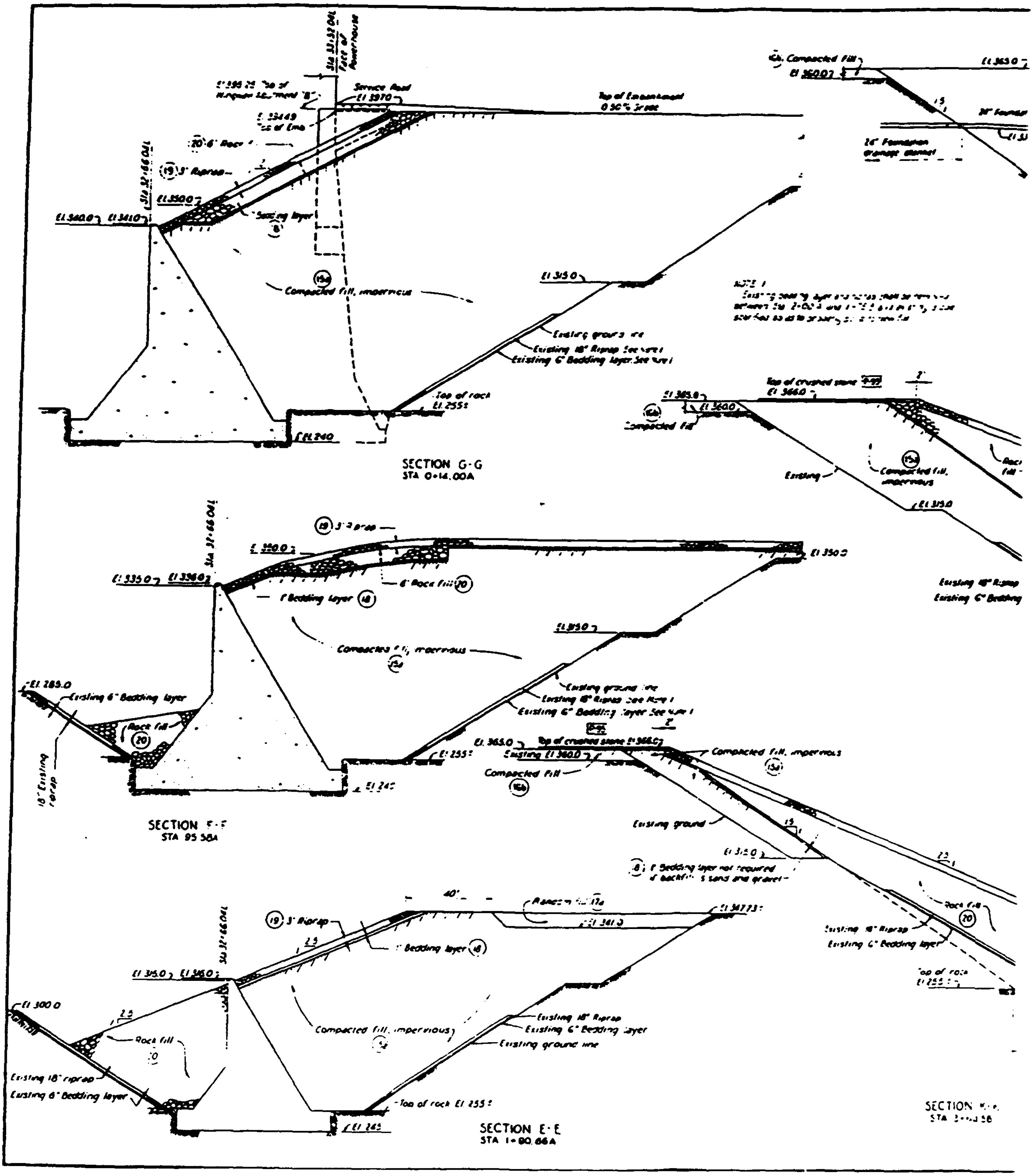
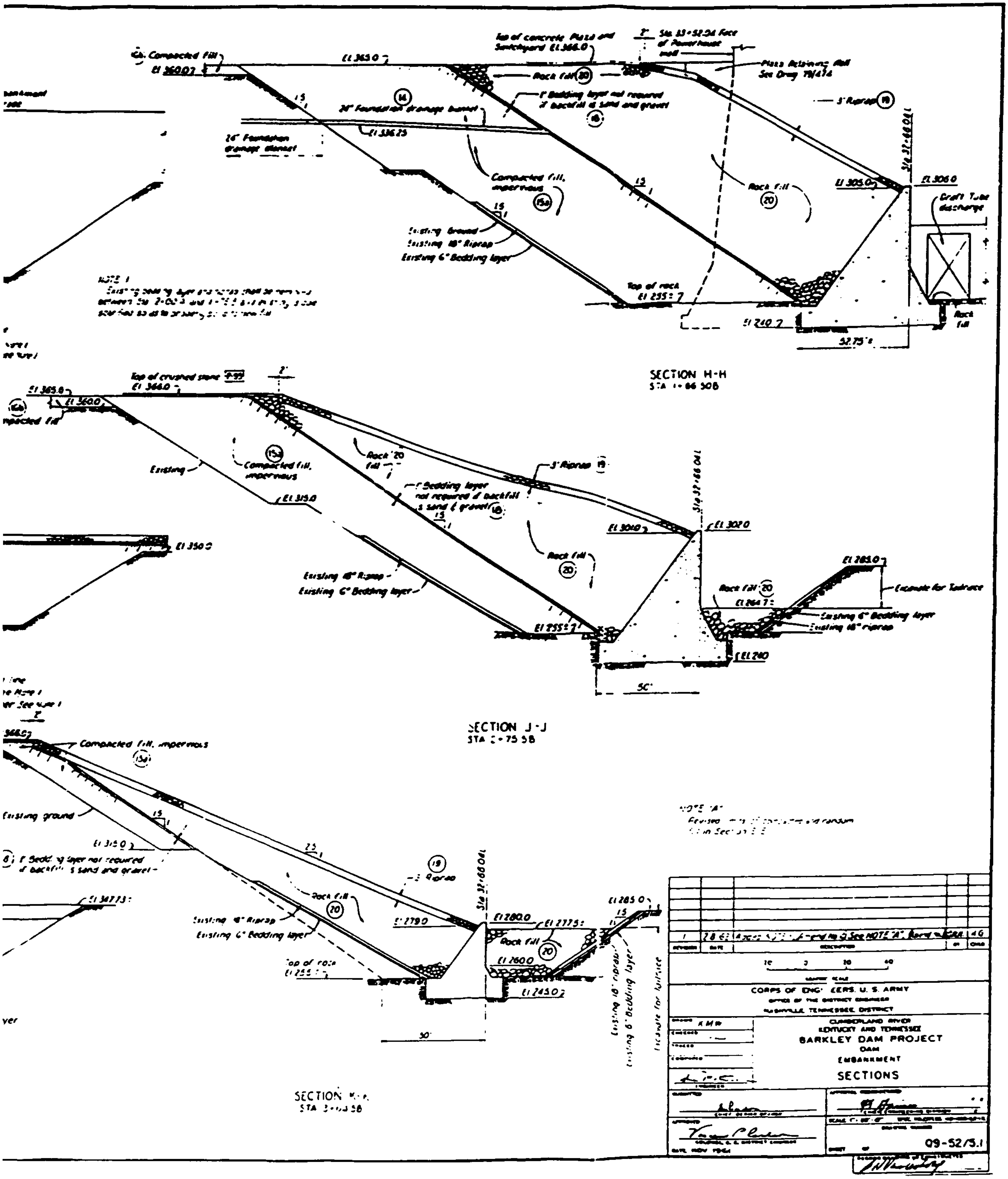


Figure 5. Upstream and downstream retaining walls ne



d downstream retaining walls near powerhouse.

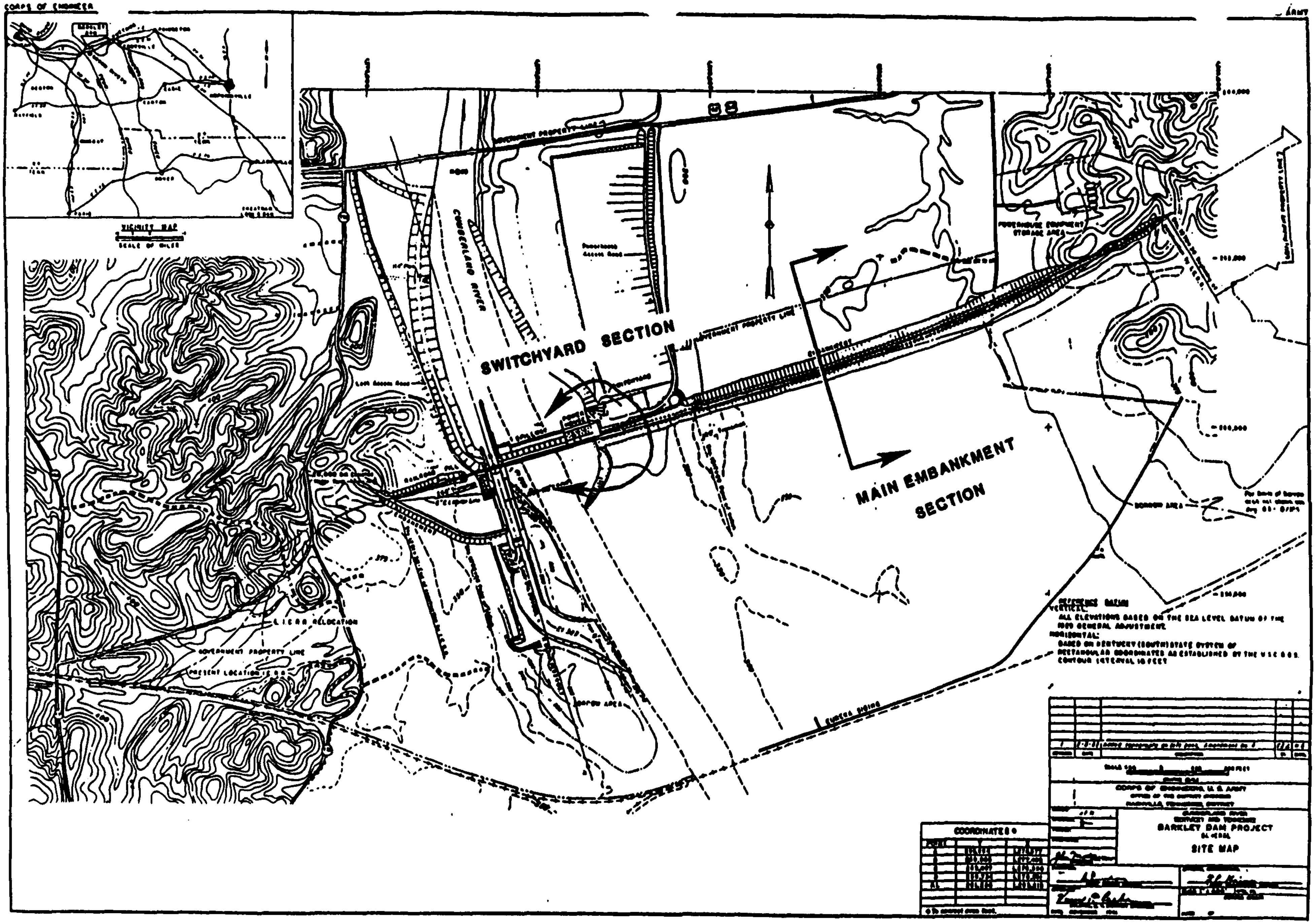


Figure 6. Plan view showing the locations of the analysis sections.

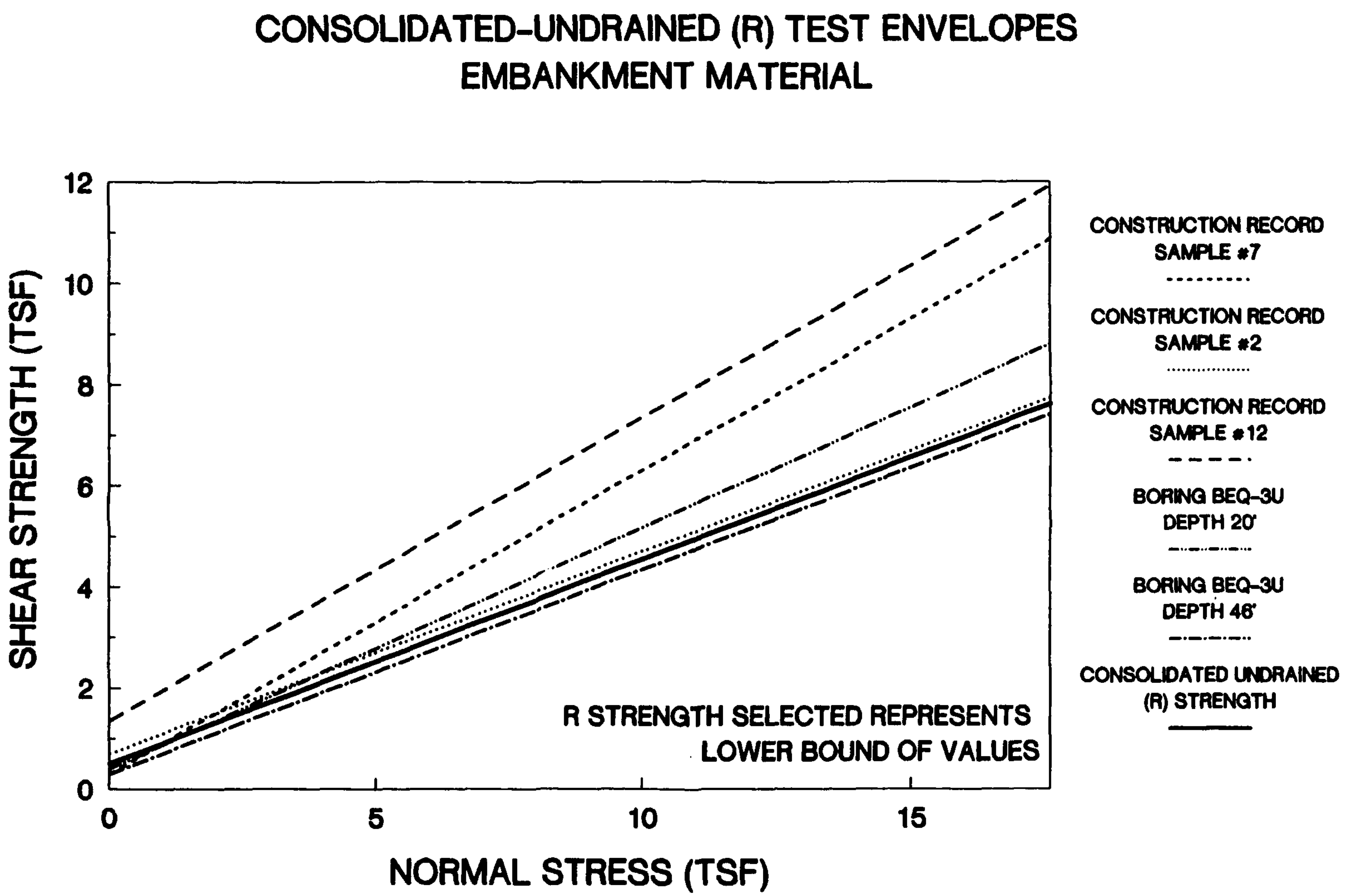


Figure 7. Consolidated - undrained (R) test envelopes for the embankment material.

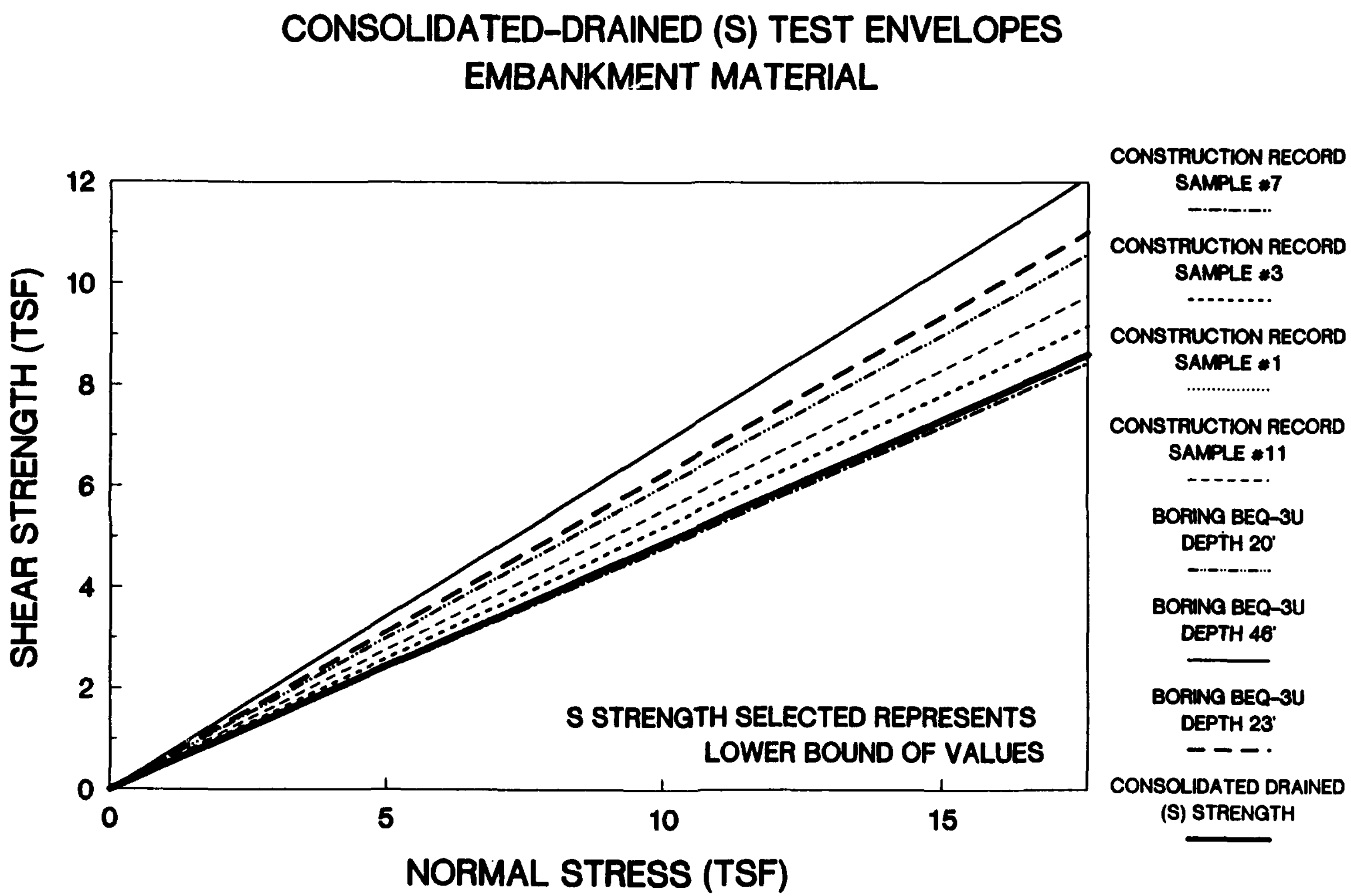


Figure 8. Consolidated - drained (S) test envelopes for the embankment material.

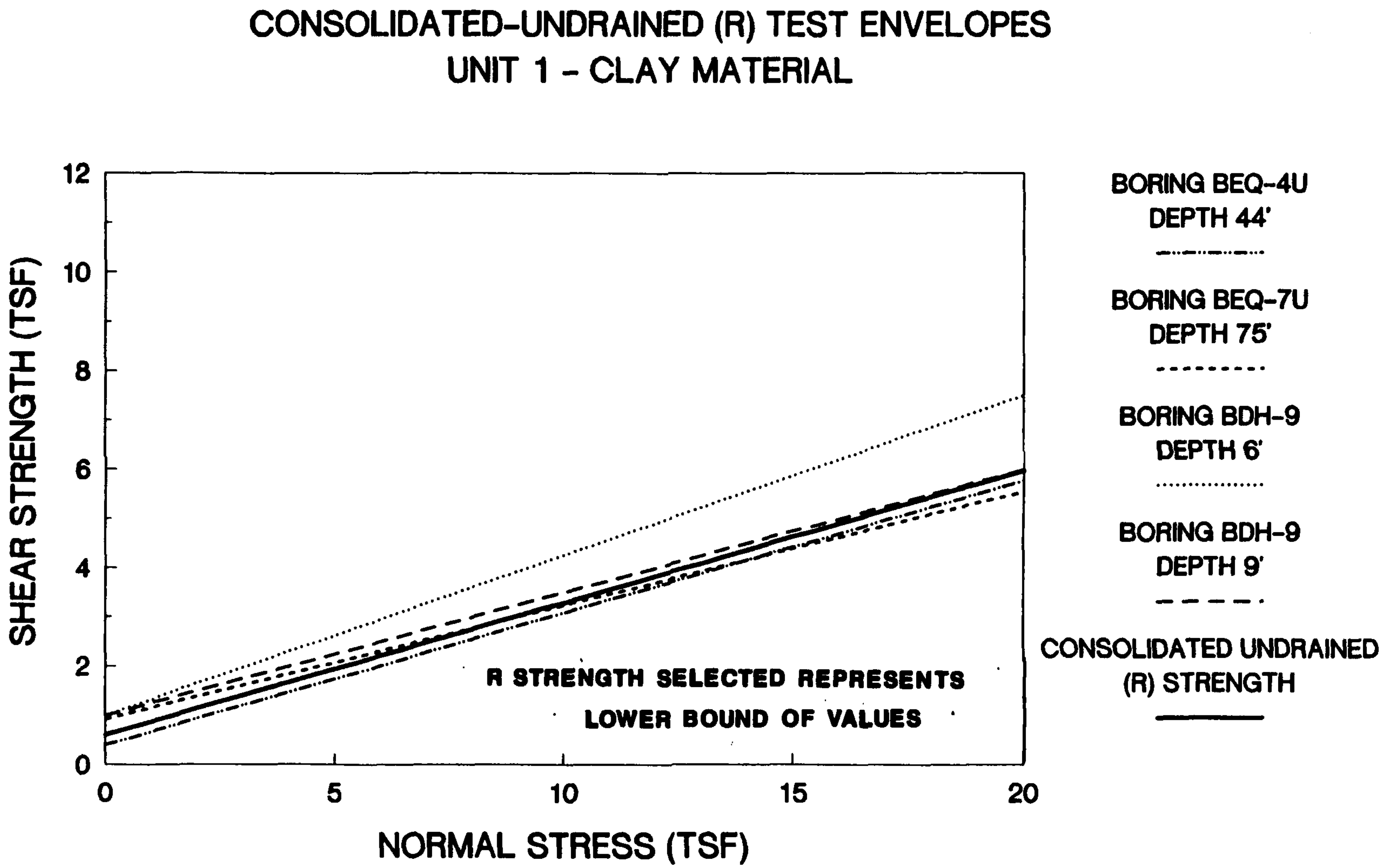


Figure 9. Consolidated - undrained (R) test envelopes for Unit 1 clay material.

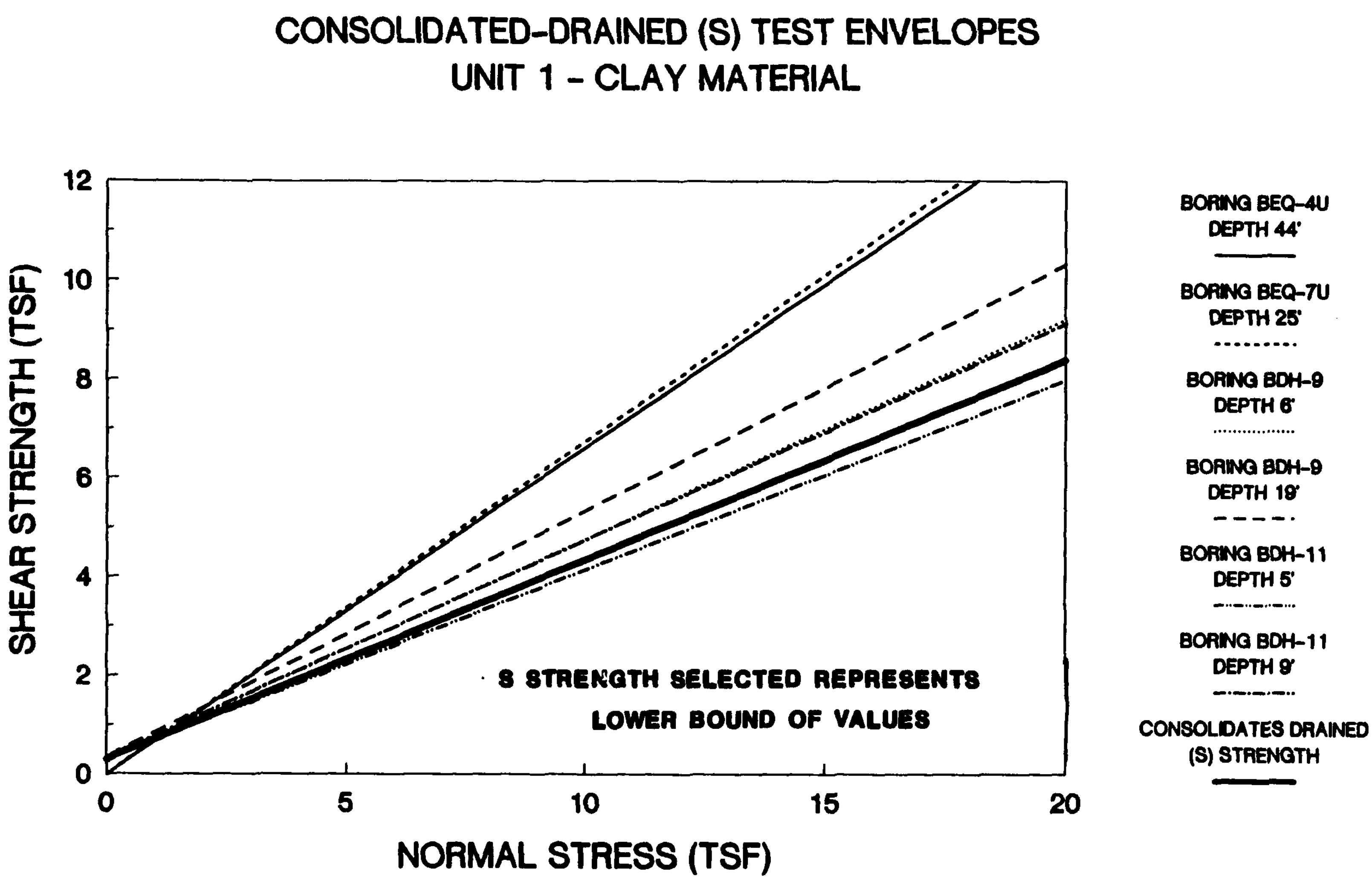


Figure 10. Consolidated - drained (S) test envelopes for Unit 1 clay material.

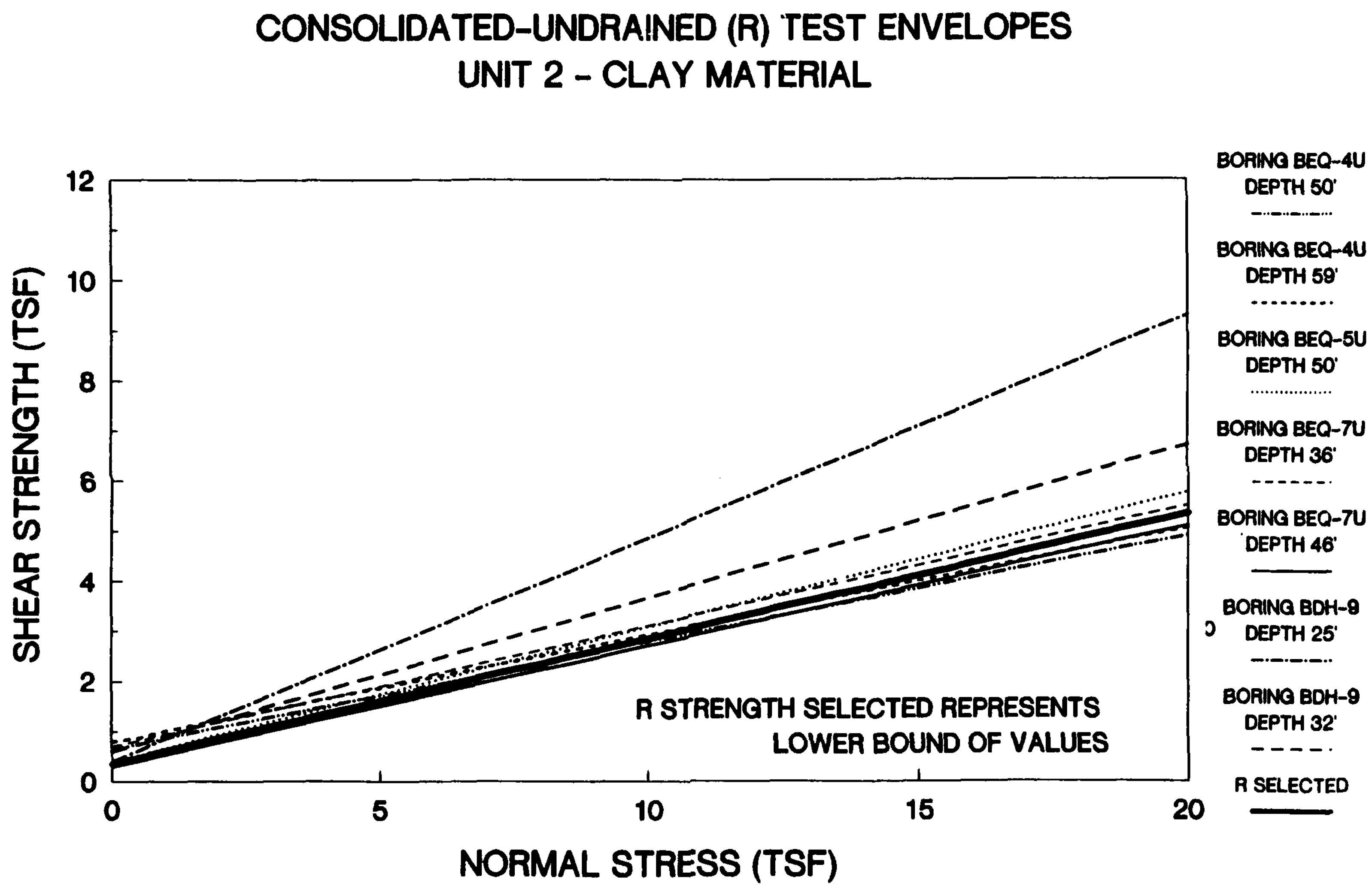


Figure 11. Consolidated - undrained (R) test envelopes for Unit 2 clay material.

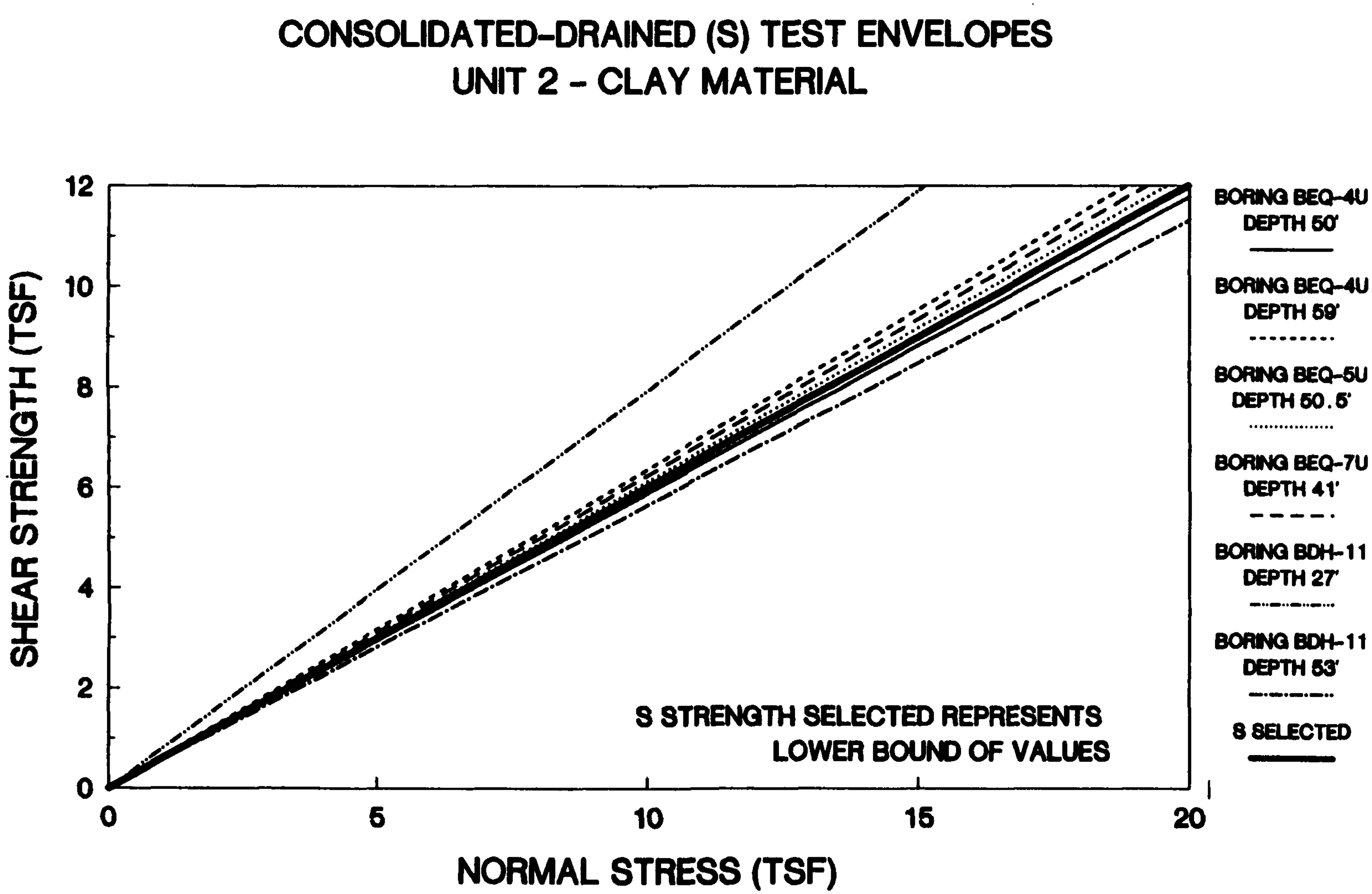


Figure 12. Consolidated - drained (S) test envelopes for Unit 2 clay material.

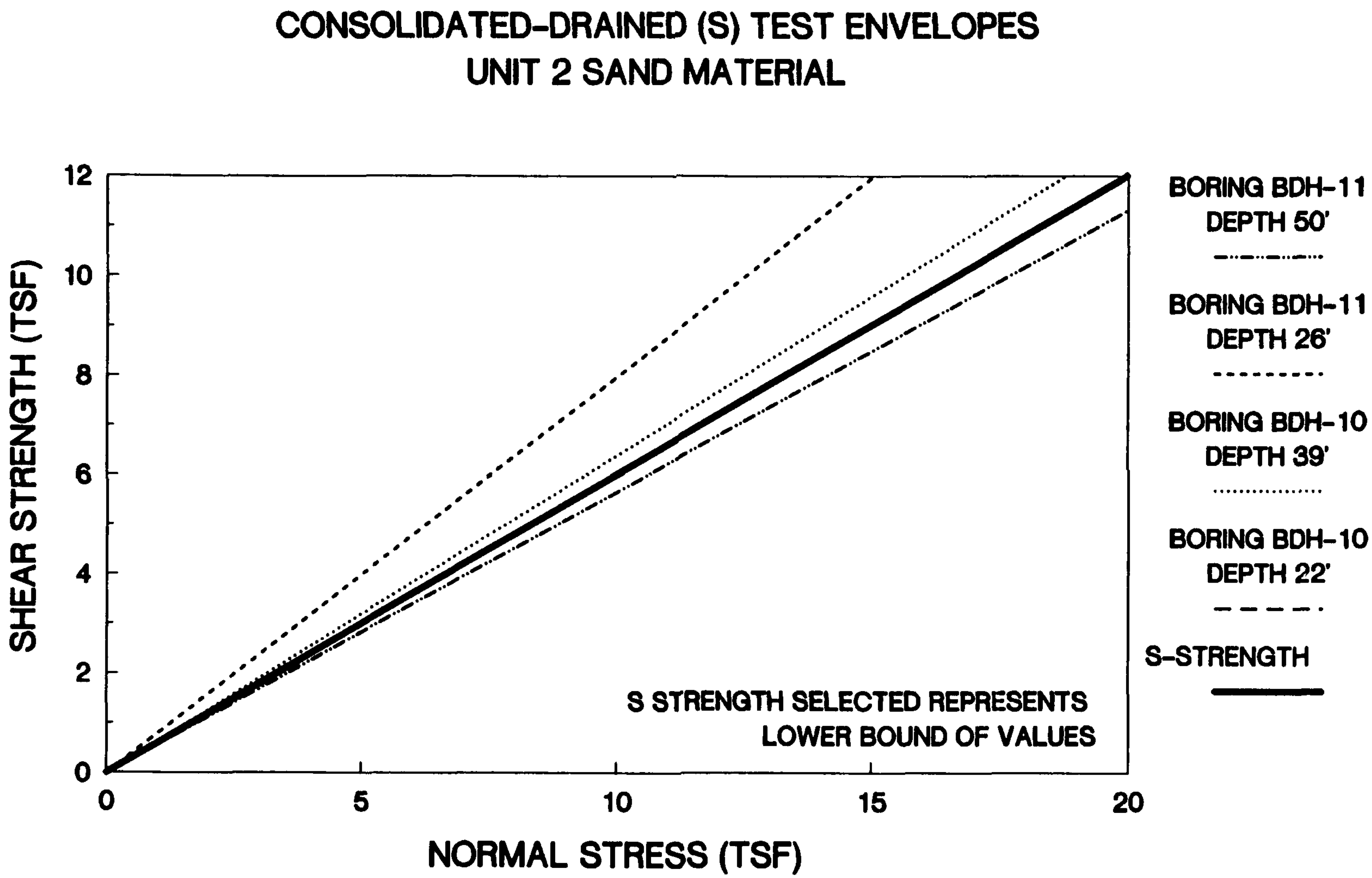


Figure 13. Consolidated - drained (S) test envelopes for Unit 2 sand material.

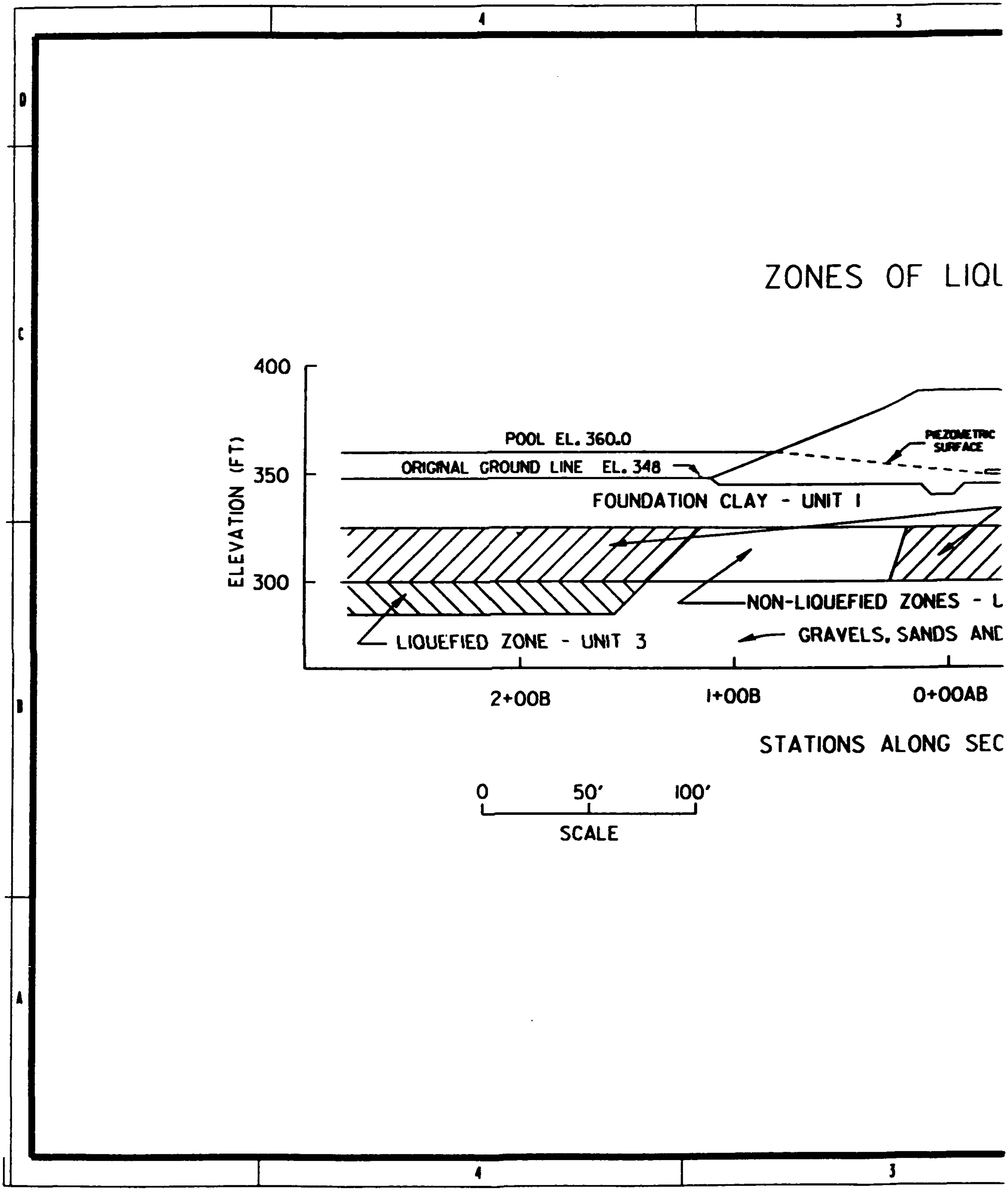
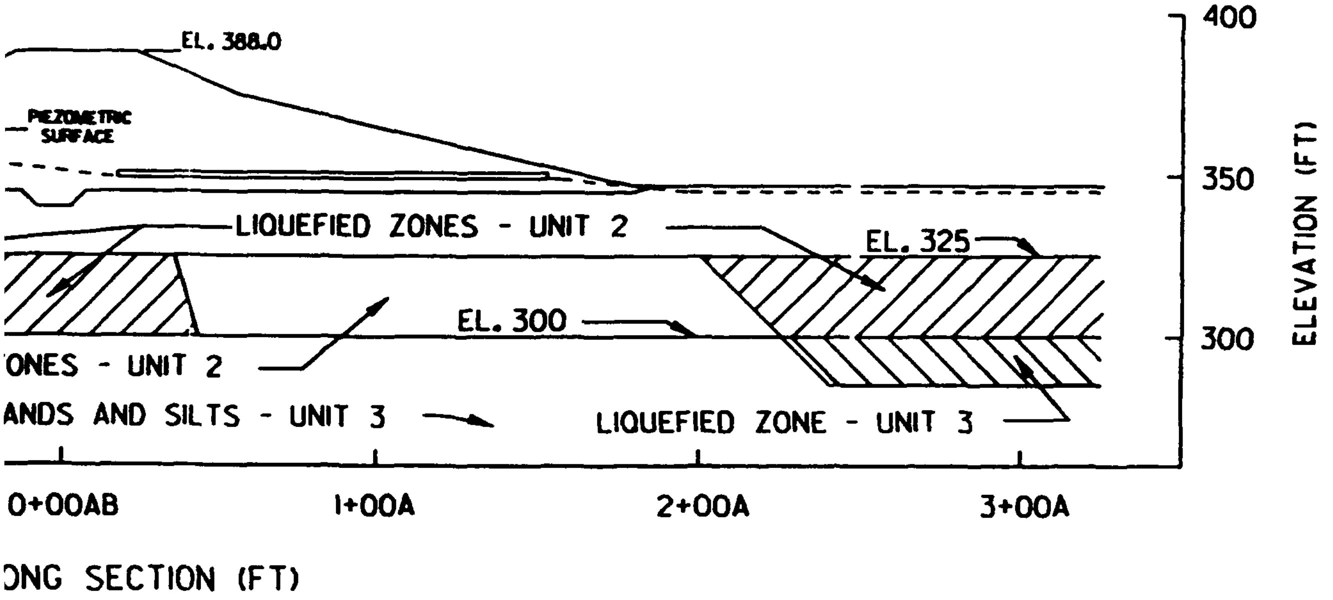


Figure 14. Main embankment

F LIQUEFACTION



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MAIN EMBANKMENT
ZONES OF LIQUEFACTION

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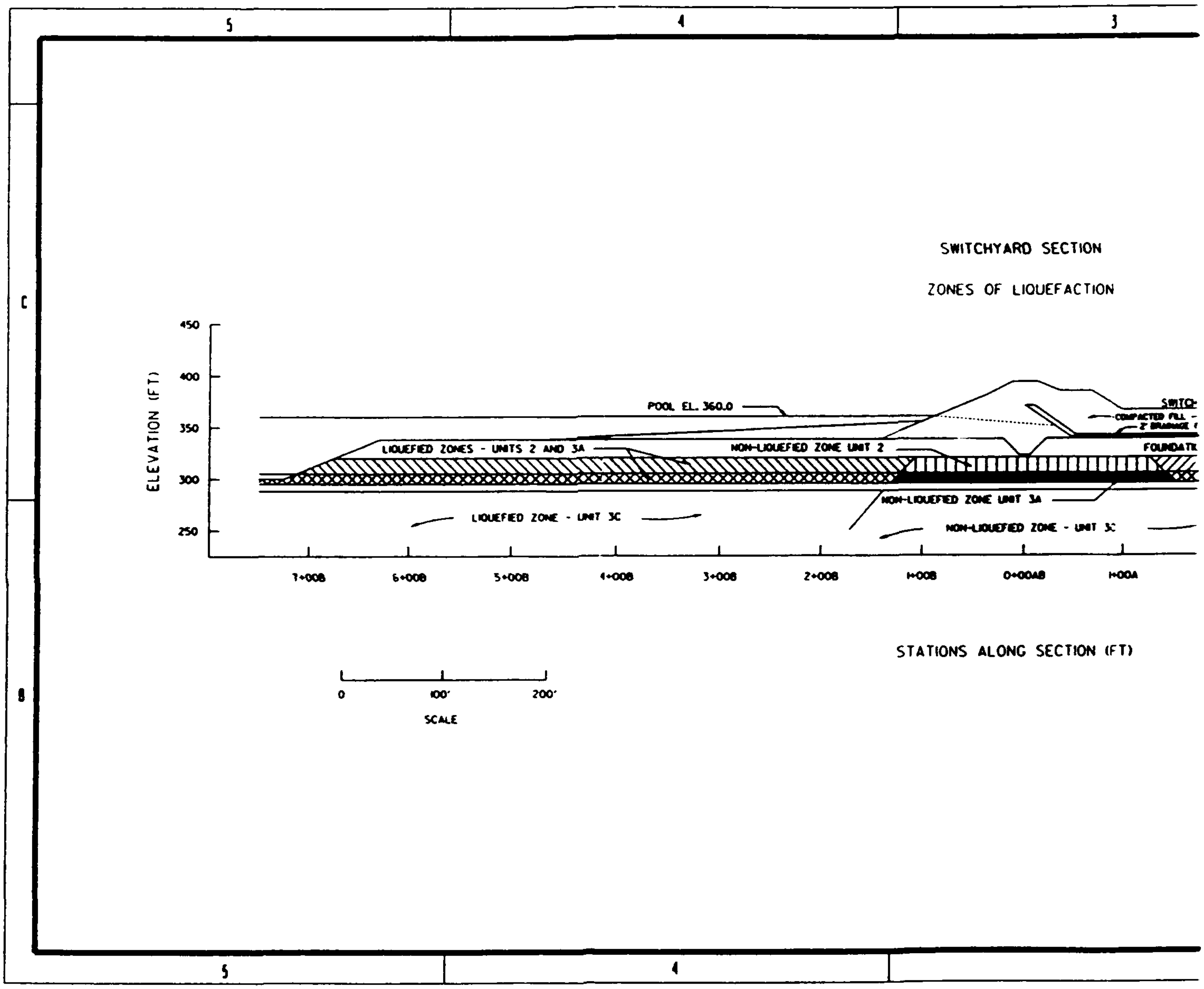
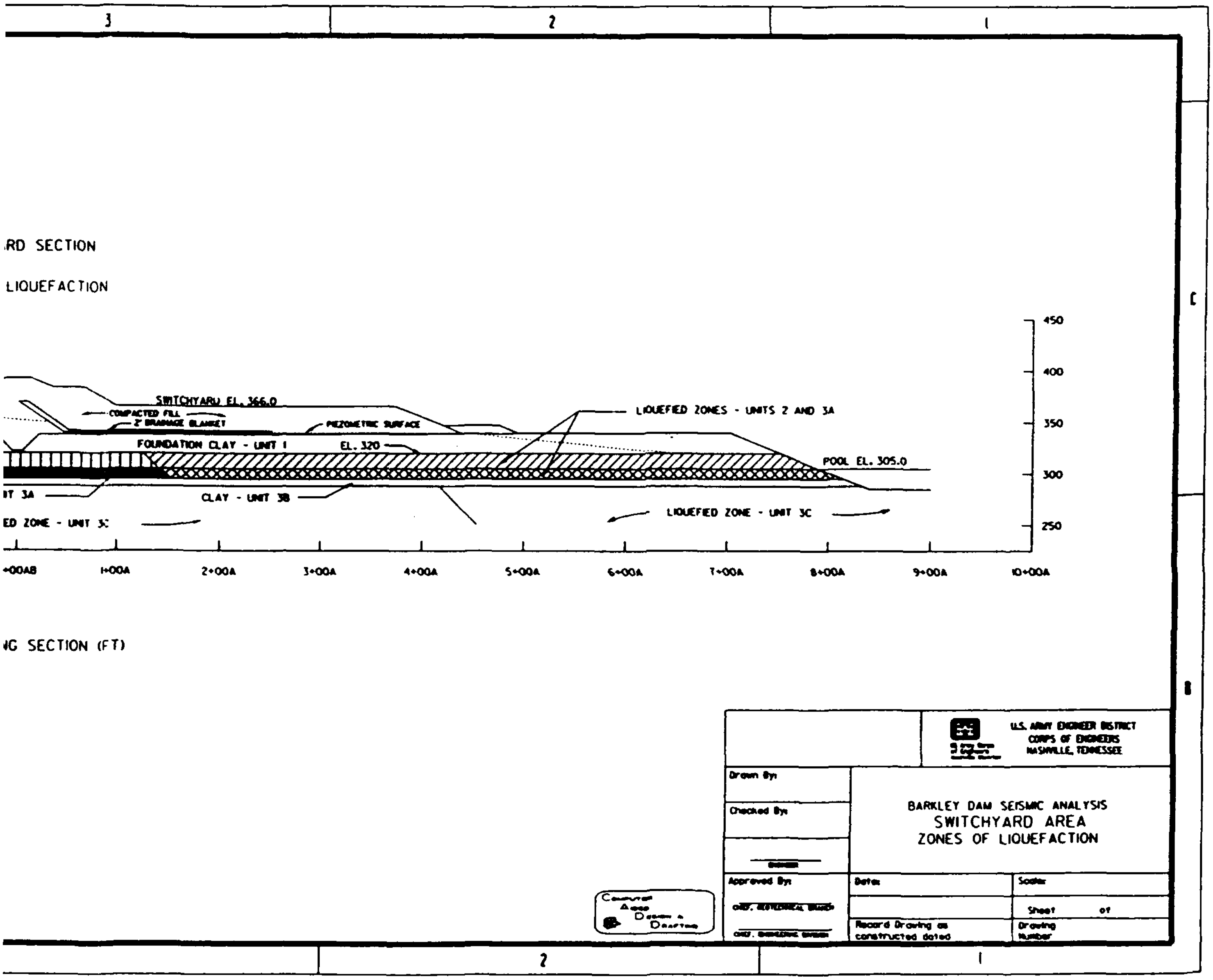


Figure 15. Zones of liquefaction f



.quefaction for the switchyard area.

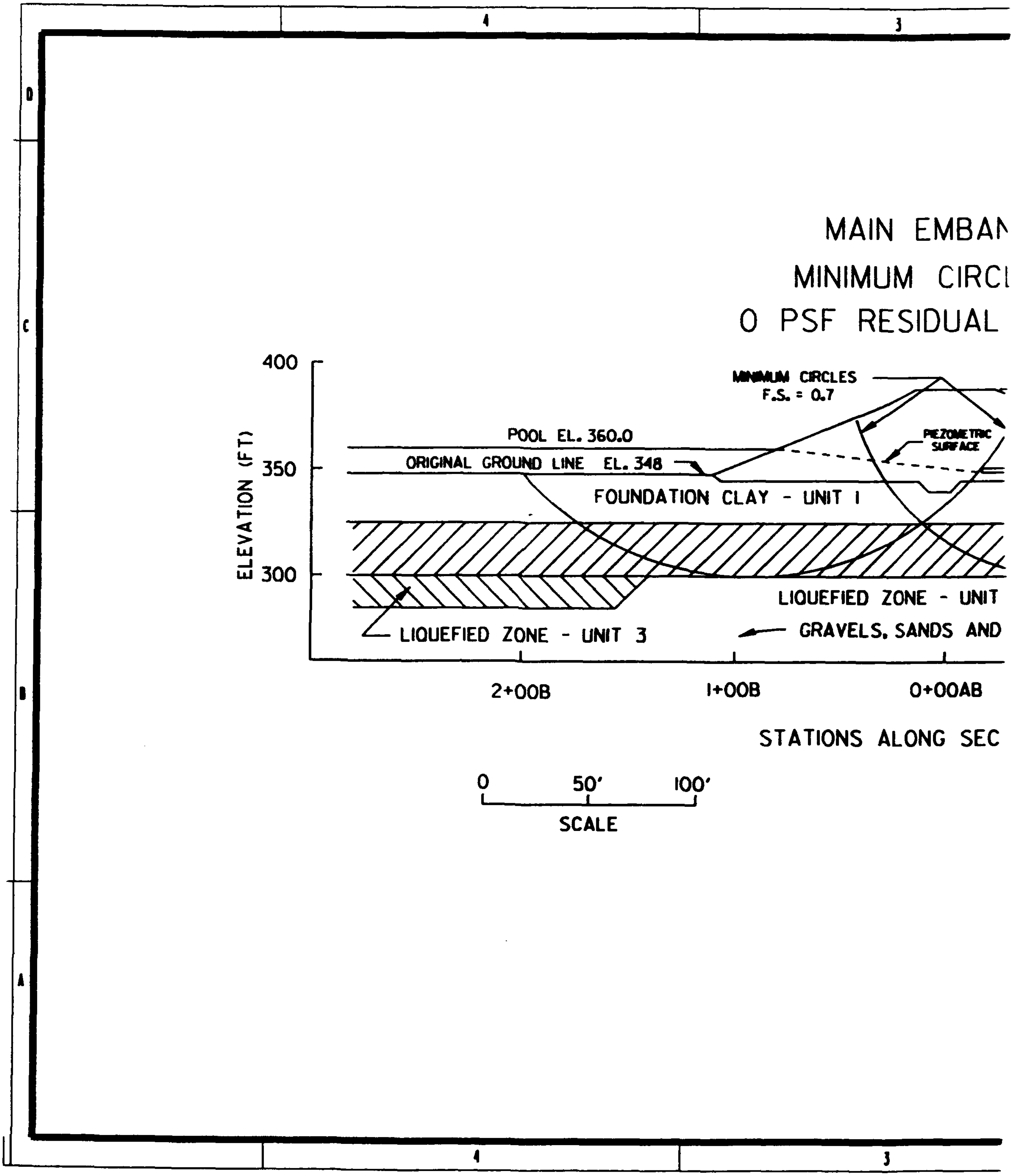
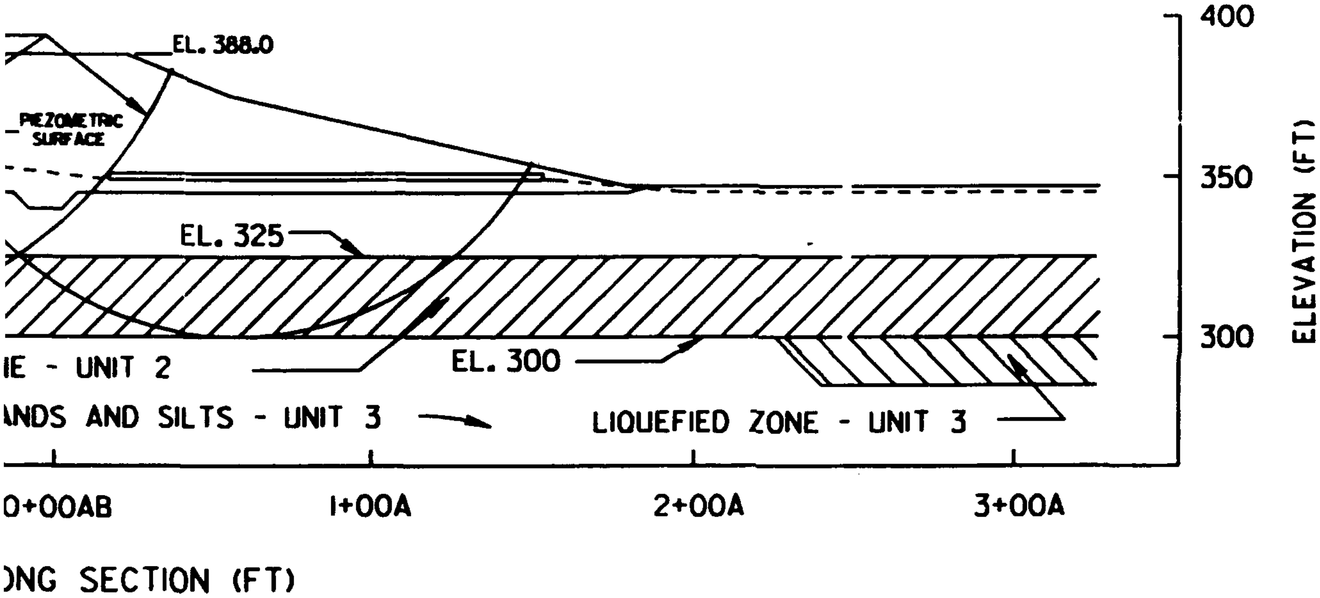


Figure 16. Minimum circles for 0 psf residu

STABILITY ANALYSIS PARAMETERS

MATERIAL	UNIT WEIGHT γ_m	UNIT WEIGHT γ_{sat}	C (PSF)	ϕ	% Ru
COMPACTED FILL	126	128	800	18	---
UNIT 1 CLAY	115	125	960	12	---
LIQUEFIED ZONE - UNIT 2	122	126	0	0	---
UNIT 3 SANDS	126	128	0	35	50
LIQUEFIED ZONE - UNIT 3	126	128	0	0	---

EMBANKMENT
CIRCLES FOR
IDUAL STRENGTH



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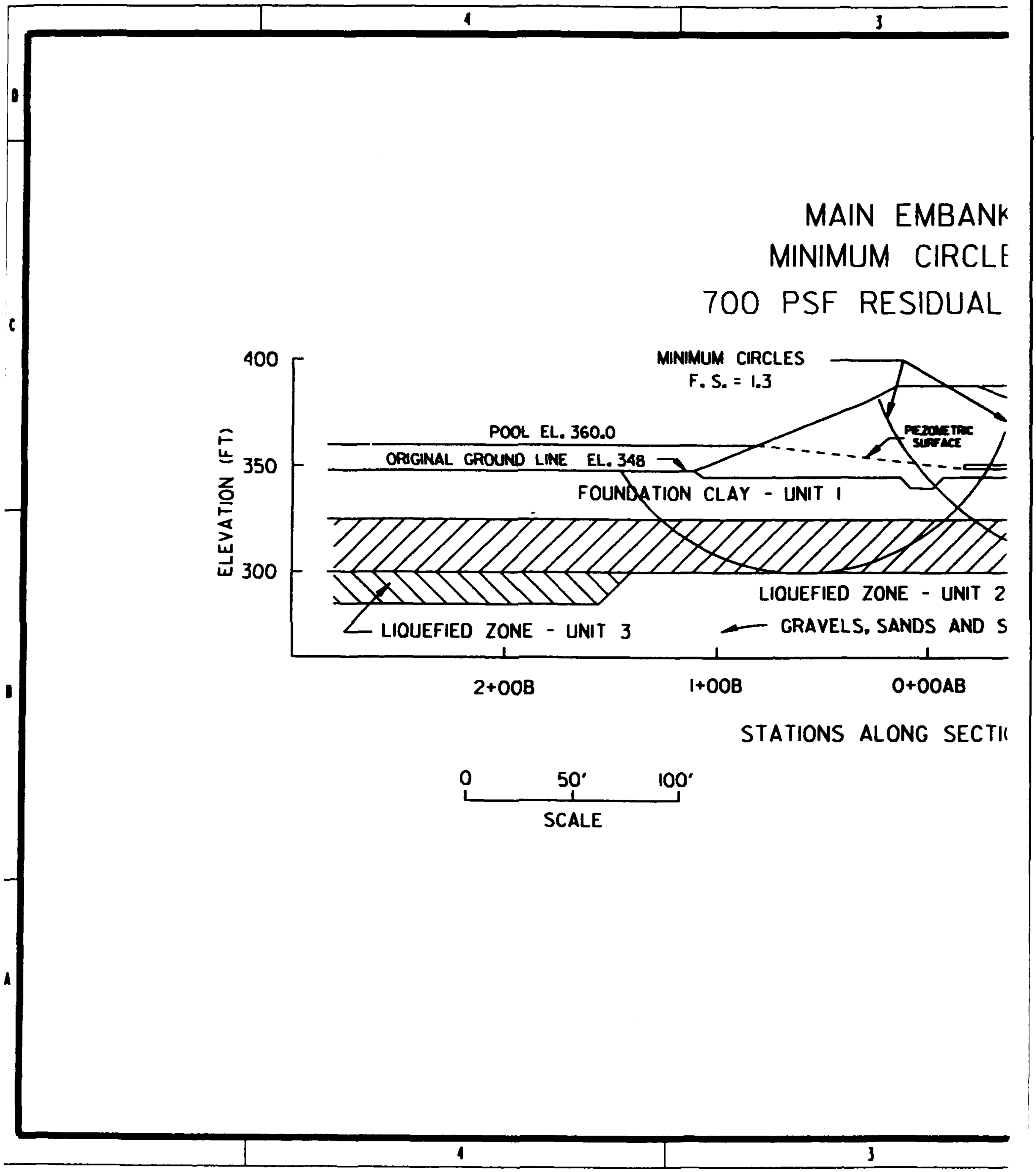
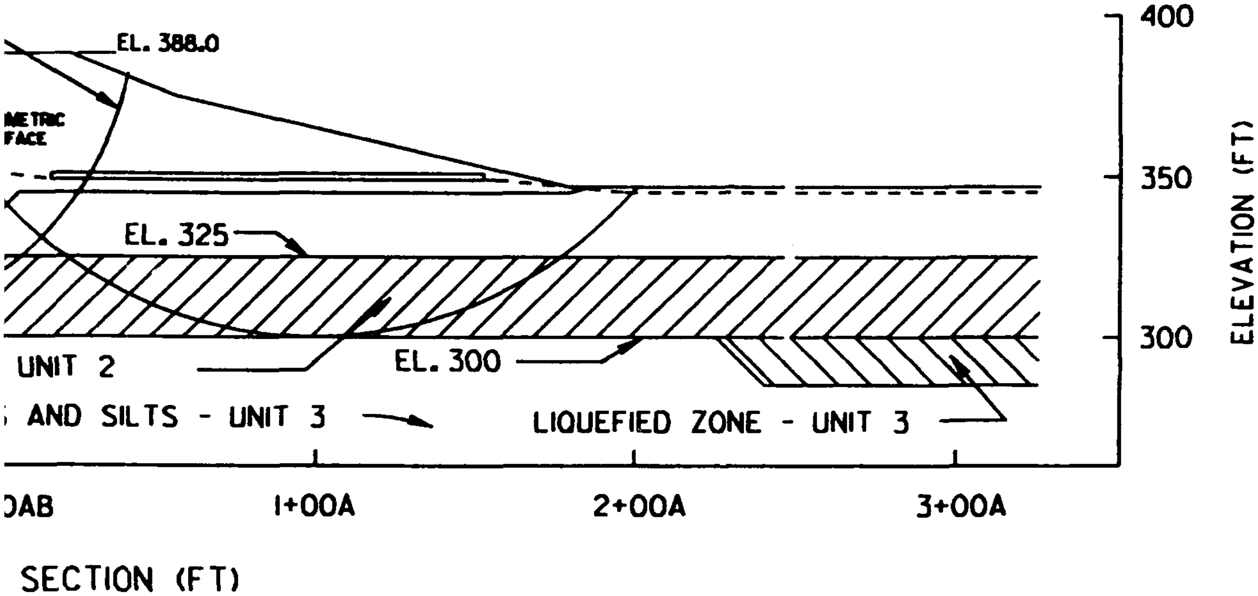


Figure 17. Minimum circles for 700 psf res

IBANKMENT
IRCLES FOR
)UAL STRENGTH

STABILITY ANALYSIS PARAMETERS

MATERIAL	UNIT WEIGHT γ_m	UNIT WEIGHT γ_{sat}	C (PSF)	ϕ	% Ru
COMPACTED FILL	126	128	800	18	---
UNIT 1 CLAY	115	125	960	12	---
LIQUEFIED ZONE - UNIT 2	122	126	700	0	---
UNIT 3 SANDS	126	128	0	35	50
LIQUEFIED ZONE - UNIT 3	126	128	800	0	---



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MINIMUM CIRCLES FOR
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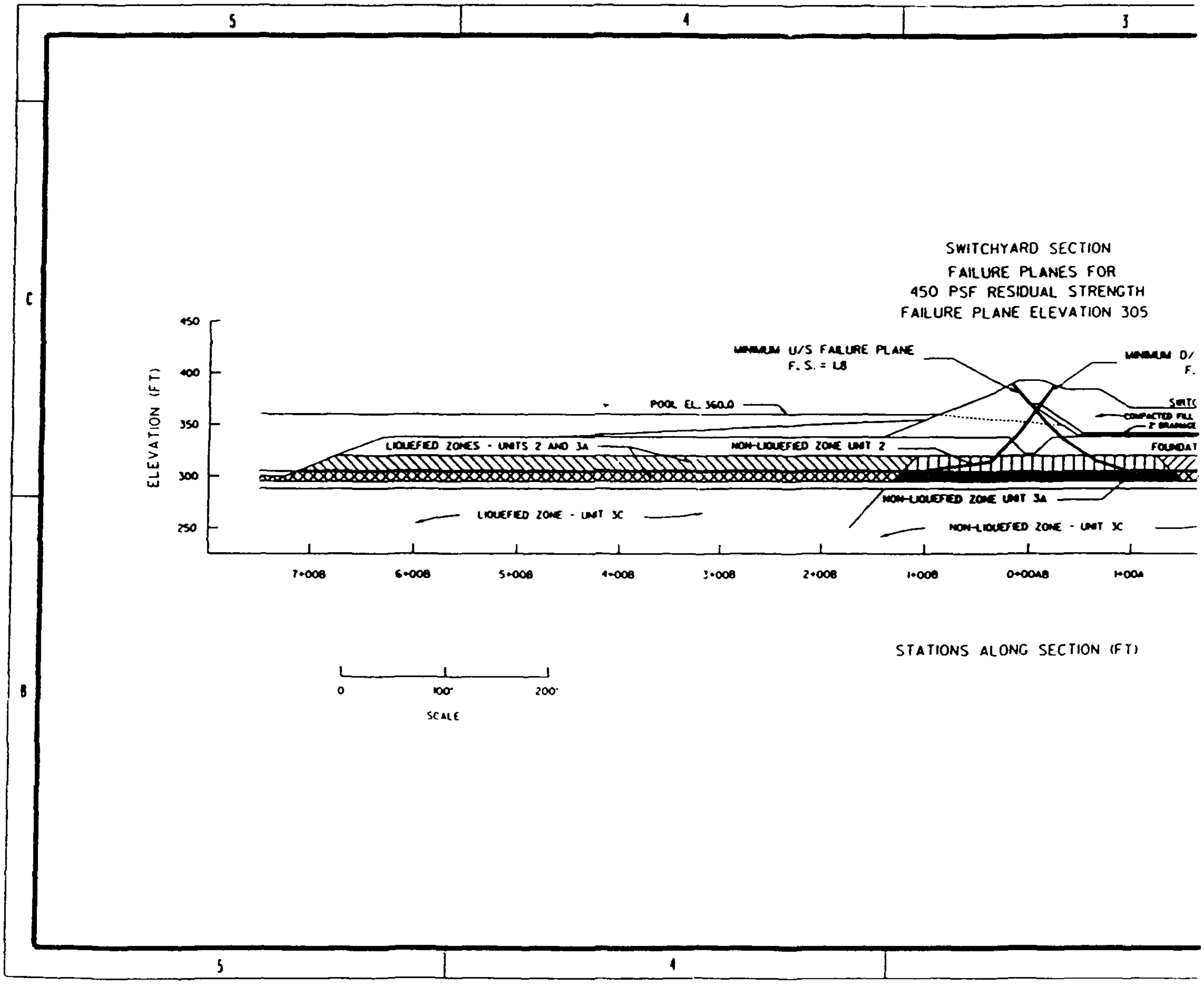


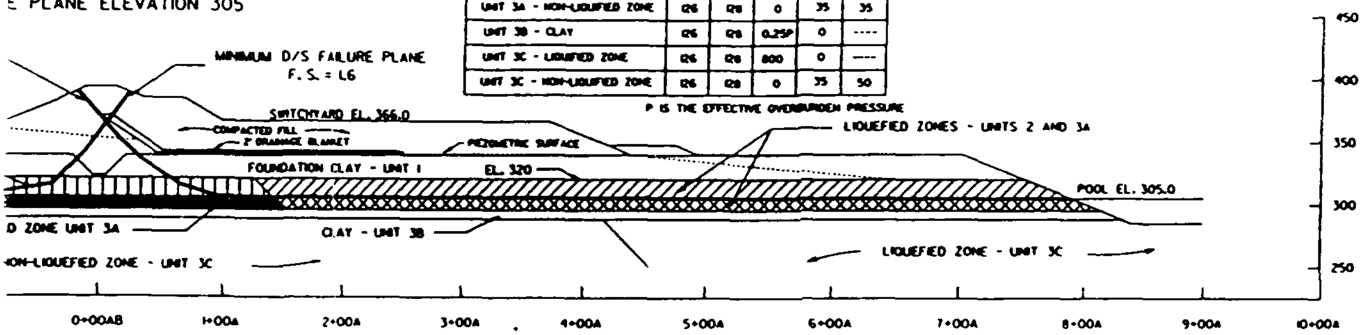
Figure 18. Minimum failure planes a
450 psf residual strength in liqu

WITCHYARD SECTION
'AILURE PLANES FOR
'SF RESIDUAL STRENGTH
E PLANE ELEVATION 305

STABILITY ANALYSIS PARAMETERS

MATERIAL	UNIT WEIGHT γ_m	UNIT WEIGHT γ_{sat}	c (PSF)	ϕ	% Ru
COMPACTED FILL	126	128	800	18	----
RANDOM FILL	126	128	320	6	----
UNIT 1 CLAY	115	125	960	12	----
UNIT 2 - LIQUEFIED ZONE	122	126	450	0	----
UNIT 2 - NON-LIQUEFIED ZONE	122	126	0.25P	0	---
UNIT 3A - LIQUEFIED ZONE	126	128	800	0	----
UNIT 3A - NON-LIQUEFIED ZONE	126	128	0	35	35
UNIT 3B - CLAY	126	128	0.25P	0	----
UNIT 3C - LIQUEFIED ZONE	126	128	800	0	----
UNIT 3C - NON-LIQUEFIED ZONE	126	128	0	35	50

P IS THE EFFECTIVE OVERBURDEN PRESSURE



NS ALONG SECTION (FT)

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SWITCHYARD AREA
450 PSF RESIDUAL STRENGTH
FAILURE PLANE ELEVATION 305

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failure planes at elevation 305 feet with
strength in liquefied zone of Unit 2.

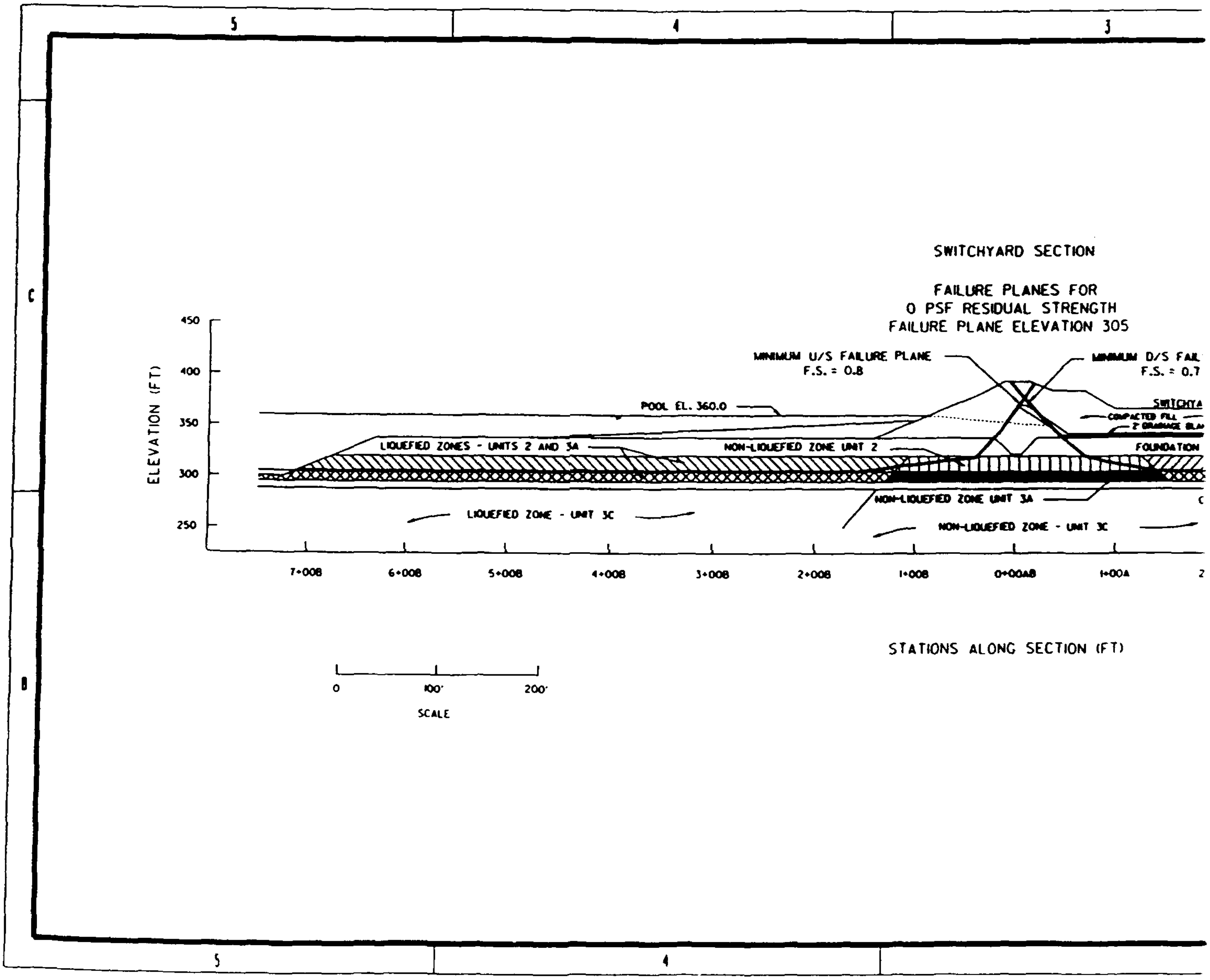


Figure 19. Minimum failure planes at e
0 psf residual strength in liquefi

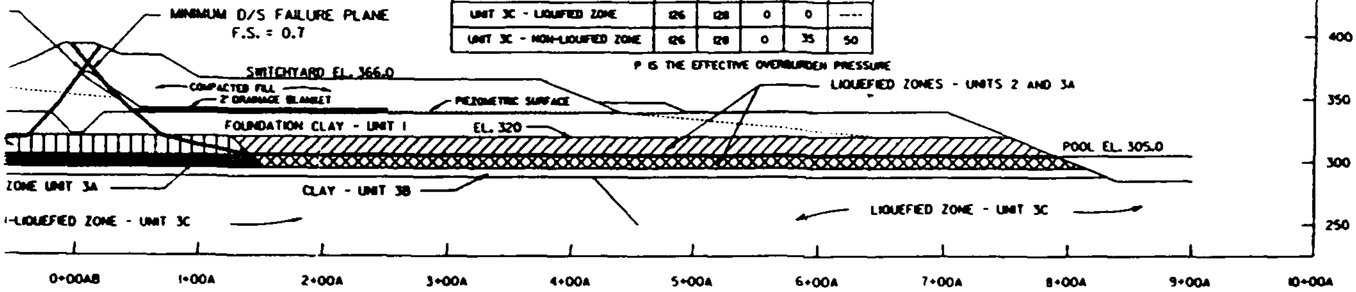
STABILITY ANALYSIS PARAMETERS

MATERIAL	UNIT WEIGHT γ_m	UNIT WEIGHT γ_{sat}	c (PSF)	ϕ	% Ru
COMPACTED FILL	126	128	800	18	----
RANDOM FILL	126	128	320	6	----
UNIT 1 CLAY	115	125	960	12	----
UNIT 2 - LIQUEFIED ZONE	122	126	0	0	----
UNIT 2 - NON-LIQUEFIED ZONE	122	126	0.25P	0	---
UNIT 3A - LIQUIFIED ZONE	126	128	0	0	----
UNIT 3A - NON-LIQUIFIED ZONE	126	128	0	35	35
UNIT 3B - CLAY	126	128	0.25P	0	----
UNIT 3C - LIQUIFIED ZONE	126	128	0	0	----
UNIT 3C - NON-LIQUIFIED ZONE	126	128	0	35	50

P IS THE EFFECTIVE OVERBURDEN PRESSURE

:CHYARD SECTION

.URE PLANES FOR
RESIDUAL STRENGTH
PLANE ELEVATION 305



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SWITCHYARD AREA
0 PSF RESIDUAL STRENGTH
FAILURE PLANE ELEVATION 305

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:lure planes at elevation 305 feet with
:ength in liquefied zone of Unit 2.

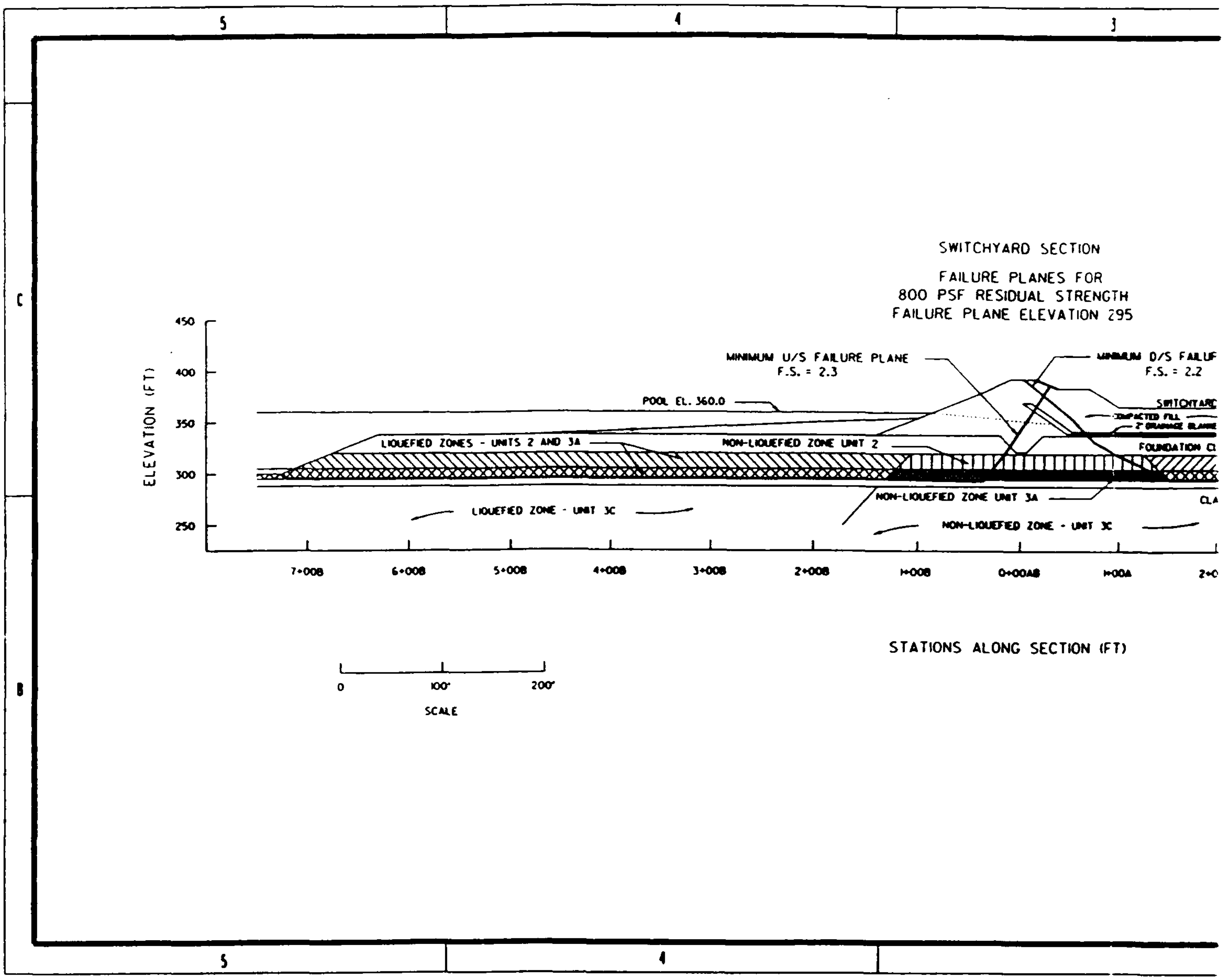


Figure 20. Minimum failure planes at
800 psf residual strength in lique

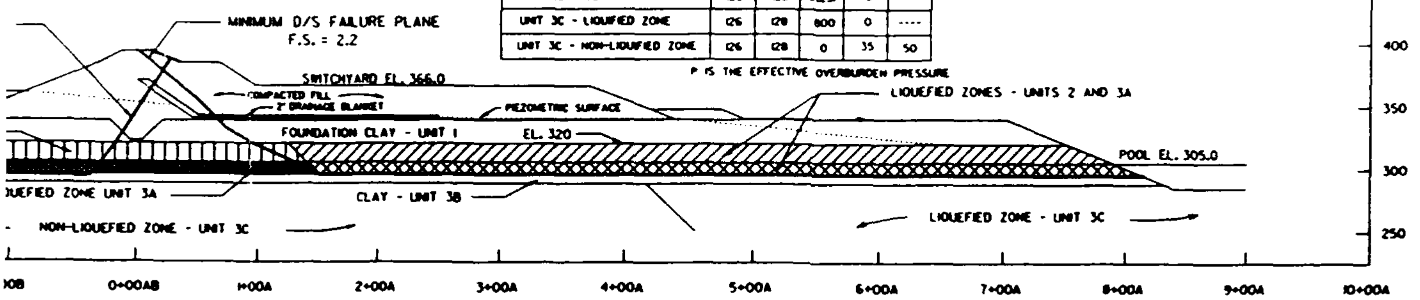
STABILITY ANALYSIS PARAMETERS

MATERIAL	UNIT WEIGHT γ_m	UNIT WEIGHT γ_{sat}	c (PSF)	φ	% Ru
COMPACTED FILL	126	128	800	18	----
RANDOM FILL	126	128	320	6	----
UNIT 1 CLAY	115	125	960	12	----
UNIT 2 - LIQUEFIED ZONE	122	126	450	0	----
UNIT 2 - NON-LIQUEFIED ZONE	122	126	0.25P	0	---
UNIT 3A - LIQUIFIED ZONE	126	128	800	0	----
UNIT 3A - NON-LIQUIFIED ZONE	126	128	0	35	35
UNIT 3B - CLAY	126	128	0.25P	0	----
UNIT 3C - LIQUIFIED ZONE	126	128	800	0	----
UNIT 3C - NON-LIQUIFIED ZONE	126	128	0	35	50

P IS THE EFFECTIVE OVERBURDEN PRESSURE

SWITCHYARD SECTION

FAILURE PLANES FOR
0 PSF RESIDUAL STRENGTH
.URE PLANE ELEVATION 295



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BARKLEY DAM SEISMIC ANALYSIS
SWITCHYARD AREA
800 PSF RESIDUAL STRENGTH
FAILURE PLANE ELEVATION 295

Date:
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Drawing Number

um failure planes at elevation 295 feet with
ıal strength in liquefied zone of Unit 3A.

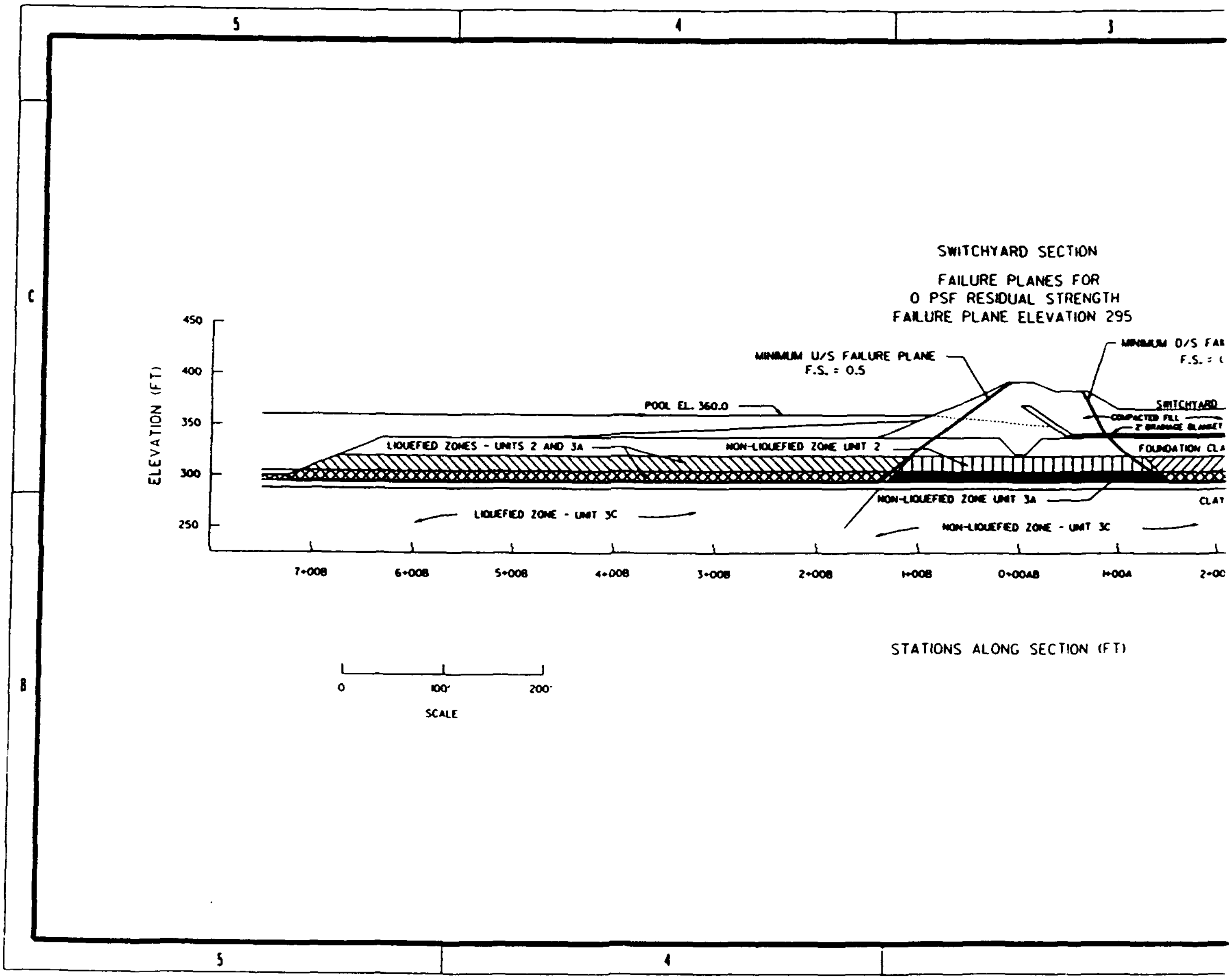


Figure 21. Minimum failure planes at e
0 psf residual strength ir

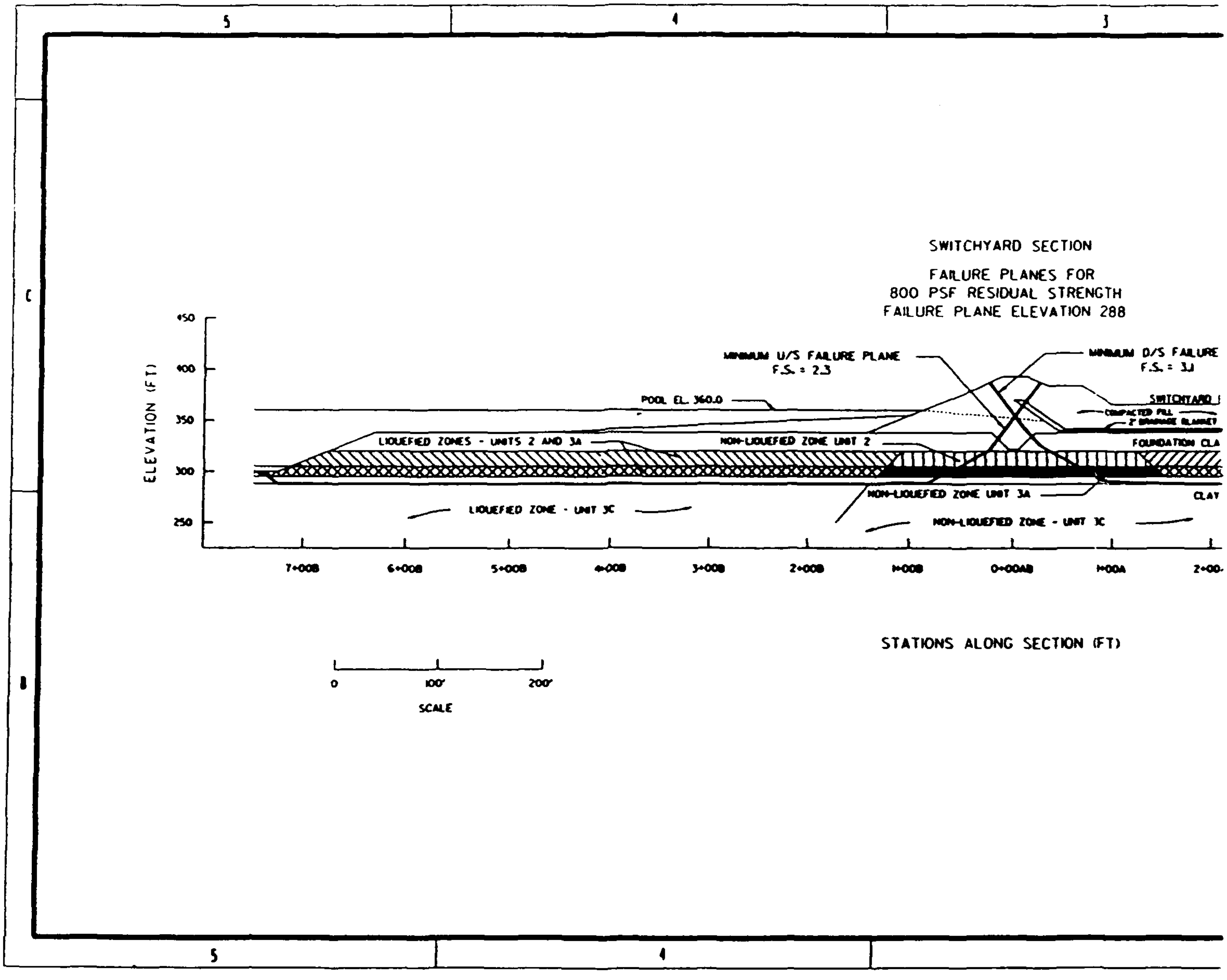


Figure 22. Minimum failure planes
800 psf residual strength in liq

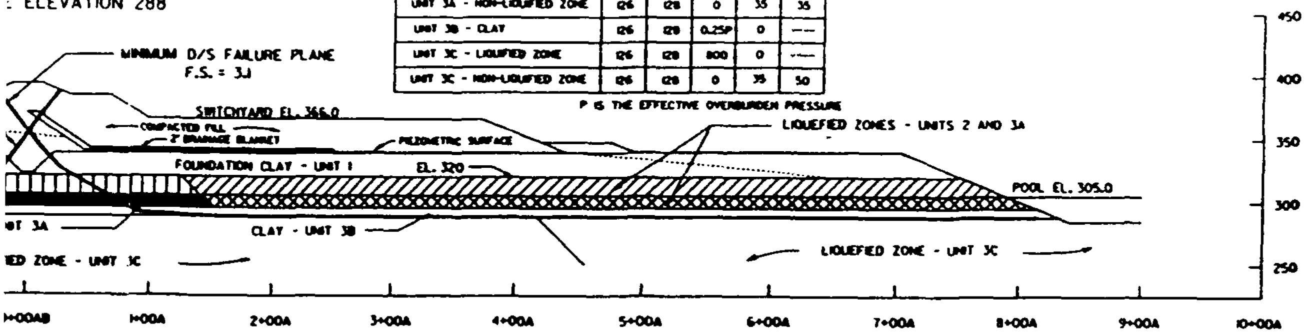
STABILITY ANALYSIS PARAMETERS

MATERIAL	UNIT WEIGHT γ_m	UNIT WEIGHT γ_{sat}	c (PSF)	ϕ	X Ru
COMPACTED FILL	126	128	800	18	----
RANDOM FILL	126	128	320	6	----
UNIT 1 CLAY	115	125	960	12	----
UNIT 2 - LIQUEFIED ZONE	122	126	450	0	----
UNIT 2 - NON-LIQUEFIED ZONE	122	126	0.25P	0	---
UNIT 3A - LIQUIFIED ZONE	126	128	800	0	----
UNIT 3A - NON-LIQUIFIED ZONE	126	128	0	35	35
UNIT 3B - CLAY	126	128	0.25P	0	---
UNIT 3C - LIQUIFIED ZONE	126	128	800	0	----
UNIT 3C - NON-LIQUIFIED ZONE	126	128	0	35	50

P IS THE EFFECTIVE OVERBURDEN PRESSURE

...ARD SECTION

...PLANES FOR
...DUAL STRENGTH
... ELEVATION 288



...NG SECTION (FT)

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NASHVILLE, TENNESSEE

Drawn By:

Checked By:

Approved By:

BARKLEY DAM SEISMIC ANALYSIS
SWITCHYARD AREA
800 PSF RESIDUAL STRENGTH
FAILURE PLANE ELEVATION 288

Date:

Scale:

Sheet of

Record Drawing as constructed dated

Drawing Number

Computer Aided Design & Drafting

failure planes at elevation 288 feet with
strength in liquefied zone of Unit 3C.

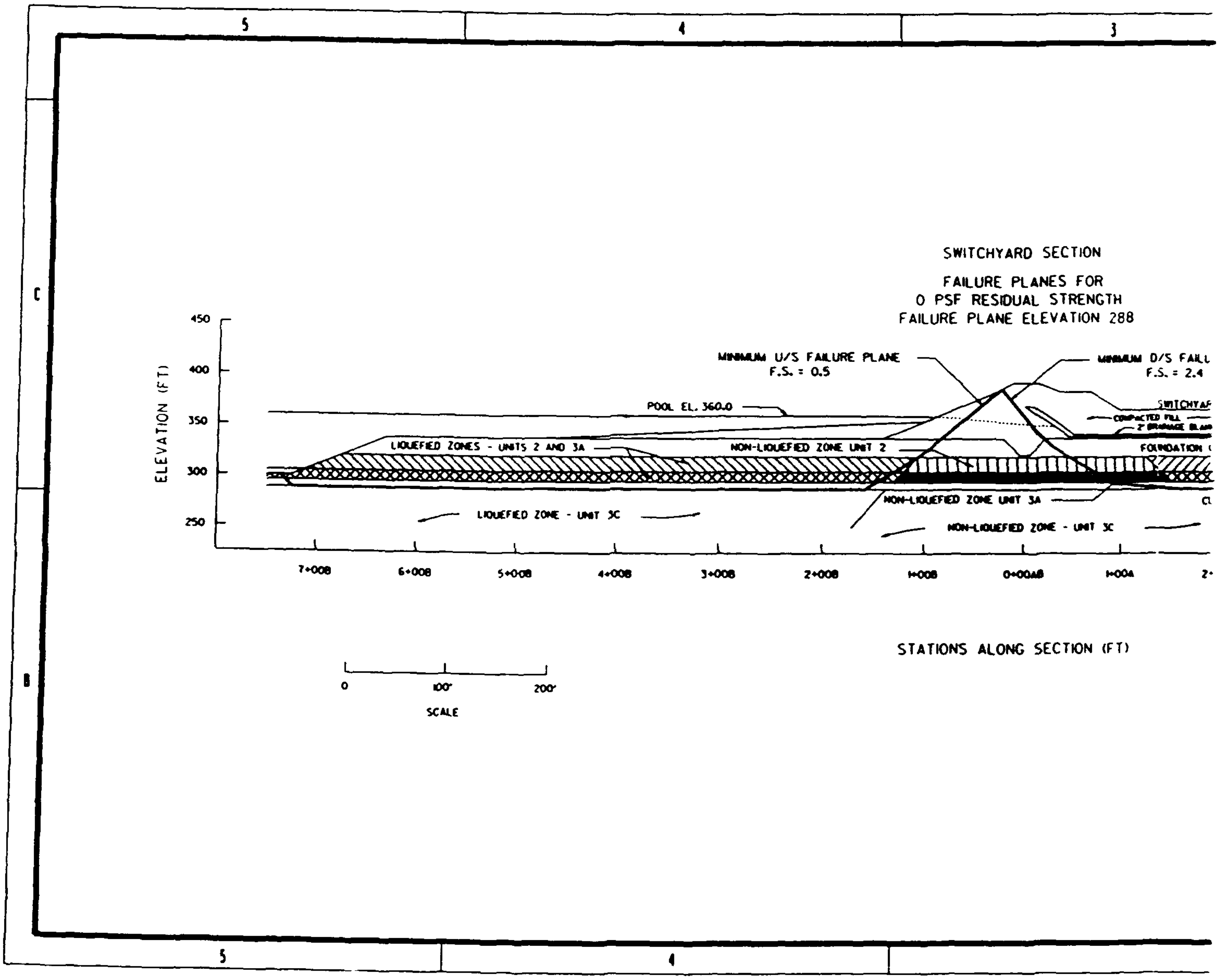


Figure 23. Minimum failure planes at
residual strength in liquef

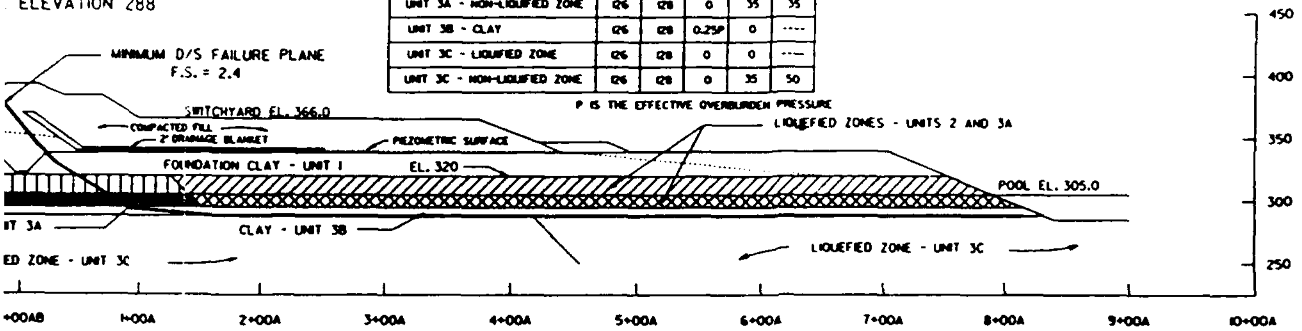
STABILITY ANALYSIS PARAMETERS

MATERIAL	UNIT WEIGHT		c (PSF)	ø	% Ru
	γ_m	γ_{sat}			
COMPACTED FILL	126	128	800	18	----
RANDOM FILL	126	128	320	6	----
UNIT 1 CLAY	115	125	960	12	----
UNIT 2 - LIQUEFIED ZONE	122	126	0	0	----
UNIT 2 - NON-LIQUEFIED ZONE	122	126	0.25P	0	---
UNIT 3A - LIQUIFIED ZONE	126	128	0	0	----
UNIT 3A - NON-LIQUIFIED ZONE	126	128	0	35	35
UNIT 3B - CLAY	126	128	0.25P	0	----
UNIT 3C - LIQUIFIED ZONE	126	128	0	0	---
UNIT 3C - NON-LIQUIFIED ZONE	126	128	0	35	50

P IS THE EFFECTIVE OVERBURDEN PRESSURE

.RD SECTION

PLANES FOR
UAL STRENGTH
ELEVATION 288



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U.S. ARMY ENGINEER DISTRICT
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NASHVILLE, TENNESSEE

Drawn By:

Checked By:

Approved By:

BARKLEY DAM SEISMIC ANALYSIS
SWITCHYARD AREA
0 PSF RESIDUAL STRENGTH
FAILURE PLANE ELEVATION 288

Date:

Scale:

Sheet of

Record Drawing as constructed dated

Drawing Number

ıre planes at elevation 288 feet with 0 psf
gth in liquefied zone of Unit 3C.

Waterways Experiment Station Cataloging-in-Publication Data

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