(2)

TECHNICAL REPORT GL-86-7

US Army Corps
of Engineers

SEISMIC STABILITY EVALUATION OF ALBEN BARKLEY LOCK AND DAM PROJECT

Volume I

SUMMARY REPORT

by

Ronald E. Wahl

Geotechnical Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

and

Paul F. Bluhm

DEPARTMENT OF THE ARMY
US Army Engineer District, Nashville
PO Box 1070, Nashville, Tennessee 37202-1070

US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

September 1992
Final Report

GEOTECHNICAL LABORATORY

Prepared for US Army Engineer District, Nashville
Nashville, Tennessee 37202-1070

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave blank)*

2. REPORT DATE September 1992

3. REPORT TYPE AND DATES COVERED Final report

4. TITLE AND SUBTITLE

Seismic Stability Evaluation of Alben Barkley Lock and Dam Project; Volume 1: Summary Report

5. FUNDING NUMBERS

6. AUTHOR(S)

Ronald E. Wahl, Paul F. Bluhm

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)

See reverse.

8. PERFORMING ORGANIZATION REPORT NUMBER

Technical Report GL-86-7

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)

US Army Engineer District, Nashville
Nashville, Tennessee 37202-1070

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES

This report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT

Approved for public release; distribution is unlimited

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

This report summarizes the results of seismological, geological, laboratory, field, and analytical investigations conducted to evaluate the seismic stability of two earth embankment sections of the Alben Barkley Lock and Dam Project, Kentucky. These represent typical sections of the powerhouse/swithyard area and the main embankment. Detailed documentation of each of the major topical areas of this study are contained in Volumes 2 through 5 of this series of reports. The design earthquake, from the New Madrid Seismic Zone, had a body-wave magnitude of 7.5. Of particular interest in this study, was the seismic performance of silty sands in the foundation and their overall effect on the stability of these embankment sections.

The results of this study indicate that large scale deformations and/or slope failure which would result in the loss of reservoir are not deformations on the order of 2 to 3 ft are expected. These deformations are relatively small in light of the fact that 28 ft of freeboard are expected to be available during the design earthquake.

14. SUBJECT TERMS

Dynamic analysis
Earthquake engineering
Finite element analysis
Liquefaction

15. NUMBER OF PAGES 302

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		

NSN 7540-01-280-5500

Standard Form 298 (Rev. 2-89)
Prescribed by ANSI Std Z39-18
298-102

7. (Continued).

USAE Waterways Experiment Station
Geotechnical Laboratory
3909 Halls Ferry Road, Vicksburg,MS 39180-6199

US Army Engineer District, Nashville
Nashville, Tennessee 37202-1070

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 1 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 discussed in this volume is a joint endeavor between ORN and WES. Mr. Paul F. Bluhm, of the Geotechnical Branch (CE-ORN-ED-G) at ORN, coordinated the contributions from ORN. Messrs. Ronald E. Wahl of Soil and Rock Mechanics Division, Richard S. Olsen and Dr. M. E. Hynes of the Earthquake Engineering and Geophysics Division (WES-GG-H), Geotechnical Laboratory (GL), WES, coordinated the work by WES. The preliminary stages of this project were directed and conducted by Dr. William F. Marcuson III, who was Principal Investigator from 1976 to 1979. From 1979 to 1988, Dr. M. E. Hynes 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, Mr. Daniel Habeeb, Mr. Melvin Seid, and Ms. Charlotte Caples 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, Director, 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. Paul Robinson, Chief, Engineering Division. Former District Commanders during the study were COL Robert K. Tenner, Cól Lee W. Tucker, COL William K. Kirkpatrick, COL Edward A. Starbird, and COL James P. King. The current District Commander is LTC Stephen M. Sheppard. 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.

CONTENTS

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 BARKELY LOCK AND DAM PROJECT

EXECUTIVE SUMMARY REPORT

PART I: INTRODUCTION

General

1. This report, the first in a series of five reports, summarizes 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 recreational facilities. The reservoir is contained by a concrete gravity section flanked by earth embankment dams. The concrete gravity section includes a gated spillway, a lock, and a switch-yard. The dam supports a railroad 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 ft, the reservoir stores 2,082,000 acre-feet, with 13 ft of freeboard (minimum crest Elevation 388 ft). For normal operation, the pool elevation varies from 354 to 359 ft, and stored volume varies from 610,000 to 869,000 acre-feet, respectively. A pool elevation of 360 ft was used for the seismic stability evaluation. A location map and plan of the project are shown in Figure 1.

3. The geological and seismological investigations conducted as part of this study revealed that an earthquake originating in the New Madrid Source Zone posed the most severe seismic threat to the site (Krinitzsky 1986). The most severe case was used as the basis for determining the parameters and

characteristics of the design earthquake used in this study. For this study, the design earthquake was specified to have a body-wave magnitude, m_b, of 7.5 occurring at a distance of 118 km in the New Madrid Source Zone. The S 48° E component of the Santa Barbara Courthouse record from the Kern County, CA earthquake of July 21, 1952 was scaled to a horizontal peak acceleration of 0.24 g to represent the ground motions expected at the site as a result of the design earthquake. The peak velocity of the scaled accelerogram was 35 cm/sec and the duration above 0.05 g was about 35 sec. The minimum elevation of the dam crest is elevation 388 ft; consequently, a pool elevation of 360 ft leaves a free board of 28 ft available at the time the design earthquake is assumed to occur.

4. The concrete gravity dam, switchyard, and lock system are 109 ft high at the maximum section and are founded on a limestone bedrock. The embankment dams are compacted rolled fill constructed of lean silty clays. Chimney and blanket drains are only present in the downstream sections of the embankment in the vicinity of the switchyard. The embankment sections are founded on alluvial deposit with a maximum thickness of approximately 120 ft. The alluvium is underlain by the limestone bedrock. This alluvium, a complex system of interbedded clays, 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. Generally, the alluvium beneath the embankment can be viewed as consisting of three units as shown on the profile in Figures 2a and 2b. The first zone, Unit 1, extends from the ground surface to a depth of 20 to 30 ft and is generally made up of a medium stiff clay with low to moderate plasticity. This material is an overbank deposit laid down on the flood plain during flooding. The second zone, Unit 2, extends from the bottom of Unit 1 to a depth of 50 to 60 ft and consists of a highly stratified sequence of clays, sands, and, silty sands. These are overbank deposits whose interbedded layers vary widely with respect to grain size and thickness. Unit 3 extends from the bottom of Unit 2 to a depth of 120 ft and consists of gravels and denser sands (denser than Unit 2) with some clay also being present. These materials are channel deposits laid down as the river swept across the valley. The different depositional environments for each of the three units described above probably resulted from changing base levels that occurred in the geologic past.

Principal Objective and Scope of Work

5. The principal objective of this study was to evaluate the seismic performance of the right embankment and its foundation in the event of the design earthquake. Detailed seismological, geological, field, laboratory, and analytical investigations were performed to achieve this objective. Additionally, an extensive effort was performed to assess the behavior of the alluvial foundation, in particular Units 2 and 3, during the design earthquake as the initial analytical investigation showed that the seismic performance of these foundation materials would control the dam's earthquake stability.

6. The investigations for the seismic evaluation of the Barkley Project are documented in a five volume report. The title of each volume is listed below:

Volume 1:	Summary Report
Volume 2:	Seismological and Geological Evaluation
Volume 3:	Field and Laboratory Investigations
Volume 4:	Liquefaction Susceptibility Evaluation and Earthquake Strength Determination
Volume 5:	Post-earthquake Stability Analysis

This volume, the Summary Report, discusses and summarizes the findings of the major investigations documented in Volumes 2 through 5. The reader is referred to the appropriate volume for more detail on a specific topical area.

PART II: GEOLOGICAL AND SEISMOLOGICAL INVESTIGATIONS

General

7. The geological and seismological evaluations were performed to determine the relevant factors for assessing the earthquake hazards at Barkley Dam. The estimation of earthquake ground motions appropriate for the seismic stability analysis of the site was the specific objective of the seismological studies.

8. The New Madrid earthquakes of 1811-1812 were the major motivating factors driving the seismic stability evaluations of Barkley Dam. The dam is located approximately 70 miles (118 km) from the source area of these earthquakes. Four major earthquake events are believed to have occurred during the 1811-1812 time frame. Two occurred on 16 December 1811, one on 23 January 1812, and one on 7 February 1812 (Street and Nuttli 1984). Street and Nuttli estimated that these events had body wave magnitudes of 7.2, 7.0, 7.1, and 7.3, respectively. These earthquakes are among the most severe in North America and, because attenuations in the central and eastern United States are lower than those in California, they were felt as far away as Canada, the East Coast, and southern Louisiana. Other more recent major earthquakes that have happened in the New Madrid seismic zone are the 5 January 1843 event near Memphis, TN, with a body wave magnitude of 6.0; the 31 October 1895 event near Charleston, MO, magnitude 6.2; and the 9 November 1968 event in south-central Illinois of magnitude 5.5. The Charleston event occurred approximately 62 miles (100 km) from the Barkley Dam. All other major earthquakes are more than 118 km from the site. Additionally, there has been continuous microearthquake activity in the New Madrid zone and miscellaneous smaller felt earthquakes in the central United States.

Earthquake Zones

9. The geological and seismological history in the general region of Barkley Dam was studied by Stearns (See Krinitzsky 1986, Appendix A). The region studied by Stearns, which includes most of the New Madrid zone and the seismic areas of southern Illinois, is shown in Figure 3.

10. Stearns examined the geological history and patterns of faulting and concluded that there were no active faults (faults with surface evidence of geologically recent fault movement) at or near Barkley Dam. Active faults were interpreted only in the New Madrid zone.

11. Stearns established four seismic zones which he deemed to be appropriate for the region in which Barkley Dam is situated. The four zones are shown in Figure 3. Zone I includes the area where a New Madrid earthquake may occur, Zone II is an area peripheral to the New Madrid zone and in which a lesser earthquake may occur, Zone III is the residual zone with the least seismicity, and Zone IV is the seismically active area of southern Illinois.

Peak Ground Motions

12. Peak ground motions specified for the Barkley Dam site were made in conformity with Engineer Regulation 1110-2-1806 which defines the earthquakes as the most severe believed possible for the site.

13. Ground motions for Zones I, II, and III were provided by Leeds (See Krinitzsky 1986, Appendix B). Motions for Zone IV were provided by Nuttli (See Krinitzsky 1986, Appendix C). The Leeds and Nuttli recommendations were made in 1978. The Modified Mercalli (MM) intensities at the dam site for maximum earthquakes in the four seismic zones were interpreted by Leeds and Nuttli in 1978 as follows:

Source	Magnitude m_b	MM Source I_o	Epicentral Distance km	Site I_s
Zone I	7.5	XI	118	IX
Zone II	6.5	IX	85	VIII
Zone III	5.5	VII	0	VII
Zone IV	6.5	IX	65	VIII

The table shows that the most severe MM Intensity at the site was judged to be I_s = IX from Zone I, the New Madrid Seismic Zone. A description of the Modified Mercalli Intensity scale is given in Figure 4.

14. Leeds and Nuttli assigned peak motions from correlations between peak ground motion parameters and site intensity at the damsite as follows:

Source	Horizontal Acceleration g	Horizontal Velocity cm/sec	Displacement cm	Duration (a>-0.05 g) sec
Zone I	0.28	40	22	10
Zone II	0.23	32	18	10
Zone III	0.18	25	15	10
Zone IV	0.23	32	18	10

15. Subsequently, new findings pertaining to the seismicity of the central United States and new data from the Imperial Valley Earthquake of 1979, caused WES personnel to consider a revised set of motions for the site. On 27 August 1980, the technical advisors to the seismological studies of Barkley Dam considered recommendations by Krinitzsky and Nuttli and together it was agreed that the dam be checked for the following earthquake which was considered to represent the worst conditions for any earthquake affecting the site. The ground motion parameters agreed upon were:

Peak Horizontal acceleration = 0.24 g at a frequency of 2 hz
Peak horizontal velocity = 50 cm/sec at a frequency of 1 hz
Duration of motion which exceeded 0.05 g = 25 sec

The S 48° E component of the Santa Barbara Courthouse record of the Kern County earthquake of July 21, 1952 with a scaled peak acceleration of 0.24 g was recommended for use in this study. The accelerogram and response spectra are shown in Figures 5 and 6. The peak velocity was about 35 cm/sec; the duration was about 60 sec. Qualitatively, the velocity being somewhat lower than the target was compensated by the fact that the duration was longer than its target. The advisors approved the record and the scaling procedures. In the dynamic response analysis of the site, the design ground motions were applied to the ground surface of a firm soil profile at the site.

16. The design ground motions were reviewed a second time by WES personnel. The review was made since the Krinitzsky and Chang motions for MM intensities were updated based on the inclusion of data from large earthquakes. The new charts were published by Krinitzsky and Marcuson (1983). The peak ground motion parameters justified by the new charts would be a peak acceleration of 0.25 g, a peak velocity of 49 cm/sec, and a duration of 64 sec. Each of these values were sufficiently close to the specified values that no changes to the specified values were considered necessary.

PART III: FIELD INVESTIGATIONS

General

17. Detailed field and laboratory investigations were performed to gather information for site characterization and idealization for the seismic analysis. These efforts focused on gathering data on the alluvial foundation soils, particularly those of Units 2 and 3, whose seismic behavior were believed to be the major factor controlling the dam's post-earthquake stability. These investigations included reviews of the geology and preconstruction records of the damsite. In situ field testing included a suite of geophysical surveys, and the performance of Standard Penetration and Cone Penetration Tests (SPT and CPT). Additionally, both disturbed and undisturbed samples were recovered from the foundation soils and a streambank exposure of a major foundation unit was evaluated during the field explorations. Laboratory tests were performed to classify the samples recovered during the field exploration program. This section of the report summarizes the information gathered from each of these sources of information.

Geology

Regional geology

18. The Barkley Project is located in the extreme northern portion of the Mississippi Embayment, which extends over an area of about 100,000 square miles as shown in Figure 7. The Mississippi Embayment covers portions of the states of Illinois, Alabama, Arkansas, Kentucky, Louisiana, Mississippi, Missouri, Tennessee, and Texas.

19. The Mississippi Embayment is essentially a downwarped trough or syncline of Paleozoic rocks in which sediments ranging in age from Jurassic to Recent have been deposited. A geologic time scale is included in Figure 8. The trough's axis plunges to the south and roughly follows the present course of the Mississippi River. The greatest thickness of post-Paleozoic sediments or rocks filling the trough is approximately 18,000 ft and occurs in the extreme southern part of the embayment, in the area of greatest subsidence and downwarping. Generally, the sediments consist of sands, silts, clays, gravels, and chalks.

Site geology

20. General: The Cumberland River, flowing near the margin of the Mississippi Embayment in the area of the Barkley Project, has completely cut through sediments which once filled the embayment and has incised itself into the underlying trough of Paleozoic rocks (See Figure 2a). The figure shows that remnant outcrops of the embayment sediments cap the hills and ridges while valley slopes are comprised of Paleozoic rocks. Alluvium is present in the valley bottoms of all major stream and rivers. The Cumberland River deposited alluvium in the valley bottoms during the Recent Epoch of the Quartenary Period.

21. The concrete structures for the Barkley Project were founded in the Mississippian Warsaw formation (Paleozoic Era)and the earth embankment sections were constructed on the alluvium. Of major interest was the evaluation of the liquefaction potential of the alluvial foundation and the effect of its performance during the design earthquake on the stability of the embankment sections.

22. Alluvial foundation: Data collected from the field investigations from SPT, CPT, undisturbed samples, and an excavated exposure of a foundation unit enabled a detailed assessment of the foundation conditions to be made. The resulting interpretation represents a synthesis of the data with consideration given to the strengths and weaknesses of all the various sources of information. The switchyard area and the main embankment area were the two main areas studied. The switchyard area is located between Sta 34+00 and Sta 43+00 and the main embankment area includes the area between Sta 43+00 and Sta 88+00.

23. The alluvial foundation underlying the embankment sections in both areas can be modeled by three basic foundation units: Unit 1, Unit 2, and Unit 3. Generally, the overall foundation thickness is approximately 120 ft. Idealized cross sections along the longitudinal axis and through the switchyard, presented in Figures 2a and 2b, respectively, show the spatial relationships of the three foundation units.

24. Unit 1 consists of medium stiff clays which typically classify as CL materials and whose thickness varies between 20 and 30 ft. In the switchyard area the thickness extends to elevation 320 and in the main embankment area it is somewhat thinner where it only extends between elevations 330

and 325. These clays, which can be viewed as a topstratum, are overconsolidated as a result of desiccation.

25. Unit 2 consists of sand and silty sand layers interbedded within a matrix of soft clays (CL). The sand layers are generally very thin, usually on the order of only a few inches thick. These layers are dirty and on the average contain about 30 percent nonplastic fines. Unit 2 lies between elevations 320 and 305 in the switchyard area (near the switchyard) and is a little thicker beneath the main dam where it generally lies between Elevations 330 and 295 ft.

26. In both the switchyard and main embankment areas, the boundary between Units 2 and 3 undulates. In the main embankment area, Unit 3 extends from the bottom of Unit 2 to elevation 230 ft and consists of sands with some layers of clay also present. In the switchyard area, Unit 3 was subdivided into three subunits: Units 3a, 3b, and 3c. Unit 3a contains dense sands and gravels which have a relatively high penetration resistance as compared to the sands layers in Unit 2. These sands typically contain less than 15 percent fines and are relatively clean compared to those in Unit 2. Unit 3a lies generally lies between elevation 305 and 295. There are some clay layers present in Unit 3a. Unit 3b, located between elevation 295 and 288, is typified by clays (CL) of low penetration resistance. These clays were detected by nearly all of the CPT's in the switchyard and appear to be a characteristic of this area of the damsite. Unit 3c lies between elevation 285 and bedrock and based on limited amounts of data appears to have characteristics similar to Unit 3a. In the main embankment area, Unit 3 was considered to consist of only one subunit whose characteristics were very much like those of Unit 3a in the switchyard.

27. The alluvium at the Barkley Project has not been preconsolidated by any overlying glacial ice as the advance of the glaciers essentially stopped at the present location of the Ohio River about 15 miles to the northwest, and the glaciers had retreated prior to the deposition of the alluvium. However, the clays of Unit 1 appear to be overconsolidated as a result of processes desiccation.

28. A preliminary seismic evaluation performed early in the study revealed that the interbedded silty sand layers in Unit 2 had the highest potential for liquefaction in the foundation. The lateral extent of liquefiable soils is an important factor in the evaluation of the dam's earthquake

stability. Based on the observed mechanics of deposition it can be inferred that alluvial deposits tend to be irregular and discontinuous in both horizontal and vertical directions while lacustrine deposits tend to be generally continuous over large areas. Thus, an issue of great importance was whether the foundation materials under the right embankment were alluvial or lacustrine deposits. A significant effort was made during the field investigations to determine if the foundation units were alluvial or lacustrine. The results indicated that the foundation was alluvial.

Design and Construction Records

Preconstruction field and laboratory investigations

29. For design of the earth embankments, the pre-construction boring program consisted of 18 drive sample holes (churn rig), generally on 400-ft centers, and 2 undisturbed Denison holes. In the areas upstream and downstream of the switchyard adjacent to the concrete section, 24 drive sample holes (churn rig) and 2 undisturbed Denison sample holes were also drilled. Numerous probings, auger, and washbore holes were also conducted. See Figure 9 for the locations of these borings. There were no SPTS tests performed in any of these borings.

30. Sieve analysis, Atterberg limits, and natural moisture contents were determined from laboratory tests on only a few of the samples recovered from the boring program. Most of the foundation samples were classified only visually in the field. In addition to the above tests, undisturbed samples were tested for specific gravity, dry density, shear strength, and permeability. Table 1 summarizes saturated and dry densities and shear strength parameters which were obtained from these tests. Zones A, B, and C in the table correspond approximately to Units 1, 2, and 3, respectively, described in Part III. The values listed in Table 1 were used in the design of the dam.

Construction of embankments

31. Construction of the dam began in 1961 with the right embankment and switchyard being built in two phases. The first phase was construction of the 800 ft embankment, switchyard, and pervious drainage blanket up to elevation 360 ft. Material used for this phase was obtained from the switchyard excavation. The second phase of construction began in 1962 and consisted of

building the reminder of the embankment and switchyard. Materials used in the embankment were obtained from several borrow sources upstream of the dam. A lean silty clay was used for sections of the impervious embankment and switchyard and was compacted in 4- to 8-in. layers with 6 passes of a 10-ton sheepsfoot roller. The pervious drainage blanket, compacted with the hauling and spreading equipment, was a crushed limestone aggregate with a maximum particle size of 1-1/2 in., a mean particle size of 1/2 in., and not more than 5 percent passing the No. 200 sieve.

32. Field density and record samples were obtained in the embankment during both phases of construction. Values used in design, determined from the field densities and tests performed on the record samples, are reported in Table 1.

Summary

33. Analysis of the geotechnical data available from the preconstruction and construction phases of the Barkley Project proved useful in developing an initial concept of the conditions at the site. However, the information obtained from the foundation soils lacked the detail necessary for characterizing the site for the seismic analysis. Hence, it was necessary to plan additional field explorations to further investigate the foundation soils and provide data which will directly support the seismic analysis of the Barkley site.

Pool Level for Seismic Analysis

34. The reservoir level selected for use in the seismic analysis was elevation 360 ft. For normal operations the reservoir is controlled using the rate curve shown in Figure 10. The curve shows that elevation 354 ft is maintained from 1 December through 31 March. The pool is gradually raised to Elevation 359 during April where it is maintained to about 1 July at which time it is gradually lowered back to Elevation 354 ft by 1 December. The reservoir is designed for flood control storage up to 370 ft. Data gathered since 1968 shows that the reservoir level of 361 ft has been exceeded only about 4 percent of the time or on the average of 2 weeks per year.

35. The tailwater elevation used in the seismic analysis was selected to be elevation 305 ft. The tailwater elevation is controlled by downstream structures on the Ohio River. The minimum tailwater elevation is 302 ft.

Stability conditions are critical when the tailwaters are at a minimum elevation. Since the tailwater is at elevation 305 ft for about half of any year, this elevation was used in the seismic analysis.

Geophysical Surveys

General

36. The purpose of the geophysical surveys was to measure the shear-wave (S-wave) velocity, V_s , and the compressional-wave (P-wave) velocity, V_p , of the embankment and foundation soils from the ground surface to bedrock. Although the V_s profiles are only low resolution indicators of stratigraphy (generally layers on the order of a few feet in thickness can be resolved though in certain cases it is possible to detect thinner layers) they are a major input parameter in performing site specific dynamic response calculations. Consequently, V_s , measurements were made at five areas to detect variations in V_s profiles along the toe and axis of and perpendicular to the dam. The V_p profiles are used primarily to distinguish between saturated and partially saturated soil zones.

37. Crosshole, downhole, P-wave surface refraction, S-wave surface refraction, and Rayleigh wave tests were performed. Specially instrumented Cone Penetrometer Test (CPT) equipment was used for the downhole tests at two of the test locations. The crosshole tests are inherently the most accurate technique for measuring seismic wave velocities, so these results were given the most weight in the development of velocity profiles for the dynamic response analysis. At two locations, namely the dam centerline and the center of the switchyard, crosshole tests were not performed. The V_s profiles were estimated in these cases from the closest reliable measurements adjusted for confining stress differences.

38. The geophysical tests which were performed at five locations are shown in the plan view in Figure 11. Table 2 summarizes the geophysical tests and includes dates, depths of investigation, boreholes used, and remarks for each of the tests performed at each of the five locations.

Results

39. The shear wave velocity profiles were correlated with the materials of Units 1, 2, and 3 at test Locations 1 though 4. These results are presented in Figure 12. The figure shows the complexities of the site stratigraphy in that these zones do not distinguish themselves with particular

S-wave velocities. The range is broad and similar for each foundation unit. Further complications in correlating these zones result from the fact that the data for Location 4 shows higher velocities which are a function of test method and not the soil properties. Also, choice of a boundary between units at particular elevations also smears zones together since the site stratigraphy is undulating. General interpretations of the zones are that the surface layers of the Unit 1 clays exhibit velocities in the range of 400 to 600 fps. Otherwise velocities in the range of 700 to 800 fps are characteristic of Unit 1. Unit 2, composed of interbedded layers of silty sands and clays, shows a broad range of velocities as would be expected from this type of stratigraphy. Unit 2 velocities range from 450 to 950 fps. The soft clays and dense sands account for the low and high ends of the range, respectively. The velocities of Unit 3 show a broad range which varies from 550 to 1,025 fps. The lower velocities account for clayey materials while the higher values account for the sands present in Unit 3.

40. Choice of average velocities would be misleading as it does not account for the variations in values caused by the complex stratigraphy. This was kept in mind while interpreting the S-wave velocities for the dynamic response analyses for representative embankment cross-sections. In conclusion, soft zones with velocities between 450 and 600 fps can be found in all units. Unit 2 is more populated with these zones than Units 1 and 3. Dense sands and gravelly sands with velocities of 900 to 1,000 fps are more characteristic of Unit 3 but also exist in Unit 2.

Piezometers

41. A total of 73 piezometers have been installed either before or during the seismic study. A plan view showing the locations of these piezometers is shown in Figure 13. Piezometer tips were placed in each of Units 1, 2, and 3.

42. The piezometric levels and fluctuations in the embankment and foundation vary, depending on their location and midtip elevation and are influenced by changes in the headwater and tailwater elevations. As discussed earlier the headwater normally varies by only about 5 ft during in a typical year but the tailwater can vary by 20 ft or more.

43. Piezometric data obtained during 1984-85 shows that for most of the length of the dam, the groundwater levels are close to the ground surface, generally being between elevation 340 and 350 ft with fluctuations of only a few feet. The data shows that piezometers in the switchyard respond to changes in the tailwater elevation. Piezometers located along the main embankment show that the piezometric levels are affected by changes in the headwater elevation. The data shows that a normal seepage gradient exists near the tailrace, but some piezometers show that a downward seepage gradient also exists.

Undisturbed Samples and Laboratory Results

General

44. Undisturbed samples were obtained from eleven borings which are listed in Table 3 and whose locations are shown in Figure 14. Undisturbed samples were used to estimate insitu density, to observe foundation stratigraphy in detail, and to perform undrained laboratory strength tests under both cyclic and monotonic loading.

45. The samples were recovered during three different field operations performed in 1977, 1979, and 1984, respectively. The 1977 and 1979 efforts were performed by WES for obtaining samples for undrained monotonic and cyclic strength testing of the foundation soils. The 1984 field work was performed by the Nashville district (ORNED) primarily for the purposes of obtaining samples for a) conventional undrained monotonic strength testing of the embankment materials and foundation alluvium, and b) to determine the steady state strength of the foundation alluvium from laboratory strength tests using a procedure developed by the firm Geotechnical Engineers, Inc.(GEI).

1977 and 1979 field and laboratory investigations

46. Undisturbed samples were obtained with a 3-in. diam Hvorslev fixed piston sampler. The hole was stabilized with drilling mud during the sampling procedures. A Pitcher sampler was used in zones where gravel was encountered in the foundation. The 1977 and 1979 work was performed by WES.

47. The undisturbed samples proved to be useful in the evaluation of the stratigraphy of Barkley Dam. Samples were split open and photographed to obtain a visual record of the nature of the soils in foundations Units 1, 2,

and 3. The photographs were used to gain an appreciation of the complex layering of the foundation soils, especially those in Units 2 and 3. Example photographs of samples retrieved from Units 2 and 3 are presented in Figures 15 and 16, respectively.

48. Undisturbed samples were drained then frozen to minimize disturbance prior to laboratory testing. The samples were divided into three basic groups according to soil type: sands, nonplastic silty sands, and specimens with plastic fines for laboratory testing. Laboratory index tests included density, specific gravity, mechanical analysis, maximum and minimum density tests, and Atterberg Limits. The triaxial tests consisted of isotopically consolidated, undrained, stress-controlled, cyclic triaxial tests (CTX) and isotopically consolidated, undrained, stress-controlled compression shear tests with pore pressure measurements ($\bar{R}$). The CTX were meant to determine cyclic strength and the $\bar{R}$ tests were performed to study the dilative and contractive behavior of the soils at various void ratios and confining stresses.

49. Ultimately, the test results from the CTX and $\bar{R}$ tests were not used in seismic analysis. This was because freezing samples with high fines content caused excessive sample disturbance and significantly altered the undrained monotonic and cyclic strength.

<u>1984 field and laboratory studies</u>

50. In 1984, the Nashville District drilled six undisturbed borings in the switchyard area. Undisturbed samples were obtained using a 3-in. diam Hvorslev fixed piston sampler and drilling mud to stabilize the hole. These samples were obtained for performing triaxial shear tests to determine: (a) the shear strength of the embankment soils, b) the clays of Units 1 and 2, and, (c) the steady state strength of the sands of Unit 2.

51. Undisturbed samples were obtained for performing triaxial shear tests to determine undrained and drained shear strength parameters of the embankment soils and the clays of Units 1 and 2. Table 4 summarizes the results of these tests (32 $\bar{R}$ and 4 R tests) which were performed by the South Atlantic Division Laboratories. The table also reports the results of index tests performed on the tested samples including the water content, the dry and moist unit weights, and the Atterberg Limits.

52. The firm, Geotechnical Engineers Inc. (GEI), performed a series of thirteen $\bar{R}$ tests on high quality undisturbed specimens to estimate the steady

state strengths of sands in the foundation (Unit 2). According to the steady state theory developed by GEI, the undrained, steady state shear strength of a particular soil depends upon its in situ void ratio. The results of the tests performed by GEI are summarized in Table 5 and in Figure 17. Table 5 and Figure 17 show the steady state strengths and void ratios measured in the laboratory and estimates of appropriate values for the foundation sands. The in situ steady state strengths were estimated to account for differences between the in situ void ratios and the as tested void ratios of the specimens. These corrections were necessary to account for unavoidable densification during sampling and consolidation. These densifications cause the as-sheared void ratio to be lower than the in situ value, resulting in a measured laboratory strength which is higher than the actual in situ strength. Based on the laboratory tests, Table 5 shows that the steady state strengths range between 5 and 94 psi. A value of 8 psi (1152 psf) was recommended by GEI for the seismic evaluation.

53. It will be shown in another section of this report that the undrained residual strength, S_{ur} , can be estimated in situ from the Standard Penetration Test (SPT) blowcounts (Seed, 1986). The S_{ur} is equivalent to the GEI steady state strength. In the post-earthquake stability analysis the S_{ur} is assigned to materials which have a high probability of liquefying during the design earthquake.

Standard Penetration Tests

General

54. During the course of the field investigations, 44 SPT borings were drilled in addition to the 11 undisturbed borings discussed in the previous section. Table 6 lists the locations, depths, and other information relevant to the performance of the SPT's at the Barkley Project. The locations are shown on the plan view of Figure 14. The SPT data include blowcounts, jar samples from split-spoon samples, and the results of index tests. The SPT data were organized and stored in a data base for the evaluation of the liquefaction potential of the foundation soils and to assist in characterizing and idealizing the site.

55. The SPT investigations were conducted during five different periods 1977, 1979, 1981, 1982, and 1984. As shown in Figure 14, all data were

obtained downstream of the centerline. Results from tests performed downstream of the centerline were assumed to apply to the upstream portion of the dam. The SPT's were performed to obtain data regarding the foundation soils in the vicinity of the switchyard (between Sta 39+00 and 44+00) and along the main embankment (between Sta 44+00 and Sta 90+00).

SPT procedures

56. The 1977 and 1979 field work was performed by WES. In the WES procedure, a 140-lb hammer was dropped 30 in. with a trip hammer to drive the split spoon through the first 18 in. of the sequence, and the hole was then advanced another 18 in. for a total depth of 3 ft, with a modified fishtail bit. The fishtail bit was equipped with baffles which deflects the mud in an upward direction. The sampler was not equipped with a liner, and the rod string between the anvil and the sampler used "N" type rods. It was estimated that the energy ratio for the trip hammer is about 80 percent. Thus, to determine equivalent SPT N-values for a rope and cathead system (an energy ratio of 60 percent), the trip hammer blowcounts need to be multiplied by a factor of 1.3.

57. The remaining SPT work for this study was performed by ORNED with a rope and cathead system. ORNED performed the SPT's with two different clean out distances. Continuous SPT's refer to borings with no clean out distance, so a continuous observation can be made of the underlying soil layers. Standard SPT's refer to borings with a clean out distance between split-spoon drives of 1 ft or greater. A column in Table 6 indicates the method of drilling for each of the SPT borings.

58. The WES procedure and standard procedures used by ORNED leave a blind spot at depths between SPT drives. This blind spot complicates attempts to correlate individual layers between borings. The problem is compounded at the Barkley site where the soil layers are intensely layered and where 18 in. is significantly greater than the thickness of a typical layer. However, cleanout distances between SPT drives of less than 1 ft may lead to disturbance of layers immediately in front of the advancing split spoon, and consequently misleadingly lower blowcounts.

Laboratory index testing on SPT samples

59. Index tests were performed on nearly all of the SPT samples. In general, the tests performed were water content, Atterberg limits, sieve and

hydrometer analysis. The information obtained from the index tests was used later in the evaluation of the liquefaction potential of the foundation.

60. Laboratory personnel logging the tests were instructed to save the entire 18-in. drive and take separate jar samples for each type of material. Because of the interbedded nature of the foundation, this was difficult to accomplish, consequently, many of the jar samples were mixtures. Thus, special care was necessary to accurately classify the foundation soils, especially those of Unit 2. Laboratory personnel were instructed to separate the different layers, if possible, and perform the index tests on the separated samples if enough material was available.

SPT data base

61. The enormous quantity of data collected from the SPT field and laboratory work was assembled into a data base designed to assist in the assessment of liquefaction resistance of the foundation soil. The following information is needed for evaluating the cyclic strength from individual blowcounts: the exact location and top-of-hole elevation of the boring it comes from, the water level at the time of sampling, the unit weights of overlying soils, the depth interval of the SPT drive, the drilling method (i.e. trip hammer or rope and cathead), the blowcounts for each 6 in. of the 18-in. drive, field classifications, and laboratory index tests for each soil layer from the jar samples. The index test data recorded for each laboratory sample was: the grain size distribution in terms of D_{60}, D_{50}, B_{30}, D_{10}, percentage passing the No. 200 sieve and the percent finer than 0.005 mm; the liquid limits (LL); the plastic limit (PL); and the natural water content (w_n); and the color of the sample.

62. The entire SPT data base actually consisted of three smaller sub-data bases. The first data base is the boring data base which identifies the name, location, and other pertinent details for each SPT boring. The second data base is the SPT sampler data base, which identifies the locations, depths, blowcounts, and number of jar samples for each SPT drive. The third is the laboratory index test data base which identifies the location and depth interval for each jar sample, and the results of the laboratory index tests. An example of the data contained in the laboratory index test data base from Boring BEQ-30 is shown in Table 7.

SPT data plots

63. Information contained in the data bases was plotted in various forms to characterize the foundation and assess the liquefaction potential. A typical SPT data plot, for BEQ-10, is shown in Figure 18. Similar plots were prepared for each of the 44 SPT borings. The following information is plotted versus depth in the figure: measured SPT blowcount, mean grain size (range and weighted average), percent passing the No. 200 sieve (range and weighted average), natural water content, and Plastic and Liquid Limits.

Streambank Excavation

64. It was determined that examination of exposures of Unit 2 in the streambanks downstream of the dam would offer a practical solution to the problem of evaluating the continuity of individual layers of sands and silty sands in Unit 2. Preliminary liquefaction analysis indicated that these materials needed further investigation and appeared to have a high potential for liquefaction. As SPT and undisturbed sampling investigations progressed, it became evident that it was not usually possible to correlate individual soil layers between borings even when spaced within 10 ft of one another. This high degree of complexity within Unit 2 made detailed mapping of the soil profile difficult at best. Since the foundation materials in question were at significant depths, roughly 15 to 55 ft, deep test pits would be very expensive and prohibitive in cost.

65. After some reconnaissance, an exposure of materials considered to be representative of those of concern in Unit 2 was found about 1.5 miles downstream of the dam on the right bank. The location of this exposure relative to the damsite is included in Figure 19. The exposure was developed during the period of 31 October to 1 November, 1983.

66. A geologic section of the face of the exposure is shown in Figure 19. Additionally, photographs of the exposure are included in Figure 19a. The face of the exposure was oriented parallel to the river, so little was learned of the nature of the soil beds in the direction perpendicular to the river. The final dimensions of the exposure were about 30 ft long by 5.5 to 6 ft high. The maximum thickness of an individual soil layer was 1.5 ft. The average thickness of the beds was be on the order of about 2 to 4 in. and was undulating in nature. Lengths of the beds varied greatly, from several inches to lengths greater than the mapped exposure (30 ft). One bed, outside the

limits of the exposure, was traced for a distance of about 150 ft before it could no longer be traced. It should be noted that some generalizations in descriptions had to be made during the logging. A bed shown in the figure may contain minor lenses or zones of material that may vary in description from what was logged. Based on this field exercise, it was concluded that significant continuity may exist in sand layers in the direction parallel to the river. This important assumption was employed in subsequent stages of the seismic stability evaluation.

Cone Penetration Tests

General

67. CPT investigation techniques were used to reveal stratigraphy and measure in situ strength. The CPT has advantages which make it particularly well suited to the Barkley site: the technique provides a continuous record, can resolve stratigraphic changes with a resolution of a few inches, and has a relatively low cost per foot. In the course of the seismic safety evaluation, it was determined that the switchyard and riverbank area were a critical zone. Thus, the CPT's were performed only in this area. In later stages of the study, the CPT results were used qualitatively for stratigraphic correlation to estimate continuity and areal extent of problem zones, and quantitatively to classify soils, predict liquefaction resistance (cyclic strength), and residual strength, S_{ur}.

Description of CPT program

68. Sixty-five CPT soundings were performed in the switchyard and riverbank area by Geoelectronics and Earth Technology Corporation (ERTEC). A summary of the 1985 testing program is provided in Table 8 and a plan view showing the locations of the CPT soundings is shown in Figure 20. The depths listed in Table 8 indicate that the CPT soundings were primarily intended to investigate the characteristics of Unit 2. Thus, most CPT soundings were terminated after limited penetration into Unit 3. Nonetheless, some data from the upper reaches of Unit 3 were recovered.

69. The CPT soundings had two main objectives, to reveal stratigraphy and to estimate strength. To study the stratigraphy, long strings of soundings were made parallel and perpendicular to the dam axis through the switchyard. As discussed in a previous section, the streambank excavation suggested

that the foundation layers extended for distances of 5 to more than 30 ft in the direction of flow, and thicknesses of the layers also varied. Continuous layers with lengths of 30 ft or greater have a more significant effect on the stability than smaller discontinuous layers. Thus, a spacing of about 25 ft between probings was estimated to be a practical limit for detecting layers that extend over lengths of about 30 ft. The spacing of most probes was about 40 ft. CPT strings parallel to the dam axis are best for detailing valley stratigraphy such as identifying channel cuts and sandbar locations and determining the variation in soil strength along these surfaces.

70. In many cases, CPT soundings were positioned near SPT and undisturbed sampling borings (within 10 ft) to relate strengths and stratigraphy observed from CPT results to observations made from other in situ sources of information. Figure 20 shows the locations of SPT and undisturbed sampling borings near to the CPT soundings.

Basic description of CPT

71. The standard CPT test involves pushing a 1.4-in. diam probe into the earth at a rate of 2 cm/sec while monitoring the cone or tip resistance, q_c , and the sleeve friction resistance, f_s . The cone resistance is a bearing capacity measurement of the cone tip. The sleeve friction is a localized strength measurement of the soil as it passes a cylindrical steel sleeve located just behind the cone tip. These simultaneous measurements are made by means of electrical strain gages bonded inside the probe unit. Continuous electrical signals are transmitted by a cable in the hollow sounding rods to electrical equipment in the CPT truck. Cone and sleeve friction resistances are recorded versus depth in both analog and digital form. A set of hydraulic rams are used to push the cone and rods into the earth. The ERTEC used a specially designed, all-terrain drive, 23-ton, heavy-duty truck to transport and house the CPT equipment.

CPT soil classifications

72. The CPT can be used as aid to classify and identify different soil types encountered by the probe. A classification scheme devised by Olsen (Olsen 1988) was used to identify the soil types encountered by the CPT probe. The chart used in the CPT soil classification scheme is shown in Figure 21. The basic idea behind this system is that soil types can be identified by the combinations of corrected values of sleeve friction and cone resistance, f_{sn} and q_{sn} . The corrected parameters, f_{sn} and q_{sn} , are the results of

adjustments made to the measured values of sleeve friction and cone resistance, f_s and q_c. These adjustments correct the measured values to overburden stress conditions of 1 tsf. In the algorithm for mapping the CPT parameters onto the soil classification chart, the output is a Soil Characterization Number (SCN) which correlates with the basic soil type. For example, a CPT SCN value in the range between 0.5 and 1.0 represents a clay, between 1 and 2 represents a silt mixture, between 2 and 4 indicates a sand. Fine sand has a SCN between 2.5 and 3.5.

CPT results

73. A database containing the results of the CPT investigations was compiled for use in the stratigraphic evaluation and the liquefaction analysis. The database includes the tip resistances, q_c and q_{cn}, the sleeve friction readings, f_s and f_{sn}, and the CPT soil classifications as functions of depth for each CPT boring. Data from CPT-36, shown in Figure 22, are an example of the information contained in the CPT data base. The left and center panels show the cone and sleeve readings (both measured and corrected) which were used to enter the charts of Figure 21 to determine the SCN's and soil types which are shown in the right panel. The depth ranges of each of the three major foundation units are superimposed on the figure. Charts similar to that for CPT-36 were developed for each of the 65 CPT soundings made at the dam site (See Appendix B to Volume 4).

74. Cross-sections were developed from the cone resistance and soil classification charts to aid in the interpretation of the stratigraphy and site idealization. For example, Figures 23 and 24 show tip resistances and CPT soil classifications from the CPT soundings located along section D-D' (see Figure 20). Similar sections showing tip resistances and CPT soil classifications were developed for CPT soundings performed along sections A-A'(along the switchyard berm) and B-B' (along the downstream toe) as shown in Figure 20 (see Volume 4).

75. Analysis of these sections enabled a refined interpretation of the foundation stratigraphy to be made. In Unit 1, the cone resistances have a general tendency to decrease with depth. Since the CPT is an index of undrained shear strength, this feature indicates that the clays are likely to be overconsolidated due to desiccation. The cross sections also show that some sands are present. The cross sections shows that the boundary between

Unit 1 and Unit 2 undulates to some degree but generally the boundary lies between elevations 325 and 320 in the switchyard area.

76. In Unit 2, CPT cone resistance profiles are characterized by spikes superimposed upon relatively low penetration resistance as was discussed earlier. The spikes are indicative of sands and the low values are indicative of clays. The points of the q_{cn} trace for the clays align themselves in a nearly vertical line which indicates that the clays in Unit 2 are normally consolidated. The CPT classifications alternate between sand mixtures and clays and show that the clays are the dominant material in Unit 2. This is consistent with the laboratory classifications of SPT samples. It was not possible to correlate the continuity of the individual sand layers from sounding to sounding in Unit 2 from the CPT data.

77. The boundary between Units 2 and 3 undulates to a minor degree in both directions but generally lies between elevations 305 and 300 in the switchyard area only. The CPT data reveal that Unit 3 is generally made up of sandy materials with some interbedded clays. Unit 3 was subdivided into three basic zones of materials based on analysis of the CPT data: Units 3a, 3b, and 3c. The interpreted boundaries of each of are shown in Figures 22 through 24. In general there is a marked increase in penetration resistance as the probe crosses the boundary between Units 2 and 3a. The increase is due to an increase in density in the sands and the presence of gravel of Unit 3a. The CPT data show that the Unit 3a sand layers are probably much thicker than the Unit 2 sand layers which is another possible reason for their higher penetration resistance values.

78. A zone of low cone resistance, designated as Unit 3b, was detected between the elevations of 295 ft and 290 ft. The cone resistance profiles of this zone have an appearance which is remarkably similar to those of Unit 2 with some thin sand lenses frequently interbedded within the clay. This zone is extensive as it was detected by nearly all of the CPT soundings performed in the switchyard, therefore it was treated as a characteristic of the site in the switchyard area. The foundation materials below the low blowcount zone were designated Unit 3c and have characteristics similar to those of Unit 3a. An analysis was performed to estimate the sand content in each foundation unit using the CPT soil classifications. The results of this analysis are shown in Figure 25. The figure shows that sand makes up less than 20 percent of Unit 2 (located between elevations 320 and 305). There is even less sand in Unit 2

in the switchyard area than in the free field area. The normally consolidated clays comprise the majority and remainder of Unit 2. In Unit 3, sand is the dominant material making up between 60 and 80 percent of the material of Unit 3 (below elevation 305). In the switchyard the presence of Unit 3b is detected between elevation 288 and 293 where the sand percentage decreases. The sand percentage in the switchyard increases in Unit 3c below elevation 288.

Summary

79. In this study, the liquefaction potential of the sand in Unit 2 was a cause for concern due to its low penetration resistance. Thus, in the stratigraphy evaluation it was essential not only to identify and classify the various material types but to map their lateral extent. It was not possible to map the extent of individual sand layers from one CPT or SPT sounding to another. However, an excavation in a downstream exposure of Unit 2 revealed that some of these layers were undulating and continuous over fairly long distances in the direction of river flow. This continuity was assumed and used in the liquefaction studies where the sandy materials of Unit 2 were treated as continuous.

PART IV: EVALUATION OF THE LIQUEFACTION POTENTIAL OF THE FOUNDATION SOILS OF BARKLEY DAM

General

80. In this study the liquefaction potential of the soils in the three foundation units beneath the embankment sections were evaluated using Seed's performance based approach (Seed et al. 1983, 1984). The following steps were applied to evaluate the liquefaction potential:

- a. Calculate the initial effective stresses existing in the embankment and foundation before the earthquake using static finite element analysis.
- b. Calculate earthquake-induced dynamic shear stresses using dynamic finite element analyses.
- c. Estimate the cyclic strengths and pore water generation characteristics using the results of insitu and laboratory tests.
- d. Compute safety factors against liquefaction from steps a, b, and c above.

The liquefaction potential is quantified by computation of safety factors against liquefaction (FS_L) which compares the cyclic strength of the soil with the dynamic stress induced by the earthquake.

81. In this study, the Seed approach was used to identify and estimate the extent of the soil deposits in the foundation most likely to liquefy as a consequence of the design earthquake. The seismic performance of two embankment sections was investigated. These were representative sections from the switchyard area and the main embankment area. Finite element techniques were used to estimate the dynamic shear stresses for the switchyard area and one-dimensional wave propagation techniques were used for the main embankment. The locations of the two analysis sections are shown in the plan view of Figure 26.

Static Analysis

General

82. A static analysis was performed to determine the state of stress existing in the embankment and foundation just before the earthquake. The

static analysis was necessary since the cyclic strengths (liquefaction resistance) of soils depend upon the pre-earthquake state of stress.

83. A static finite element analysis was performed to determine the stress state of the switchyard section. The static stresses estimated from this analysis were used in the determination of the cyclic strengths of soils in the cross-sections of both the switchyard and main embankment sections. The procedures for estimating cyclic strengths of soils for the Barkley project will be discussed in a later section of this volume.

84. The computer program FEADAM84 developed by Duncan, Seed, Wong, and Ozawa (1984) was used to perform the static analysis of Barkley Dam. FEADAM84 is a two-dimensional, plane strain, finite element solution developed for the calculation of the static stress, strains, and displacements in earth and rockfill dams and their foundations. The program uses a nonlinear hyperbolic constitutive model to estimate the nonlinear stress-strain behavior of soils.

Finite element inputs

85. Seven material types were modeled in the switchyard section. These materials included:

- a. Random embankment fill.
- b. Compacted embankment fill.
- c. Submerged compacted embankment fill.
- d. Unit 1 - lean alluvial clay.
- e. Units 2 and 3a - silty sand.
- f. Units 2 and 3b - clay.
- g. Unit 3c - sands and gravels.

The distribution of these materials is shown in Figure 27. The hyperbolic parameters listed for each material are listed in Table 9. Submerged unit weights were used for all materials below the phreatic line. The finite element mesh used in the static analysis is shown in Figure 28 and had 265 elements and 293 nodal points. This mesh is different than the one used in the dynamic analysis which will be discussed in a following section.

Results of static analysis

86. The results of the static finite element analysis are presented in the form of contour plots of vertical effective stress, shear stress on horizontal planes, and alpha (α) values. The contour plots for these stresses are presented in Figures 29 through 31, respectively. The α contours of

Figure 31 represent the ratio of initial static shear stresses acting on horizontal planes to the vertical effective stresses.

87. The static stresses were used to determine the cyclic strengths at various locations in the foundation soils, since the cyclic strength at a particular location is dependent on the vertical effective stress and the α value at that point.

Dynamic Response Analyses

General

88. A two-dimensional plane strain dynamic finite element analysis was performed using the computer program FLUSH (Lysmer et al. 1973) to calculate the dynamic response of the switchyard section to the motions of the design earthquake. The objectives of the analysis were to determine dynamic shear stress histories, peak accelerations at selected points in the cross section, earthquake-induced strain levels, and the fundamental period of the dam at the earthquake-induced strain levels. The information gained from the dynamic analysis is required to evaluate the liquefaction potential of soils in the idealized section.

89. FLUSH solves the equations of motion using the complex response technique assuming total stress conditions. Non-linear soil behavior is approximated using the equivalent linear constitutive model which relates shear modulus and damping ratio to the dynamic strain level developed in the soil. In FLUSH, the differential equations of motion are solved in the frequency domain and an iterative procedure is used to determine the appropriate modulus and damping values which are compatible with the developed level of strain.

90. A series of one-dimensional wave propagation calculations using SHAKE (Schnabel, Lysmer, and Seed 1972) were performed to approximate the dynamic response of the main embankment section to the design earthquake. SHAKE was also used to develop a site specific free field ground surface accelerogram which could be used directly as input into the FLUSH analysis. SHAKE solves the wave equation in the frequency domain through the use of the Fast Fourier Transform. The nonlinear strain dependent soil properties of shear modulus and damping are handled with a similar equivalent linear procedure used by FLUSH.

91. SHAKE uses on the following assumptions:

a. All layers in the profile are horizontal and of infinite lateral extent. Level ground conditions are assumed to exist, thus prior to the earthquake there are no static shear stresses existing on horizontal planes.

b. Each soil layer in the profile is defined and described by its shear modulus, damping, total unit weight and thickness.

c. The response of the soil is caused by horizontally polarized shear waves propagating vertically through the soil layers in the system.

d. The acceleration history which excites the soil profile consists of shear waves.

e. The equivalent linear procedure satisfactorily models the nonlinear strain dependent modulus and damping of the soils in the profile.

Site specific ground motions

92. Free field ground surface motions and rock outcrop motions were required for input into the dynamic response analysis of the switchyard and main embankment sections, respectively. The site's free field ground surface motions were input to FLUSH. The accelerogram presented in Figure 5 was recommended for use in this study (see Part II). However, these motions represent the response that would be measured at the surface of a firm soil site in the vicinity of the Barkley Project. Since the natural alluvial soils of the Barkley Project's free field are soft, it was necessary to develop an accelerogram from the original accelerogram of Figure 5 which would be suitable for the free field at the dam. Additionally, a rock outcrop accelerogram was the most expedient means of exciting the series of one-dimensional SHAKE profiles in approximating the dynamic response analysis of the main embankment.

93. A deconvolution process using SHAKE, was used to develop the ground motions for the site's free field ground surface and at a rock outcrop location. The process is illustrated with the aid of Figure 32. The numbers at various locations on the plot are keyed to the following discussion. The profile on the left side of the figure represents the firm soil profile on which the accelerogram of Figure 5 was actually recorded (Santa Barbara recording station) and the profile on the right represents the natural alluvial soil present in the free field of the Barkley Project. The original accelerogram was input to SHAKE at ground surface (Point No. 1) and deconvolved through the firm soil profile to obtain the acceleration histories at baserock

(Point No. 2) and at the rock outcrop location (Point No. 3). In a second SHAKE run, the accelerogram from Point No. 3 was input to SHAKE and propagated through the Barkley free field soil profile, to obtain the accelerogram at the ground surface (Point No. 5). The accelerograms at Points No. 3 and No. 5 were the desired outputs of this process. Details of the Santa Barbara (firm) soil profile and the Barkley free field profile are presented in Figure 33 and 34, respectively. Values of peak acceleration resulting from the deconvolution process at five locations are shown in Figure 35.

94. The rock outcrop accelerogram of Point No. 3 and its 5 percent damped response spectrum are shown in Figures 36 and 37. The ground surface accelerogram of the Barkley free field (Point No. 5) and its 5 percent damped response spectrum are shown in Figure 38 and 39, respectively. The accelerograms shown in Figures 36 and 38 were used as input to the one-dimensional analyses of the main embankment and the finite element analysis of the switchyard sections, respectively.

Finite element analysis of the switchyard section

95. Inputs: The finite element analysis of the switchyard section was performed using FLUSH. The mesh used in the analysis is presented in Figure 40. This mesh has 531 elements and 571 nodes. The element heights in the mesh were designed according to Lysmer's criteria (Lysmer et al. 1972) to insure that the frequencies in the range of interest propagated satisfactorily through the mesh.

96. The key material properties input to FLUSH include the total unit weight, low-strain amplitude shear modulus, and strain dependent modulus degradation and damping relationships for each element. The unit weights, shear-wave velocities, and shear modulus for each of the seven material types are shown in the cross-section of Figure 41 and are listed in Table 10. The modulus degradation curves used for both the one- and two-dimensional analyses are shown in Figure 42. These curves were developed by Seed et al. (1984) for cohesionless soils.

97. Results of the FLUSH analysis: The switchyard section was excited by applying the accelerogram shown in Figure 38 to a control point located at the ground surface of the site's free field. The motions of the free field response were transmitted to the finite element mesh across transmitting boundaries which separate the free field from the finite element mesh as shown in

Figure 32. The effective earthquake induced accelerations, peak accelerations at selected points, dynamic shear stresses, earthquake-induced shear strains, and the fundamental period were the most important items of information sought from these calculations.

98. Contours of the effective earthquake-induced dynamic shear stress are shown in Figure 43. The effective dynamic shear stresses can be thought of as an average uniform cyclic stress which is equivalent to the nonuniform cyclic stresses imposed by the earthquake. The effective shear stress is 65 percent of the peak shear stress in the nonuniform stress history. Safety factors against liquefaction were later calculated using the dynamic shear stresses shown in this plot.

99. Peak accelerations at selected nodal points are presented in Figure 44. The data in the figure show that there is a general trend for the peak acceleration to decrease with depth below the ground surface. The maximum peak acceleration was 0.28 g at the crest of the dam.

100. The effective strain levels determined in the finite element analysis are shown in Figure 45. The cross hatched area indicates the zone of elements which had the largest earthquake effective cyclic shear strains in the FLUSH analysis. The effective strains in these elements ranged from 0.7 to 1 percent. This area coincides largely with the foundation soils of Units 2 (interbedded sands within clay) and 3b (weak clay layer). This area extends completely across the section from the upstream in the vicinity of the switchyard to the downstream sides of the dam. The modulus degradation curves in Figure 42 show that for these levels of strain the modulus would degrade to about 8 percent of its maximum value. Thus, the finite element analysis predicts that significant earthquake induced strains and material softening can be expected in the foundation as a result of the design earthquake.

101. The lengthening of the embankment fundamental period during earthquake shaking is another measure of strain softening of the embankment materials. The pre-earthquake period of the embankment and foundation system were estimated using a simplified procedure developed by Sarma (1979). FLUSH was used to compute the fundamental period of the embankment and its foundation at the earthquake induced strain levels. The Sarma technique indicates that the pre-earthquake fundamental period of the switchyard section was about 0.75 sec. The period at the earthquake induced strain levels was determined to be 1.75 sec. A comparison of the two values indicates that the period

lengthens by a factor greater than two as a result of the straining caused by the earthquake.

Analysis of main embankment

102. General: The dynamic response of the main embankment was modelled using a series of 1-D wave propagation calculations to approximate a 2-D dynamic response. The 1-D analyses were performed using SHAKE which was described earlier in this chapter. The principal objective of these analyses was to determine the earthquake induced shear stresses in the foundation units beneath the embankment. The section selected to represent the main embankment is shown in Figure 46 and in the plan view in Figure 26.

103. One-dimensional soil profiles: Four 1-D soil profiles were developed to approximate the dynamic response of the representative cross section. The locations of these profiles are shown in Figure 46. Detailed information concerning the sublayering and total unit weights, and the shear-wave velocity properties used for each of the four profiles are presented in Figure 47 through 50. The shear-wave velocities in Units 2 and 3 of Profiles 2, 3, and 4 were adjusted to account for the increased overburden stresses caused by the overlying embankment.

104. Results of SHAKE analyses: Each of the four profiles was excited by the acceleration history shown in Figure 36 which corresponds to the rock outcrop location at Point No. 3 in Figure 32. The dynamic shear stresses (65 percent of the peak value) as functions of depth for each profile were the most important output items sought from the dynamic responses for each profile.

105. The dynamic shear stresses induced by the input accelerogram are plotted in Figures 51 through 54 for Profiles 1 through 4, respectively. The effective shear stresses plotted in these figures represent 65 percent of the peak dynamic shear stress for each layer in the profile. The plots show that for each of the four cases the effective shear stresses increased with depth. The dynamic shear stresses for Profile 1 were applied to locations in the free field and at the toe of the dam (See Figure 46.) In a subsequent section, the liquefaction potential of the foundation soils were evaluated by comparing the cyclic strengths of the sands with the dynamic effective shear stresses shown in Figures 51 through 54.

Estimation of Cyclic Strength of Foundation Soils

General

106. The Seed approach (Seed et al. 1983 and Seed, Tokimatsu, Harder, and Chung 1984) was used to estimate the cyclic strength of foundation soils at the Barkley site. Seed's methods use SPT blowcounts to determine the cyclic strength (liquefaction resistance) of a soil deposit. The procedures were developed for evaluating the liquefaction potential of sand, silts, silty sands, and gravels. Additionally, screening criteria are included for distinguishing liquefiable clays from nonliquefiable clays.

107. The cyclic strengths estimated from Seed's procedure depend mainly upon the SPT blowcounts and the fines content of the material. At the Barkley site, the application of this approach is complicated by the fact that the alluvial foundation consists of many thin silty sand layers which have varying amounts of fines. In deposits of this sort it is difficult to estimate the "true" SPT blowcounts. The CPT with its abilities to provide greater detail and a continuous record of information was used to overcome some of these difficulties. Additionally, procedures were developed to convert the CPT penetration resistances into equivalent SPT blowcounts so that Seed's performance based chart could be used to estimate the cyclic strength. The chart used in this study is shown in Figure 55.

108. Data from both the SPT and CPT data bases, discussed in Part III, were used to evaluate the characteristics of the alluvial foundation soils so that their cyclic strengths could be determined. The SPT blowcounts were used to evaluate the cyclic strength of the foundation alluvium beneath the main embankment and CPT predicted blowcounts were used to evaluate the cyclic strength of the foundation soils in the switchyard area.

SPT blowcounts

109. Data reduction procedures: Seed's method requires measured blowcounts to be corrected to determine the value that would be obtained had the Standard Penetration Test been performed under a specified set of standard conditions. Blowcounts obtained at standard conditions can be compared directly on a one to one basis. Factors which have a major influence on the measured SPT blowcounts include: the energy efficiency of the drilling procedure, the effective overburden (vertical) stress, and the fines content of the soil. The standard conditions to which all blowcounts are reduced include: a

vertical effective stress of 1 tsf, 60 percent energy efficiency, and a fines content of less than or equal to 5 percent. Blowcounts reduced to standard conditions are designated N_{1c} . The following paragraphs discuss the data reduction procedures employed to determine the blowcounts for standard conditions.

110. The field measured blowcounts were corrected to a vertical effective stress of 1 tsf by multiplying by the factor C_N shown in Figure 56 (Bieganousky and Marcuson 1975). The overburden corrected blowcount, N_1 , is determined using:

$$N_1 = N_{meas} \times C_N \tag{1}$$

where

N_1 = SPT blowcount corrected to an effective stress of 1 tsf

N_{meas} = field measured SPT blowcount

C_N = Overburden correction factor of Figure 56

The field measured blowcounts were corrected to an energy delivery of 60 percent efficiency in accordance with guidelines described by Seed, Tokimatsu, Harder, and Chung, 1984. This accounts for the efficiencies of the different types of equipment used in the field. No correction was made to account for continuous Standard Penetration Testing, that is, testing with no clean-out space between 18-in. drives. Upon comparison of continuously measured SPT's and standard SPT's with a clean-out depth of at least 1 ft, no corrections seem to be warranted. As discussed previously, SPT blowcounts obtained with the WES trip hammer rig were multiplied by a factor of 1.3 to adjust the measured blowcounts to standard energy conditions. Blowcounts adjusted to account for overburden stress and for the energy efficiency of the drilling equipment were designated $(N_1)_{60}$.

111. An additional adjustment was made to each blowcount, $(N_1)_{60}$ to correct for the effect of the fines content (percentage passing the No. 200 sieve). The corrected blowcount is termed the equivalent sand blowcount and designated as N_{1c} . The subscript c indicates both the correction for energy efficiency as well as fine content. The equivalent sand blowcount, N_{1c} , has the same cyclic strength as the $(N_1)_{60}$. For example, the N_{1c} value is 15 blows/ft for a material having a $(N_1)_{60}$ value of 12 blows/ft and a fines

content of 15 percent as illustrated in Figure 55. The determination of N_{1c} for all SPT blowcounts allows SPT data obtained in materials having different fines contents to be compared on a one-to-one basis.

112. <u>SPT data analysis:</u> Analysis of the SPT data was difficult due to the complexities of the three foundation units at the damsite, particularly those of Unit 2. Meaningful results could only be obtained by keeping the findings of the stratigraphy evaluation in mind during the analysis of the data. Thus, the data were analyzed with the view that the interbedded sands and clays were distinguishable components and that the characteristics of each should be examined separately. The entire SPT data base was scanned boring-by-boring and the data were evaluated according to a set of criteria designed to establish the liquefaction potential of the materials in the foundation, particularly Unit 2.

113. The SPT data analysis of the sandy portion of the foundation was performed for the purpose of determining the blowcounts $(N_1)_{60}$ and fines content. The cyclic strength of the foundation sands could then be determined using Seed's chart of Figure 55.

114. The SPT data analysis of the clayey samples of the foundation was performed to assess their potential for liquefaction. Clayey soils having the potential for liquefaction were identified using empirically developed criteria recommended by Seed (1983) based on the findings of Wang (1981). These are commonly referred to as the "Chinese Criteria". These criteria are discussed in detail later in this part of the report.

115. <u>SPT blowcounts in sands:</u> The SPT data base was scanned on a boring by boring basis to identify each sample which classified as a sand, silt, or silty sand. The blowcounts for these samples were corrected using the data reduction procedures discussed previously to yield values for $(N_1)_{60}$ and ultimately N_{1c}.

116. In querying the data base, sand samples were considered to be those having between 0 and 50 percent of their material passing the No. 200 sieve. An analysis of samples meeting this criterion was performed to estimate the statistical distributions of D_{50} and fines contents of sandy soils of Units 2 and 3.

117. In the main embankment area where most of the SPT's were performed, analyses were performed to determine the statistical distributions of $(N_1)_{60}$ blowcounts, percentage passing the No. 200 sieve (fines content), and

the N_{1c} blowcounts. The main embankment encompasses the area between Sta 44+00 and 90+00. Histograms showing the statistical distributions of $(N_1)_{60}$, fines content, and N_{1c} for Unit 2 (between elevations 295 and 320 in main embankment area) are presented in Figures 57 through 59, respectively. Statistical analysis indicates that the distribution of $(N_1)_{60}$ has a mean value of 15.8 blows/ft and a standard deviation of 8.3 blow/ft. The distribution in Figure 58 shows that fines content has a mean value of 23.2 percent and a standard deviation of 16.2 percent. A histogram showing the distribution of equivalent sand blowcounts for the sandy soils of Unit 2 for the main embankment is shown in Figure 59. The mean and standard deviation for this set of data was 20.7 and 6.5 blows/ft based on data from 83 samples. Additionally, in Unit 2, the data base shows that most samples meeting this criterion classified as silty sands (SM). The average mean grain size of the sandy component of Unit 2 was 0.15 mm.

118. Similar histograms were developed for the $(N_1)_{60}$, fines content, and N_{1c} values of Unit 3 (below elevation 295). These distributions of data are shown in Figure 60 through 62, respectively. The distribution of $(N_1)_{60}$ had a mean value of 23.8 blows/ft and a standard deviation of 8.9 blows/ft. The fines content had a mean value of 16.1 percent and a standard deviation of 10.6 percent. The typical soil classification of sandy materials in Unit 3 was SM and the D_{50} was 0.20 mm. The histogram showing the distribution of N_{1c} or the sands in Unit 3 is shown in Figure 62. For N_{1c} , the mean and standard deviation were estimated to be 24.3 and 9.9 blows/ft based on 258 samples. The data in Figure 62 were obtained from SPT borings located across the entire site which includes the switchyard and main embankment areas while the data in Figures 57, 58, 59, 60, and 61 were obtained from borings located only in the main embankment area.

119. The statistical analyses of the SPT data for $(N_1)_{60}$ and fines content of the main embankment areas for Units 2 and 3 are summarized in Table 10. Table 12 summarizes the mean values of N_{1c} and its standard deviations for the alluvium beneath the switchyard area, main embankment area, and the total site. The information in Tables 11 and 12 was used to determine the cyclic strengths and residual strengths in the main embankment area. In the switchyard area, there was an insufficient number of SPT blowcounts in materials meeting the criteria for sand to have an adequate statistical base for determining the cyclic strengths. Hence, it was decided to use the CPT data

to estimate equivalent SPT blowcounts. The CPT predicted blowcounts were then used to determine the cyclic and residual strengths.

CPT predicted blowcounts

120. General: The CPT penetration resistances were used to estimate the equivalent SPT blowcounts (i.e. the SPT blowcount that would have been obtained in thick layers of the same soils) and fines contents so that the cyclic strength of the alluvial foundation in the switchyard area could be evaluated using Seed's empirical chart of Figure 55. The scheme for converting the CPT data points into their equivalent SPT blowcounts was devised by Olsen (1986). Basically, the procedure enables various combinations of CPT tip and sleeve resistances to predict the $(N_1)_{60}$ and N_{1c} blowcounts of a soil deposit.

121. The chart in Figure 63 was used to determine equivalent $(N_1)_{60}$ blowcounts from the CPT data. To use this chart, the tip and friction sleeve resistances, q_c and f_s, must be adjusted to the equivalent values for overburden stresses of 1 tsf to q_{cn} and f_{sn} as discussed in a previous section on CPT soil classifications. Contours of $(N_1)_{60}$ blowcounts which are functions of the corrected cone parameters are laid out in the figure. The value of the equivalent $(N_1)_{60}$ depends upon which contour the CPT data point falls on.

122. Similarly, the CPT data were also used to estimate the SPT fines corrected blowcounts, N_{1c} . The chart of Figure 64 was used to make this conversion. The equivalent N_{1c} depends upon which contour the adjusted tip and friction sleeve data points, q_{cn} and f_{sn} , fall on. The chart was developed from the concept that the fines correction of a soil can be predicted from CPT data. The fines correction, ΔN_1 , was estimated from the corrected CPT data using Figure 65. The ΔN_1 contours represent the difference in blowcounts between the $(N_1)_{60}$ contours of Figure 63 and the N_{1c} contours of Figure 64. This chart was developed with the aid of the Soil Classification Chart of Figure 21 and Seed's liquefaction chart of Figure 55. Details concerning the development of the charts of Figures 63 through 65 are given in Olsen, 1984 and Olsen and Farr, 1986.

123. Analysis of CPT predicted blowcounts: A statistical analysis was performed on the distributions of CPT predicted $(N_1)_{60}$ and N_{1c} blowcounts to determine the appropriate values to use for the determination of cyclic strength for the sands in Unit 2. The CPT predicted blowcounts for $(N_1)_{60}$ and

N_{1c} for sandy soils were estimated from the tip and sleeve resistances obtained from the Cone Penetration Tests using Olsen's charts in Figures 63 and 64, respectively. Sandy soils were those whose Soil Characterization Numbers (SCN) were greater than 2 (see Figure 21). In querying the CPT data base, Unit 2 was assumed to lie between elevations 320 and 305. A histogram showing the distribution of $(N_1)_{60}$ is shown in Figure 66. This distribution includes 1,119 data points, has a mean value of 14.5 blows/ft and a standard deviation of 8.9 blows/ft. This distribution is skewed to the right. The histogram showing the distribution of CPT predicted N_{1c} is shown in Figure 67, determined from the same 1,119 basic CPT data points as the $(N_1)_{60}$ histogram, has a mean value of 19.0 blows/ft and standard deviation of 7.2 blows/ft. This histogram shows that CPT predicted N_{1c} has nearly a symmetrical triangular distribution. The CPT predicted average fines content (percent passing the No. 200 sieve) is estimated to be about 15 percent from the mean values of N_{1c} and $(N_1)_{60}$ and use of Seed's cyclic strength chart in Figure 55.

124. In like manner to that of Unit 2, the CPT was used to predict the SPT $(N_1)_{60}$ and N_{1c} blowcounts for Unit 3 sands. In the statistical analysis of the CPT data from Unit 3, sands were considered to be those materials whose CPT combinations, q_{cn} and f_{sn}, gave Soil Classification Number's (SCN) which were greater than two. Unit 3 was considered to below elevation 305. The histogram for CPT predicted $(N_1)_{60}$ is shown in Figure 68. The distribution, from 7,037 combinations of CPT tip and sleeve readings, has a mean of 24.3 blows/ft and a standard deviation of 7.6 blows/ft. The histogram of CPT predicted equivalent sand blowcounts, N_{1c}, is shown in Figure 69. This distribution derived from the same 7,037 samples has a mean of 25.7 blows/ft and a standard deviation of 7.6 blows/ft. The relatively large number of data points in Unit 3 (as compared to Unit 2) indicates that sands constitute a greater percentage of Unit 3 than in Unit 2. The CPT predicted fines content (percentage passing the No. 200 sieve) is estimated to be about 7 percent from the mean values of N_{1c} and $(N_1)_{60}$ and use of Seed's cyclic strength chart in Figure 68. Comparison of the fines contents from either the CPT or SPT split spoon samples of Units 2 and 3 demonstrates that the Unit 3 sands have less fines than those of Unit 2.

125. A summary of the statistical analysis for CPT predicted blowcounts is listed on Table 13. The table gives means and standard deviations for the

CPT predicted $(N_1)_{60}$ and N_{1c} blowcounts for the sandy materials of Units 2 and 3. The CPT predicted fines contents were estimated using Figure 55 from the mean values of $(N_1)_{60}$ and N_{1c} .

126. It is important to note that for Unit 2, in the switchyard area (where most of the CPT's were conducted), the CPT predicted values for N_{1c} (mean of 20.7 blows/ft from Table 12) are significantly greater than the SPT N_{1c} values (mean 13.6 blows/ft from Table 11). The difference can be attributed to the sensitivity of the CPT to detect the very thin sand layers present in Unit 2 and the SPT's relative insensitivity to the detection of these same layers. In this situation, it is believed that the increased sensitivity of the CPT to the interbedded sand and clay layers of Unit 2 enabled the CPT to make a better measurement of the "true" penetration resistance of the sands in Unit 2 than is possible for the SPT.

127. Additionally, Table 12 shows that in Unit 3 the mean SPT value for N_{1c} is 24.3 blows/ft. Table 13 shows that the mean CPT predicted value for N_{1c} is 25.7 blows/ft. These results are in good agreement and demonstrate that the CPT technique for estimating the SPT blowcounts is reasonable in the Unit 3 sands and improves the overall confidence in applying the technique to Unit 2 where the sand layers are thicker. Thus, in Unit 2 in the switchyard area it was concluded that equivalent SPT blowcounts derived from the CPT were more useful for estimation of cycle strength than were the actual SPT blowcounts.

Estimation of cyclic strength

128. General: The cyclic strengths of foundation Units 2 and 3 were estimated from the SPT and CPT predicted equivalent clean sand blowcounts, N_{1c} , and Seed's correlations in Figure 55. The SPT blowcounts were used for the foundation sands beneath the main embankment and the CPT predicted blowcounts were used in the switchyard area. The 30 percentile (mean minus one-half standard deviation) of the statistical distribution for the CPT predicted N_{1c} blowcounts was judged to be the appropriate level of conservatism for the basis upon which to select cyclic strengths. The cyclic strengths obtained using Seed's chart in Figure 55 apply to standardized conditions where the magnitude is 7.5, the ground surface is level, and the vertical effective stress is equal to 1 tsf. The next section describes the factors that are used to extrapolate the chart strengths to non-standard conditions.

129. <u>Modification factors for cyclic strength:</u> Equations (2) and (3) show that adjustments must be made for magnitude, vertical effective stress, initial static shear stresses, and layer thickness to determine the cyclic strength for non-standard conditions.

$$\left[\frac{\tau}{\sigma_v'}\right]_{\substack{\alpha \neq 0 \\ \sigma_v' \neq 0 \\ M \neq 7.5}} = K_M \times K_\sigma \times K_\alpha \times K_{layer} \times \left[\frac{\tau}{\sigma_v'}\right]_{\substack{\alpha = 0 \\ \sigma_v' = 1 \\ M = 7.5}} \qquad (2)$$

$$\tau_{str} = \sigma_v' \; x \; \left[\frac{\tau}{\sigma_v'}\right]_{\substack{\alpha \neq 0 \\ \sigma_v' \neq 1\ tsf \\ M \neq 7.5}} \qquad (3)$$

where

σ_v' = effective vertical stress

α = initial shear stress ratio, the initial static horizontal shear stress (s_{xy}) divided by the effective vertical stress (r_v')

K_M = adjustment factor to correct cyclic strength for different earthquake magnitudes and different numbers of cycles

K_σ = overburden correction factor to adjust cyclic strength for confining stress other than 1 tsf

K_α = adjustment factor to correct cyclic strength for non-zero initial horizontal static shear stress conditions

K_{layer} = adjustment factor to account for the inability of thin sand layers to develop the full penetration resistance of the Cone Penetrometer in the CPT

$\left[\frac{\tau}{\sigma_v'}\right]_{\substack{\alpha = 0 \\ \sigma_v' = 1\ tsf \\ M = 7.5}}$ = cyclic shear stress ratio (liquefaction resistance) determined from empirical charts which correspond to σ_v' = 1 tsf, α = 0, and M = 7.5.

$\left(\frac{\tau}{\sigma_v'}\right)_{\substack{\alpha \neq 0 \\ \sigma_v' \neq 1\ \mathrm{tsf} \\ M \neq 7.5}}$ = cyclic shear stress ratio (liquefaction resistance) determined from empirical charts which correspond to $\sigma_v' \neq 1$ tsf, $\alpha \neq 0$, and $M \neq 7.5$.

130. The adjustment factor K_M adjusts the cyclic strength to account for earthquakes whose Richter magnitudes are not 7.5. The specified Richter, M, for the Barkley project is 8.5 (equivalent to m_b = 7.5). The K_M factor for an M of 8.5 is 0.89 (Seed 1983).

131. The overburden correction factor K_σ is applied to the cyclic strength to account for the nonlinear relationship between liquefaction and confining stress. Seed's charts give cyclic stresses which correspond to effective confining stress of 1 tsf. The relationship between K_σ and effective vertical stress is given in Figure 70.

132. The factor K_α is applied to the cyclic shear strength to adjust for the increase in liquefaction resistance due to the presence of pre-earthquake static shear stresses, typically caused by sloping ground surfaces. The value α is the ratio of the initial horizontal shear stress to the effective vertical stress. Both of these stresses were determined at various locations within the critical cross sections from the static analysis using FEADAM84. The relationship between α and K_α used in this study is presented in Figure 71. The curve for materials having relative densities of 55 percent was used because it was estimated that the sands of Units 2 and 3 had relative densities of at least 55 percent.

133. The thin sand layers in Unit 2 posed a major problem with respect to measuring the "true" CPT tip resistance in these sands since the measurements were influenced to a large degree by the softer underlying clay soils. It is believed that this resulted in the underestimation of the true penetration tip resistance. In the switchyard area, the cyclic strengths determined from Unit 2 as determined from Seed's charts were increased by 25 percent (K_{layer} = 1.25) to account for the inability to measure the true penetration resistance of the thin sand layers in Unit 2. (See Seed's letter report dated 3 February 1986 in Appendix D of Volume 4). In the main embankment area, a value of 1.0 was chosen for K_{layer} since the sand layers there are thicker than in the switchyard area.

134. <u>Selection of cyclic stress ratios for sands in the switchyard area of Unit 2:</u> The cyclic strengths in the switchyard area were determined from the CPT predicted blowcount data. The statistical analysis presented for the

CPT predicted blowcounts presented earlier in this section showed that the statistical distribution for N_{1c} had a mean value of 19.0 blows/ft and a standard deviation of 7.2 blows/ft (see Table 12). As stated earlier, the 30 percentile level of the N_{1c} distribution was adopted for selecting the cyclic stress ratio from Seed's chart in Figure 55. The 30 percentile is equal to the blowcount value one-half standard deviation less than the mean value. Thus the 30th percentile value for N_{1c} is:

$$N_{1c} = 19.0 - (7.2/2)$$

$$= 15.4 \text{ blows/ft} \approx 15 \text{ blows/ft}$$

Entering Figure 55 at 15 blows/ft results in a cyclic stress ratio of 0.165 for the Unit 2 sands. The cyclic stress ratio was corrected to a value of 0.184 after multiplying by the factors K_M (0.89) and K_{1ayer} (1.25) to account for the effects of a Magnitude 8+ event and the thin layers.

135. Selection of cyclic stress ratios for sands in the main embankment area of Unit 2: The cyclic strengths of the Unit 2 sands beneath the main embankment area were determined directly from the statistical analysis of the SPT blowcounts presented earlier in Table 12. This analysis indicated that the distribution of N_{1c} blowcounts had a mean value of 20.7 blows/ft and a standard deviation of 6.5 blows/ft. Thus, the 30 percentile value for N_{1c} was estimated to be:

$$N_{1c} = 20.7 - 6.5/2$$

$$= 17.45 \text{ blows/ft} \approx 17.5 \text{ blows/ft}$$

The cyclic stress ratio for an effective vertical stress of 1 TSF and level ground conditions was estimated to be 0.195 based on Seed's chart in Figure 55. The cyclic strength stress ratio was corrected to a value of 0.174 after multiplying by the K_M factor of 0.89 to account for a Magnitude 8+ event and by considering that K_{1ayer} was equal to one beneath the main embankment.

136. Selection of cyclic stress strength ratio for the sands in Unit 3a and 3c for both the switchyard and main embankment areas: The cyclic strength of the Unit 3 sands in both the switchyard and main embankment areas were

estimated from the CPT predicted N_{1c} blowcounts. In manner similar to that for Unit 2, the statistical analysis of the distribution for the CPT predicted N_{1c} blowcounts of Unit 3 sands showed that the mean value was 25.7 blows/ft and that the standard deviation was 7.6 blows/ft. Thus, at the 30th percentile the equivalent sand blowcount, N_{1c}, was 21.9 blows/ft which was rounded to 22 blows/ft. From Seed's charts in Figure 55, 22 blows/ft results in a cyclic stress ratio of 0.241. The cyclic stress ratio was corrected to a value of 0.214 after multiplying by the K_M factor of 0.89 to account for a Magnitude 8+ event. The value of 1.0 was used for K_{layer} in Unit 3. In the switchyard area these results were applied to Units 3a and 3c which were the sandier portions of Unit 3. Unit 3b was treated as a nonliquefiable clay. In the main embankment area this cyclic strength was applied over the entire thickness of Unit 3.

Evaluation of the lique-
faction potential of clayey soils

137. General: Thus far all of the discussion has centered around the evaluation of the cyclic strength and liquefaction potential of the sandy components of the alluvium at the Barkley site. The CPT data analysis gave indication that about 80 percent of the material in Unit 2 (switchyard area) consisted of the clay component (see Figure 25). Thus, the evaluation of the liquefaction potential of the clays would necessarily have significant implications regarding the dam's stability.

138. Seed (1983) provided screening criteria designed to distinguish liquefiable clays from non-liquefiable clays. Seed defined clayey soils as those which plotted above the "A-line". These criteria are listed below:

a. Liquid limit (LL) < 35 percent.

b. Natural water content > 0.9 times the LL.

c. Percent finer than 0.005 mm < 15 percent.

The above are referred to as the "Chinese Criteria" since they were developed as a result of experience obtained during Chinese earthquakes (Wang, 1979). In the analysis, it was assumed that fine-grained foundation soils meeting all three of these criteria would liquefy as a result of the design earthquake.

139. Analysis of clay samples: The data base of samples (disturbed and undisturbed) classified in the laboratory was queried to identify all of the samples which classify as clays. The liquid limit, water content, and fines

content (percentage finer than 0.005 mm) of each sample were compared against the Chinese Criteria.

140. Plots of the liquid limits versus plasticity index for the laboratory samples from Units 2 and 3 which were identified as clays are shown in Figures 72 and 73, respectively. The figures show that the clay soils from both units plot above the "A-Line" and classify as CL according to the Unified Soil Classification System. Also the data from both Units 2 and 3 fall within the same range indicating that Units 2 and 3 are very similar. The data shows that the liquid limit of most of the samples from Units 2 and 3 is less than 35 percent which meets the first (a) of the three Chinese criteria for being considered liquefiable. The average liquid limit of these samples was about 30 percent.

141. Histograms of the ratio of water content to liquid limit (w_n/LL) were used to evaluate the second criterion which states that the ratio must be greater than 0.9 for the clay to be considered liquefiable. The w_n/LL histograms for Units 2 and 3 are shown in Figure 74 and 75, respectively. As was the case with the PI vs LL plots these histograms show the w_n/LL distributions for Units 2 and 3 are very similar and fall within the same range. The mean w_n/LL ratio is about 0.80 for Unit 2 and 0.85 for Unit 3 which shows that the clays in these units have high water contents. The w_n ratios are only marginally lower than that given by criterion (b) for being considered a liquefiable soil.

142. The third criterion involved determining whether the percentage passing the 0.005 mm of the clays was typically less than 15 percent. This criterion is checked in the form of charts which show w_n/LL ratio plotted against the percentage of material finer than 0.005. The charts for Units 2 and 3 are shown in Figure 76 and 77, respectively. Since it has been established that the average insitu liquid limit is less than 35 percent, any point falling within the cross hatched boxes on these figures would classify as a liquefiable clay which meets all three of the "Chinese criteria". The plots show that the percentage passing 0.005 mm fall within the same range for both Units 2 and 3 which indicates the "clays" within these two zones are similar. The plots also show that nearly all the data points fall outside the shaded region; this indicates the clays are not liquefiable.

143. In summary, analysis of SPT data from samples of clayey soils shows that the clayey soils of Unit 2 and 3 are very similar having liquid

limits, w_n/LL ratios, and percentages passing 0.005 mm size which fall within the same ranges. Additionally, the clays in these units were considered to be non-liquefiable since all three of the Chinese criteria are not met. The major factor in arriving at this conclusion is the fact that SPT samples show that almost invariably more than 15 percent of these clay materials are finer than 0.005 mm. Additionally, the CPT data showed that the soils of Unit 3b were predominately clayey in nature; thus, in the remainder of the analysis these soils were treated as nonliquefiable clays.

Evaluation of liquefaction potential
of the sands in foundation Units 2 and 3

144. General: Safety factors against liquefaction, FS_L, were computed to evaluate the liquefaction potential and pore pressure generation characteristics of the sands in Units 2 and 3 for the representative cross section of both the switchyard (see Figure 2a) and main embankment areas (see Figure 46). The FS_L is defined as the ratio between cyclic strength and earthquake induced shear stress as expressed in Equation 4) below:

$$FS_L = \frac{\tau_{str}}{\tau_{dyn}} \tag{4}$$

145. For the switchyard area the FS_L's were computed at the centroids for each of the foundation elements of the finite element mesh shown in Figure 40 for Units 2, 3a, and 3c. The cyclic strengths, τ_{str}, for each of these elements were evaluated using Equations 2 and 3). The cyclic stress ratios (adjusted for magnitude) estimated from the CPT predicted equivalent sand blowcounts were 0.184 for Unit 2 and 0.214 for Units 3a and 3c. Values of K_σ and K_α were determined from the effective vertical stress and α ratio from the static analysis (see Figures 29 and 31) and the relationships shown in Figures 70 and 71, respectively. The earthquake induced stresses, τ_{dyn}, determined from the dynamic analysis using FLUSH are shown in Figure 43. Having determined τ_{str} and τ_{dyn} the safety factor for each element was evaluated with Equation 4.

146. For the main embankment area, the FS_L's were computed at the centers of the layers for each of the four one-dimensional soil profiles shown in Figures 47 through 50. As for the switchyard area, the cyclic strengths

were computed using Equations 2 and 3). The magnitude adjusted cyclic strengths used for the sands of Units 2 and 3 were 0.174 and 0.214, respectively. The K_{layer} factor for Unit 2 was set equal to one because the sands layers in Unit 2 beneath the main embankment are thicker than those beneath the switchyard. A K_{layer} factor equal to one was also assigned to Unit 3. The values of K_α were estimated based on the static finite element analysis of the switchyard area. Static stresses as determined from the FEADAM analysis at locations upstream of the centerline were applied to corresponding locations on the upstream side of the main embankment because the upstream slope geometry and foundation conditions of the switchyard area approximate those of the main embankment. The earthquake induced stresses used in the evaluation of the liquefaction potential for the foundation beneath the main embankment were presented earlier in Figures 51 through 54.

147. <u>FS_L in Switchyard Area:</u> Contours of the computed safety factors against liquefaction were drawn on the idealized cross section and shown in Figure 78. The cross hatched zones on the drawing represent zones where liquefaction is expected (zones where FS_L values were less than one). The figure shows that liquefaction is predicted throughout Unit 2 except for two zones located directly beneath the sloping section of the switchyard berm (between locations 450 and 325 ft downstream of the centerline) and directly beneath the main portion of the embankment from locations about 125 ft downstream to 100 ft upstream of the centerline. Safety factors in the first reach have FS_L values which are up to 1.2 and in the second reach FS_L values are up to 1.4.

148. In Unit 3, liquefaction is predicted in the sands of 3a and 3c at locations near the upstream and downstream toes. The downstream zone of liquefaction extends from the free field area to about 500 ft downstream of the centerline. The upstream zone of liquefaction extends from the free field to about 225 ft upstream of the centerline. A significant portion of Unit 3 located between the two liquefiable zones is not predicted to liquefy. The FS_L values within this zone tend to increase in directions toward the centerline and reach a maximum near the centerline where the FS_L contours reach values of about 1.4. As stated previously, the clay layer which comprises Unit 3b was treated as a nonliquefiable clay and safety factors against liquefaction were not computed for it.

149. FS$_L$ Main Embankment Area: The results of the computed FS$_L$'s of Units 2 and 3 are presented in Tables 14 through 18 for the one-dimensional profiles shown in Figure 46. All of the parameters and factors used in evaluating the foundation FS$_L$'s of each profile are listed in these tables. The values of α were determined using the data in Figure 31. Values of α for Profile 4 at the centerline and in the free field portion of Profile 1 were set equal to zero because shear stress on horizontal planes are zero at these locations.

150. The safety factors against liquefaction from Tables 14 through 18 were synthesized into the cross section shown in Figure 79. In Unit 2, the analysis indicates that liquefaction can be expected in both the upstream and downstream free field areas of the main embankment. The FS$_L$'s improve to values ranging between 1.1 and 1.25 beneath the sloping sections of the embankment where the contribution of initial shear stresses have their greatest effect increasing the cyclic strength of the foundation sands. Figure 79 shows that liquefaction is predicted in Unit 2 near the centerline where the FS$_L$'s are only about 0.9. Some liquefaction is predicted to occur in the upper reaches of the Unit 3 sands between elevations 285 and 295 ft beyond the upstream and downstream toes of the dam. Liquefaction is not expected in Unit 3 directly beneath the embankments where the analysis shows that FS$_L$'s are greater than 1.3.

Determination of post-earthquake strength conditions

151. General: The strengths postulated to exist in the various zones of the foundation immediately after the end of the earthquake are required items of information for the post-earthquake stability analysis. Post-earthquake strengths were recommended for the soils of the switchyard and embankment sections. For each section, these strengths were selected by independent consideration of the characteristics of Units 2 and 3 (including subunits). Post-earthquake strengths were first selected for each of the major material types in the foundation which include: the normally consolidated clays of Units 2 and 3b and the sands of the Units 2 and Units 3a and 3c. Finally, a set of criteria was adopted which lead to conservative choices in the assignment of the post-earthquake strengths of the foundation soils in the idealized cross sections.

Post earthquake strengths

152. Normally consolidated clays: The clayey portions of Units 2 and 3b were determined to be normally consolidated based on the results of the CPT's. The clays were assumed to have c/p ratios equal to about 0.31. If the clays are assumed to lose 20 percent of their strength due to the earthquake shaking (Thiers and Seed 1968). Thus, the post-earthquake c/p ratio will be equal to 0.25 (0.31 × 0.80). According to criteria to be described later elements determined to be controlled by clay were assigned the post-earthquake c/p ratio of 0.25.

153. Sands having FS_L less than one: Sands which have a safety factor, FS_L , less than one are assumed to have liquefied due to the motions of the design earthquake. The post-earthquake strengths of the liquefied sands in Units 2 and Units 3a and 3c were estimated from the CPT predicted equivalent sand blowcounts, N_{1c} . A performance based chart that relates SPT blowcounts to post-earthquake strength for (materials that liquefied) evaluation of post-earthquake slope stability was introduced by Seed (1986). This chart, shown in Figure 80, was developed from back-calculated undrained strengths, S_{ur}. The back-calculated residual strengths, S_{ur} , were related to SPT blowcounts observed at the sites having slopes that have failed by sliding due to liquefaction. The data from which the chart was developed came primarily from sandy sites.

154. Seed's work (1986) demonstrated that the correlation between SPT blowcounts and S_{ur} was dependent upon the fines content of the liquefied soil. Thus, the effective blowcount value, N_{1eff} , upon which the S_{ur} value of a liquefied soil is based is arrived at by adjusting the overburden corrected blowcount, N_1 , for the percentage of material passing the No. 200 sieve. The fines correction made for the determination of S_{ur} is different from that made for cyclic strength. The value for N_{1eff} is determined using the following equation:

$$N_{1eff} = (N_1)_{meas} + \Delta N_1 \tag{5}$$

where

N_{1eff} = fines corrected blowcount used to determine the residual strength of a liquefied soil

$(N_1)_{meas}$ - measured blowcount corrected to an effective vertical stress of 1 TSF. In this study, $(N_1)_{meas}$ was assumed to be equal to $(N_1)_{60}$

N_1 - correction for fines content

Values for ΔN_1 for different fines contents are listed in Table 19. Conservatively, the lower bound curve in Figure 80 was used to estimate the S_{ur} values from the N_{1eff} blowcounts for the liquefied sands in Units 2 and 3 for both the switchyard and main embankment areas.

155. Switchyard area: The residual strengths of the liquefied sands in Units 2 and 3 of the switchyard area were based upon the mean CPT predicted values for $(N_1)_{60}$ and fines contents listed in Table 13 which were derived from the statistical analysis of CPT data. The use of the mean value was justified because no correction was applied to increase the residual strength to account for the effect of thin layers on penetration resistance as was done for the cyclic strength discussed previously. The CPT predicted fines contents rather than the SPT fines contents were used in the main embankment area because their use resulted in a smaller ΔN_1 which in turn yielded a conservative value for S_{ur} . The N_{1eff} value was determined to be 15.5 blows/ft based on an $(N_1)_{60}$ value of 14.5 blows/ft and fines content of 15 percent (for a ΔN_1 value of 1 blow/ft). Thus, the S_{ur} of liquefied sands in Unit 2 was approximated to be 450 psf from the chart of Figure 80.

156. Similarly, in Unit 3 the N_{1eff} was determined to be 25.0 blow/ft based on an $(N_1)_{60}$ of 24.3 blows/ft and a fines content of 7 percent (for a ΔN_1 value of 0.7 blows/ft). From this, the S_{ur} value was estimated as 800 psf. This choice stays within the confines of Seed's basic data in Figure 80.

157. The recommended values of S_{ur} to be applied to liquefied sands in Units 2 and 3 of the switchyard area are summarized in Table 20.

158. Main embankment area: The post-earthquake undrained residual strength, S_{ur} , of the liquefied sands of Unit 2 was determined using mean values of the SPT blowcounts and fines contents from the statistical analysis discussed earlier. The data upon which S_{ur} was based is presented in Table 11. Based on an $(N_1)_{60}$ blowcount of 15.8 blows/ft and a fines content of 23.1 percent (for an ΔN_1 value of 1.7 blows/ft), N_{1eff} was determined to be 17.5 blows/ft. Thus, the S_{ur} of the liquefied sands of Unit 2 was estimated to be 700 psf. The S_{ur} of Unit 3 in the main embankment area was estimated to be 800 psf on the same basis as that for the switchyard area.

The recommended values of S_{ur} to be applied to liquefied sands in Units 2 and 3 of the switchyard area are summarized in Table 20.

Sands having FSL greater than one
pore pressure generation characteristics

159. Pore pressure generation can occur in sands even though the deposit does not liquefy. Tokimatsu and Yoshimi (1983) showed that the earthquake induced residual excess pore pressures (normalized with respect to vertical effective stress) are related to the normalized stress ratio (ratio of induced shear stress ratio to the stress ratio causing liquefaction) as shown in Figure 81. Tokimatsu and Yoshimi found that for most sands the relationship between normalized parameters falls within the shaded area. The FS_L is represented by the reciprocal of the normalized stress ratio on the abscissa. In this analysis, the elements with FS_L greater than one were assigned excess pore pressure ratios, r_u, according to the relationship defined by the dashed line in the center of the shaded area in Figure 81. In this study r_u, is the ratio of excess pore pressure to effective vertical stress σ_v' as defined by Equation 9 below:

$$r_u = \frac{u_e}{\sigma_v'} \tag{6}$$

160. The excess pore pressures generated by the earthquake were essential information to be input into the post earthquake seismic stability analysis. In this analysis, for Units 2 and Units 3a and 3c, the strength of nonliquefied sands was determined from the r_u value, the effective vertical stress, and a friction angle of 31 deg. The friction angle of 31 deg was used in the design of the dam as reported in the Design Memorandum (U.S. Army Engineer District, Nashville, TN, 1960).

Results for switchyard area

161. General: Post-earthquake strengths were assigned to each element according to criteria developed specifically for each of foundation Units 2, 3a, 3b, and 3c. The criteria account for the material types found in each unit and were reasonably conservative as to the final strength selected for each element.

162. <u>Criteria for Unit 2:</u> The criteria for foundation Unit 2 were developed under the premise that both sands and clays were likely to be present in any element. Hence, depending on the conditions it was possible to assign either the residual undrained strength, S_{ur} , or a Mohr-Coulomb strength (friction ratio of 31 deg with a r_u value) for sands, or a strength based on the post-earthquake c/p ratio of 0.25 for normally consolidated clays. The assignment of the proper earthquake strength first depended upon whether or not the sands in the element were determined to liquefy.

163. If the safety factor, FS_L , of the element was less than one (i.e. sands liquefied), the strength assigned to the element was the lower of the residual strength (S_{ur} = 450 psf) and the undrained strength, c , based on a post-earthquake c/p ratio of 0.25. In the comparison, the c/p based strength was determined by multiplying the effective vertical stress by 0.25.

164. If the safety factor, FS_L , was greater than one and if the S_{ur} was greater than the Mohr-Coulomb based strength (after allowing for the excess pore pressure) then the post-earthquake strength was based on the minimum value of the c/p based strength and the residual strength. This criterion controlled for cases where the safety factor was only marginally greater than one and guaranteed the element would be assigned a post earthquake strength at least as great as that of either the S_{ur} or the c/p based value. If the safety factor, FS_L , was greater than one and if the S_{ur} was less than the Mohr-Coulomb strength than the post-earthquake strength was assigned based on the smaller of the Mohr-Coulomb and c/p based strengths. This criterion controlled for FS_L values which were more than just nominally greater than one.

165. <u>Criteria for Units 3a and 3c:</u> These Units were treated as consisting only of sands and thus the logic for assigning post-earthquake strength is somewhat simpler than that for Unit 2. Units 3a and 3c were treated identically. Again, the criteria depended on the whether or not the sands in Units 3a and 3c are predicted to liquefy. If the FS_L is less than unity, the post-earthquake strength is assigned the undrained residual strength value, S_{ur} , of 800 psf. If the value of FS_L was greater than one then the post- earthquake strength assigned to the element under consideration was the larger of the Mohr-Coulomb based strength and the residual strength value. This criterion guaranteed that the strength of the sands in these

units would be at least as large as the S_{ur} for safety factors only slightly greater than one.

166. Criteria for Unit 3b: Unit 3b was determined to consist of normally consolidated nonliquefiable clays. Hence, the strengths were based on the post-earthquake c/p ratio of 0.25 only.

167. Recommended strengths for switchyard area: The post-earthquake strengths ultimately assigned to the switchyard cross section upon which the finite element analysis was based are shown in Figure 82. Zones (groups of elements) where the residual strength controlled are indicated by the value of S_{ur} . Zones where the post-earthquake c/p ratio governed are also indicated by the c/p value of 0.25. Areas in the foundation where the Mohr-Coulomb strengths controlled are indicated by the excess pore pressures values, r_u , to be assigned to these areas. A friction angle of 31 deg should be assigned to these areas of the cross section. The post-earthquake strengths shown in Figure 82 are recommended for the stability analysis of the switchyard section.

Main Embankment Section

168. The one-dimensional dynamic response and liquefaction analysis of the four profiles of the main embankment were discussed previously in Part IV. The results were presented in terms of the safety factors against liquefaction, FS_L , given in Tables 13 through 16 and in Figures 79.

169. The liquefaction analysis indicates that widespread liquefaction of Units 2 and 3 is not expected beneath the main embankment. Nonetheless, as a conservative measure undrained residual strengths are recommended for both Units 2 and 3 in the post-earthquake stability analysis. The recommended strengths are presented on the cross-section in Figure 83. A residual strength, S_{ur} , of 700 psf is recommended for the entire breadth of Unit 2 and 800 psf is recommended for Unit 3.

PART V: POST-EARTHQUAKE STABILITY EVALUATION

General

170. Analyses were performed to evaluate the stability of the switchyard and main embankment sections in their post-earthquake conditions. The stability analyses were performed using the computer program, UTEXAS2 (Wright et al. 1987). Spencer's method was selected over other methods because it satisfies all requirements for complete static equilibrium. The method assumes that the side forces are inclined at the same angle for each slice. The method has the capability of computing factors of safety against sliding for both circular and wedge shaped planar surfaces. During the analysis it was assumed that when the earthquake occurs the embankment has reached a steady state seepage condition which corresponds to a consolidated undrained condition.

171. The locations of the two sections analyzed are shown in the plan of Figure 26. Cross-sectional views of the switchyard and main embankment sections are shown in Figures 2b and Figure 46, respectively. The plan view shows that the switchyard section actually curves from the reservoir (upstream) to the tail race channel (downstream). Material properties and strengths assigned to each section were based on the recommended post-earthquake strengths presented earlier in Figures 82 and 83.

Material Properties for Stability Analysis

Pre-earthquake conditions

172. Table 22 summarizes the material properties postulated to exist just prior to the design earthquake event. This table lists the unit weights (moist and saturated), and shear strength parameters for consolidated-drained (S-strength) and consolidated-undrained (R-strength) conditions. The properties listed in the table were obtained from construction data in General Design Memorandum No. 3C (U.S. Army Engineer District, Nashville, TN 1960) and from the results of tests performed on undisturbed samples during this seismic investigation. Table 22 lists properties applicable to the stability analysis for materials associated with the embankment and each of the major foundation units.

Piezometric and pool levels

173. In the analysis of both sections, the upstream pool was assumed to be at elevation 360 ft. Since the crest elevation is 388 ft, 28 ft of freeboard are anticipated during the occurrence of the design earthquake. A normal line of seepage was assumed through the dam and a ground water elevation of 345 ft was assumed beyond the toe for the main part of the dam. In the switchyard section, the normal line of seepage was assumed through the switchyard meeting the tailwater elevation of 305 feet. The corresponding piezometric lines for the two sections analyzed are shown in Figures 27 and 46.

Post-earthquake conditions

174. General: The criteria for strength selection and the recommended strengths for the post-earthquake stability analysis were discussed previously in Part IV of this report. Table 23 summarizes the material properties ultimately used in the post-earthquake stability evaluations of the two embankment sections analyzed. The following paragraphs outlines the post-earthquake conditions for each section.

175. Main embankment section: The strength conditions presented in Figure 83, were used in the post-earthquake stability analysis of the main embankment dam.

176. Post-earthquake strengths for the embankment and Unit 1 were determined from the results of tests reported for construction record samples and from recent laboratory and insitu tests performed in samples from borings made for the seismic analysis. The following reas ning was employed to arrive at the strength parameters for Unit 1. No cyclic tests were performed on the materials of 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 percent can be expected for 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 percent reduction in their strengths after the earthquake motions had ceased.

177. In Unit 3, Figure 83 shows that an undrained residual strength of 800 psf was recommended for the analysis. It is believed that this represents a conservative choice because Figure 79 shows that in Unit 3 liquefaction is not expected across the entire cross section.

178. Switchyard section: The post-earthquake strength conditions of Figure 82 were used in the stability analysis. An excess pore pressure ratio of 50 percent was conservatively assigned to all non-liquefied zones in Unit 3. In the stability analysis, in Unit 2, all liquefied zones were assigned a residual strength of 450 psf except for the nonliquefied area near the centerline of the dam.

179. For the same reasons as given for the main embankment section, the assumption was made that the materials of the embankment and Unit 1 would experience a 20 percent reduction in strength as a result of the earthquake shaking.

Stability Analysis

180. The principal objective in performing the stability analysis is to calculate the minimum factor of safety against sliding if the section of interest is in its weakened post-earthquake condition. The procedures followed in this analysis were based on those suggested by Seed, 1987.

181. Additionally, the procedure allows a means for making a rough estimate of the deformations which might be expected for an embankment constructed on a liquefiable foundation. The problem of estimating deformations of an embankment following liquefaction of the foundation is difficult and not well defined. Limited research by Ledbetter and Finn (1991) indicated that for dams on liquefiable foundations deformations that result from shaking are much smaller than those that result from post-earthquake gravity loading especially if the foundation experiences large strength degradation. In this case, the deformations were estimated by analogy to observed embankment and foundation deformations reported by Seed et al. (1975) and Seed (1987). The type of deformations predicted using this technique occur after the end of the earthquake shaking when only static stresses are acting on the embankment.

Conditions and assumptions of analysis

182. The selection of appropriate failure surfaces for analysis was based on considerations of the surface geometry and the foundation stratigraphy. Searches were then made to find the minimum factor of safety against sliding with the embankment sections in their post-earthquake conditions.

183. For the main embankment section, circular failure surfaces were assumed. In Unit 2, the post-earthquake strengths of soils in the zones where liquefaction occurs are controlled by the residual strength of the sand. This

is conservative because any failure circle passing through a liquefied zone of Unit 2 must also intersect the soft clays which compose about 80 percent of the unit. Liquefaction of these clays is not anticipated.

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

185. Procedure: Evaluating the stability of the embankment under the above conditions was complex. Accordingly, Seed (1987) has proposed the following procedure for evaluating the stability of embankments after liquefaction as occurred in the foundation:

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

b. If, in condition (a), failure of the dam does not occur, then assume that the strength in the liquefied zone is zero. If 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 Barkley), then the stability of the embankment is controlled by the nonliquefied soil and the deformations of the embankment will be small.

c. If, in condition (b), the factor of safety is not significantly greater than one, then the residual strength required 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 to 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 estimated.

Results of the analysis

186. Main embankment: Figure 84 shows that the minimum safety factors against sliding for the upstream and downstream slopes were each 1.3 when the residual strength of 700 psf for the liquefied soil was used in the analysis (Item a). When the analysis was performed with zero strength in these zones; 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 ft (Item b). The pre-earthquake safety factors for the upstream and downstream circles were each 3.2.

187. The results suggest that sliding and large scale deformations are not anticipated, however, both the upstream and downstream portion of the main dam are expected to undergo large strains. Based on judgment, Seed estimated that the strain required to mobilize the residual strength was about 25 percent (see Seed's letter report dated 3 February 1986 in Appendix). Results of residual strength tests indicate that a strain of about 20 to 25 percent is required to reach the residual strength of these materials and this is what the dam is expected to undergo. The results of the dynamic analysis indicates that high strains can be expected in foundation Unit 2.

Switchyard section

188. Because liquefaction can occur in Units 2, 3a and 3c, stability analyses were performed on failure planes at elevations of 305, 295 and 288 ft (corresponding to the elevations of the these Units). 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 upstream and downstream failure circles for elevations 305, 295, and 288 are show in Figures 85 through 87, respectively.

189. Figure 86 shows that the minimum failure plane occurs at elevation 305 ft with factors of safety of 1.8 and 1.6 for the upstream and downstream 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). The pre-earthquake factors of safety for the upstream and downstream failure wedges were 4.4 and 5.6, respectively.

190. At failure plane elevations of 295 ft the factors of safety with the residual strengths mobilized in the liquefied zones are 2.3 and 2.2 for

the critical upstream and downstream failure surface, respectively, as shown in Figure 86. The corresponding factors of safety if zero strength is applied to the critical zone are 0.5 and 0.6 for the upstream and downstream failure surfaces, respectively. The pre-earthquake factors of safety for the upstream and downstream factors of safety are 4.4 and 7.2, respectively.

191. For failure plane elevations of 288, the factors of safety with the residual strengths mobilized in the liquefied zones are 2.3 and 3.1 for the upstream and downstream failure surfaces, respectively, as shown in Figure 87. The corresponding factors of safety if zero strength is applied to the critical zone are 0.5 and 2.4 for the upstream and downstream failure surfaces, respectively. The pre-earthquake factors of safety for the upstream and downstream failure surfaces were each 5.2.

192. The pre- and post earthquake safety factors discussed above are summarized in Table 24 for the main embankment and switchyard sections.

Estimated deformations

193. 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 ft and the zone consists of 20 percent sand, then 2 to 3 ft of horizontal movement can be expected. In the switchyard area where the primary zone of liquefaction is 15 ft thick and contains 20 percent sand, the expected horizontal deformations will be about 1 to 2 ft.

194. 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 of stability analysis

195. 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. The results of the stability analysis indicate that wide scale deformations or slope failure which would result in loss of the reservoir are not expected to occur as a result of the design earthquake in either the main embankment or switchyard areas. Vertical deformations on the order of 2 to 3 ft can be expected on the slopes

of the main portion of the dam. In the switchyard area vertical deformations of about 1 to 2 ft can be expected, but loss of the reservoir will not occur.

PART VI: SUMMARY AND CONCLUSIONS

General

196. This report summarizes the results of seismological, geological, field, laboratory, and analytical investigations which were conducted to evaluate the seismic stability of the earth embankments and foundations of the Barkley Lock and Dam Project, KY. The seismic performance of two representative areas were investigated to a specified design earthquake. These were the switchyard (near the concrete section and switchyard) area and the main embankment area (between Sta 43+00 and Sta 80+00). Of specific interest in these investigations were the evaluation of the seismic performance of the alluvial soils which make up the foundation of these two sections of the dam and the overall effect on these stability embankment sections.

Seismological Studies

197. The geological and seismological investigations conducted as part of this study revealed that an earthquake originating in the New Madrid Source Zone posed the most severe seismic threat to the site. The most severe case was used as the basis for determining the parameters and characteristics of the design earthquake used in this study. For this study, the design earthquake was specified to have a body-wave magnitude, m_b, of 7.5 occurring at a distance of 118 km in the New Madrid Source Zone. The peak ground motions assigned to the design event were an acceleration of 0.24 g, a peak velocity of 35 cm/sec, and a duration (time from the first occurrence to the last occurrence above 0.05 g) was 35 sec. The motions were to be applied to the ground surface of a firm soil site near the Barkley Project. The S 48°E component of the Santa Barbara record from the Kern County, CA earthquake of July 21, 1952 scaled to these parameters was selected to be representative of the motions at the site for the dynamic analysis.

Stratigraphy Analysis

198. The field and laboratory investigations provided essential elements of data for an analysis of the foundation stratigraphy. Understanding

the nature of the foundation stratigraphy was essential in evaluating the behavior of the foundation alluvium during the design earthquake event. This analysis was based on data collected from Standard Penetration Tests (SPT), Cone Penetration Tests (CPT), undisturbed samples, and an excavation of an exposure of a critical foundation material. The data were analyzed in order to clarify the conditions in the foundation at the dam site. The analysis showed that the foundation could be modeled by three basic foundation units designated as Units 1, 2, and 3. Generally, the overall foundation thickness was approximately 120 ft. Each unit had distinguishing characteristics. Foundation Unit 1 consists predominantly of clays which classify as CL materials. These clays which can be viewed as a topstratum, are overconsolidated due to desiccation. Generally speaking, Unit 1 is 30 ft thick and lies between the elevations of 350 and 320 ft. Unit 2 was considered to consist of two separate and distinct component materials. These were silty sand layers interbedded within a matrix of soft normally consolidated clays (CL). The sand layers are generally very thin being on the order of only a few inches thick. These sand layers are dirty and contain on the order of 30 percent non-plastic fines. Unit 2 generally lies between elevation 320 and 305 in the switchyard area and is a little thicker beneath the main dam where it extends to approximately elevation 295 ft. In the switchyard area, Unit 3 is subdivided into three subunits: Units 3a, 3b, and 3c. Unit 3a contains dense sands and gravels which have a relatively high penetration resistance as compared to the sands layers in Unit 2. These sands typically contain less than 15 percent fines and are relatively clean compared to those in Unit 2. Unit 3a lies generally lies between elevation 305 and 295 in the switchyard area. Unit 3b, located between elevation 295 and 285, is typified by clays (CL) of low penetration resistance. Unit 3c lies between elevation 285 and bedrock and based on limited amounts of data appears to have characteristics similar to Unit 3a. In the main embankment area, Unit 3 was considered to consist of only one subunit which was very much like Unit 3a described for the switchyard area.

199. Analytical studies showed that the sandy materials in foundation Unit 2 had a high potential for liquefaction and should be the main focus of this investigation. Thus, in the stratigraphy evaluation it was essential not only to identify and classify these materials but to map the lateral extent of individual sand layers. Unfortunately, it was not possible to map the extent

of individual sand layers from one CPT or SPT sounding to another. However, an excavation in a downstream exposure of Unit 2 revealed that some of these layers were undulating and continuous over fairly long distances in the direction of river flow. This result was carried into the liquefaction studies where the sandy materials in Unit 2 were conservatively treated as continuous.

Dynamic Response Analysis

200. Dynamic response analyses of two representative cross sections was conducted to evaluate the overall performance of two representative cross sections due to the design earthquake. One cross section was located in the switchyard area near the switchyard and the second was representative of the main embankment area. Two dimensional static and dynamic finite element analyses were performed using the programs FEADAM84 and FLUSH to evaluate the performance and response of the representative switchyard section. This cross section was deemed critical due to the low SPT blowcounts that were measured in Unit 2 in the area of the switchyard. The cross section was excited with the motions of the design accelerogram specified in the seismological studies. Earthquake induced shear stresses were determined from the dynamic finite element analysis for later use in the liquefaction analysis. The results of the dynamic analysis predicted that cyclic strain levels on the order of about 1 percent are expected to occur in Unit 2 due to the design earthquake. This zone of large strains extends completely across the section from the downstream free field to the upstream free field. A comparison of the dam's pre (0.75 sec) and effective earthquake fundamental periods (1.75 sec) with the input accelerogram shows that the dam will pass through resonance if subjected to these motions. The lengthening of the period indicates that significant strain softening of materials will occur in the foundation units.

201. A series of one-dimensional dynamic response calculations using SHAKE were performed to approximate the dynamic response of the representative section of the main embankment. Dynamic shears stresses from this series of analysis were used later to evaluate the liquefaction potential of the foundation soils beneath the main embankment. Similar to the switchyard section, these analyses indicated that large strains could be expected to develop in Unit 2 as a result of the design event.

202. In the analyses, the minimum elevation of the dam crest is elevation 388 ft. A pool elevation of 360 ft was selected for the analytical studies. This leaves approximately 28 ft of freeboard since the crest of the dam is at elevation 388.

Evaluation of the Liquefaction Potential of the Foundation Alluvium

203. The cyclic strengths were determined from Seed's empirically developed approach which uses blowcounts from the SPT. However, due to the complex nature of the thin interbedded sand lenses in Unit 2 - good estimates of the "true" SPT blowcounts were unattainable in the switchyard area where the critical section was located. Hence, a technique developed by Olsen (1984) was used to estimate from CPT test data the SPT blowcounts that would have been obtained in thick strata of the same soil. The CPT offered the advantage of high resolution and excellent sensitivity in detecting the thin sand lenses and in estimating their "true" penetration resistance for the evaluation of cyclic strength.

204. For both the main embankment and switchyard section, the liquefaction potential was evaluated by computation of safety factors against liquefaction in which the cyclic strengths are compared with the dynamic shear stresses.

Switchyard section

205. Safety factors against liquefaction for the switchyard section are shown in Figures 78. This figure shows that liquefaction is predicted in the foundation sands of Unit 2 over three general areas: the upstream free field, beneath the switchyard berm, and the downstream free field. Liquefaction is not expected to occur in Unit 2 just under the sloping section of the embankment and in the area near the centerline of the dam. Liquefaction is predicted in the sands of Unit 3 in the upstream and downstream free field areas. Liquefaction is not expected to occur in Unit 3 sands beneath the entire embankment from the upstream toe to the downstream toe.

Main embankment section

206. The results of the liquefaction analysis of the main embankment area are represented in Figure 79. These analyses show that liquefaction was predicted in the upstream and downstream free field and in the area immediate

to the centerline of Unit 2. Liquefaction was not predicted in Unit 2 in areas directly beneath the slopes of the embankment. Liquefaction was also expected in the upper reaches of Unit 3 of the free field between elevations 285 and 295. Liquefaction was not expected in all other areas of Unit 3.

Post-earthquake Strengths

207. Post-earthquake strengths were determined for both the switchyard and main embankment sections. The strengths were determined based on the results of the liquefaction analysis and the CPT predicted and SPT blowcounts. Seed's empirical technique which uses SPT blowcounts was used to estimate undrained residual strengths in areas where liquefaction of the sands was predicted. Earthquake induced excess pore pressures and clay strengths were determined for areas where liquefaction is not expected. The post-earthquake strengths recommended for use in the stability analysis for the switchyard section are presented in Figure 82. The recommended post-earthquake strengths for the main embankment section are presented in Figure 83.

Post-Earthquake Stability Analysis

208. 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. The stability analyses was based on procedures suggested by Seed (1987).

209. During the course of this study many assumptions were made before arriving at the final results of the stability analysis. These assumptions had analytical implications and involved uncertainties in the interpretation of the site conditions. Generally, the assumptions were chosen to be conservative so as to error on the side of the safety of the embankment. In this study, the most conservative assumptions were:

a. the treatment of sand layers in Unit 2 as continuous. It was not possible to conclusively prove from data collected at the site the continuous or discontinuous nature of any individual sand layer in this analysis.

b. the treatment of the sand layers of Unit 2 as flat lying in the stability analysis. The downstream exposure revealed that these layers undulated.

_c_. the treatment of liquefied zones in Unit 2 as being composed entirely of sand. The CPT indicated that this Unit 2 was comprised of about 80 percent clay which in the liquefaction analysis was shown to be nonliquefiable.

210. 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 ft can be expected on the slopes of the main portion of the dam. In the switchyard area deformations of about 1 to 2 ft can be expected, but loss of the reservoir will not occur. These estimated deformations are relatively small in light of the fact that 28 ft of freeboard are expected to be available during the earthquake.

REFERENCES

Bieganousky, Wayne A., and Marcuson, William F. III. 1975. "Liquefaction Potential of Dams and Foundations - Report 1: Laboratory Standard Penetration Tests on Ried Bedford Model and Ottawa Sands," WES Report S-76-2, Oct., 1976.

Bluhm, P. F., Wahl, R. E., and Olsen, R. S. 1991. "Seismic Stability Evaluation of Alben Barkley Dam and Lake Project: Stability Evaluation of Geotechnical Structures,", US Army Engineer Waterways Experiment Station, Technical Report GL-86-, Volume 5, Vicksburg, MS. (IN PUBLICATION)

Duncan, J. M., Seed, R. B., Wong, W. S., and Ozawa, Y. 1984. "FEADAM84: A Computer Program for Finite Element Analysis of Dams,: Research Report No. SU/GT/84-03. Stanford University.

Ellis, W. and Hartman, V. B. 1967. "Dynamic Soil Strength and Slope Stability," Journal of The Soil Mechanics and Foundations Division, ASCE, Vol 93, No. SM4, July, 1967.

Krinitizsky, E. L. and Marcuson, W. F., III. 1983. "Principles For Selecting Earthquake Motions In Engineering Design," Bulletin, Association of Engineering Geologists, Vol VXX, No. 3, pp 253-265.

Krinitzsky, E. L. 1986. "Seismic Stability Evaluation of Alben Barkley Dam and Lake Project: Geological and Seismological Evaluation," US Army Engineer Waterways Experiment Station, Technical Report GL-86-7, Volume 2, Vicksburg, MS.

Ledbetter, R. H., and Finn, W. D., Personal communication, February 22, 1991.
Olsen, Richard S. 1984 July. "Liquefaction Analysis Using the Cone Penetrometer Test (CPT)," Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, California.

Olsen, Richard S. 1984 July. "Liquefaction Analysis Using the Cone Penetrometer Test (CPT)," Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, California.

Olsen, Richard S. and Farr, John V. 1986. "Site Characterization Using the Cone Penetrometer Test," Proceedings of INSITU 86, ASCE Specialty Conference on the Use of In-situ testing in Geotechnical Engineering, Blacksburg, VA.

Olsen, R. S., Bluhm, P. F., Hynes, M. E., and Yule, D. E. 1991. "Seismic Stability Evaluation of Alben Barkley Dam and Lake Project: Field and Laboratory Investigations," US Army Engineer Waterways Experiment Station, Technical Report GL-86-, Volume 3, Vicksburg, MS. (IN PUBLICATION)

Robertson, P. K., Campanella, R. G., and Wightman, A. 1983. "SPT-CPT Correlations," Journal of Geotechnical Engineering, ASCE, Vol 109, No. GT11, November 1982, pp 1449-1459.

Rollins, Kyle M. 1987. "The Influence of Buildings on Potential Liquefaction Damage," Dissertation for Doctor of Philosophy, University of California, Berkeley.

Schmertmann, John H. 1979a. "Energy Dynamics of SPT," Journal of Geotechnical Engineering, ASCE, Vol 105, No. GT8, Aug. 1979.

Schmertmann, John H. 1979b. "Statics of the SPT," Journal of Geotechnical Engineering, ASCE, Vol 105, No. GT5, May 1979, pp 655-670.

Schnabel, Per B., Lysmer, John, and Seed, H. Bolton. 1972. "SHAKE a Computer Program for Earthquake Response Analysis of Horizontally Layered Sites," Report No. EERC 72-12, Earthquake Engineering Research Center, University of California at Berkeley, December 1972.

Seed, H. Bolton 1983 May. "Earthquake-Resistant Design of Earth Dams," Proceedings of the Seismic Design of Embankments and Caverns, ASCE, Philadelphia, Pennsylvania, pp 41-64.

Seed, H. B., Idriss, I. M., and Arango, I. 1983. "Evaluation of Liquefaction Potential Using Field Performance Data," Journal of Geotechnical Engineering, ASCE, Vol 109, No. GT3, March 1983, pp 458-482.

Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, Riley M. 1984. "The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Report No. UCB/EERC-84/15, University of California at Berkeley, Earthquake Engineering Research Center, October 1984.

Seed, H. B., Wong, R. T., Idriss, I. M., and Tokimatsu, K. 1984. "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils." Report No. EERC 84-14. Earthquake Engineering Research Center, University of California, Berkeley, CA.

Seed, H. B. 1986. "Design Problems in Soil Liquefaction," Report No. UCB/EERC-86/02, University of California at Berkeley, Earthquake Engineering Research Center, February 1986.

Street, R., and Nuttli, O. W. 1984 (Apr). "The Central Mississippi River Valley Earthquakes of 1811-12," Paper Presented at the New Madrid Seismic Zone Symposium, Cape Girardeau, MO.

Szerdy, F. 1986. "Flow Slide Failures Associated with Low Level Vibrations," Dissertation for Doctor of Philosophy in Civil Engineering, University of California, Berkeley.

Thiers, C. and Seed, H. B. 1968. "Cyclic Stress-Strain Characteristics of Clay," Journal of Soil Mechanics and Foundations Division, ASCE, Vol 94, No. SM2, Mar. 1968.

Tokimatsu, K. and Yoshimi, Y. (1983). "Empirical Correlation of Soil Liquefaction based on SPT N-value and Fines Content," Soil and Foundations, Vol 23, No. 4, pp 56-74.

US Army Engineer District, Nashville, TN. 1960. "Design Memorandum No. 3C - Soil Explorations and Right Bank Earth Structures", Barkley Dam Project, KY.

Wahl, R. E., Olsen, R. S., Bluhm, P. F., Yule, D. E., and Hynes, M. E., 1991. "Seismic Stability Evaluation of Alben Barkley Dam and Lake Project: Liquefaction Susceptibility Evaluation and Post-earthquake Strength Determination," US Army Engineer Waterways Experiment Station, Technical Report GL-86-, Volume 4, Vicksburg, MS. (IN PUBLICATION)

Wang, W. 1979. "Some Findings In Soil Liquefaction," Water Conservancy and Hydroelectric Power Scientific Research Institute, Beiging, China, Aug, 1979.

Table 1

Design Strength Test Data

Material	Saturated Unit Weight pcf	Moist Unit Weight pcf	Drain Shear Strength (S) from DCD Tests		Consolidated-Undrained Shear Strength (R) from TCU Tests		Unconsolidated-Undrained Shear Strength (Q) from TUU Tests	
			tan ϕ	c (tsf)	tan ϕ	c (tsf)	tan ϕ	c (tsf)
Impervious fill	128	126	0.500	0	0.300	0.40	0.065	0.50
Random fill	128	126	0.250	0	0.150	0.20	0.033	0.25
Zone A - 0-19'	128	126	0.413	0	0.285	1.00	0.221	0.70
Zone B - 19-53'	128	126	0.602	0	0.380	0.38	0.276	0.44
Zone C - 53-90'	128	120	0.710	0.15	0.700	0.10	--	--

TCU - Triaxial Consolidated Undrained.
DCD - Direct Shear.
TUU - Triaxial Unconsolidated Undrained.
Table from DM 3C - Soil Explorations and Right Bank Earth Structures Zones A, B, and C are from test results from holes BDH-9 and 11.

Table 2

Summary of Geophysical Tests Performed at Barkley Dam

Study Area	Location	Date	Test Type	Wave Type	Line ID	Line Length ft	Remarks
			Surface Techniques				
Location 1	STA 64+006	Dec 1977	Surface Refraction	P-Wave	RS-1	625	Downstream, parallel to toe
				P-Wave	RS-2	625	Downstream, perpendicular to toe
				P-Wave	RS-3	625	Downstream, parallel to toe
				S-Wave	RS-3	625	Downstream, parallel to toe
				P-Wave	RS-4	165	Along dam crest
				S-Wave	RS-4	165	Along dam crest
			Surface vibratory	Rayleigh wave	RY-1	22	Downstream toe, near crosshole set

Study Area	Date	Test Type	Borings	Station L	Offset B	TOH Elevation	Depth ft	Remarks
				Subsurface Techniques				
Location 1	Dec 77	Crosshole (3-hole set)	BEQ 1U	64+20	2+31	349.6	127.2	P-wave and S-wave at downstream toe
			BEQ 2U	64+00	2+51	350.2	121.9	
			BEQ 6	64+20	2+51	350.3	132.2	
		Downhole	BEQ 2U	64+00	2+51	350.2	121.9	S-wave successful P-wave unsuccessful
Location 2	Apr 84	Crosshole (2-hole set)	WES 1-1	36+00	0+38	387.1	127.3	P-wave and S-wave on downstream slope
			WES 1-2	36+11	0+39	387.6	126.6	
Location 3	Apr 84	Crosshole (2-hole set)	WES 2-1	34+45	4+95	341.3	88.3	P-wave and S-wave downstream of switchyard near tailrace slope
			WES 2-2	34+45	4+85	341.3	88.0	
Location 4	May 85	Downhole	CPT 12	38+70	2+07	365.7	86.6	S-wave successful P-wave unsuccessful
Location 5	May 85	Downhole	CPT 26	34+56	4+98	341.5	67.3	S-wave successful P-wave unsuccessful

Table 3

Locations of Undisturbed Borings

Boring No.	Date Drilled	Location L (ft)	Location B (ft)	Elevation TOH (ft)	Depth Soil (to rock) (ft)	No. of Samples
BEQ-1U	3 Nov 1977	64+20	2+31	349.6	114.2(127.2)	39 Soil 2 rock
BEQ-2U	Nov 1977	64+00	2+51	350.2	114.2(121.9)	37 Soil 1 rock
DS-1	23 May 1979	63+80	2+31	349.2	114.3(122.0)	39 Soil 1 rock
DS-2	2 Jun 1979	63+60	2+31	350.5	118.3(124.4)	39 Soil 1 rock
DS-3	9 Jun 1979	34+30	4+81	340.3	86.3 (94.1)	16 Spt 16 soil
BEQ-3U	6 Dec 1984	37+00	0+44	385.2	47.6	5 Soil
BEQ-4U	31 Oct 1984	37+20	1+50	366.2	79.3	19 Soil
BEQ-5U	8 Nov 1984	34+61	4+86	341.7	54.7	15 Soil
BEQ-6U	14 Nov 1984	34+74	4+96	341.9	41.1	13 Soil
BEQ-7U	29 Nov 1984	37+00	5+20	347.7	78.7	21 Soil
BEQ-8U	16 Nov 1984	34+43	4+70	341.6	38.1	6 Soil

Table 4

Shear Strength Results

Material	Natural Water Content percent	Dry Unit Weight pcf	Moist Unit Weight pcf	Atterberg Limits percent		Consolidated-Undrained Shear Strength (R)		Drained Shear Strength (S)		Effective Friction Angle
				LL	PL	c (tsf)	tan ϕ	c (tsf)	tan ϕ	ϕ'
Embankment, Switchyard	16.2	114.2	132.7	36	19	0.45	0.503	0	0.617	32°
CL (Unit 1)	22.4	103.1	126.0	38	20	0.57	0.279	0	0.653	33°
CL (Unit 2)	24.0	99.8	123.5	33	19	0.54	0.233	0	0.640	33°

Table 5

Measured and Estimated Insitu Steady-State Shear Strengths and Void Ratios of Foundation Sand

				Insitu Values Estimated from Laboratory Values Corrected for Sample Volume Change			
		Values Measured In Laboratory		Assumption: Uniform Volume Change In Tube Sample		Assumption: Sand Undergoes All Measured Compression But No Expansion	
Test No.	Material Group (% Fines)	Void Ratio e	Steady-State Shear Strength S_{us} psi	Void Ratio e	Steady-State Shear Strength S_{us} psi	Void Ratio e	Steady-State Shear Strength S_{us} psi
R-4	12-16	0.749	66	0.776	46	0.805	28
R-6	12-16	0.684	99	0.742	45	0.741	43
R-8	12-16	0.733	133	0.752	105	0.761	95
R-9	12-16	0.680	129	0.726	77	0.737	66
R-13	12-16	0.618	40	0.667	17	0.680	13
R-1	18-44	0.721	26	0.746	20	0.755	14
R-3	18-44	0.617	35	0.677	9	0.692	6
R-5	18-44	0.630	71	0.703	20	0.703	20
R-7	18-44	0.692	64	0.780	15	0.790	11
R-10	18-44	0.548	15	0.579	9	0.598	6
R-11	18-44	0.730	62	0.784	29	0.794	22
R-12	18-44	0.759	55	0.875	7	0.894	5

Table 6

SPT Borings

SPT No.	Date Drilled	Location L (ft)	Location B (ft)	El. TOH* (ft)	Depth (ft)	No. Samp	Method of Drilling	Drilling Agency
BEQ-1	5 Oct 1977	44+50	2+10	345.6	124.0	40	Standard	WES
BEQ-2	10 Oct 1977	54+00	2+10	347.2	119.0	40	Standard	WES
BEQ-3	12 Oct 1977	64+00	2+00	349.6	120.0	39	Standard	WES
BEQ-4	18 Oct 1977	74+00	2+60	351.7	115.7	38	Standard	WES
BEQ-5	20 Oct 1977	84+00	4+80	343.6	61.5	12	Standard	WES
BEQ-6	18 Nov 1977	64+20	2+51	350.3	132.2	36	Standard	WES
DS-3	9 Jun 1979	34+28	4+81	340.3	94.1		**	WES
BEQ-7	7 Nov 1982	34+33	4+86	341.5	60.0	53	Continuous	Nashville
BEQ-8	10 Nov 1982	39+40	4+81	349.8	66.5	33	Standard	Nashville
BEQ-9	17 Nov 1982	49+25	2+15	350.5	74.0	42	Standard	Nashville
BEQ-10	23 Nov 1982	54+00	2+10	347.2	60.0	79	Continuous	Nashville
BEQ-11	1 Dec 1982	59+00	2+30	347.0	61.5	62	Standard	Nashville
BEQ-12	9 Dec 1982	69+00	2+40	348.5	61.5	61	Standard	Nashville
BEQ-13	20 Jan 1983	74+06	2+60	344.0	60.0	90	Continuous	Nashville
BEQ-14	13 Jan 1983	79+05	3+10	345.0	64.0	57	Standard	Nashville
BEQ-15	9 May 1984	35+60	1+52	365.7	86.5	83	Standard	Nashville
BEQ-16	22 May 1984	35+60	1+48	365.7	84.0	126	Continuous	Nashville
BEQ-17	17 Apr 1984	36+95	1+52	366.1	86.5	84	Standard	Nashville
BEQ-18	26 Apr 1984	36+95	1+47	366.1	84.5	89	Standard	Nashville
BEQ-19	31 May 1984	39+85	1+75	364.4	81.5	77	Standard	Nashville
BEQ-20	13 Jun 1984	39+85	1+70	364.7	81.0	131	Continuous	Nashville
BEQ-21	12 Jul 1984	34+35	4+96	341.5	61.5	55	Standard	Nashville
BEQ-22	22 Jul 1984	34+35	4+91	341.5	58.5	103	Continuous	Nashville
BEQ-23	12 Mar 1984	36+95	5+00	347.3	66.5	59	Standard	Nashville
BEQ-24	9 Apr 1984	37+00	5+00	347.3	67.0	23	Standard	Nashville
BEQ-25	20 Jun 1984	39+50	4+86	349.8	69.0	62	Standard	Nashville
BEQ-26	2 Jul 1984	39+50	4+75	349.8	67.5	110	Continuous	Nashville
BEQ-27	30 Jul 1984	34+35	7+00	342.7	59.0	56	Standard	Nashville

(Continued)

* Top of hole.

** Boring was alternating SPT-Undisturbed.

Table 6 (Concluded)

SPT No.	Date Drilled	Location L (ft)	Location B (ft)	El. TOH* (ft)	Depth (ft)	No. Samp	Method of Drilling	Drilling Agency
BEQ-28	7 Aug 1984	34+35	7+05	342.7	61.5	101	Continuous	Nashville
BEQ-29	14 Aug 1984	36+95	7+00	347.2	64.0	65	Standard	Nashville
BEQ-30	22 Aug 1984	36+94	7+05	347.7	67.5	103	Continuous	Nashville
BEQ-31	28 Aug 1984	39+80	6+90	350.0	69.0	67	Standard	Nashville
BEQ-32	5 Sep 1984	36+85	6+85	350.0	65.0	106	Continuous	Nashville
BEQ-33	20 Sep 1984	39+84	2+76	362.9	78.0	123	Continuous	Nashville
BEQ-34	24 Sep 1984	39+84	2+64	362.7	79.0	76	Standard	Nashville
BD-1	2 Sep 1981	65+10	2+20	348.5	39.0	25	Continuous	Nashville
BD-2	8 Sep 1981	65+10	2+00	349.7	40.0	23	Continuous	Nashville
BD-3	15 Sep 1981	65+60	2+35	349.8	39.0	26	Continuous	Nashville
BD-4	17 Sep 1981	66+00	2+27	348.5	39.0	26	Continuous	Nashville
BD-5	9 Sep 1981	[illegible]	2+71	349.4	39.0	26	Continuous	Nashville
BD-6	14 Sep 1981	63+00	2+30	348.6	39.0	25	Continuous	Nashville
BD-7	23 Sep 1981	66+00	2+35	348.0	39.0	26	Continuous	Nashville
BD-8	28 Sep 1981	65+50	2+30	348.6	31.6	21	Continuous	Nashville
BD-9	5 Oct 1981	64+50	0+60	374.0	64.5	42	Continuous	Nashville

Table 7

Example of Information Contained in SPT Data Base from Boring BEQ-30

BEQ-30

Barkley dam

Depth	Elev	CUS	Soil		(--Z---			)	(------D50	------	)	(--P200	--	)	(-----P005	----	)	(SPT)		(-N1c--)
feet	feet	tsf	#	ID	PL	Wn	LL	Wn/LL	L	Aver	H	L	Aver	H	L	Aver	H	N	N1	P200
1.0	346.7	2.8	0	_														0	0	
25.0	322.7	1.5	3	C	14	21	27	0.79		0.04		53	60	72		24.8		7	6	
26.5	321.2	1.6	3	CS	14	21	25	0.85	0.06	0.08	0.11	44	51	56	22.3	24.0	24.8	6	5	
28.0	319.7	1.7	3	C		23						64	67	72				5	4	10.0
29.5	318.2	1.7	2		15	23	26	0.87	0.03	0.12	0.20	15	38	57		23.0		6	5	
31.0	316.7	1.8	3	CS		23			0.05	0.12	0.20	15	43	63		19.5		5	4	10.0
32.5	315.2	1.8	2	C		26				0.20		15	53	72				4	3	9.3
34.0	313.7	1.9	2			26				0.20		15	48	83				3	2	8.6
35.5	312.2	1.9	3	CS		24			0.11	0.14	0.20	15	56	77		17.5		6	4	10.5
37.0	310.7	2.0	3	C	15	24	24	1.01	0.02	0.07	0.20	15	49	63	21.0	22.6	24.5	3	2	
38.5	309.2	2.0	2	C	14	24	22	1.11	0.14	0.17	0.20	15	36	53		17.0		5	4	9.8
40.0	307.7	2.0	1	S		24			0.18	0.19	0.20	15	20	39		12.1		9	6	10.8
41.5	306.2	2.1	2	C	17	25	28	0.90	0.02	0.13	0.20	15	45	72		23.0		6	4	
44.5	303.2	2.2	3	C		25			0.01	0.11	0.25	5	51	83	30.0	33.9	37.0	13	9	
46.0	301.7	2.2	1	S		24			0.25	0.25	0.25	5	6	7				31	21	21.4
47.5	300.2	2.3	3	SC	32	41	58	0.71	0.24	0.25	0.25	5	30	90				10	7	
49.0	298.7	2.4	3	CS		25			0.32	5.41	10.58	3	26	56				49	32	32.5
50.5	297.2	2.4	1			21				6.90			5					37	24	24.2
52.0	295.7	2.4	2	S		17				0.27		12	14	16				48	31	32.5
53.5	294.2	2.5	2	S		25				0.19		14	15	15				21	13	17.7
55.0	292.7	2.5	3	SC		29						31	62	87				12	8	13.9
56.5	291.2	2.6	3	CS	17	30	34	0.88		0.02		69	72	77		23.0		6	4	
58.0	289.7	2.6	1	S		24				0.14		27	27	27				17	11	16.3
59.5	288.2	2.7	2	S		21			0.23	0.24	0.24	7	8	8				49	30	31.5
61.0	286.7	2.7	2	_		15						19	15	20				59	36	32.5
62.5	285.2	2.8	2	S		20				0.60		10	10	10				23	14	16.2

Table 8

CPT Locations and Testing Program

CPT No.	Location L ft	Location B ft	Top of Ground Elevation ft	Instrument No.*	Data Measurements (ft) Depth of q_c and f_s ft	Pore Pressure ft	Conductivity	Dielectric
1	38+71	0+65	378.5	080	97.1	--	--	--
2	38+71	1+10	365.7	080	81.9	--	--	--
3	35+54	1+52	365.7	070,076	81.7	50.2-81.7	81.7	--
4	36+05	1+50	365.7	070	83.2	--	83.2	--
5	36+45	1+50	365.8	076	79.7	--	79.7	--
6	37+05	1+50	365.8	070,076	86.6	34.0-86.6	--	--
7	37+61	1+50	365.8	076	83.1	--	83.1	--
8	38+17	1+50	365.7	229,070	61.0	--	62.4	--
9	38+71	1+50	365.8	070,076	79.9	30.0-79.9	79.9	--
10	39+27	1+50	365.8	076	83.1	--	83.1	--
11	39+85	1+60	365.2	076	83.0	--	83.0	--
12	38+70	2+07	365.7	080	86.6	--	--	--
13	39+85	2+15	363.6	080	79.9	--	--	--
14	38+55	2+68	365.5	070,076	82.3	35.1-82.3	82.3	--
15	34+94	2+81	365.5	076	83.1	--	83.1	--
16	38+41	3+13	364.7	080	79.2	--	--	--
17	39+85	3+20	361.7	080	78.9	--	--	--
18	34+94	3+41	365.6	076	80.1	--	80.1	--
19	38+34	3+57	364.7	076,076	78.9	44.8-78.9	78.9	--
20	39+35	3+70	361.4	080	79.0	--	--	--
21	34+52	4+51	340.7	080	58.6	--	--	--
22	38+07	4+22	348.4	080	65.1	--	--	--
23	39+85	4+30	350.5	080	68.8	--	--	--

(Continued)

* Instruments:
080 subtracting design probe.
229 tension design probe.
070 subtracting probe with conductivity unit.
076 subtracting probe with conductivity unit and piezo element.

Table 8 (Continued)

CPT No.	Location L ft	Location B ft	Top of Ground Elevation ft	Instrument No.*	Data Measurements (ft) Depth of q_c and f_s ft	Pore Pressure ft	Conductivity	Dielectric
24	34+46	4+65	341.1	076	60.0	--	60.0	--
25	34+46	4+65	341.5	229	59.7	--	--	--
26	34+56	4+98	341.5	080	67.1	--	--	--
27	34+97	4+91	342.2	080	43.4	--	--	--
28	35+29	4+91	343.4	070,080	59.8	30.0-59.8	59.8	--
29	35+61	4+92	344.8	070	59.9	--	--	--
30	35+93	4+93	345.9	229	46.5	--	--	--
31	36+25	4+96	346.6	076-080	49.6	35.1-49.7	--	--
32	36+55	4+98	346.9	229	50.1	--	--	--
33	36+83	5+00	347.2	076	66.8	--	--	--
34	37+85	5+00	348.0	070,076, 080	65.8	34.9-65.8	65.9	--
35	38+70	5+00	349.9	076	66.3	--	66.3	--
36	39+50	4+90	349.8	070,076	67.7	30.5-67.7	67.7	--
37	40+00	4+96	350.4	076	73.1	--	73.1	--
38	40+50	5+00	350.9	080	69.1	--	--	--
39	41+50	5+00	351.7	076	76.5	--	76.5	--
40	42+50	5+00	352.8	080	85.0	--	--	--
41	34+38	5+03	341.7	070,076	56.9	29.8-56.9	56.9	--
42	34+35	5+37	341.8	076	63.1	--	63.1	--
43	34+35	5+68	342.0	229,076	44.2	--	59.9	--
44	37+61	5+53	348.0	076	69.6	--	69.6	--
45	40+00	5+48	350.0	080	69.8	--	--	--
46	34+35	6+00	342.2	070	59.7	--	59.7	--
47	34+35	6+30	342.7	080	62.3	--	--	--
48	37+32	6+35	347.5	076,080	69.1	34.9-69.1	69.1	--
49	40+00	6+22	349.8	080	64.1	--	--	--
50	34+35	6+60	342.8	070	46.9	--	--	--
51	34+35	6+90	342.7	070,076	47.2	--	46.9	--

(Continued)

(Sheet 2 of 3)

Table 8 (Concluded)

CPT No.	Location L ft	Location B ft	Top of Ground Elevation ft	Instrument No.*	Data Measurements (ft) Depth of q_c and f_s ft	Pore Pressure ft	Conductivity	Dielectric
52	37+05	7+00	347.1	076	60.0	--	60.0	--
53	40+00	6+90	347.8	070,076, 080	69.4	25.0-69.4	69.9	
54	34+71	4+92	341.5	070,076	58.3	30.1-58.3	58.3	3.1-42.7
55	34+68	4+85	341.5	229	44.1	--	--	--
56	34+58	4+76	341.1	076	66.7	--	66.7	0.5-41.0
57	34+51	4+71	341.1	070	59.0	--	59.0	--
58	33+62	5+04	341.9	229	43.4	--	--	--
65	39+92	2+69	362.5	080	76.3	--	--	--

(Sheet 3 of 3)

Table 9

Properties for Static Analysis Using FEADAM

		Young's Modulus					Bulk Modulus					
MAT	Description	Unit Wt	K	KUR	N	RF	KB	M	C	PHI	DPHI	KO
1	Random fill	.0660	90.0	90.0	.45	.70	80.00	.20	.20	30.00	.00	.50
2	Compacted embankment fill	.1260	120.0	120.0	.45	.70	110.00	.20	.30	30.00	.00	.50
3	Unit 1 - Clay	.0660	120.0	120.0	.45	.70	110.00	.20	.30	30.00	.00	.50
4	Unit 2 + 3a - Sandy material	.0640	300.0	300.0	.25	.70	250.00	.00	.00	32.00	4.00	.50
5	Unit 3b - Clayey material	.0640	300.0	300.0	.25	.70	250.00	.00	.00	32.00	4.00	.50
6	Unit 3c - Sands + gravels	.0660	300.0	300.0	.40	.70	75.00	.20	.00	36.00	5.00	.40
7	Submerged compacted embankment fill (same as 2)	.0660	120.0	120.0	.45	.70	110.00	.20	.30	30.00	.00	.50

Notes:

(a) Unit weight is in kcf.
(b) C is in ksf.
(c) PHI and DPHI are in degrees.

Table 10

Material Properties for Dynamic Analysis

Material No.	Description	Total Density pcf	Shear Wave Velocity fps	Shear Modulus ksf
1	Random fill	129	500	1,001
2	Compacted embankment fill	126	600	1,408
3	Unit 1 Lean clay	129	775	2,406
4	Units 2a and 3a Sandy material	127	700	1,932
5	Unit 3b Clayey material	127	800	2,525
6	Unit 3c Sands and gravel	129	900	3,245
7	Compacted	129	775	2,406

Table 11

Statistical Analysis of $(N_1)_{60}$ and Fines Content of SPT's Performed in Main Embankment Area

Location	Number of Samples	$(N_1)_{60}$ (Blows/ft)		Fine Content percent	
		Mean	STD	Mean	STD
Unit 2	82	15.8	8.3	23.2	16.2
Unit 3	48	23.8	8.9	16.1	10.6

Table 12

Summary of Statistical Analysis of Equivalent Sand Blowcounts $(N_1)_c$, Obtained from Standard Penetration Tests

Area	Unit	Elevation	Mean $(N_1)_c$, Blows/ft	Standard Deviation	Number of Samples
Switchyard	2	305-320	13.6	3.0	22
Main embankment	2	305-320	20.7	6.5	83
Total site (switchyard and main embankment)	3	Below 305	24.3	9.9	258

Table 13

Summary of CPT Predicted Blowcounts and Fines Contents

	CPT Predicted Values				
	$(N_1)_{60}$ Blows/ft		$(N_1)_c$ Blows/ft		
Unit	Mean	STD	Mean	STD	Fines %
2	14.5	8.9	19.0	7.2	15
3	24.3	7.6	25.7	7.6	7

Table 14

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1 at Upstream Free Field of Main Embankment

Layer No.	Depth ft	Elevation ft	Unit No.	σ_v' tsf	α	N_1 Blows/ft	τ_{str}/σ_v' *	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS_L
6	27.5	322.5	2	0.92	0.23	17	0.187	0.89	1.04	1.00	318	384	0.82
7	32.5	317.5	2	1.08	0.22	17	0.187	0.89	0.94	1.00	338	447	0.76
8	37.5	312.5	2	1.25	0.20	17	0.187	0.89	0.92	1.00	382	501	0.76
9	42.5	307.5	2	1.40	0.19	17	0.187	0.89	0.90	1.00	419	553	0.75
10	47.5	302.5	2	1.56	0.17	17	0.187	0.89	0.87	1.00	451	598	0.75
11	55.0	294.5	3	1.80	0.15	22	0.242	0.89	0.84	1.00	651	632	1.03
12	65.0	284.5	3	2.12	0.14	22	0.242	0.89	0.80	1.00	731	641	1.14
13	75.0	274.5	3	2.45	0.12	22	0.242	0.89	0.74	1.00	781	693	1.13
14	87.5	262.0	3	2.85	0.11	22	0.242	0.89	0.72	1.00	883	691	1.28
15	102.5	247.0	3	3.35	0.11	22	0.242	0.89	0.71	1.00	1,020	829	1.24

* Applies to conditions where $\alpha = 0$ and $\sigma_v' = 1$ tsf.

Table 15

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1 at Upstream Toe of Main Embankment

Layer No.	Depth ft	Elevation ft	Unit No.	σ_v' tsf	α	N_1 Blows/ft	τ_{str}/σ_v' *	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS_L
6	27.5	322.5	2	0.92	0.23	17	0.187	0.89	1.04	1.76	560	384	1.95
7	32.5	317.5	2	1.08	0.22	17	0.187	0.89	0.94	1.73	585	447	1.31
8	37.5	312.5	2	1.25	0.20	17	0.187	0.89	0.92	1.69	647	501	1.29
9	42.5	307.5	2	1.40	0.19	17	0.187	0.89	0.90	1.67	700	553	1.26
10	47.5	302.5	2	1.56	0.17	17	0.187	0.89	0.87	1.63	736	598	1.23
11	55.0	294.5	3	1.80	0.15	22	0.242	0.89	0.84	1.54	1,004	632	1.59
12	65.0	284.5	3	2.12	0.14	22	0.242	0.89	0.80	1.49	1,089	641	1.70
13	75.0	274.5	3	2.45	0.12	22	0.242	0.89	0.74	1.44	1,125	693	1.62
14	87.5	262.0	3	2.85	0.11	22	0.242	0.89	0.72	1.42	1,255	691	1.82
15	102.5	247.0	3	3.35	0.11	22	0.242	0.89	0.71	1.42	1,455	829	1.76

* Applies to conditions where $\alpha = 0$ and $\sigma_v' = 1$ tsf.

Table 16

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1 one-third of the Way Upslope of the Main Embankment

Layer No.	Depth ft	Elevation ft	Unit No.	σ_v' tsf	α	N_1 Blows/ft	τ_{str}/σ_v' *	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS_L
9	40.5	322.5	2	1.42	0.22	17	0.187	0.89	1.70	1.70	707	619	1.14
10	45.5	317.5	2	1.58	0.20	17	0.187	0.89	1.68	1.68	768	686	1.12
11	50.5	312.5	2	1.75	0.20	17	0.187	0.89	1.68	1.68	819	737	1.11
12	55.5	307.5	2	1.91	0.20	17	0.187	0.89	1.68	1.68	876	785	1.12
13	60.5	302.5	2	2.07	0.18	17	0.187	0.89	1.65	1.65	910	820	1.11
14	68.0	295.0	3	2.31	0.16	22	0.242	0.89	1.60	1.60	1,209	850	1.42
15	78.0	285.0	3	2.63	0.13	22	0.242	0.89	1.45	1.45	1,215	856	1.42
16	88.0	275.0	3	2.96	0.13	22	0.242	0.89	1.45	1.45	1,331	858	1.55
17	100.5	262.5	3	3.36	0.13	22	0.242	0.89	1.45	1.45	1,469	868	1.69
18	115.5	257.5	3	3.84	0.13	22	0.242	0.89	1.45	1.45	1,833	868	1.89

* Applies to conditions where $\alpha = 0$ and $\sigma_v' = 1$ tsf.

Table 17

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1

Two-thirds of the Way Upslope of the Main Embankment

Layer No.	Depth ft	Elevation ft	Unit No.	σ_v' tsf	α	N_1 Blows/ft	$\frac{\tau_{str}}{\sigma_v'}$*	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS_L
10	53.5	322.5	2	2.35	0.15	17	0.187	0.89	0.78	1.55	947	773	1.23
11	58.5	317.5	2	2.52	0.15	17	0.187	0.89	0.76	1.55	988	820	1.20
12	63.5	312.5	2	2.70	0.14	17	0.187	0.89	0.74	1.50	998	866	1.15
13	68.5	307.5	2	2.80	0.13	17	0.187	0.89	0.72	1.49	1,000	903	1.11
14	73.5	302.5	2	3.00	0.12	17	0.187	0.89	0.71	1.45	1,028	932	1.10
15	81.0	295.0	3	3.24	0.11	22	0.242	0.89	0.68	1.42	1,347	957	1.41
16	91.0	285.0	3	3.60	0.10	22	0.242	0.89	0.66	1.38	1,412	970	1.45
17	101.0	275.0	3	3.90	0.09	22	0.242	0.89	0.66	1.35	1,496	986	1.52
18	133.5	262.5	3	4.30	0.09	22	0.242	0.89	0.64	1.35	1,600	984	1.63
19	128.5	247.5	3	4.80	0.09	22	0.242	0.89	0.63	1.35	1,759	1,045	1.68

* Applies to conditions where $\alpha = 0$ and $\sigma_v' = 1$ tsf.

Table 18

Computation of Safety Factors Against Liquefaction, FS_L, for Profile 1 at the Centerline of the Main Embankment

Layer No.	Depth ft	Elevation ft	Unit No.	σ_v' tsf	α	N_1 Blows/ft	τ_{str}/σ_v' *	K_m	K_σ	K_α	τ_{str} psf	τ_{dyn} psf	FS_L
8	65.5	322.5	2	3.31	0	17	0.187	0.89	0.70	1.0	771	885	0.87
9	0.5	317.5	2	3.47	0	17	0.187	0.89	0.69	1.0	797	883	0.90
10	75.5	312.5	2	3.36	0	17	0.187	0.89	0.68	1.0	806	925	0.87
11	80.5	307.5	2	3.79	0	17	0.187	0.89	0.68	1.0	858	944	0.91
12	85.5	302.5	2	3.96	0	17	0.187	0.89	0.67	1.0	883	972	0.91
13	93.0	295.0	3	4.20	0	22	0.242	0.89	0.65	1.0	1175	1001	1.17
14	103.0	285.0	3	4.52	0	22	0.242	0.89	0.64	1.0	1246	1020	1.22
15	113.0	275.0	3	4.84	0	22	0.242	0.89	0.63	1.0	1313	1035	1.27
16	125.5	262.5	3	5.25	0	22	0.242	0.89	0.60	1.0	1357	1072	1.27
17	140.5	247.5	3	5.73	0	22	0.242	0.89	0.50	1.0	1457	1064	1.37

* Applies to conditions where $\alpha = 0$ and $\sigma_v' = 1$ tsf.

Table 19

Approximate Values of ΔN_1

Fines Content (%)	ΔN_1 Blows/ft
10	1
25	2
50	4
75	5

(Reference: Seed 1986).

Table 20

Residual Undrained Strengths of Liquefied Sands in the Foundation of the Switchyard Area

Foundation Unit	CPT Predicted $(N_1)_{60}$ Blows/ft	CPT Predicted Fines Content percent	ΔN_1 Blows/ft	N_{1eff} Blows/ft	S_{ur} psf
2	14.5	15	1	15.5	450
3a and 3c	24.3	7	0.7	25.0	800

Notes: S_{ur} values for Units 2 and 3 based upon CPT predicted SPT Data on Table 10.

Table 21

Residual Undrained Strengths of Liquefied Sands in the Foundation of the Main Embankment Area

Foundation Unit	$(N_1)_{60}$ Blows/ft	Fines Content percent	ΔN_1 Blows/ft	N_{1eff} Blows/ft	S_{ur} psf
2	15.8	23.1	1.7	17.5	700
3	24.3	7	0.7	25.0	800

Notes: 1. S_{ur} values for Unit 2 based upon SPT Data on Table 8.
2. S_{ur} values for Unit 3 based upon CPT Data on Table 10.

Table 22

Pre-earthquake Material Properties

Soil Type	Unit Weights (pcf) Moist	Sat	R Strengths C (psf)	phi	S Strengths C (psf)	phi (degrees)
Embankment and switchyard	126	128	1,000	22	0	26.5
Random fill*	126	128	400	8.5	0	14
Unit 1 – Clay	115	125	1,200	15	600	22
Unit 2 – Clays	122	126	700	14	0	31
Sands	122	126	—	—	0	31
Unit 3A – Dense sands, gravels*	126	128	200	35	300	35
Unit 3B – Clays	122	126	700	14	0	31
Unit 3C – Dense sands, gravels*	126	128	200	35	300	35

* Indicates estimated values from design Memorandum No. 3C.

Table 23

Parameters Used in Post Earthquake Stability Analysis

Soil Type	Unit Weights (pcf) Moist	Unit Weights (pcf) Sat	Strength C (psf)	Strength phi (degrees)	Excess Pore Pressure Ratio
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	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	128	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%

P is the effective overburden pressure.

Table 24

Summary of Stability Analyses

	Factors of Safety						Residual Strength Required, PSF	
	Pre Earthquake		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	—	—
S'Yard								
El 305	4.4	5.6	1.8	1.6	0.8	0.7	200	200
El 295	4.4	7.2	2.3	2.2	0.5	0.6	200	250
El 288	5.2	5.2	2.3	3.1	0.5	0.6	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.

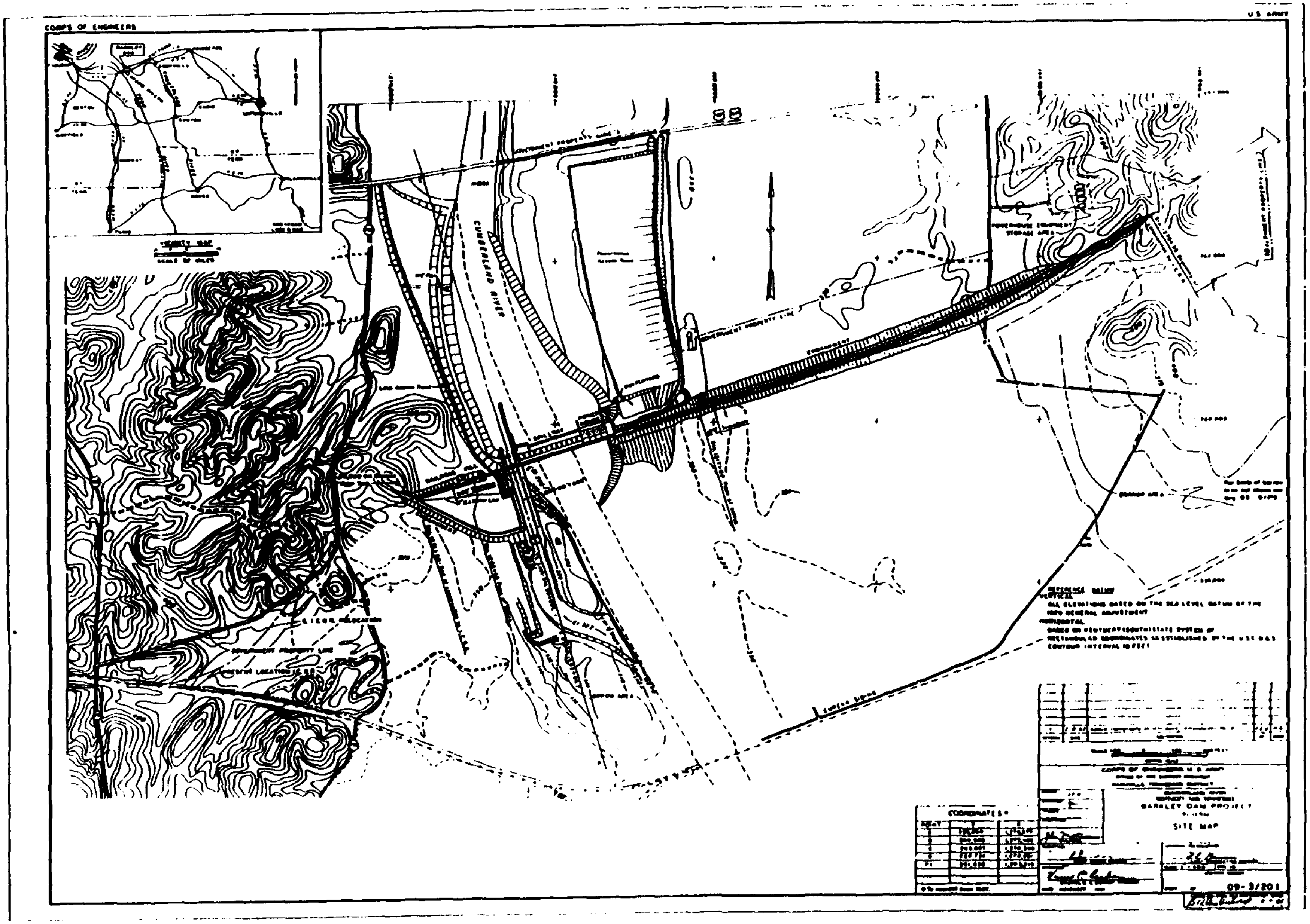


Figure 1. Plan view and locality sketch of Barkley Lock and Dam Project.

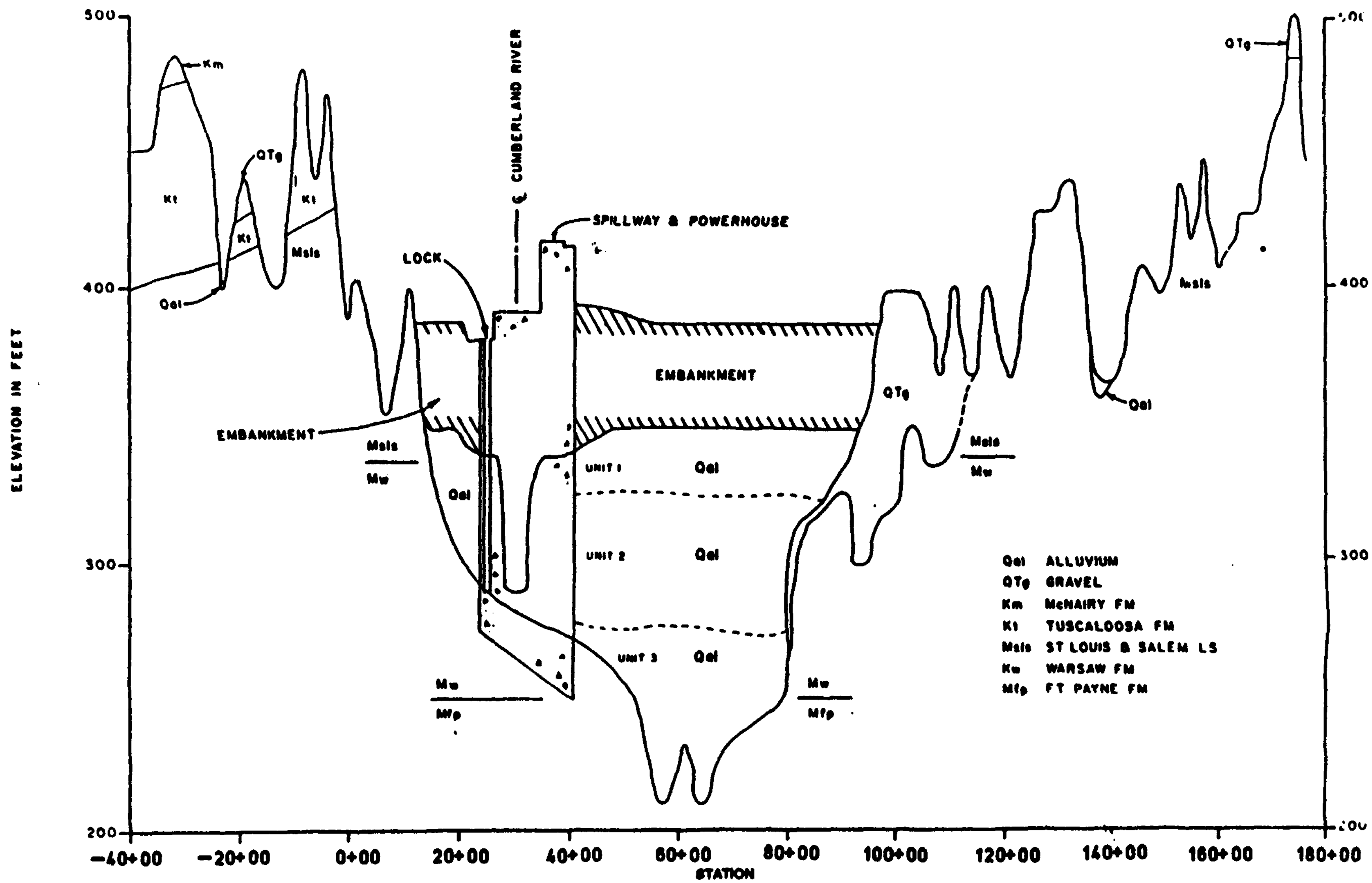


Figure 2. Generalized geologic cross section along the toe of the dam showing the three major foundation units.

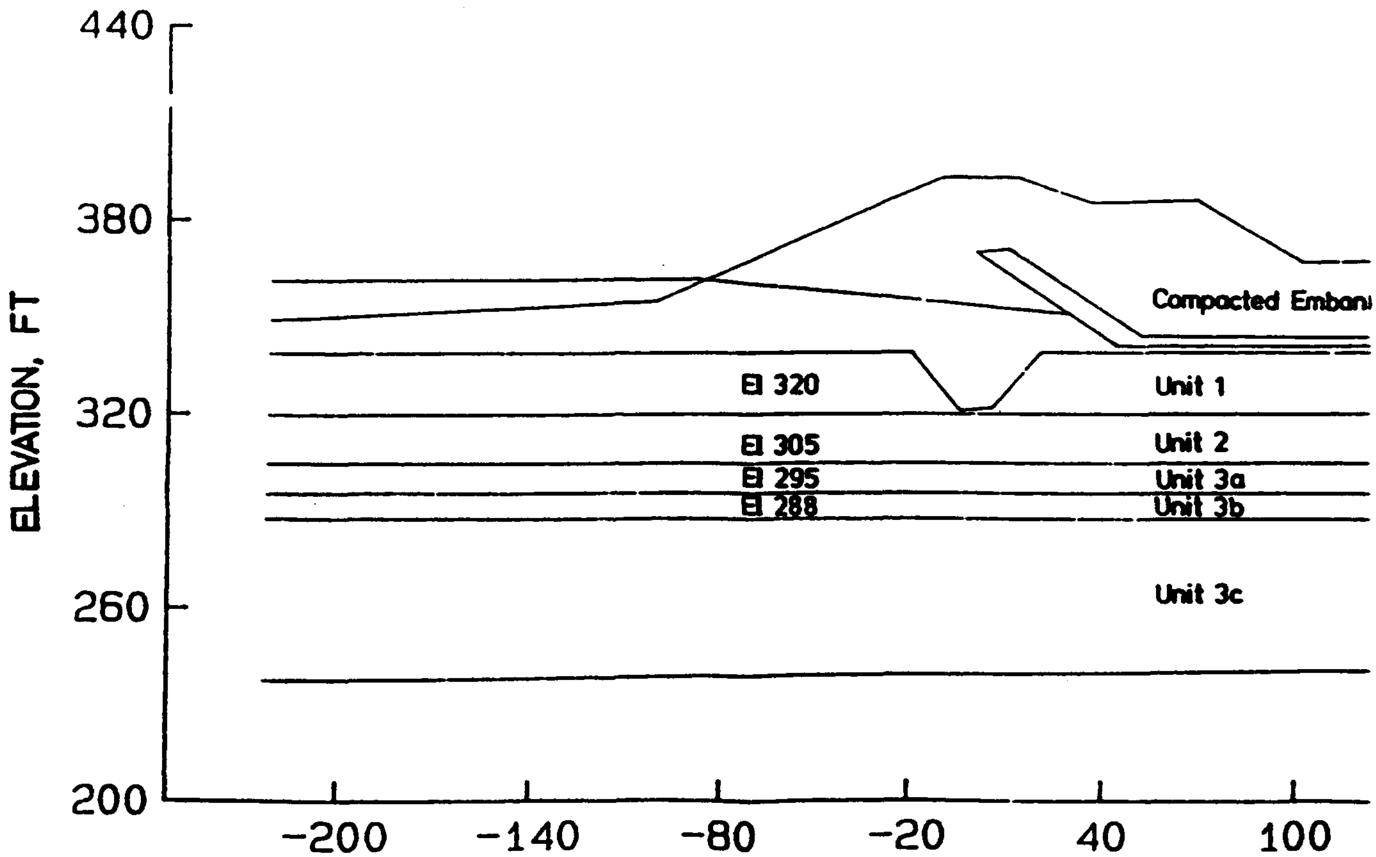
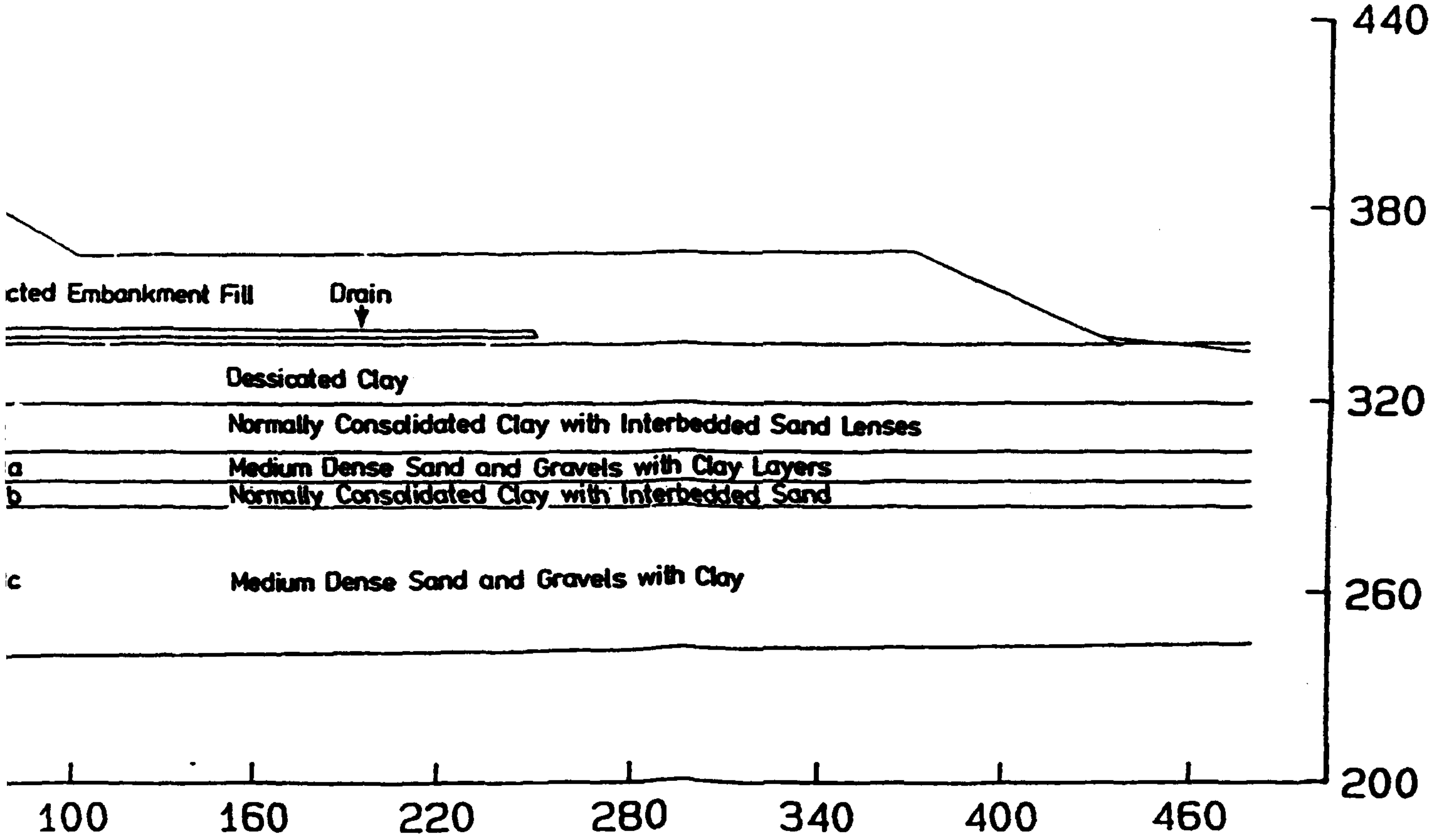


Figure 2a. Cross section showing th

.A



howing the foundation units in the switchyard area.

B

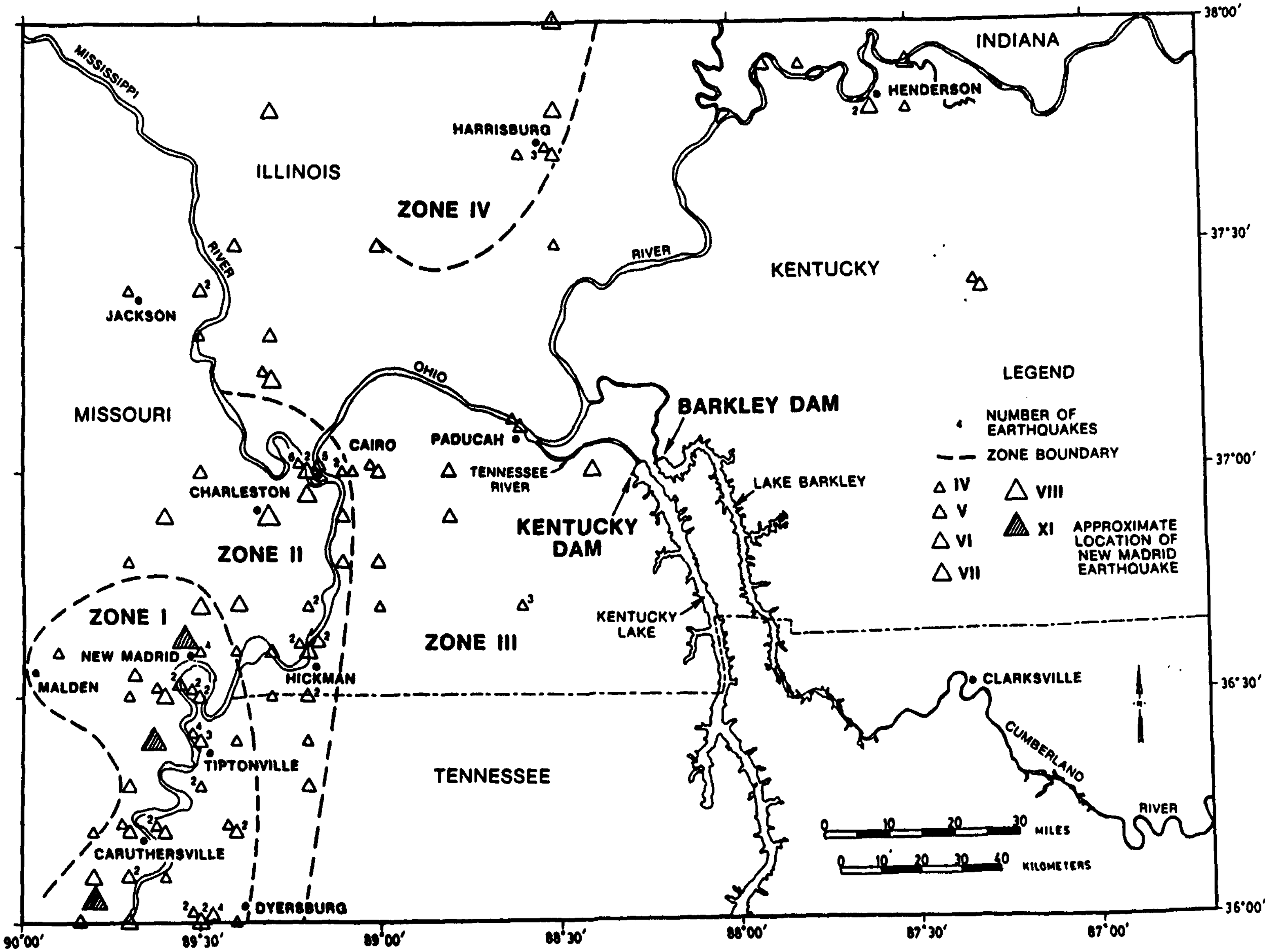


Figure 3. Earthquake history and seismic zones in the region of Barkley Dam. (From Kriwitzsky, 1986)

MODIFIED MERCALLI INTENSITY SCALE OF 1931

(Abridged)

I. Not felt except by a very few under especially favorable circumstances.

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.

III. Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.

IV. During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls made cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.

V. Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles and other tall objects sometimes noticed. Pendulum clocks may stop.

VI. Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Disturbed persons driving motor cars.

IX. Damage considerable in specially designed structures; well designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.

XI. Few, if any (masonry), structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.

XII. Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

Figure 4. Modified Mercalli Intensity scale.

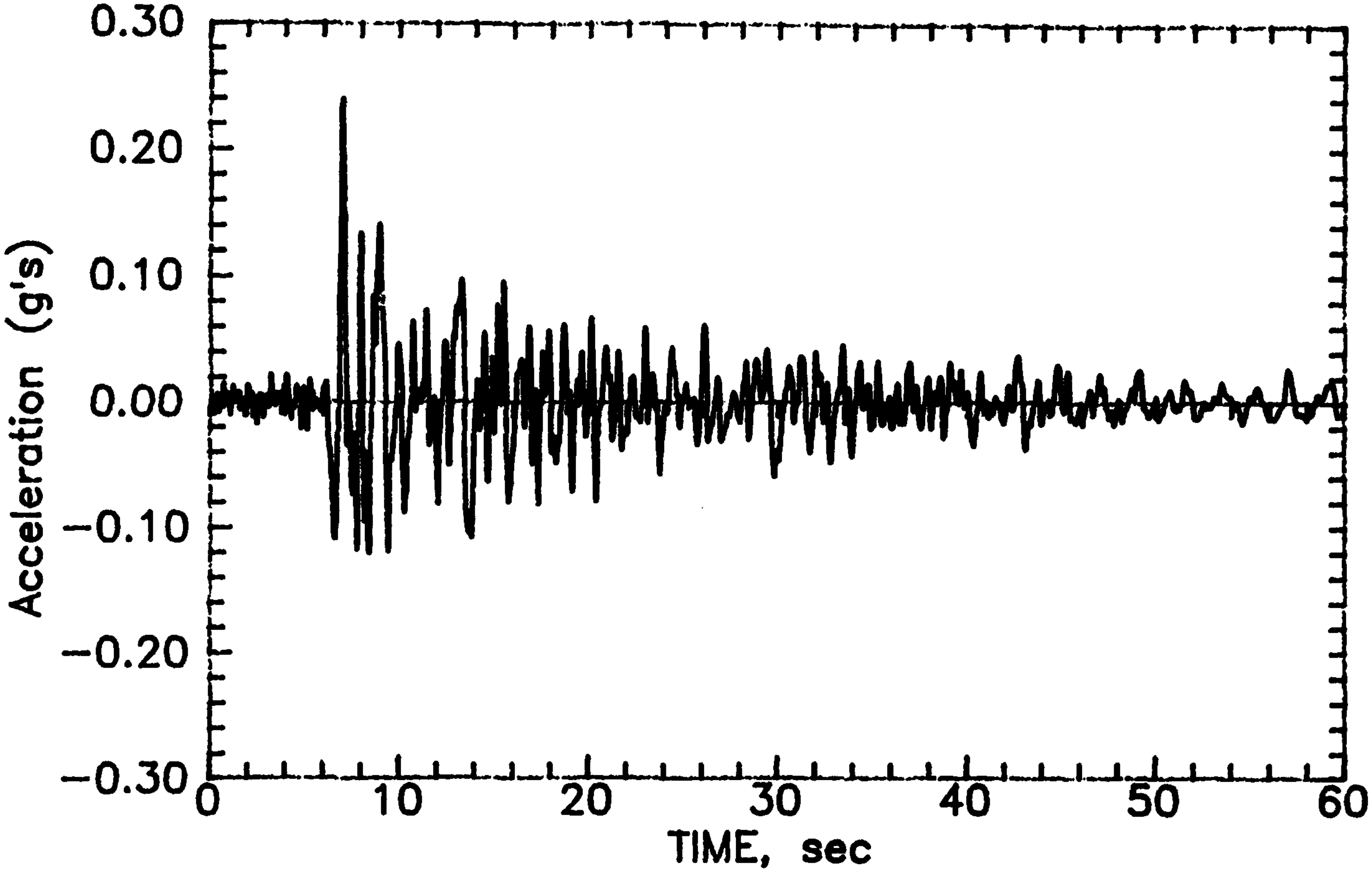


Figure 5. Accelerogram recommended for analysis - Kern County California Earthquake of July 21, 1952 - S48°E component recorded at the Santa Barbara Courthouse scaled to 0.24 g.

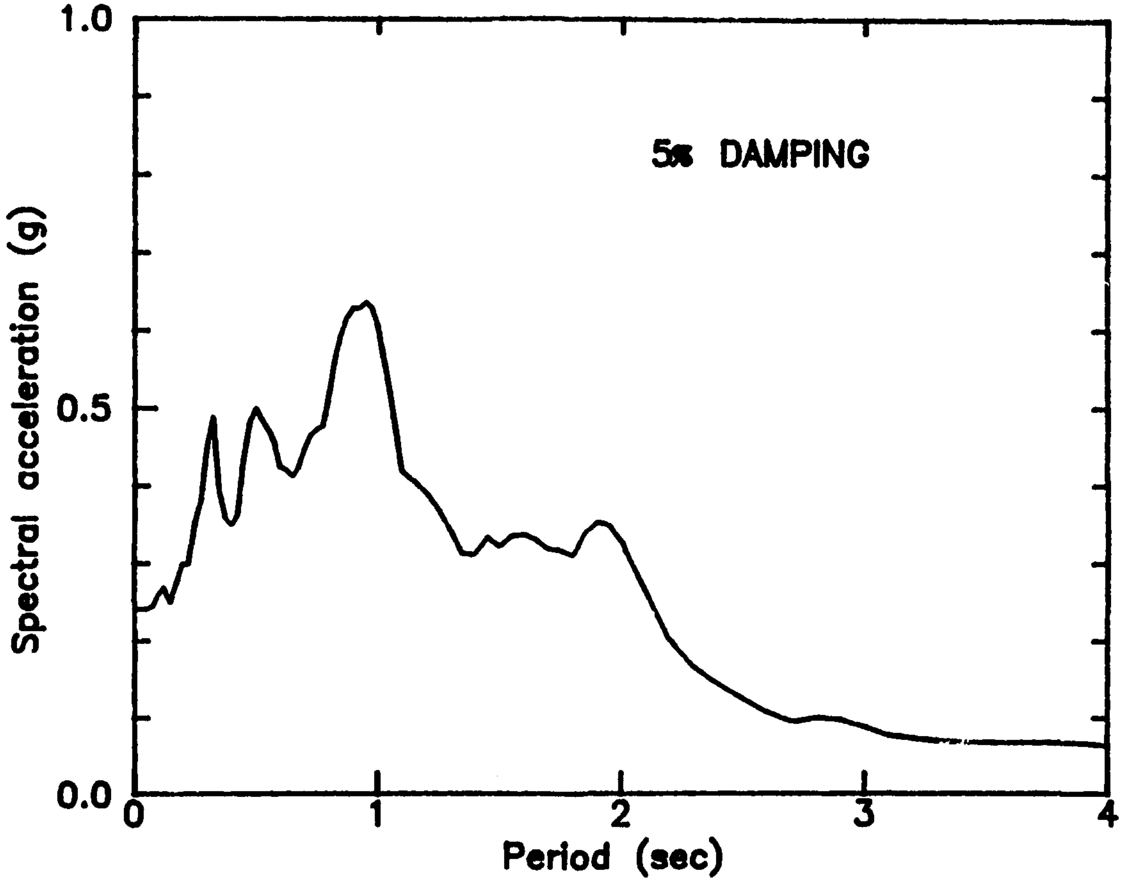


Figure 6. Response spectra of recommended acceleration history at the 5% level of damping.

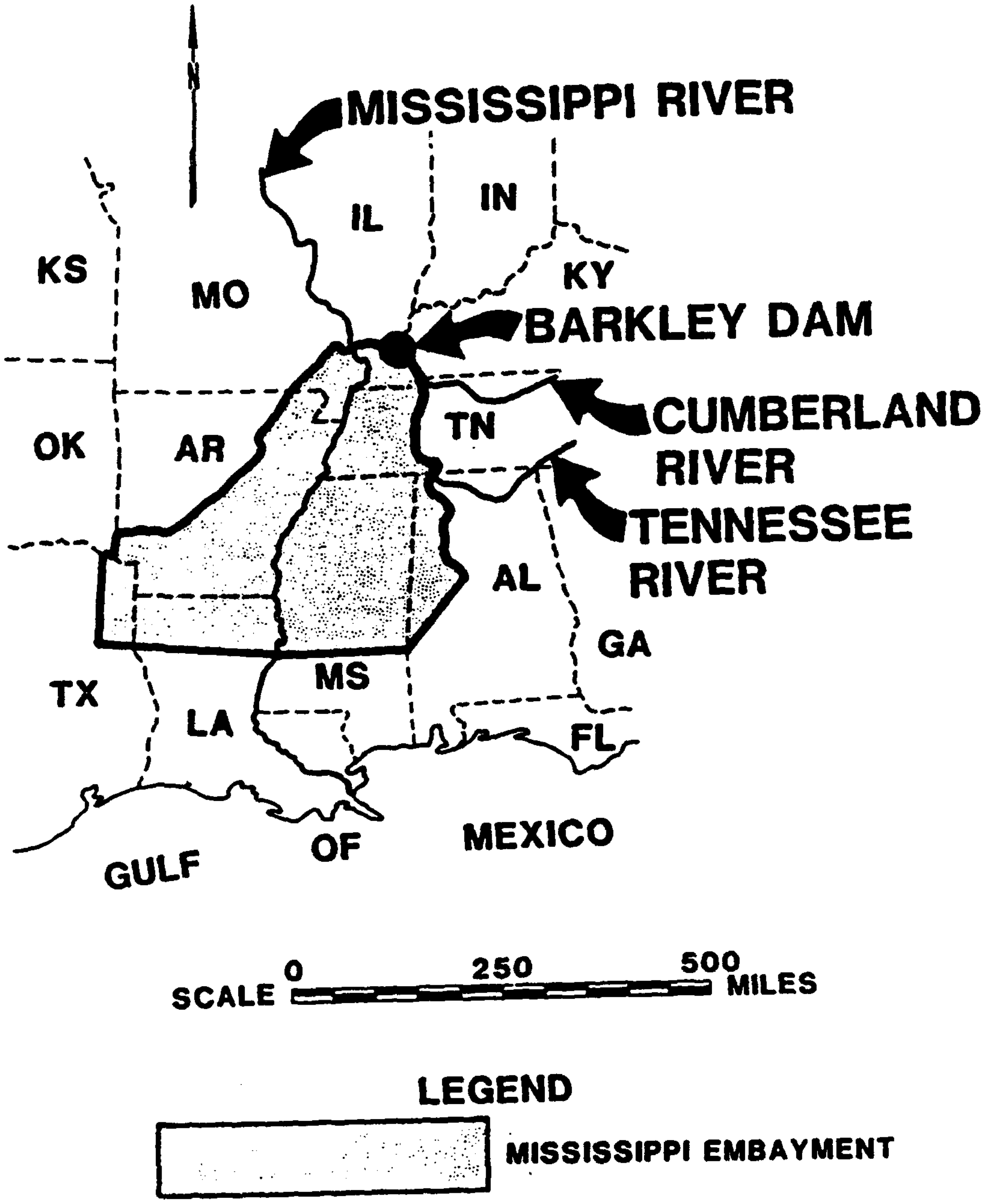


Figure 7. Map of the Mississippi Embayment.

ERA	PERIOD/SYSTEM (TIME) (ROCK)	EPOCH/SERIES (TIME) (ROCK)	GROUP	FORMATION	ROCK TYPE	THICKNESS (FT)	AGE (MY)
CENOZOIC	QUATERNARY	RECENT		ALLUVIUM (1)	SILT, CLAY, SAND & GRAVEL	0-120 (3)	0.01
		PLEISTOCENE		LOESS/CONTINENTAL DEPOSITS	SILT/SAND	0-7	
				GRAVEL	UNCONSOLIDATED GRAVEL & SAND	0-100	1.6
	TERTIARY	PLIOCENE?		GRAVEL	UNCONSOLIDATED GRAVEL & SAND		5.3
		MIOCENE					23.7
		OLIGOCENE					36.6
		EOCENE					57.8
		PALEOCENE					70
MESOZOIC	CRETACEOUS	LATE/UPPER	NAVARRO	McNAIRY	UNCONSOLIDATED SAND & WEAKLY CONSOLIDATED GRAVEL	0-40+	
			WOODBINE	TUSCALOOSA	UNCONSOLIDATED SAND & WEAKLY CONSOLIDATED GRAVEL	0-110+	135
	JURASSIC						208
	TRIASSIC						245
PALEOZOIC	PERMIAN						286
	PENNSYLVANIAN						315
	MISSISSIPPIAN	MERAMECIAN		UPPER ST. LOUIS	LIMESTONE	170+	
				LOWER ST LOUIS/SALEM	LIMESTONE	340-370	
				WARSAW (2)	LIMESTONE	200± (35±) (3)	
		OSAGE		FT. PAYNE (2)	CHERTY LIMESTONE	600±	350
	DEVONIAN	LATE/UPPER		CHATTANOOGA	SHALE	150±	
		EARLY/LOWER		CLEAR CREEK/BAILEY	CHERTY LIMESTONE	800±	400
	SILURIAN	LATE/UPPER	BASS ISLANDS	DECATUR	LIMESTONE	300	
		MIDDLE	BROWNSPORT	BROWNSPORT	LIMESTONE & SHALY LIMESTONE		
				LOUISVILLE	LIMESTONE		
			WAYNE	WALDRON	SHALE		
				LAUREL	LIMESTONE		
				OSGOOD	LIMESTONE		
		EARLY/LOWER		BRASSFIELD	CHERTY LIMESTONE		440
	ORDOVICIAN	CINCINNATIAN		MAQUOKETA	SHALE	300±	
		CHAMPLAINIAN		KIMMSWICK	LIMESTONE	4200	
				PLATTIN	LIMESTONE		
				JOACHIM	LIMESTONE		
				DUTCHTOWN	LIMESTONE		
		CANADIAN		UPPER KNOX	DOLOMITE		500
	CAMBRIAN	ST CROIXIAN		LOWER KNOX	DOLOMITE	1300±	
				BONTERRE	DOLOMITE & HARD SHALE	1500±	
				LAMOTTE & OLDER SEDIMENTS	DOLOMITIC & LIMY HARD SANDSTONE & SILTSTONE	3000±	600
	PRECAMBRIAN			—	HARD IGNEOUS & METAMORPHIC ROCKS		

(1) FOUNDATION FOR EMBANKMENT
(2) FOUNDATION FOR CONCRETE STRUCTURE
(3) THICKNESS AT DAM

Figure 8. Geologic time scale.

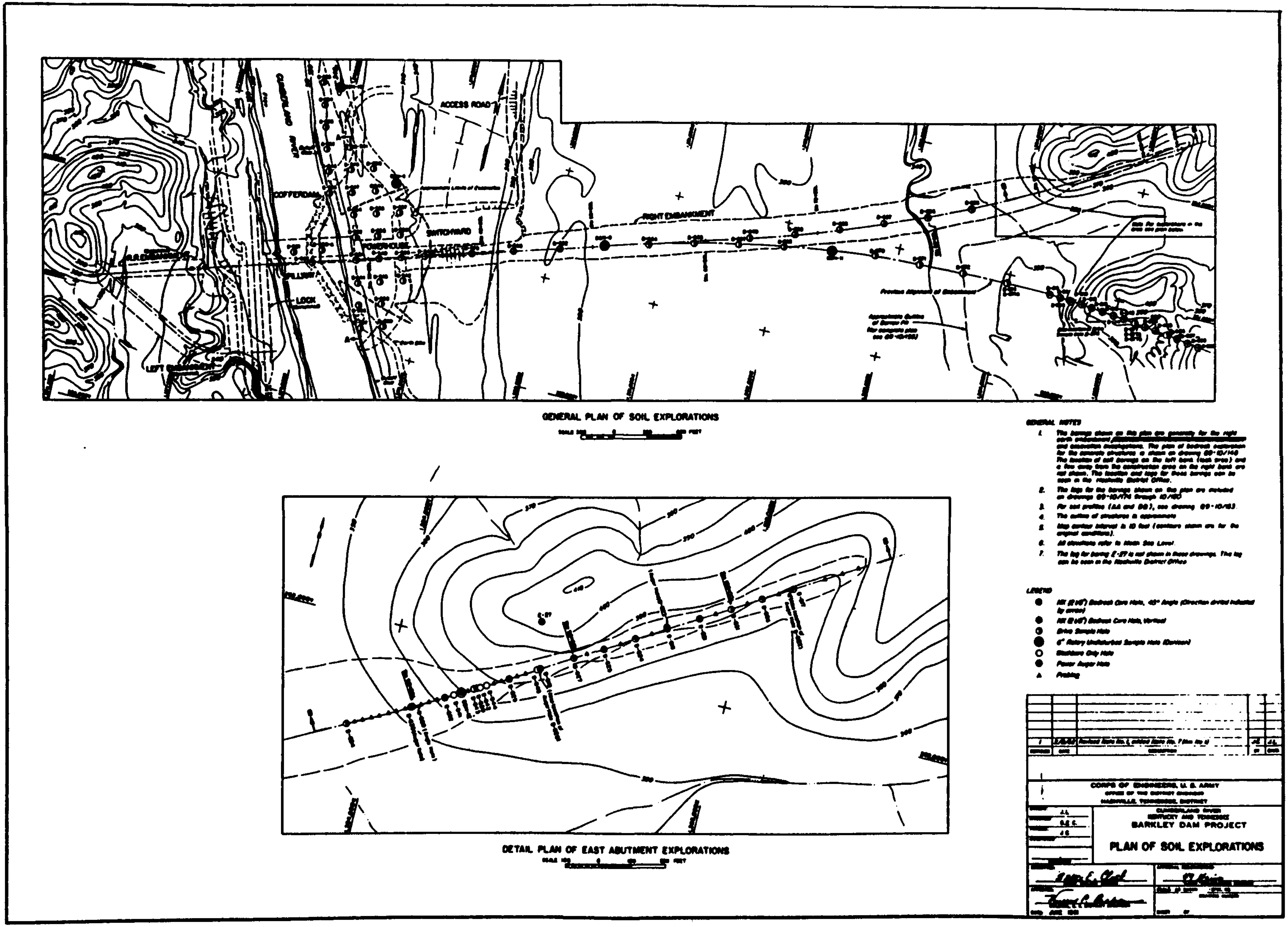


Figure 9. Plan of preconstruction soil explorations.

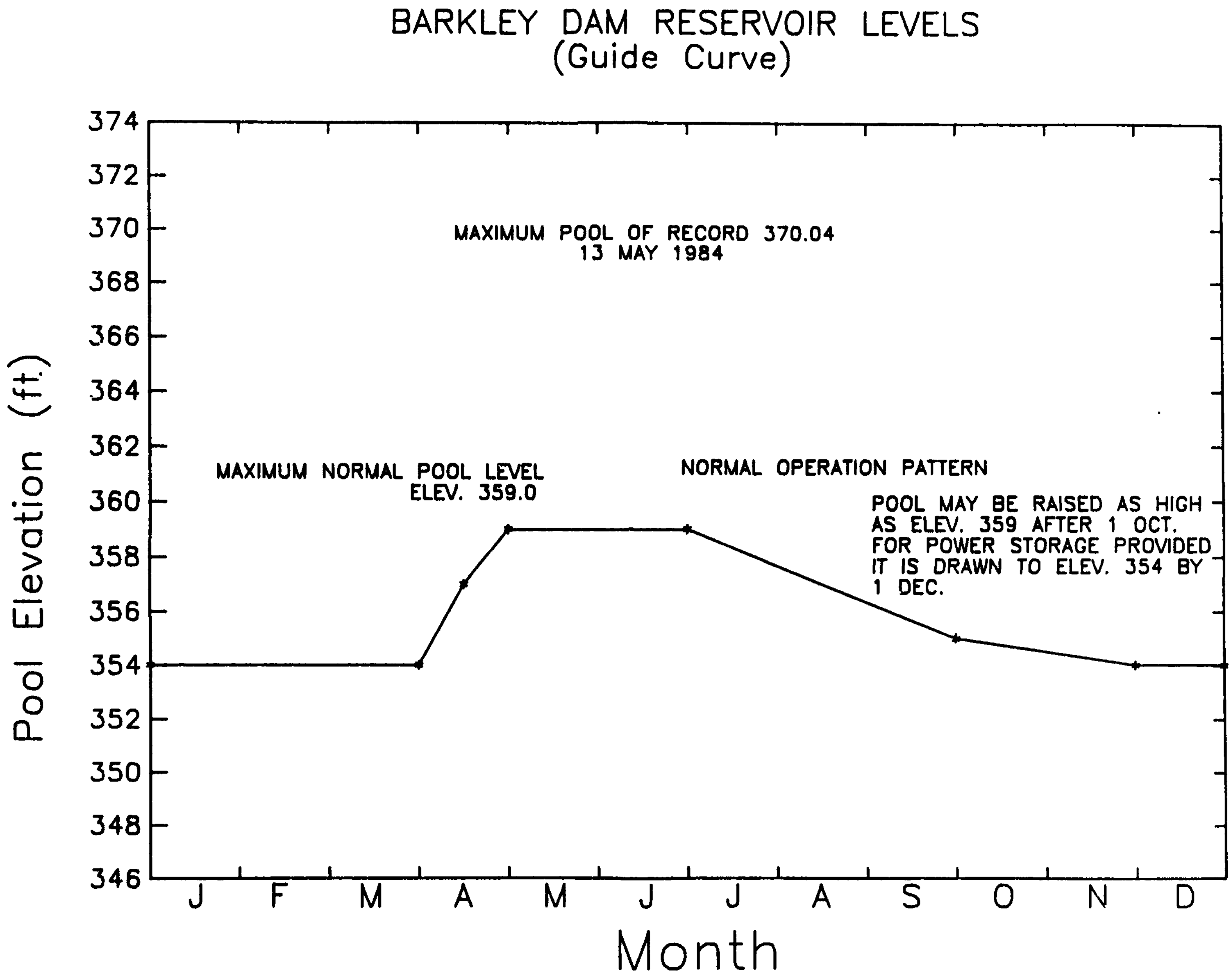


Figure 10. Guide curve for reservoir levels at Barkley Dam.

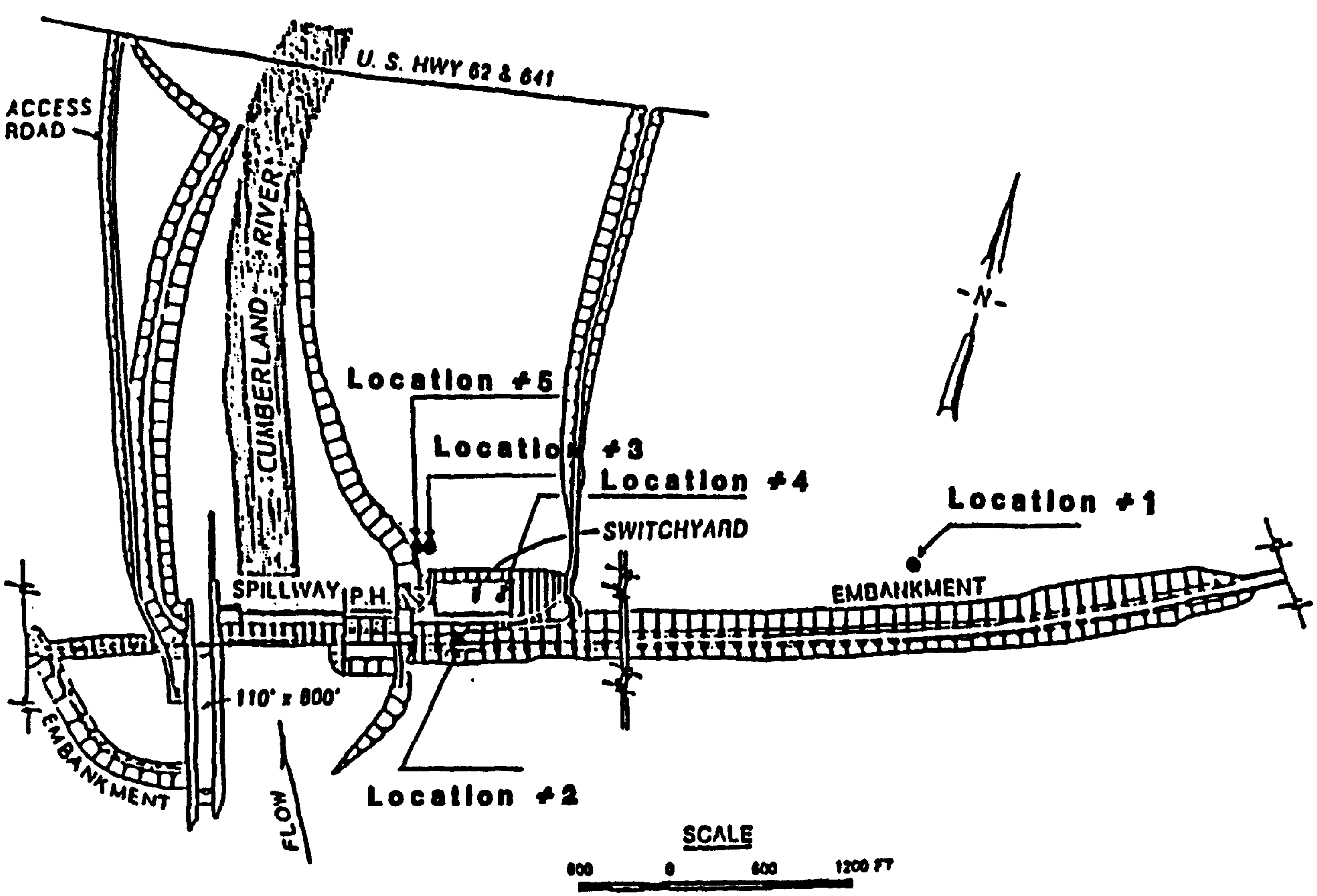


Figure 11. Plan view showing the locations where geophysical tests were performed.

SHEAR WAVE VELOCITIES
(ft/s)

DEPTH (ft)	ELV 350	LOCATION 1	LOCATION 2	LOCATION 3	LOCATION 4	TOTAL
0–15	UNIT 1	420-770 665	700-800 765	550-715 620	890-1095 995	420-1095 760
15–65	UNIT 2	485-770 620	725-940 785	445-715 585	610-1495 980	445-1495 745
65–	UNIT 3	545-1025 870	650-900 785	560-680 630	740-1190 940	545-1190 805
	MAXIMUM TEST DEPTH	115	127	65	86	
	TEST METHOD	CROSSHOLE	CROSSHOLE	CROSSHOLE	DOWNHOLE	

RANGE AND MEAN SHEAR WAVE VELOCITIES CORRELATED TO UNIT 1-3

Figure 12. Comparison of shear wave velocities of Units 1-3.

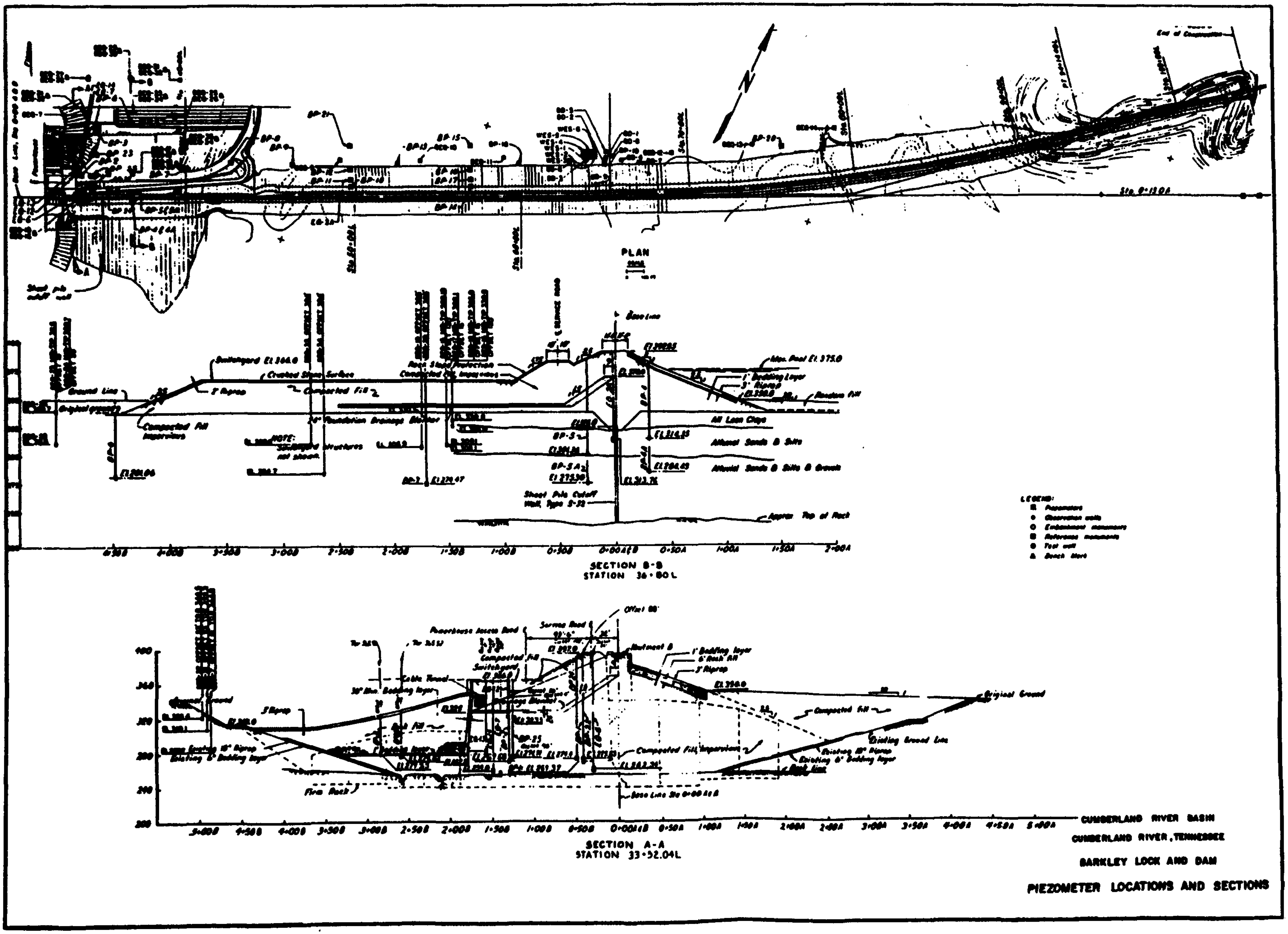


Figure 13. Plan view showing piezometer locations.

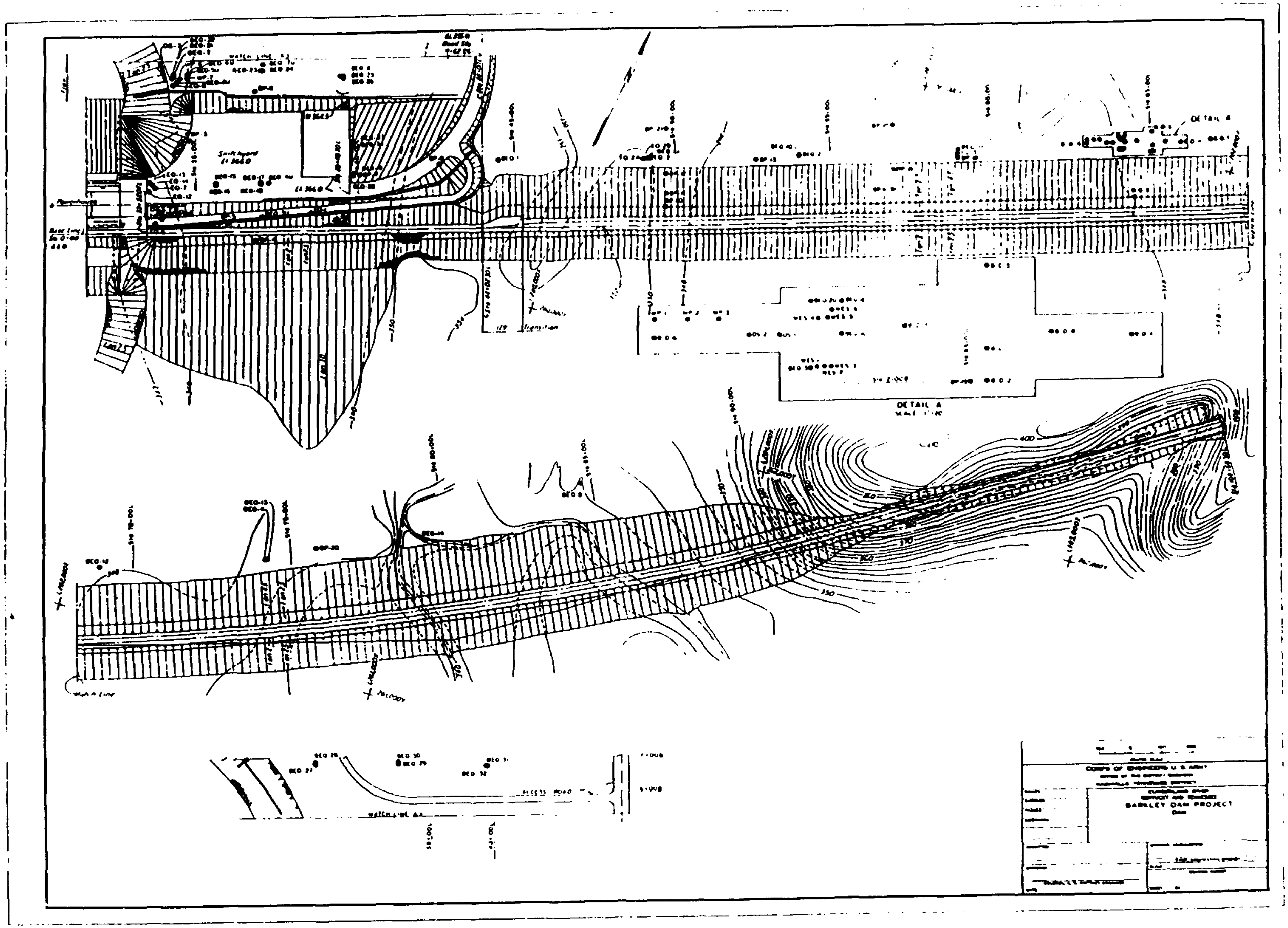


Figure 14. Plan view showing the locations of the borings used to recover undisturbed samples and SPT borings.

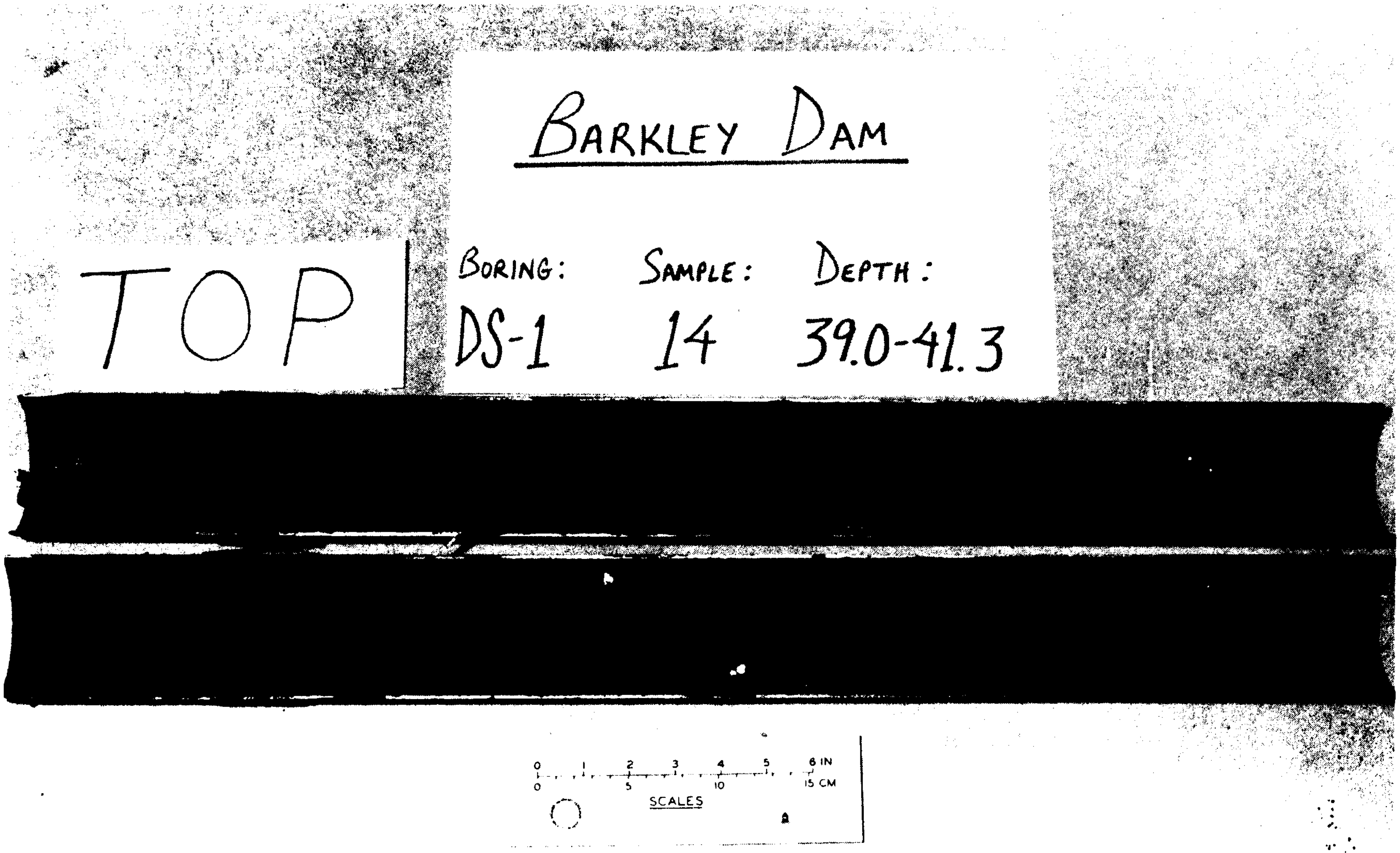


Figure 15. Photograph of undisturbed sample recovered from Unit 2.

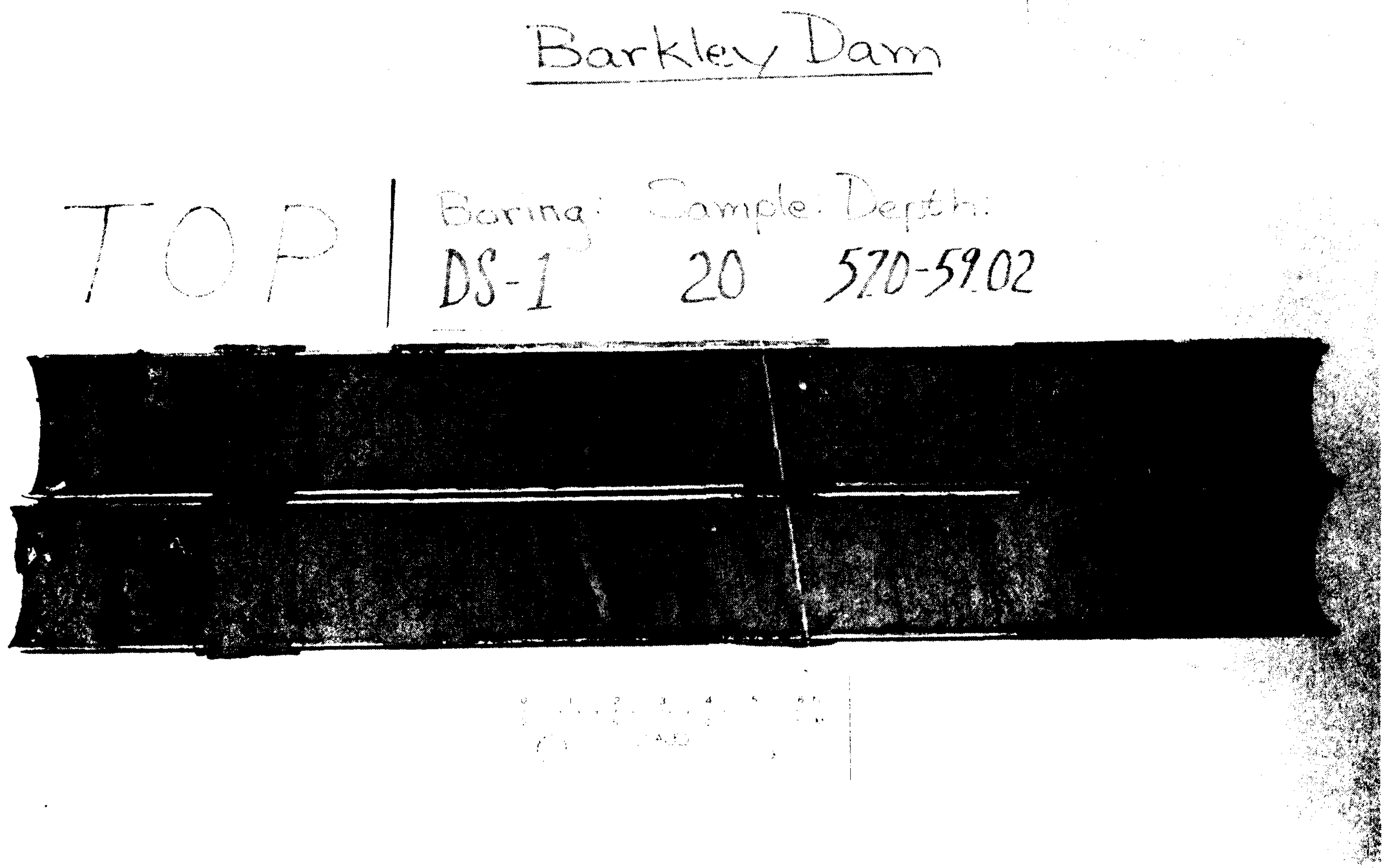


Figure 16. Photograph of undisturbed sample recovered from Unit 3.

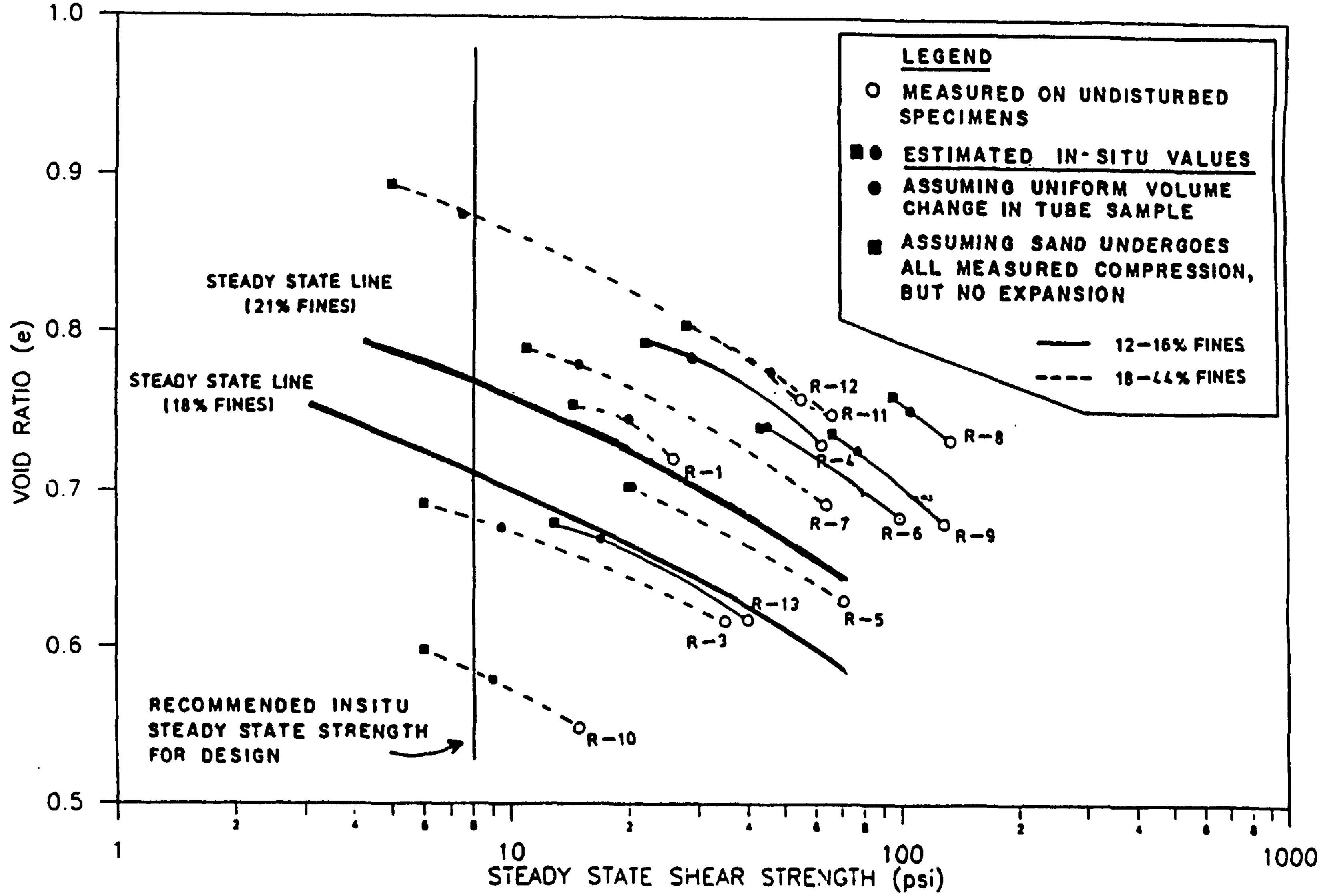


Figure 17. Void ratio versus steady state shear strength from laboratory testing of undisturbed samples of the sands in the foundation of Barkley Dam.

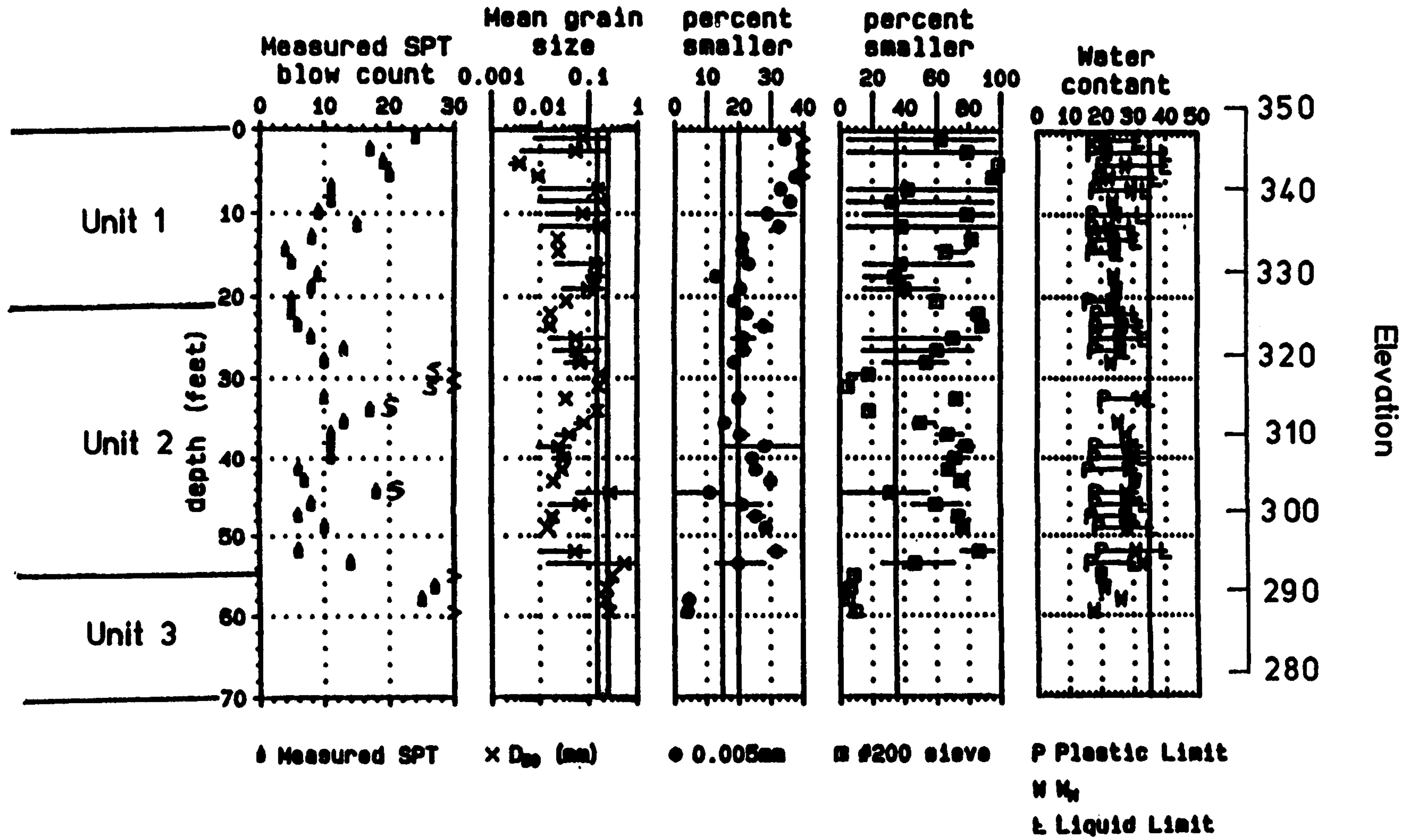


Figure 18. Example of SPT data plot from Boring BEQ-10.

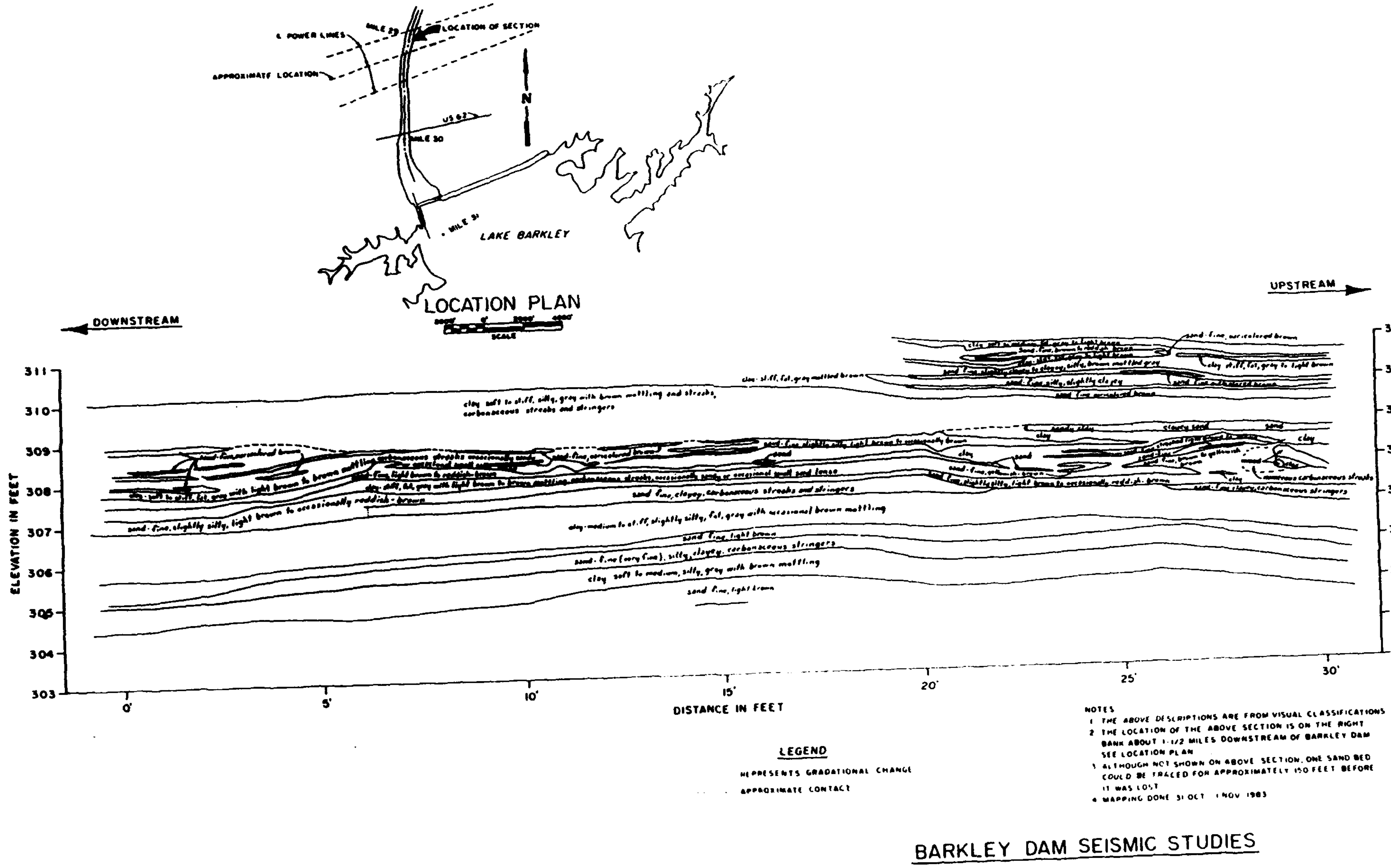
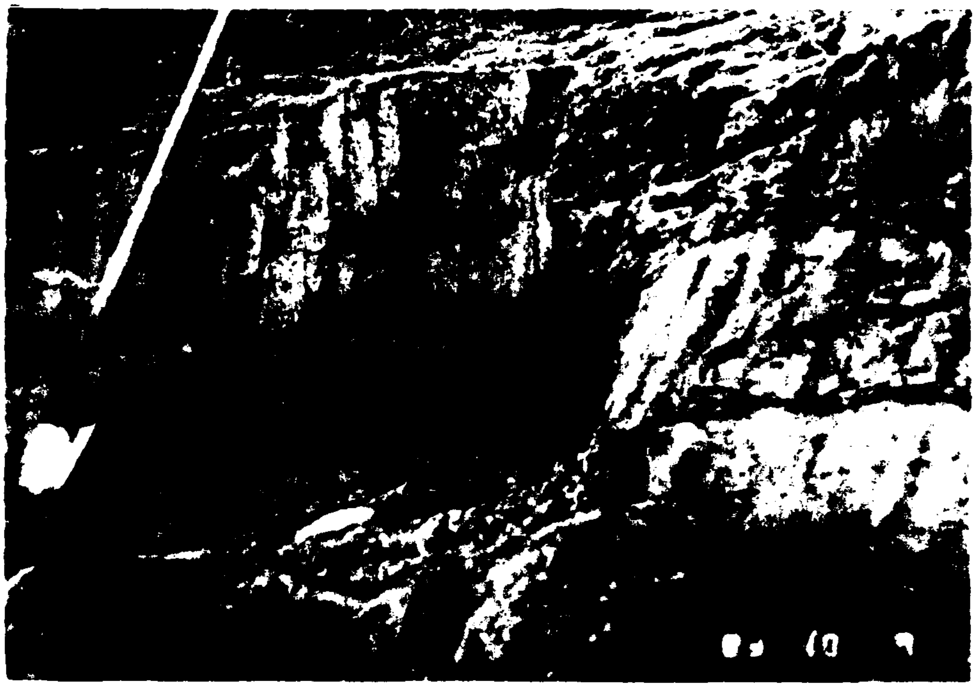


Figure 19. Map of soil profile at downstream streambank exposure.



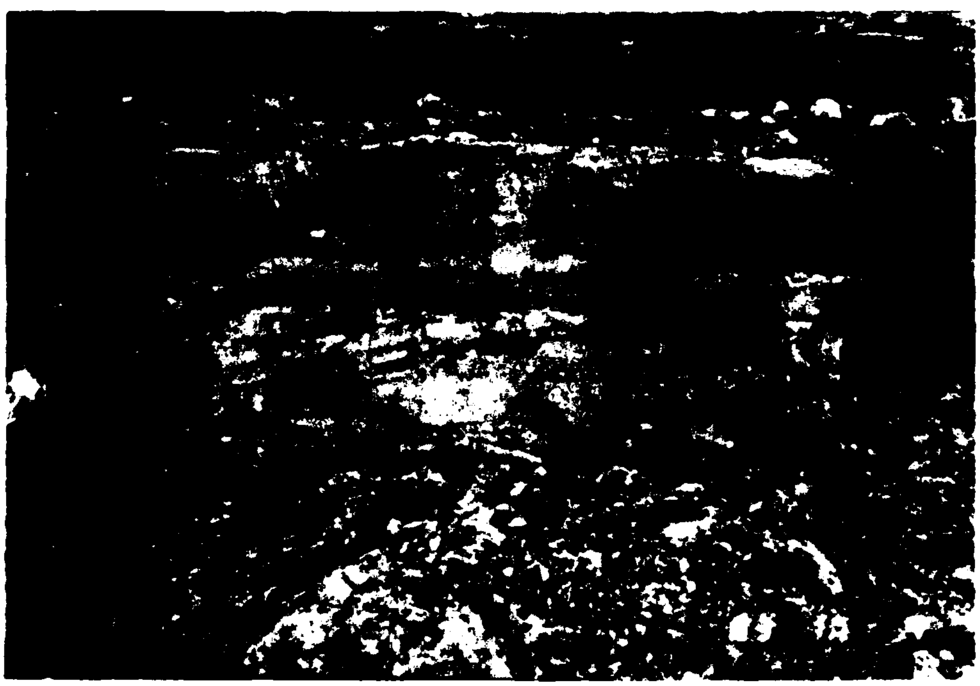


Figure 19a. Photographs of streambank exposure of Unit 2.

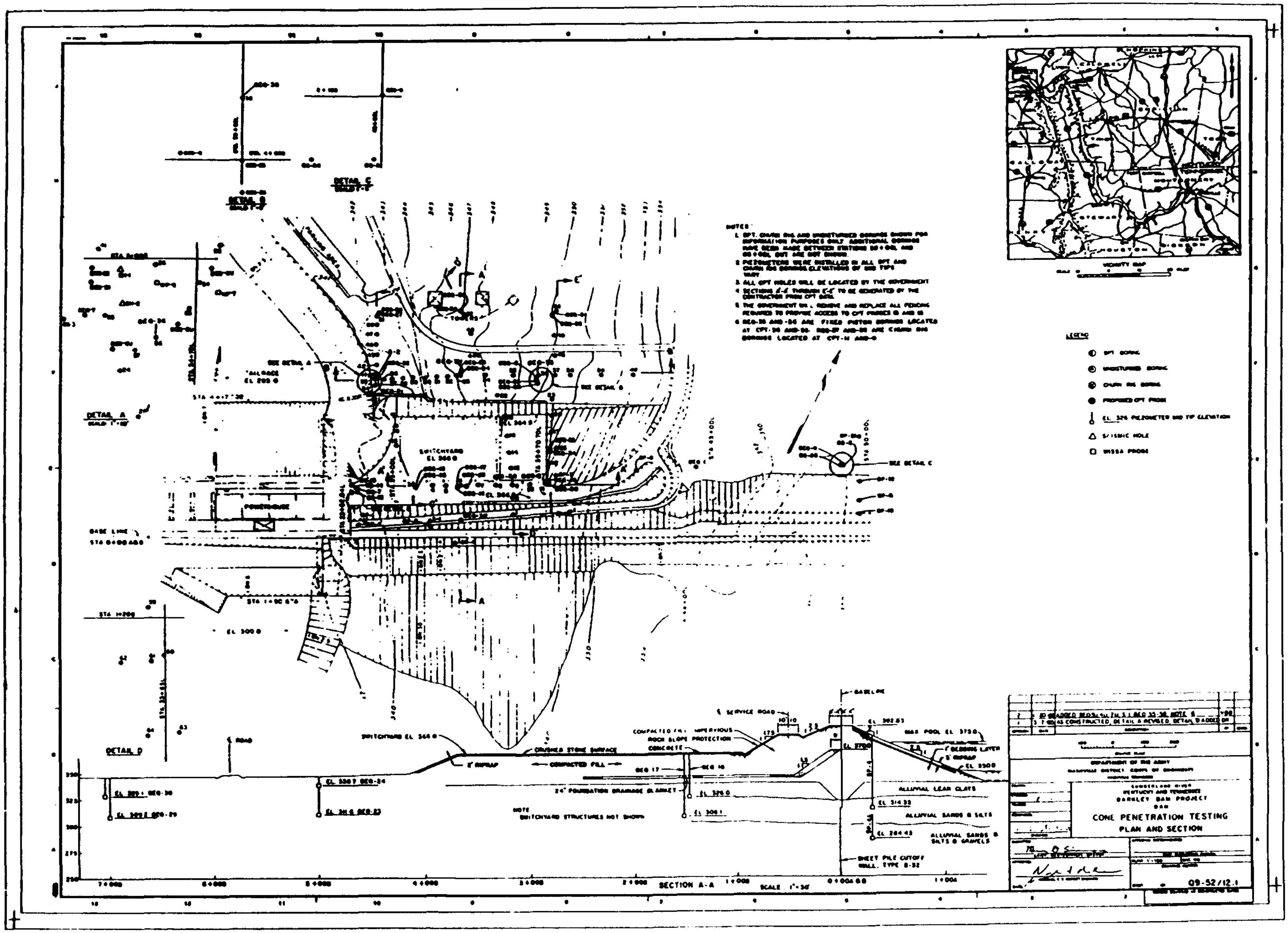
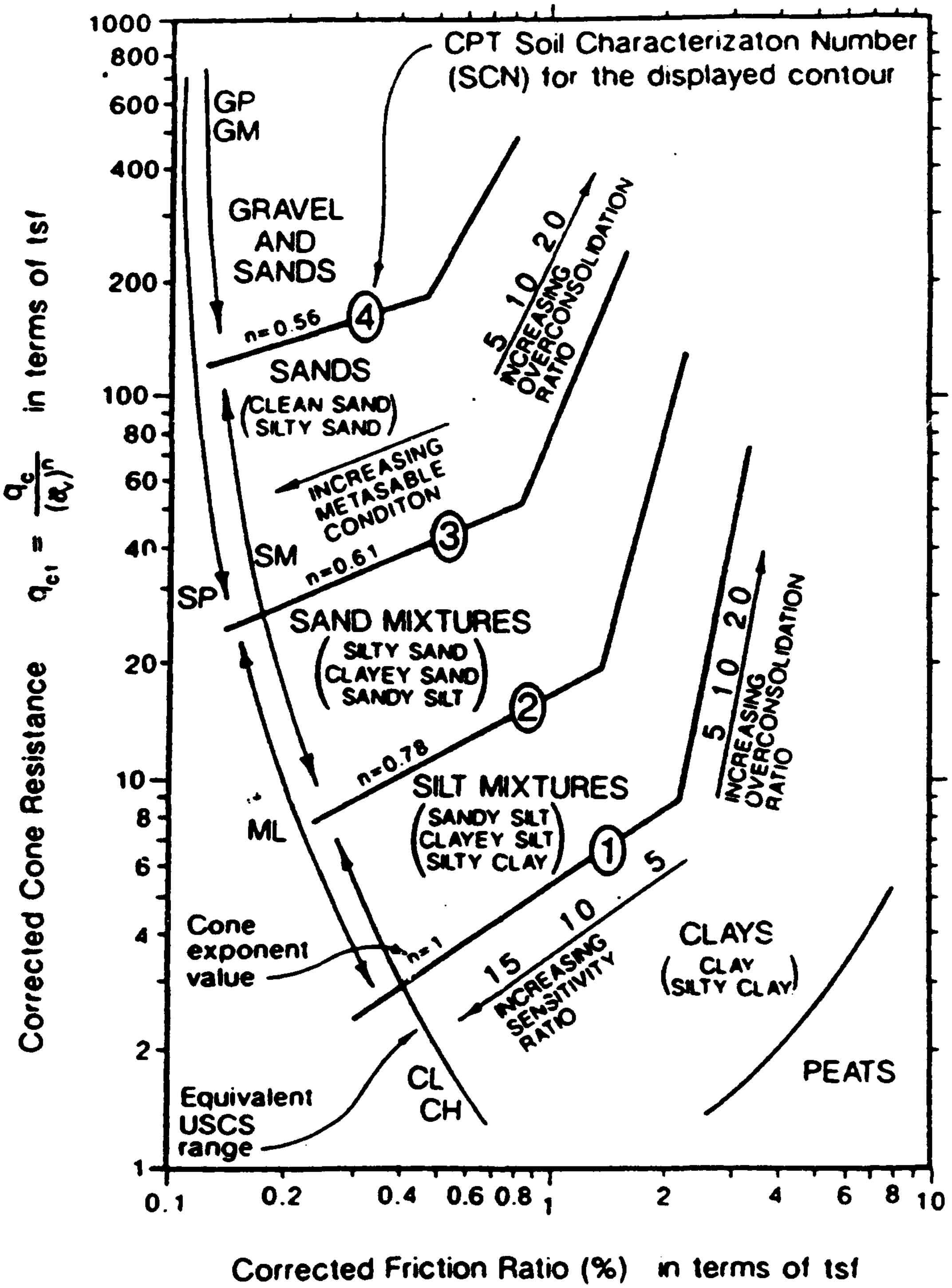


Figure 20. Layout of CPT investigations.



$$FR_1 = \frac{f_{s1}}{q_{c1}} 100 = \frac{f_s/\bar{\sigma}_v}{q_c/(\bar{\sigma}_v)^n} 100 = \frac{f_s}{q_c} \frac{1}{(\bar{\sigma}_v)^{(1-n)}} 100$$

Figure 21. CPT soil classification chart (Olsen, 1988).

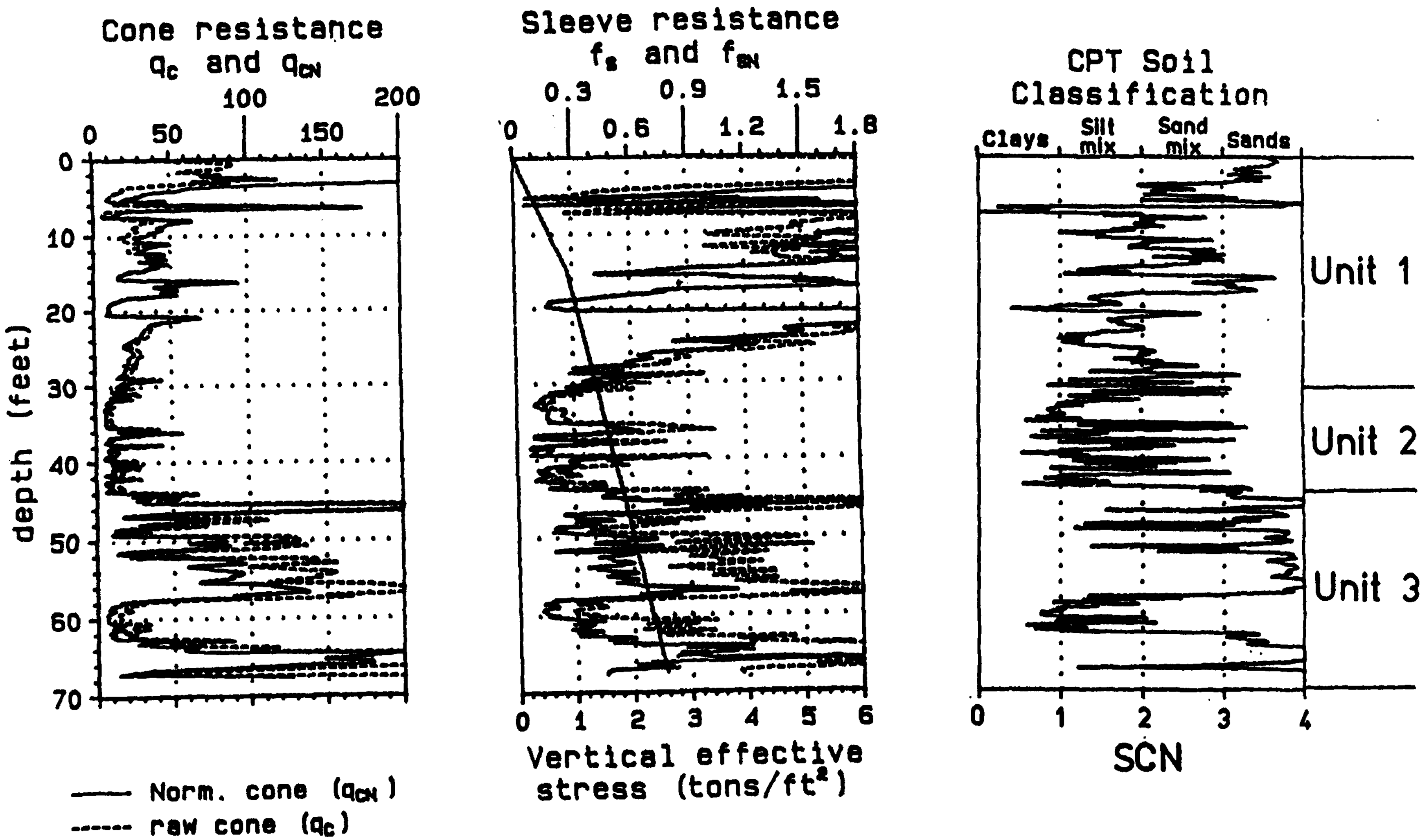


Figure 22. Example of data contained in CPT data base from sounding CPT-36.

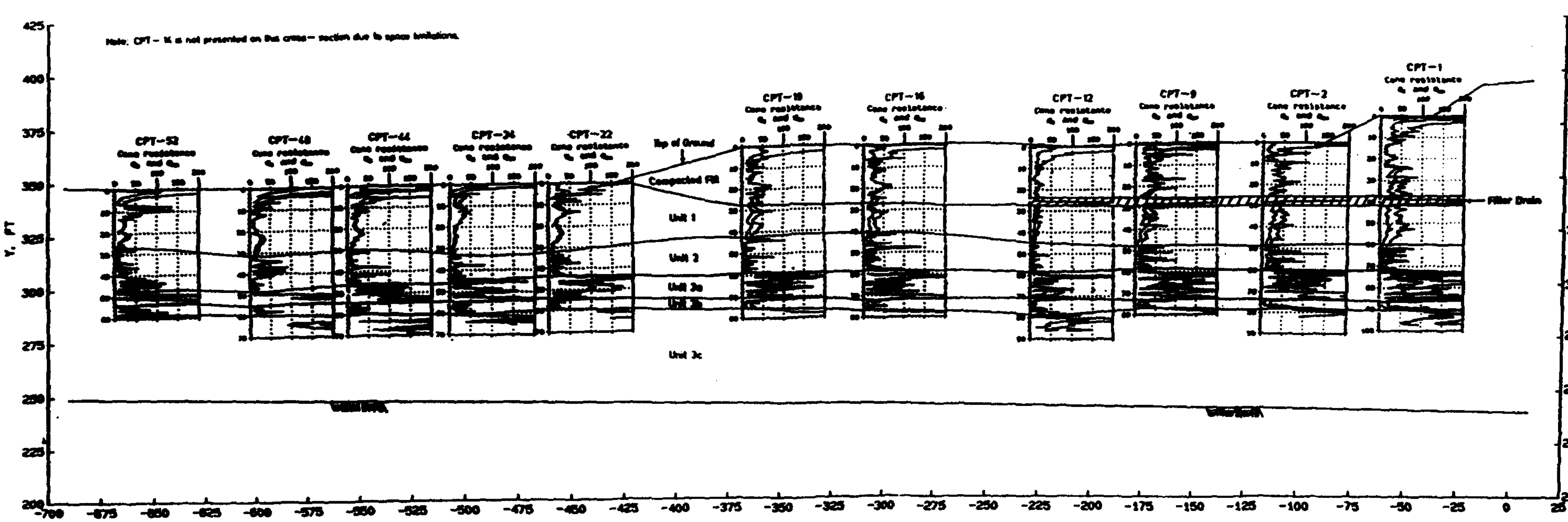


Figure 23. CPT tip resistances along section *D - D'* running perpendicular to the axis of the dam.

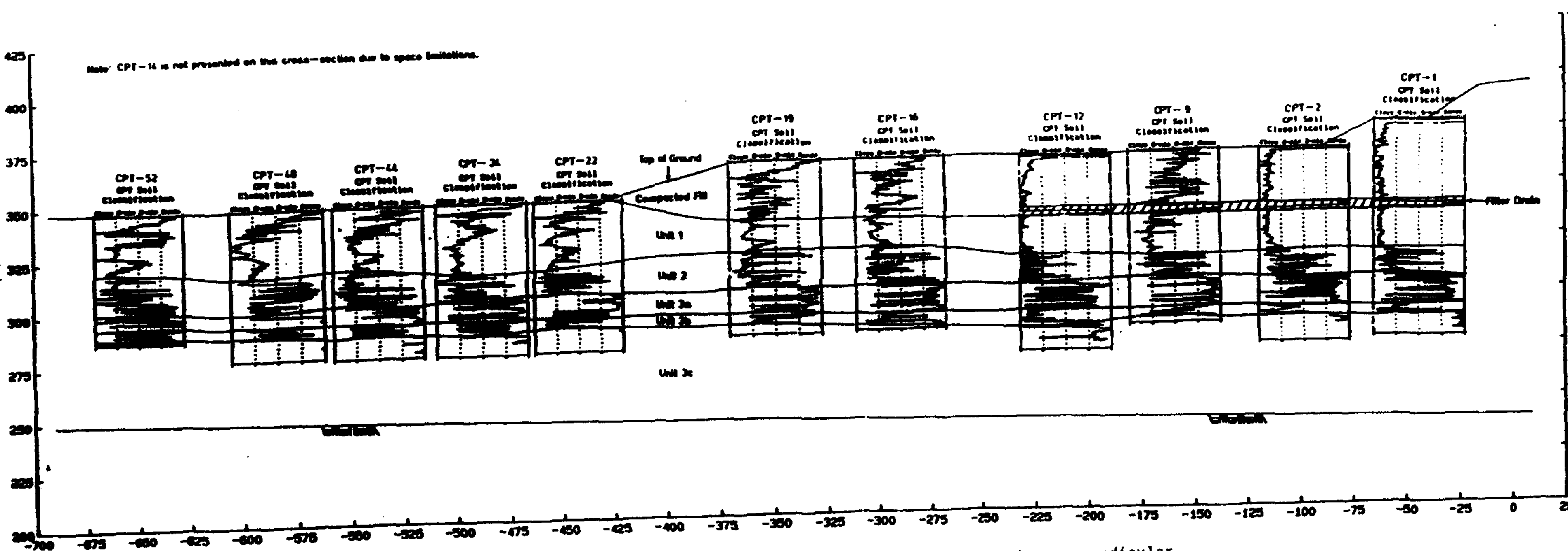


Figure 24. CPT soil classifications along section D-D' running perpendicular to the axis of the dam.

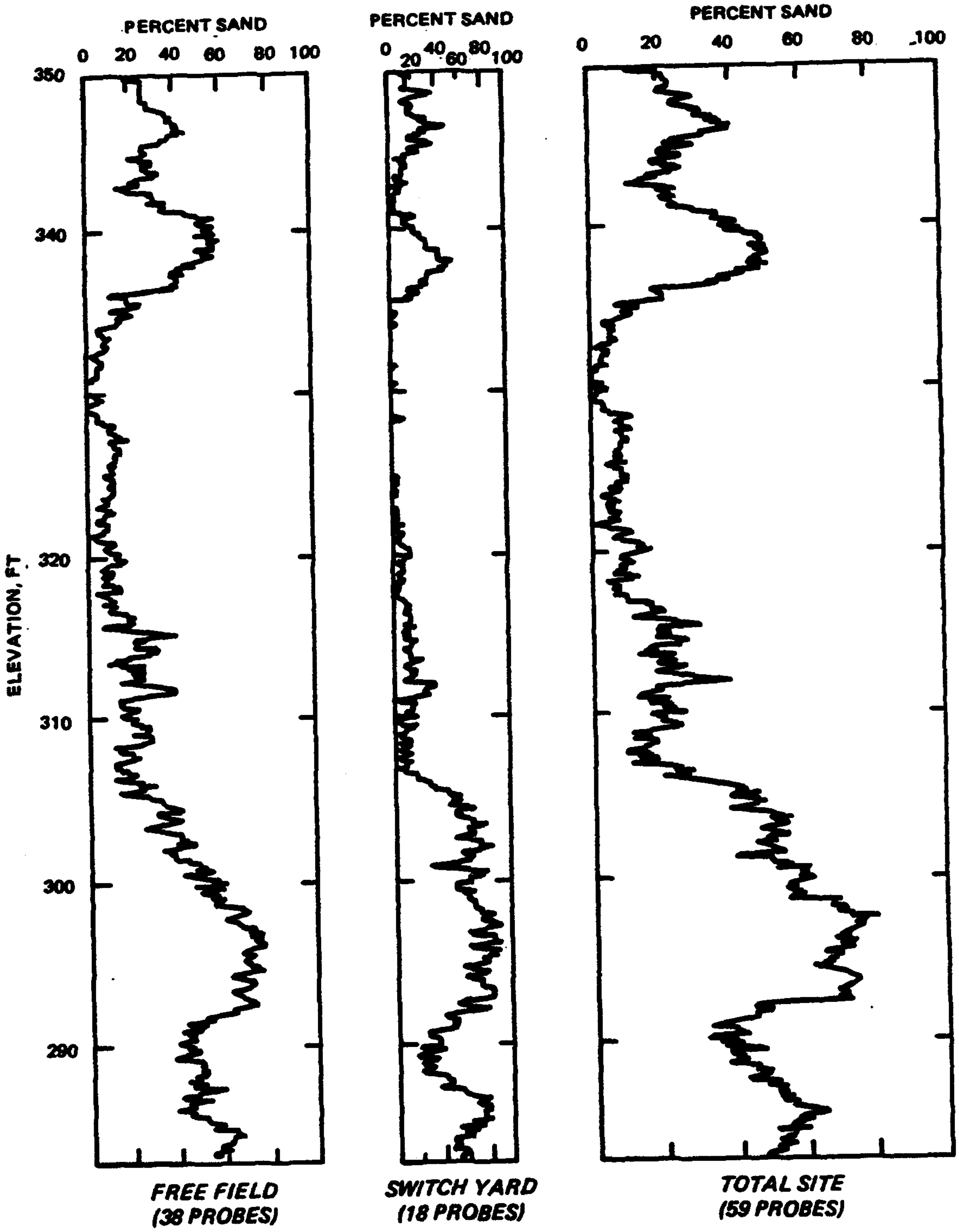


Figure 25. Sand percentages as functions of elevation across the site.

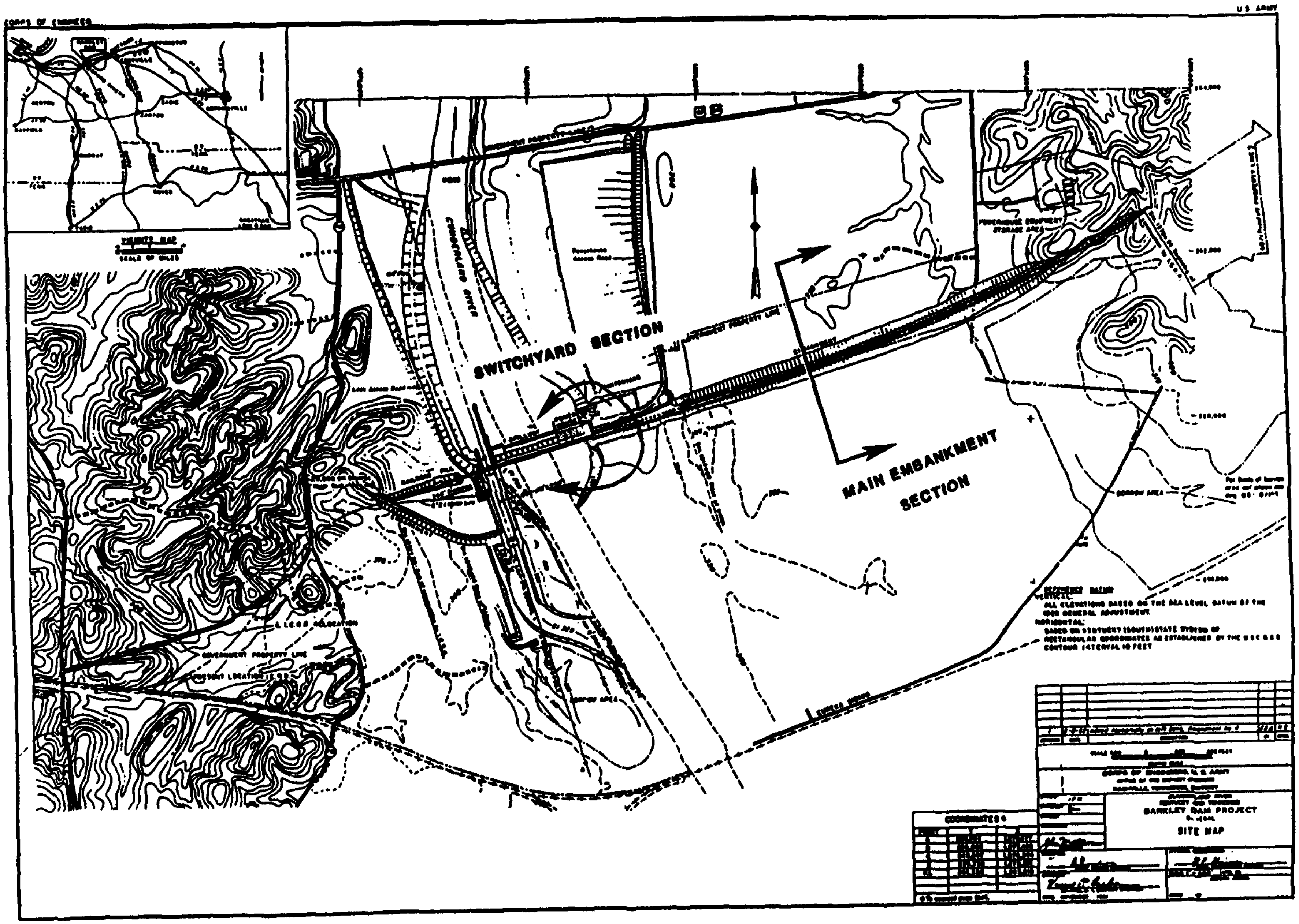


Figure 26. Plan view showing the locations of the analysis sections.

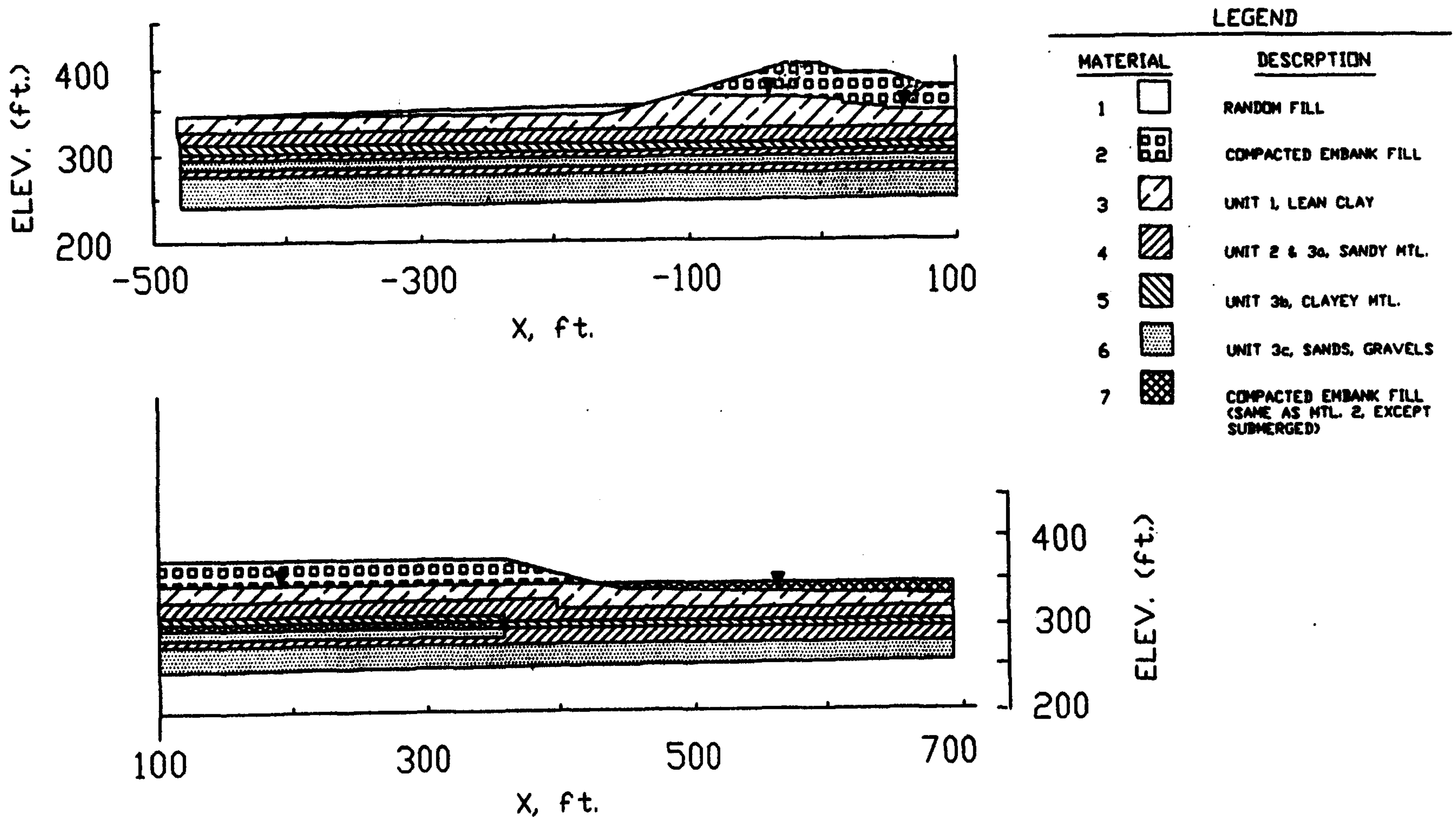


Figure 27. Material distribution for static analysis.

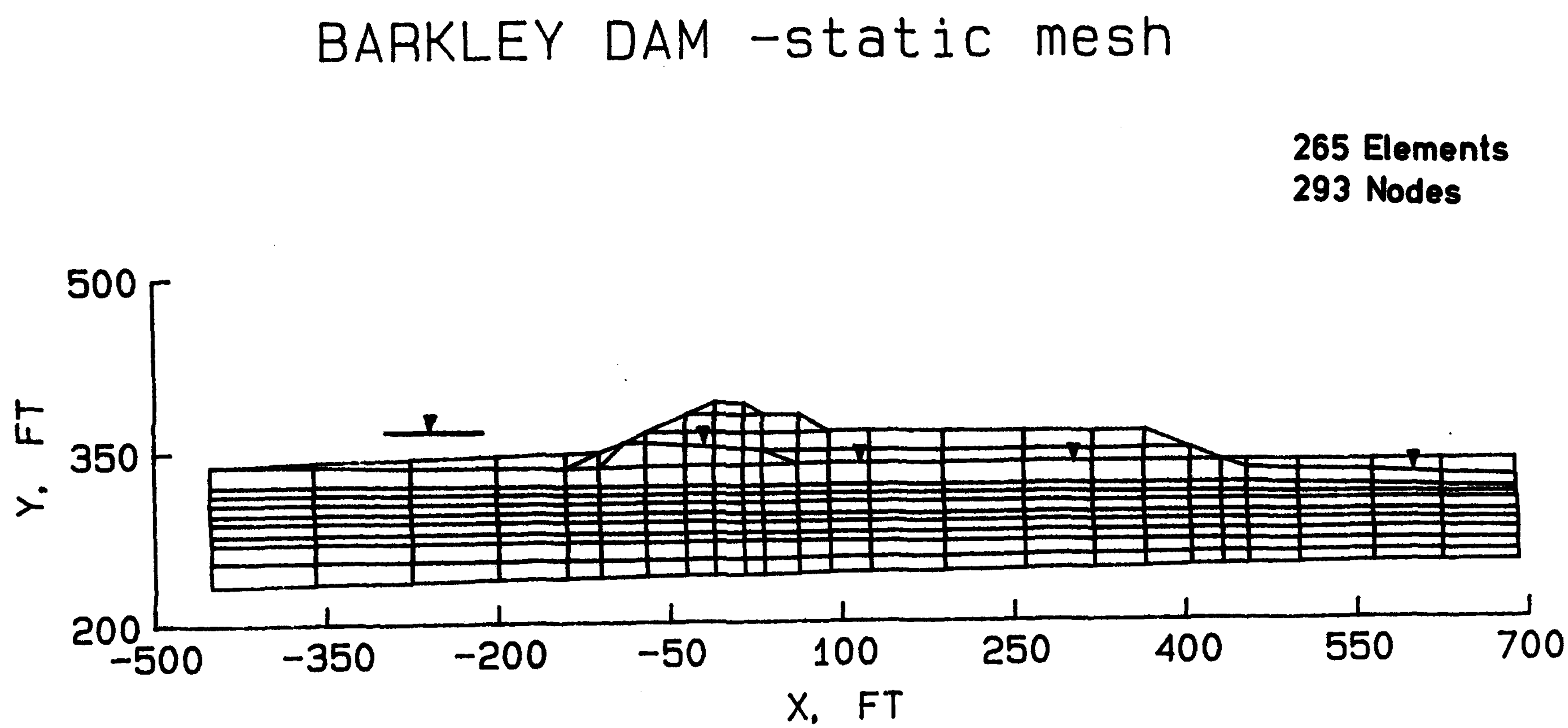


Figure 28. Finite element mesh for static analysis.

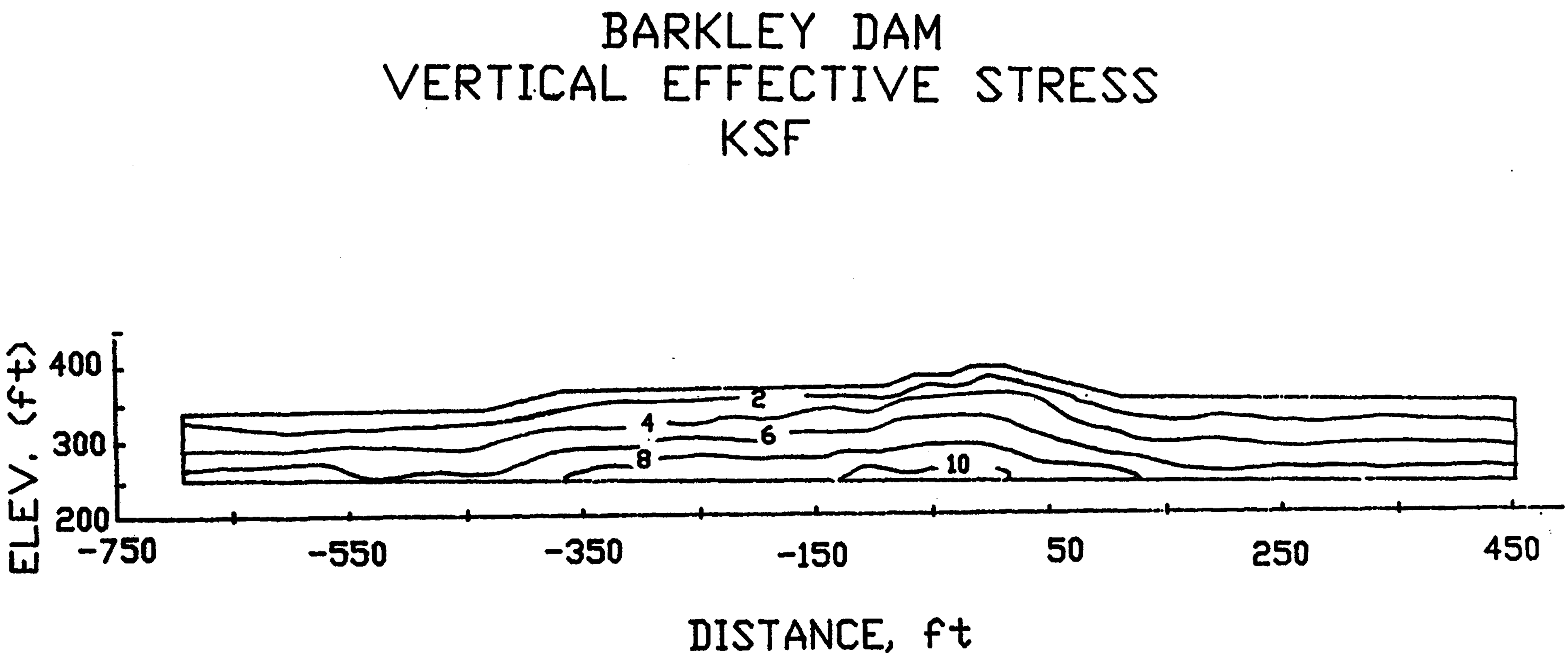


Figure 29. Contours of vertical effective stress, ksf.

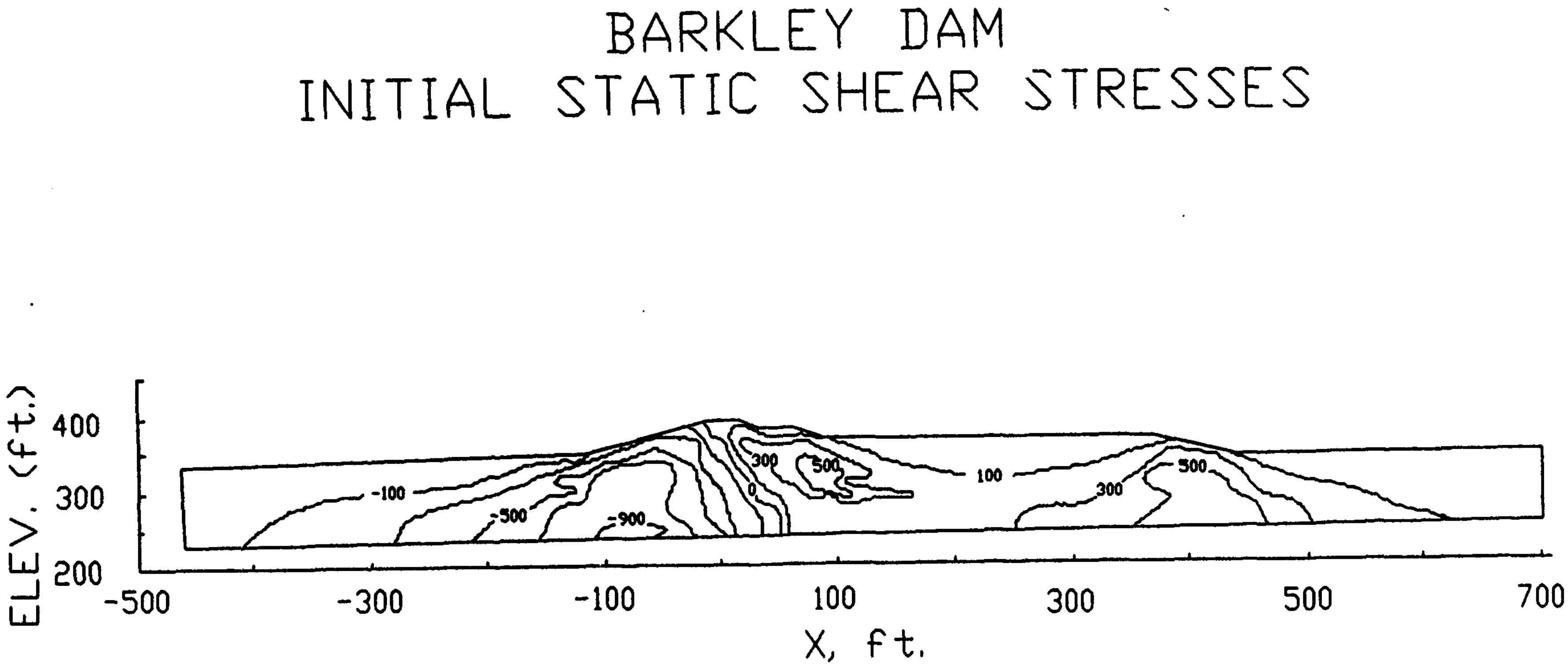


Figure 30. Initial static shear stresses on horizontal planes.

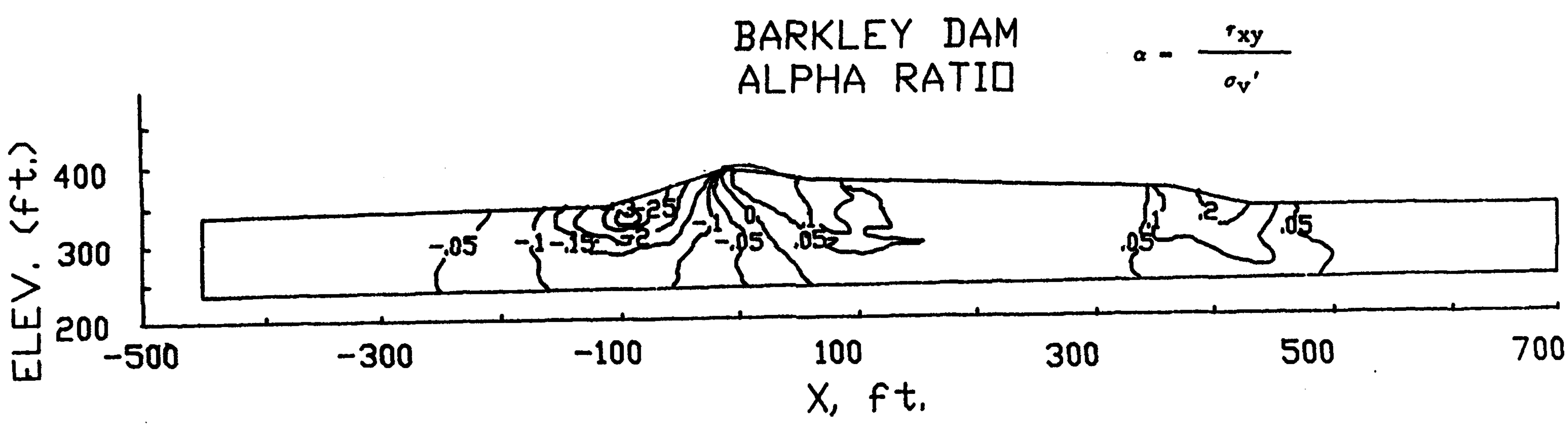


Figure 31. Contours of alpha ratio.

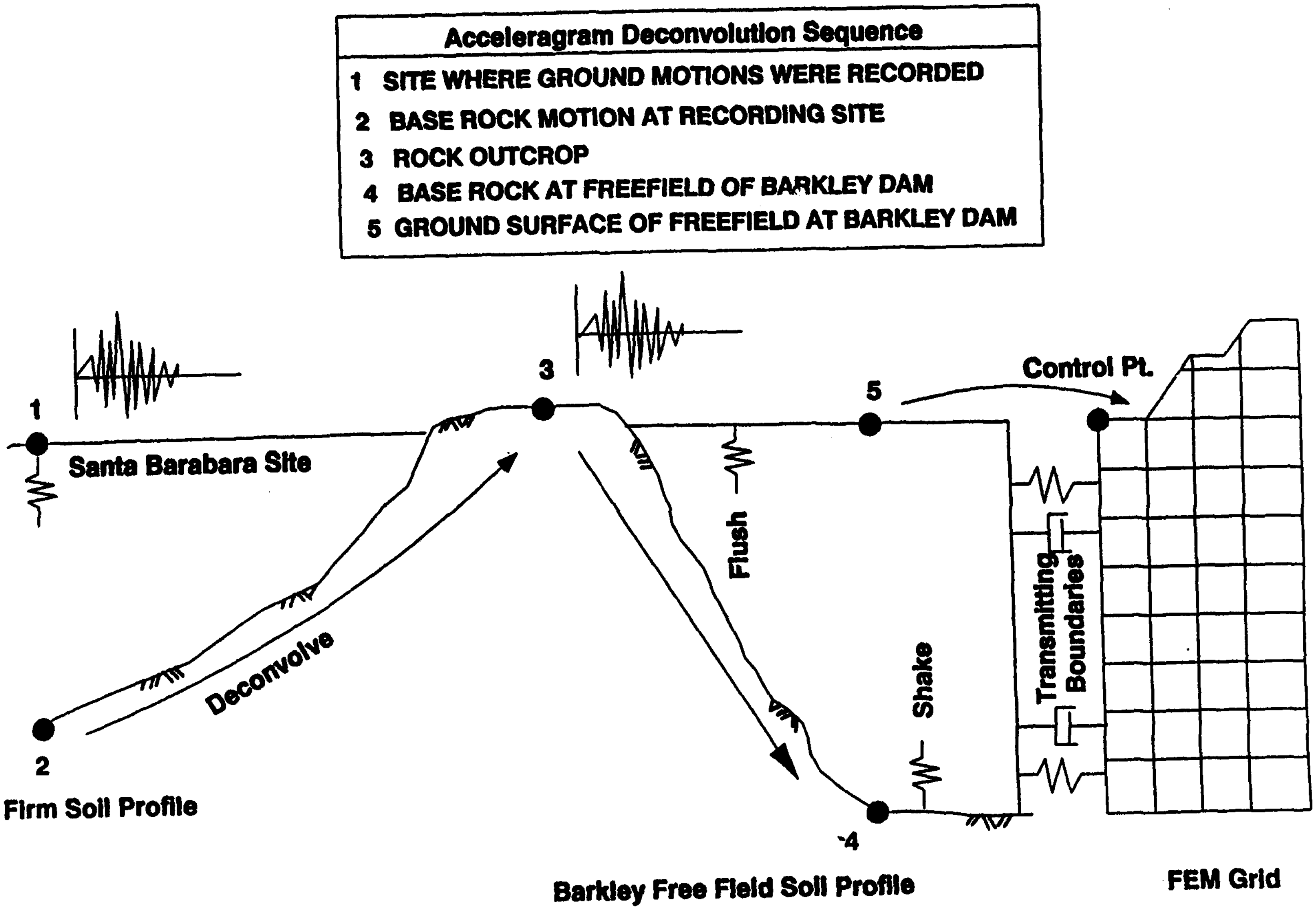
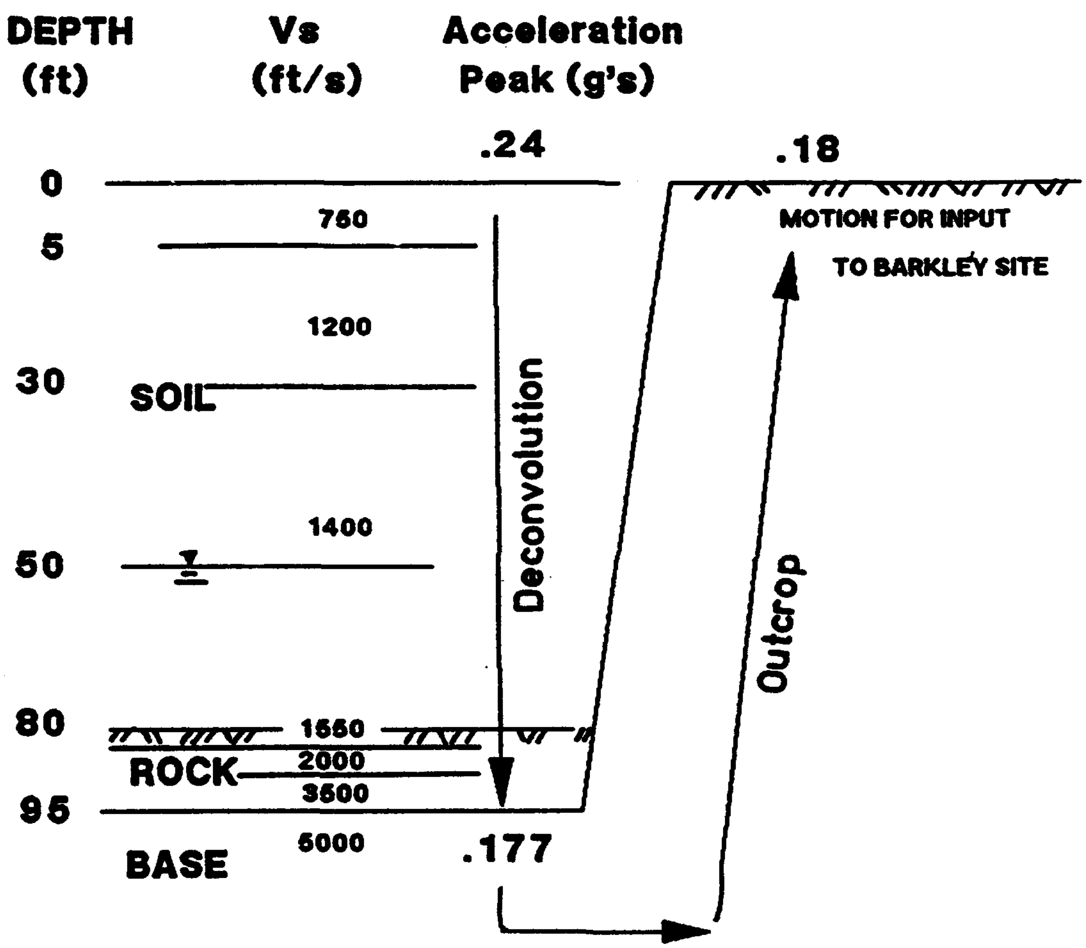


Figure 32. Schematic showing how input acceleration was developed.

DECONVOLUTION



SANTA BARBARA SITE PROFILE

MODIFIED : TRANSITION TO HARD ROCK BASE

Figure 33. Santa Barbara soil profile.

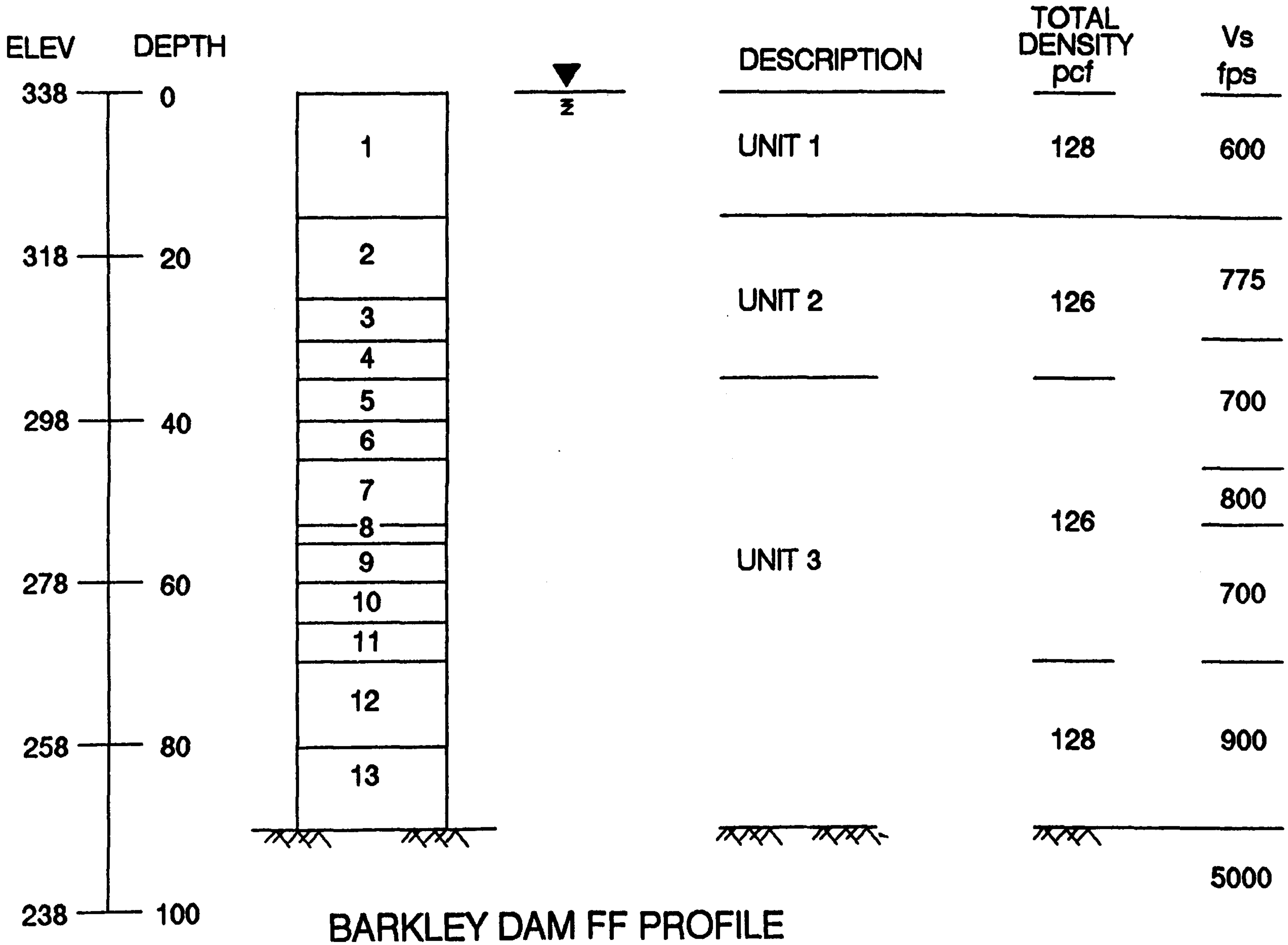


Figure 34. Free field soil profile at Barkley Dam.

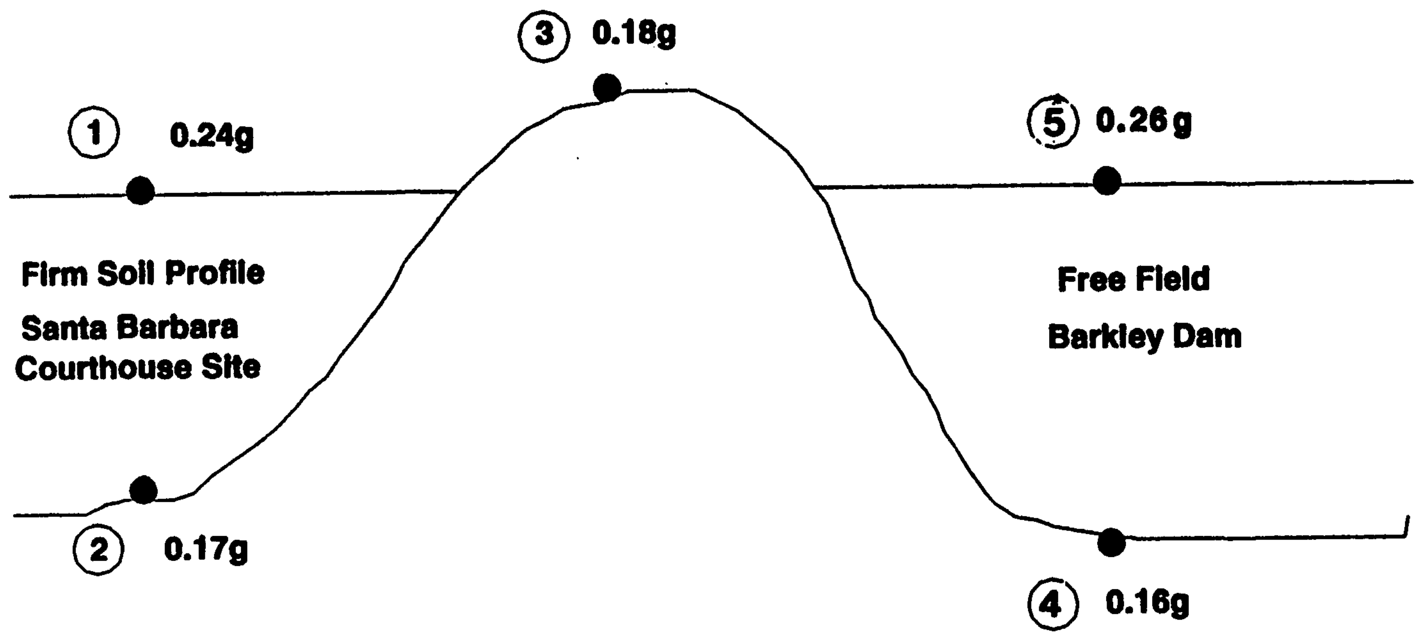


Figure 35. Peak acceleration values at key locations in deconvolution process.

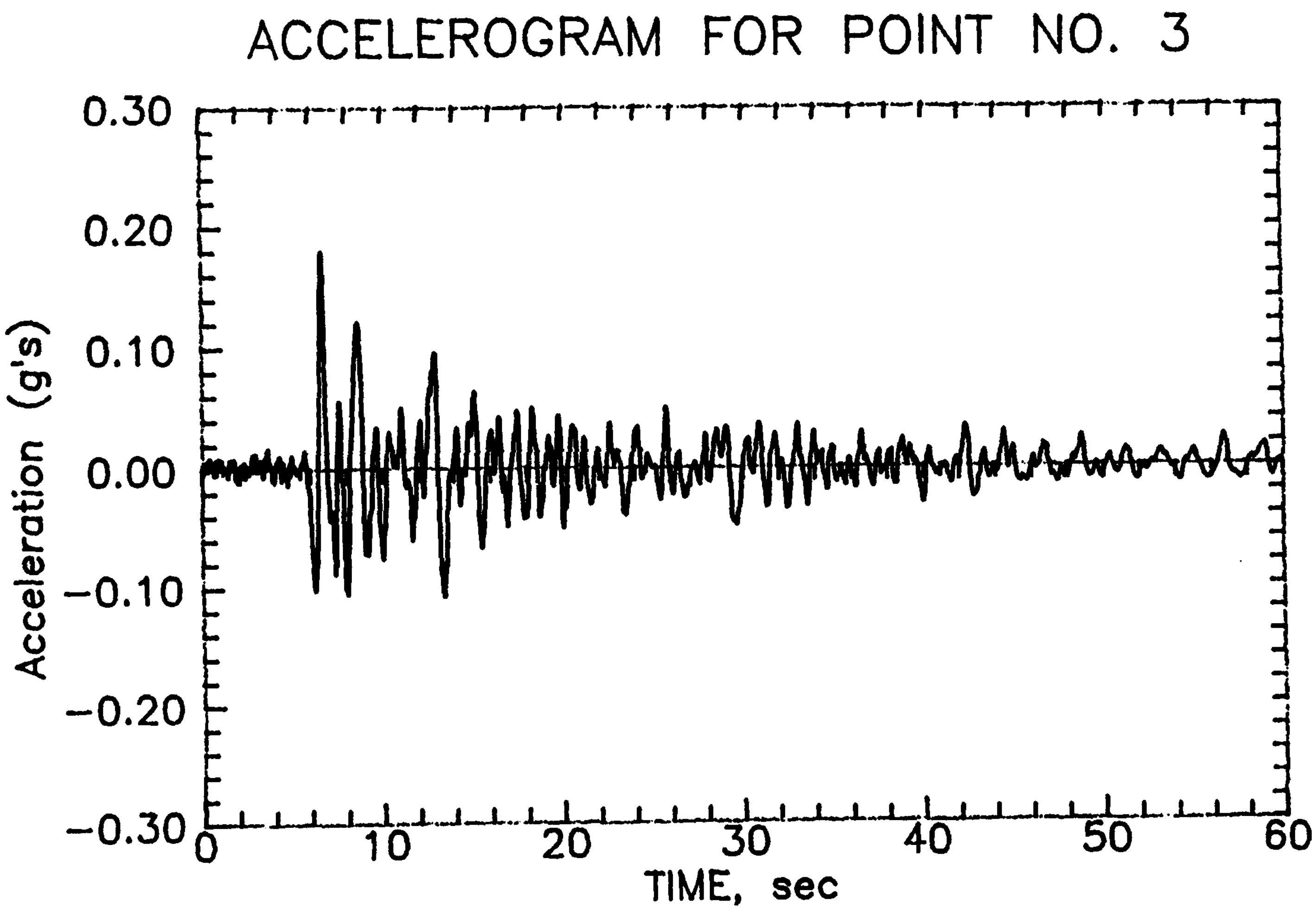


Figure 36. Rock outcrop accelerogram for Barkley Project at Point #3 and used as input to the dynamic response analysis of the main embankment section.

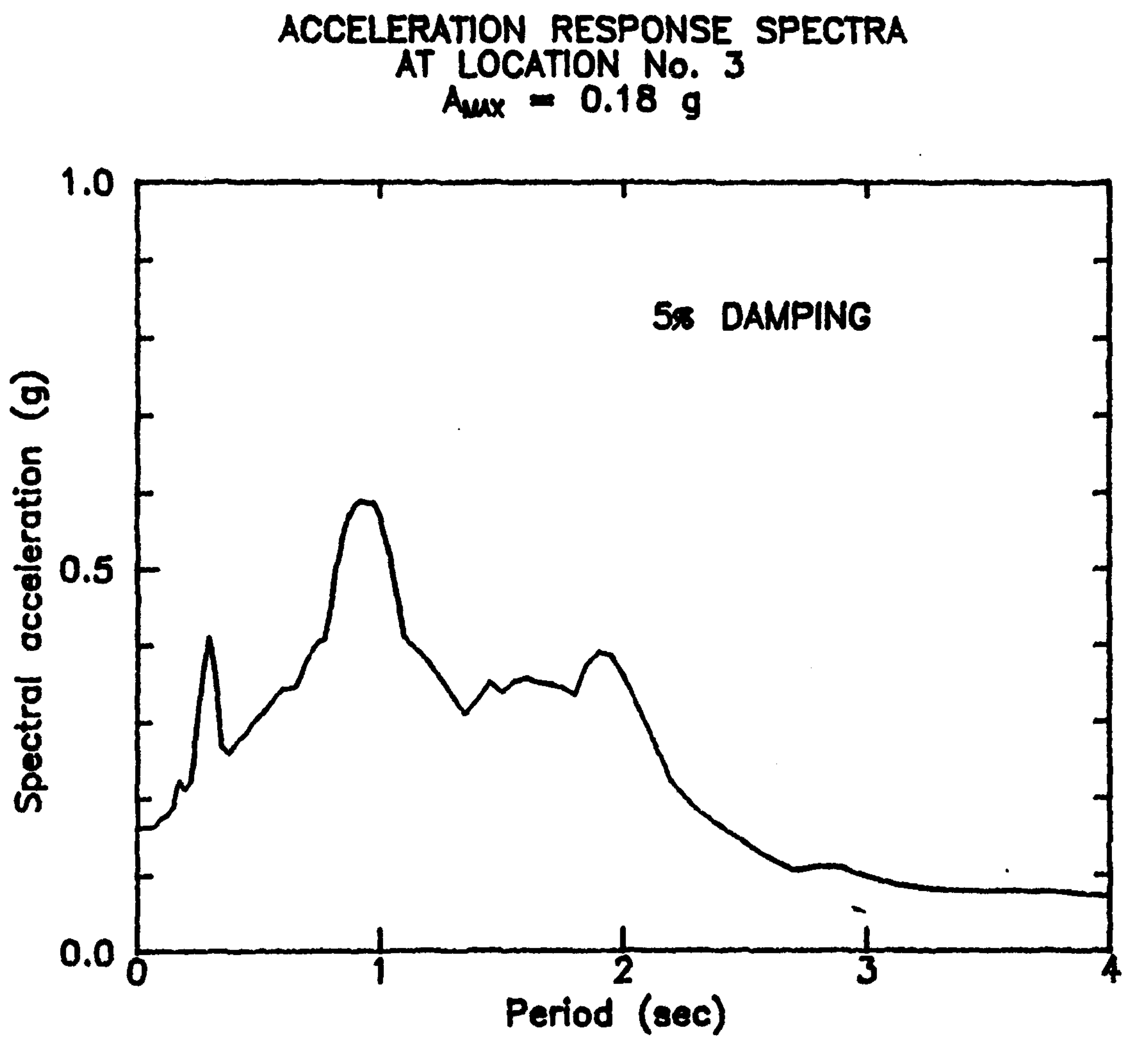


Figure 37. Response spectra for rock outcrop accelerogram at Point #3 at the 5% damping level.

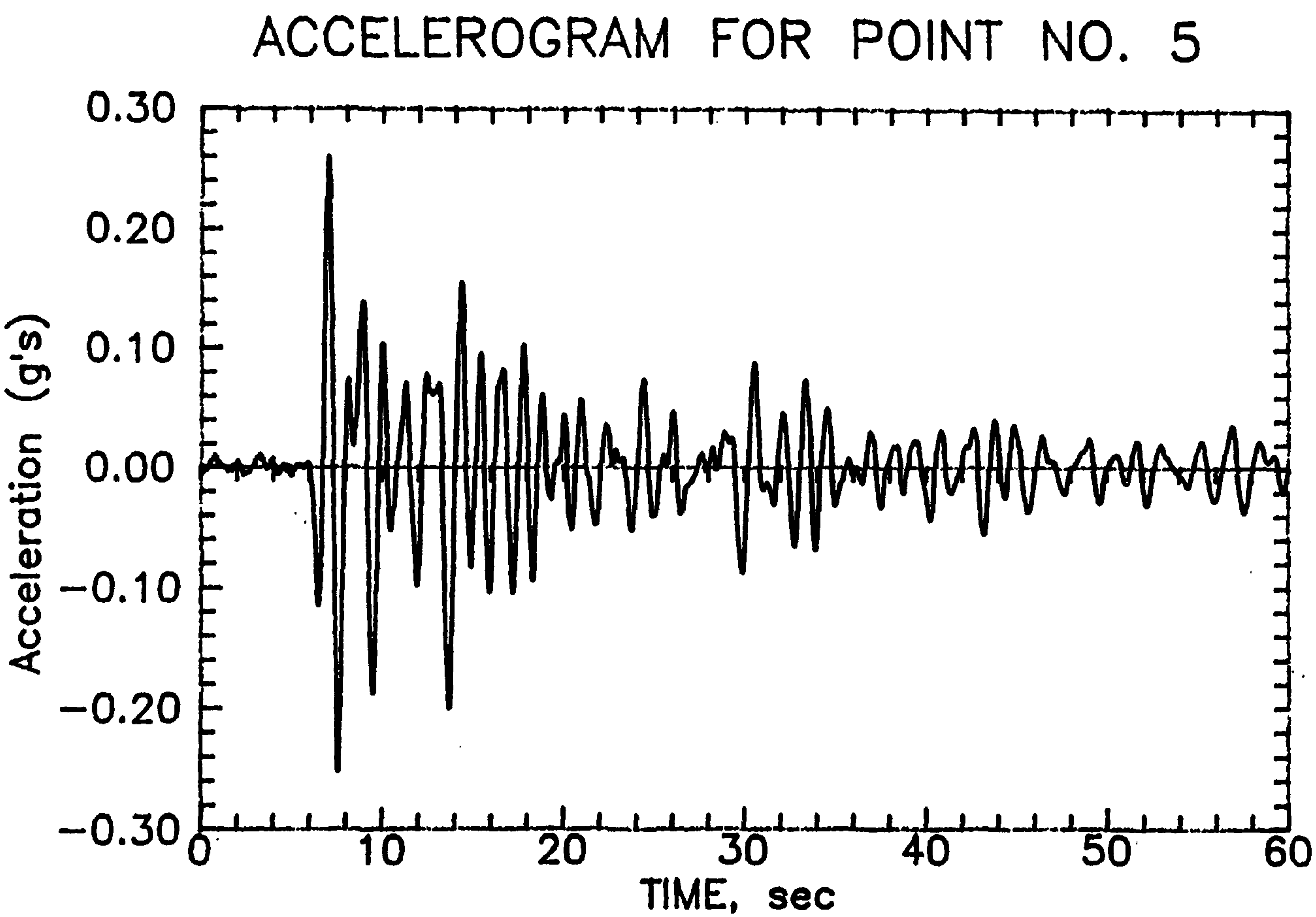


Figure 38. Barkley free field ground surface accelerogram at Point #5 and used as input to dynamic finite element analysis of the switchyard section.

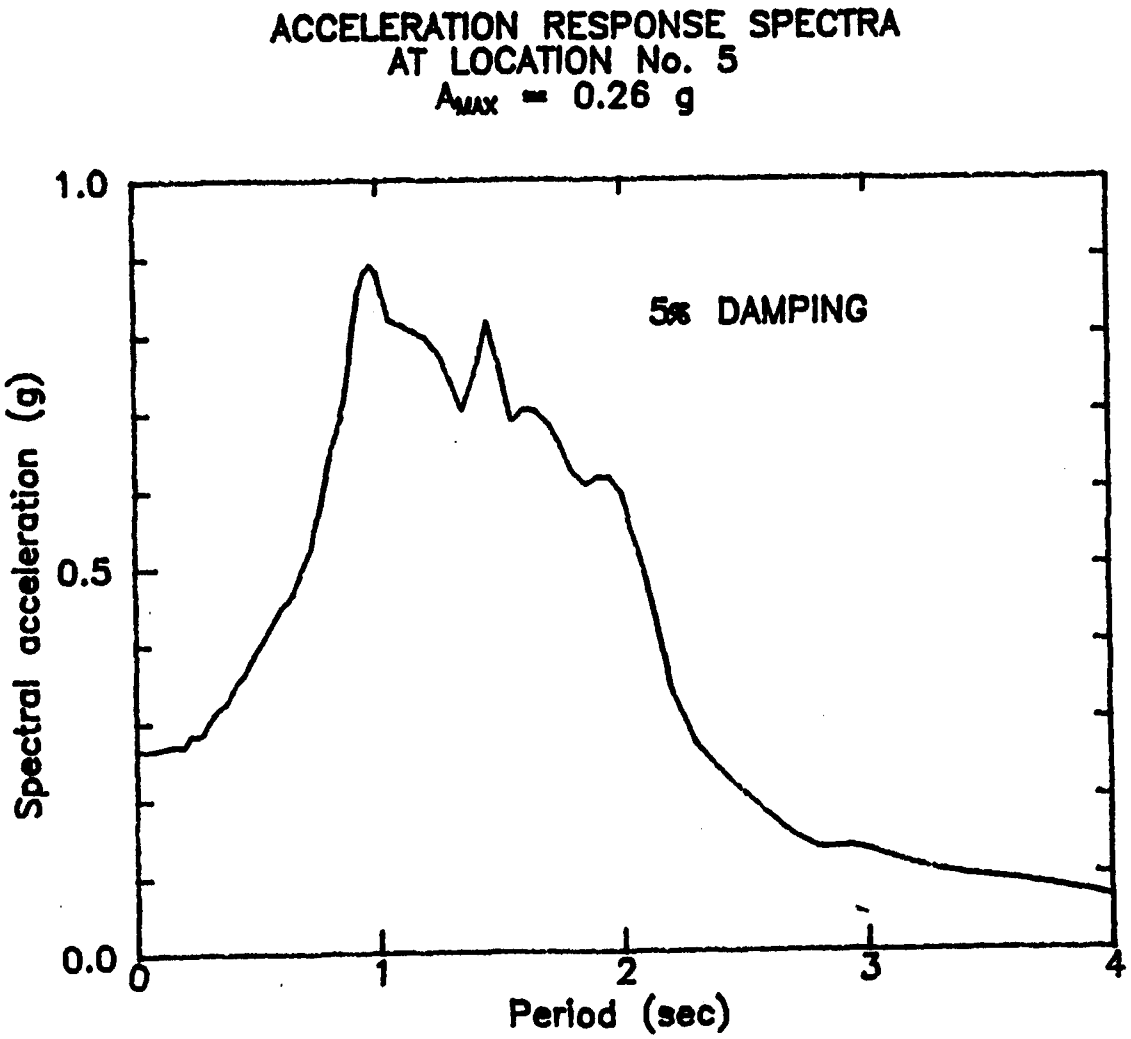


Figure 39. Response spectra of Barkley free field ground surface accelerogram at the 5% damping level.

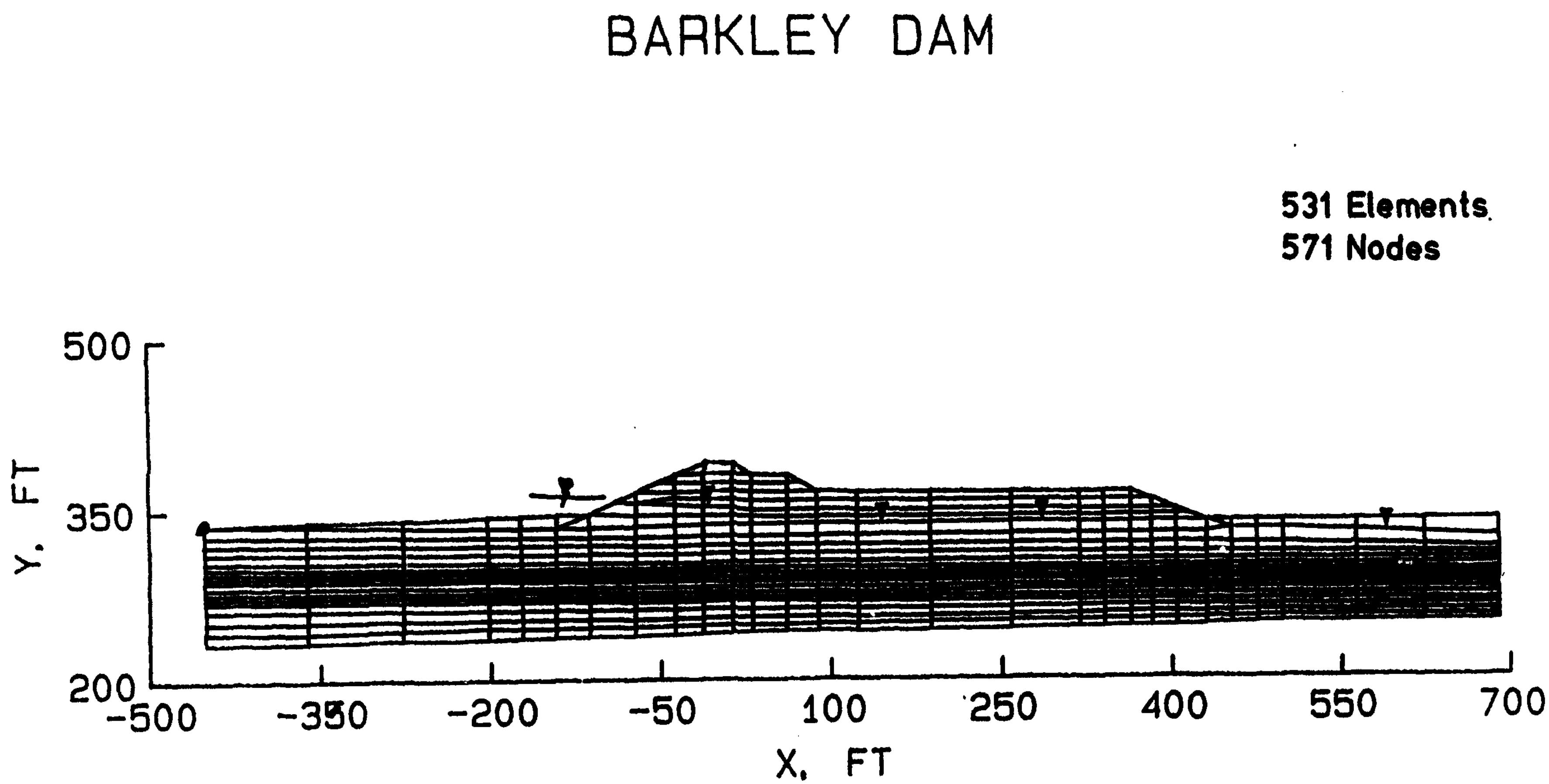


Figure 40. Finite element mesh used in the dynamic analysis of the switchyard section.

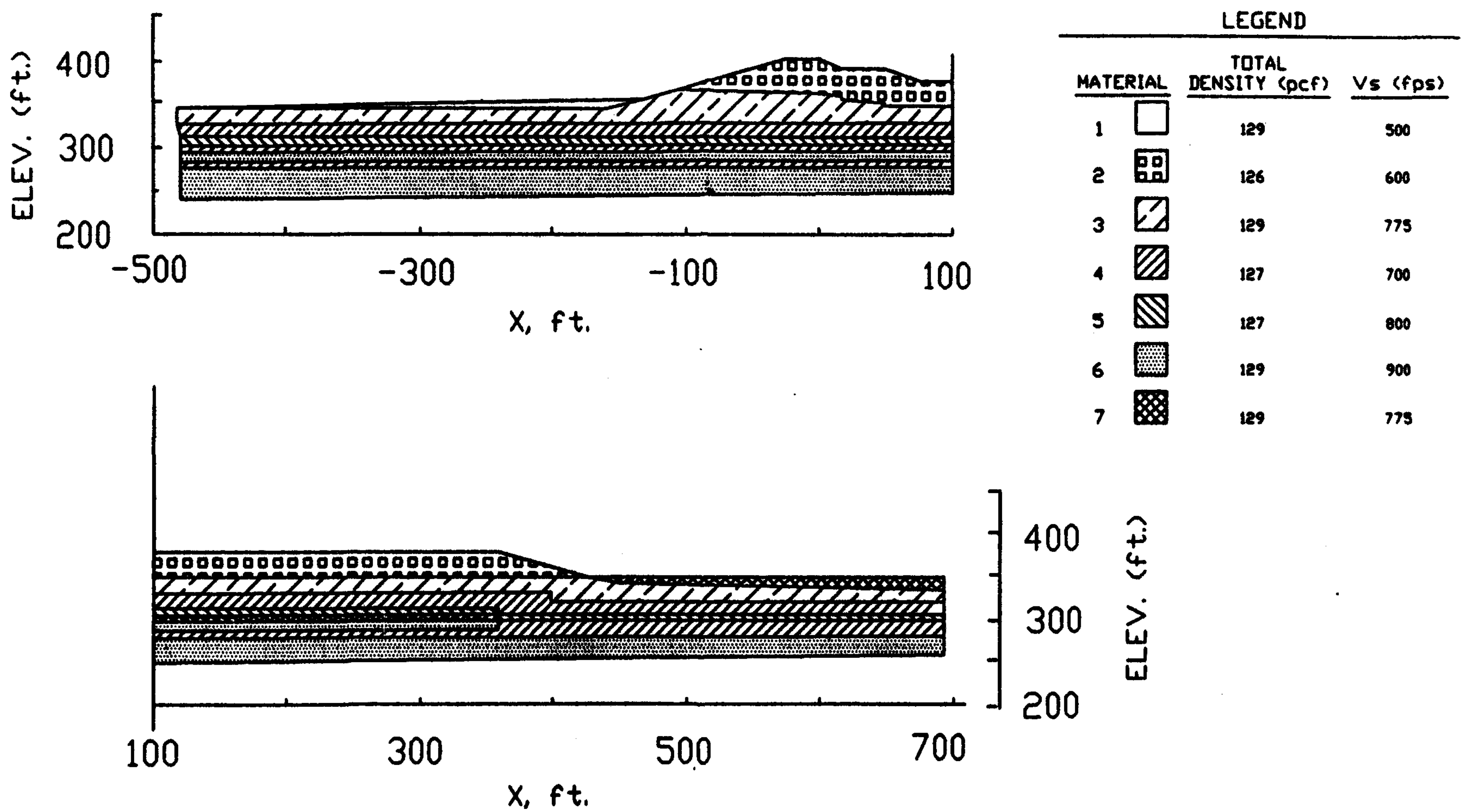


Figure 41. Material properties used in the dynamic finite element analysis of the switchyard section.

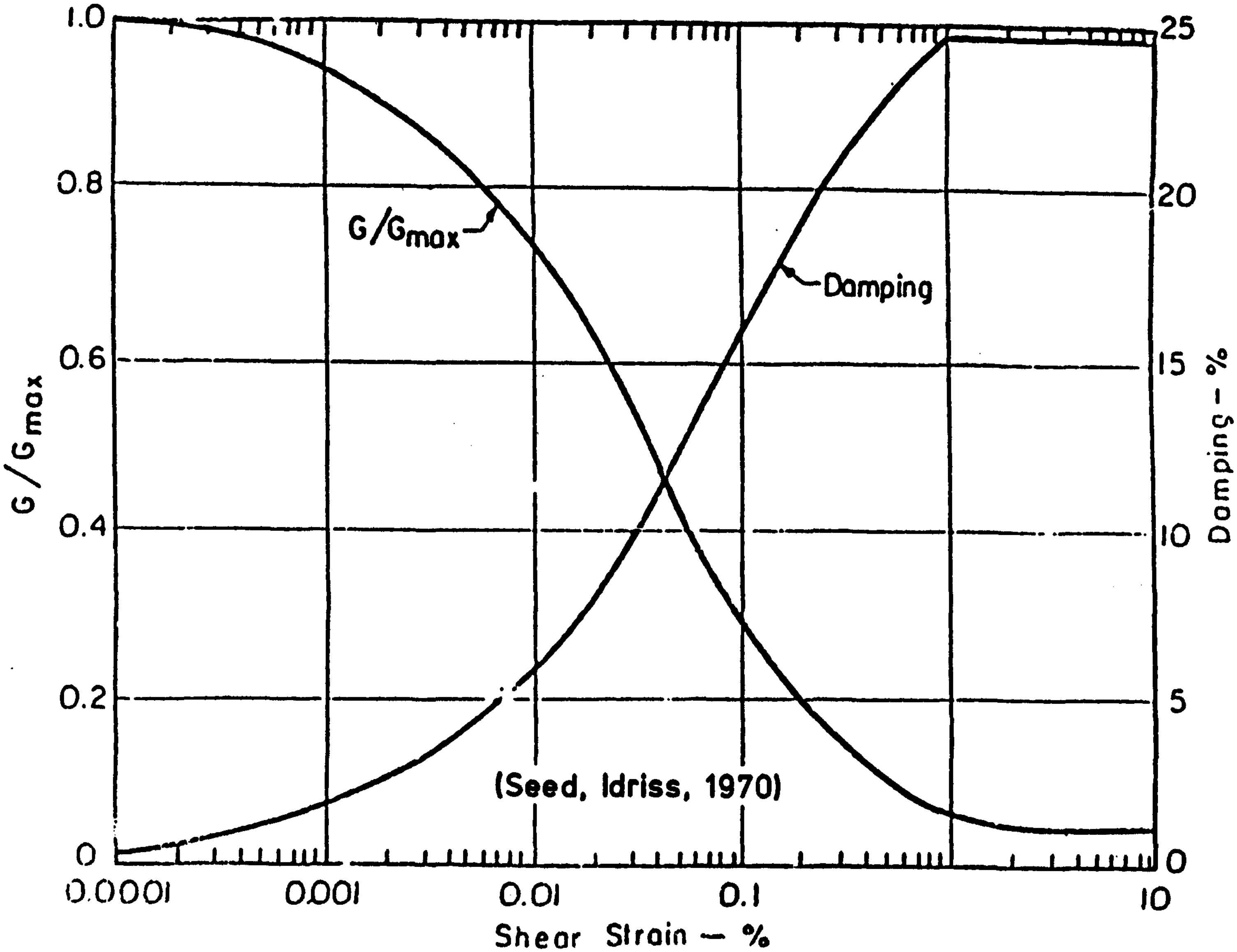


Figure 42. Modulus degradation and damping curves used in one-dimensional and finite element dynamic response analyses.

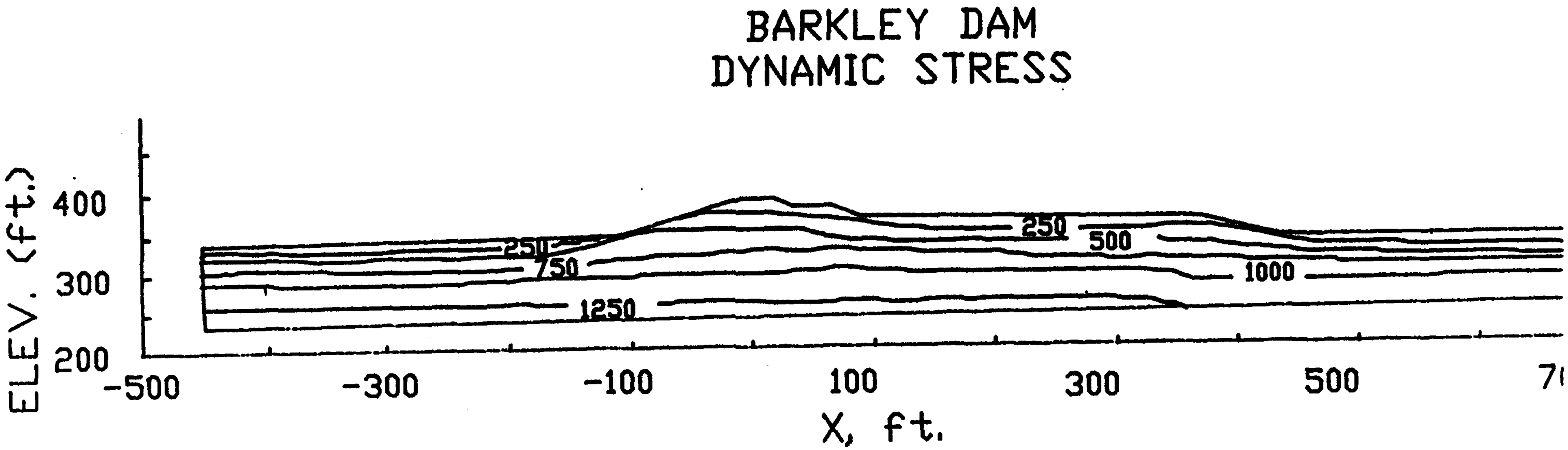


Figure 43. Effective earthquake-induced shear stresses.

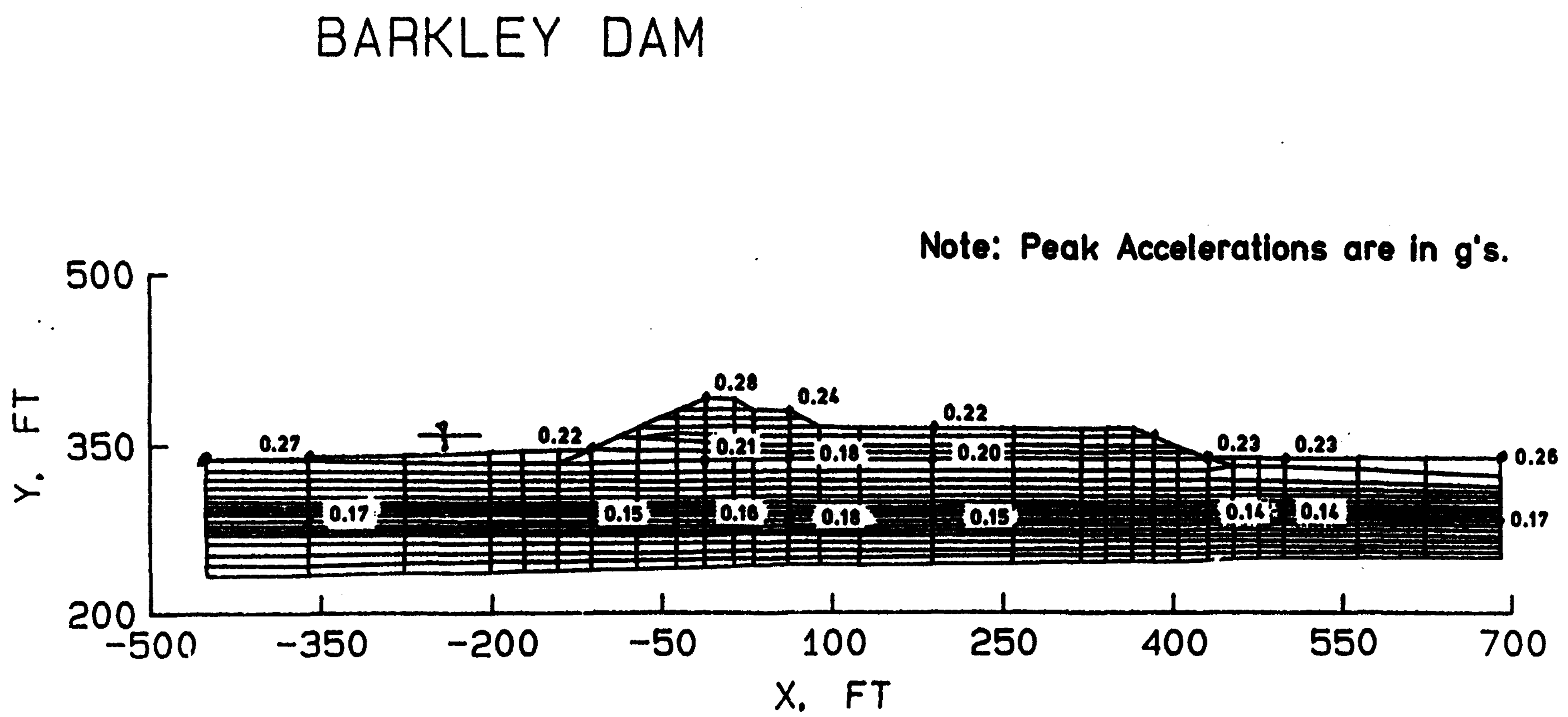


Figure 44. Peak accelerations at selected nodal points.

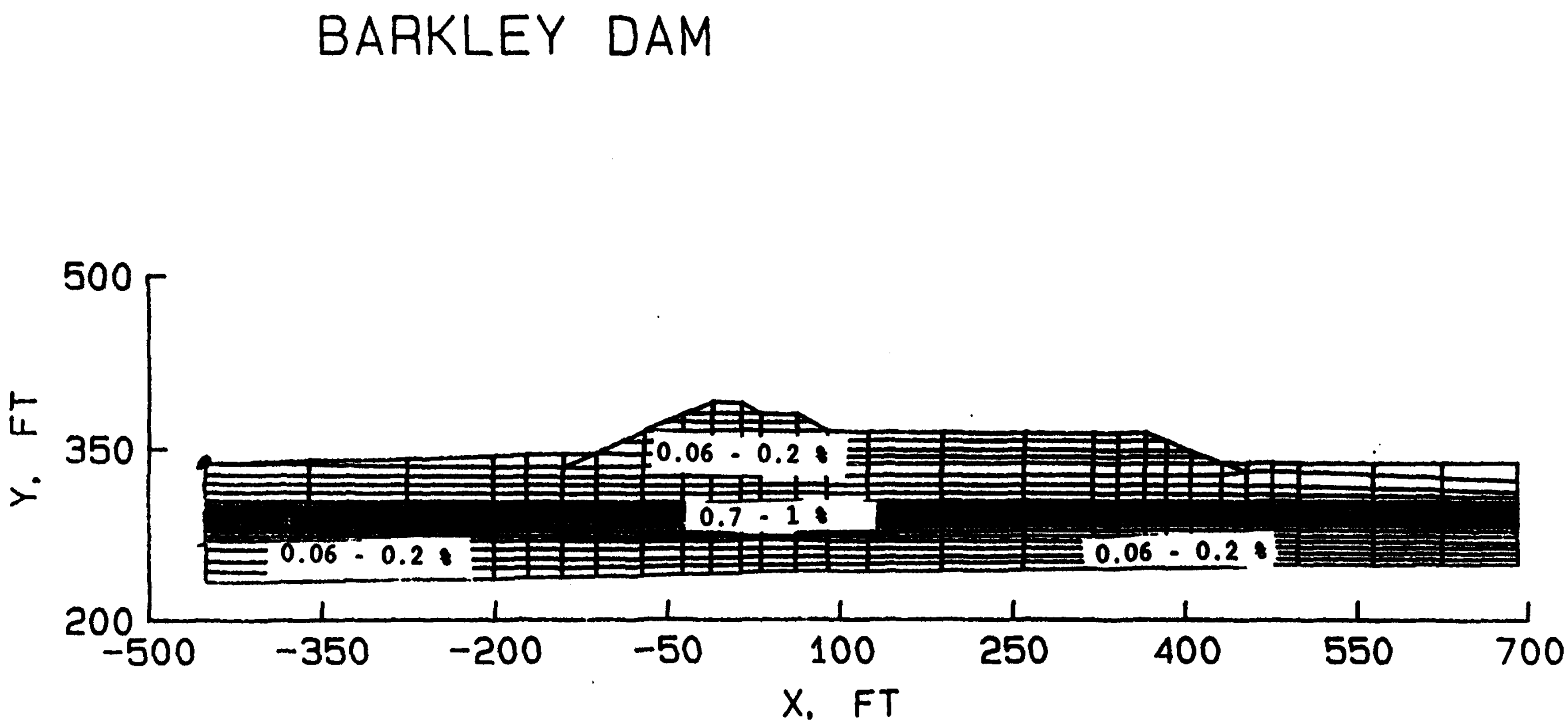


Figure 45. Effective cyclic stains levels determined from the finite element analysis.

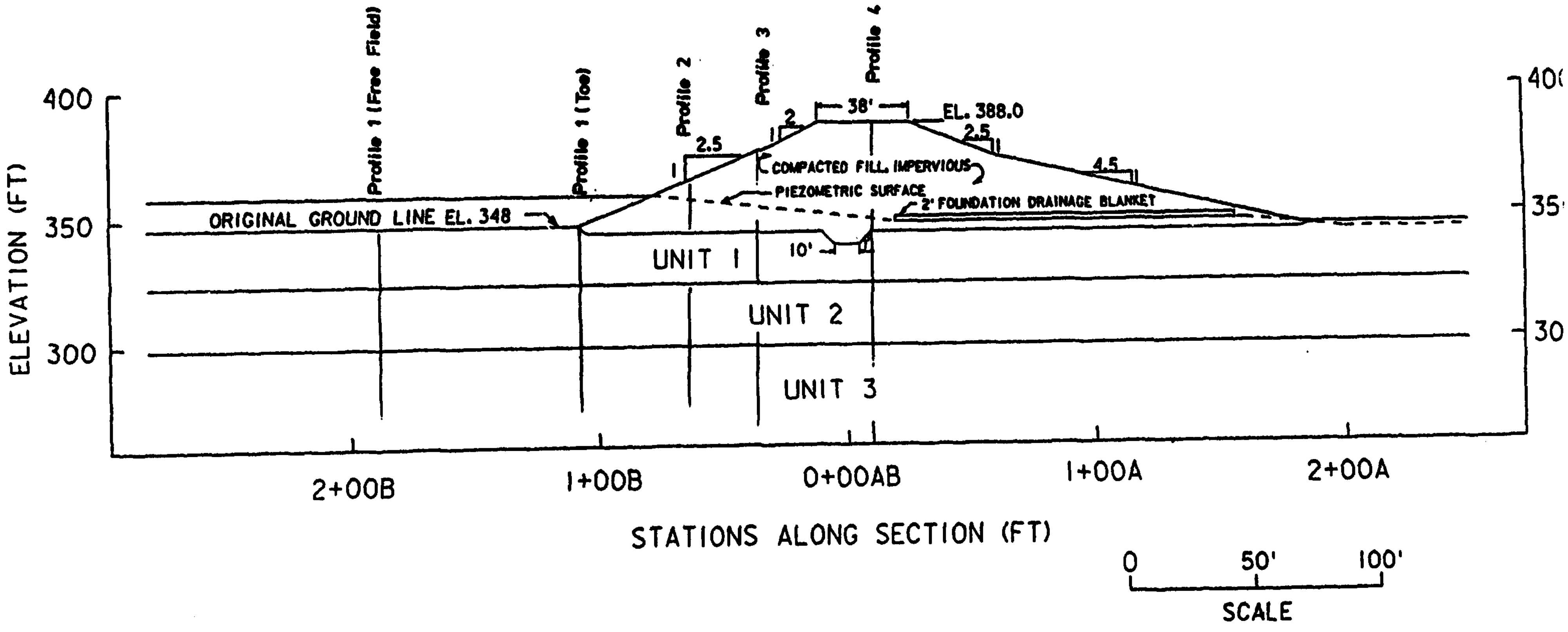


Figure 46. Representative cross-section and location of one-dimensional profiles used to approximate the dynamic response of the main embankment.

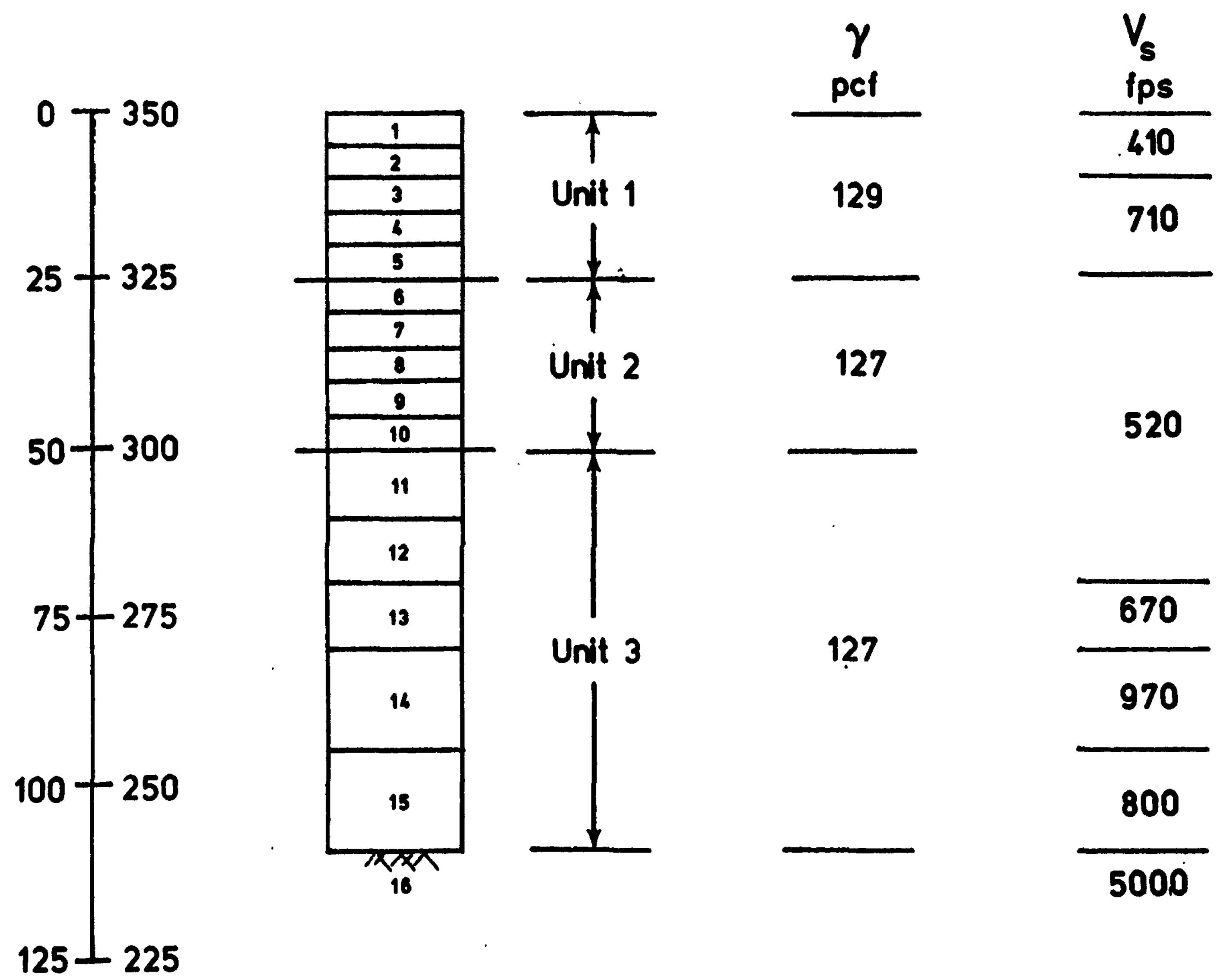


Figure 47. Profile 1 - upstream free field of main embankment.

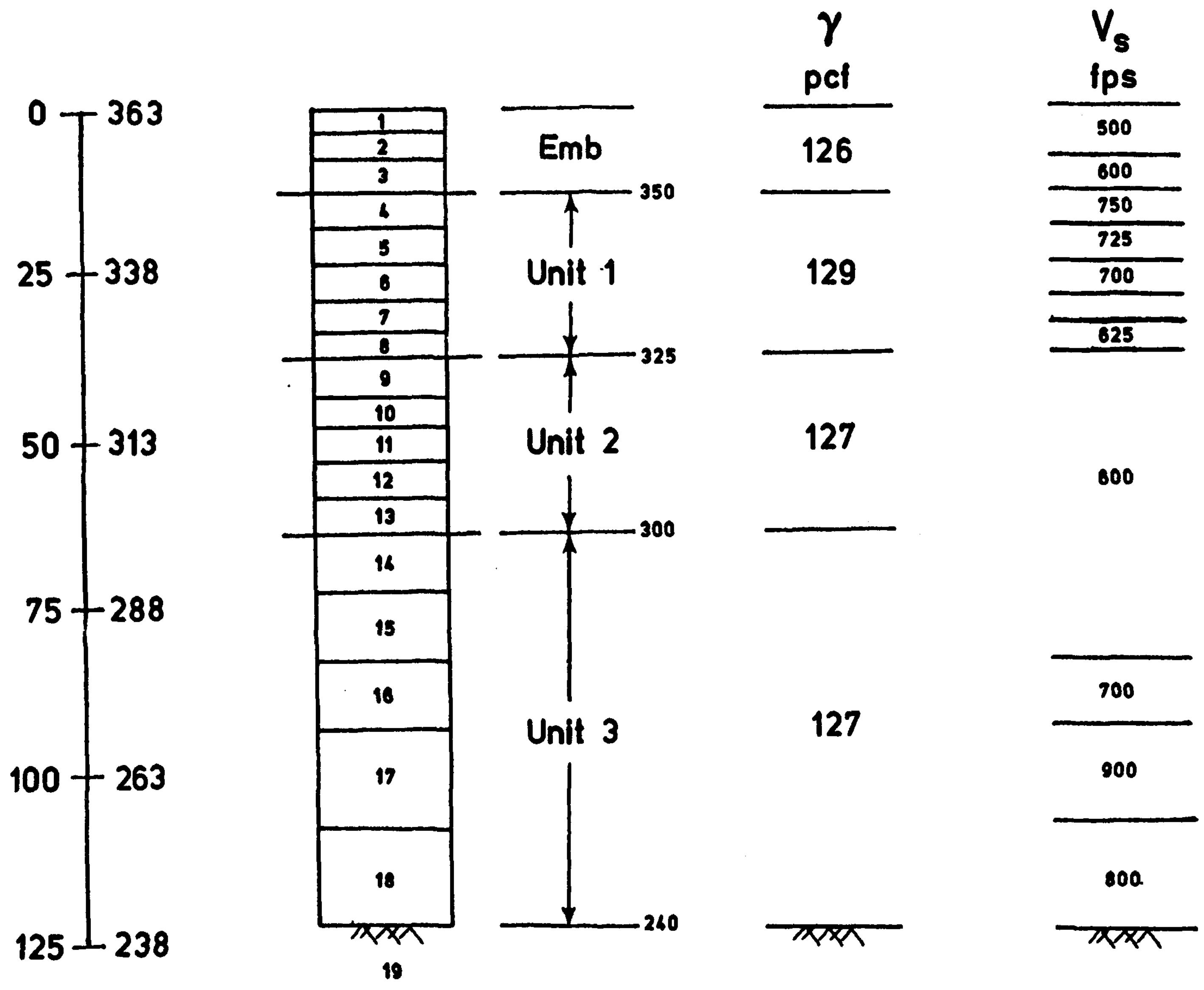


Figure 48. Profile 2 - main embankment profile located at a point one-third of the way up the face of the upstream side of the embankment.

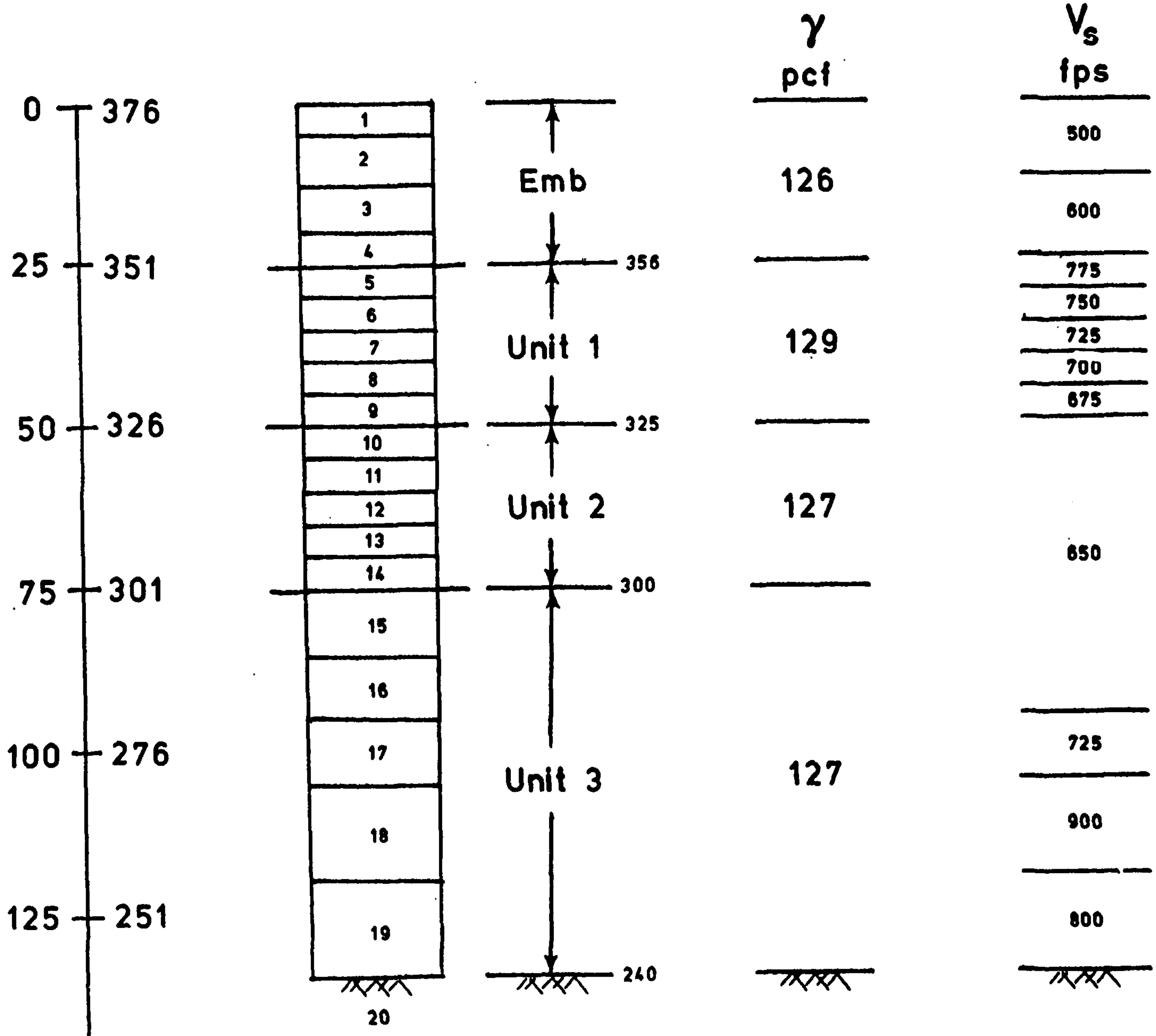


Figure 49. Profile 3 - main embankment profile at a point two-thirds of the way up the face of the embankment.

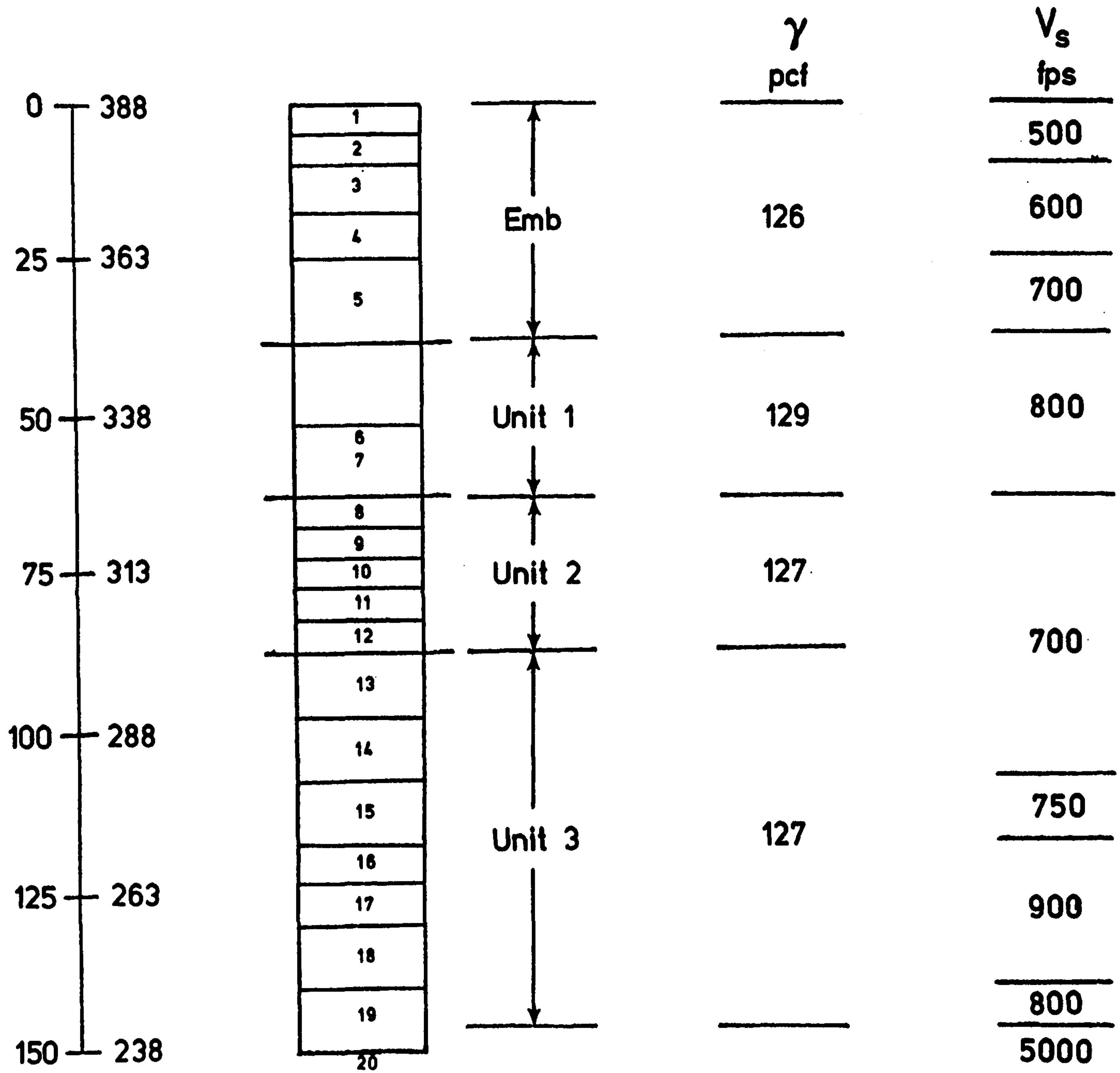


Figure 50. Profile 4 - main embankment profile at a point on the centerline of the embankment.

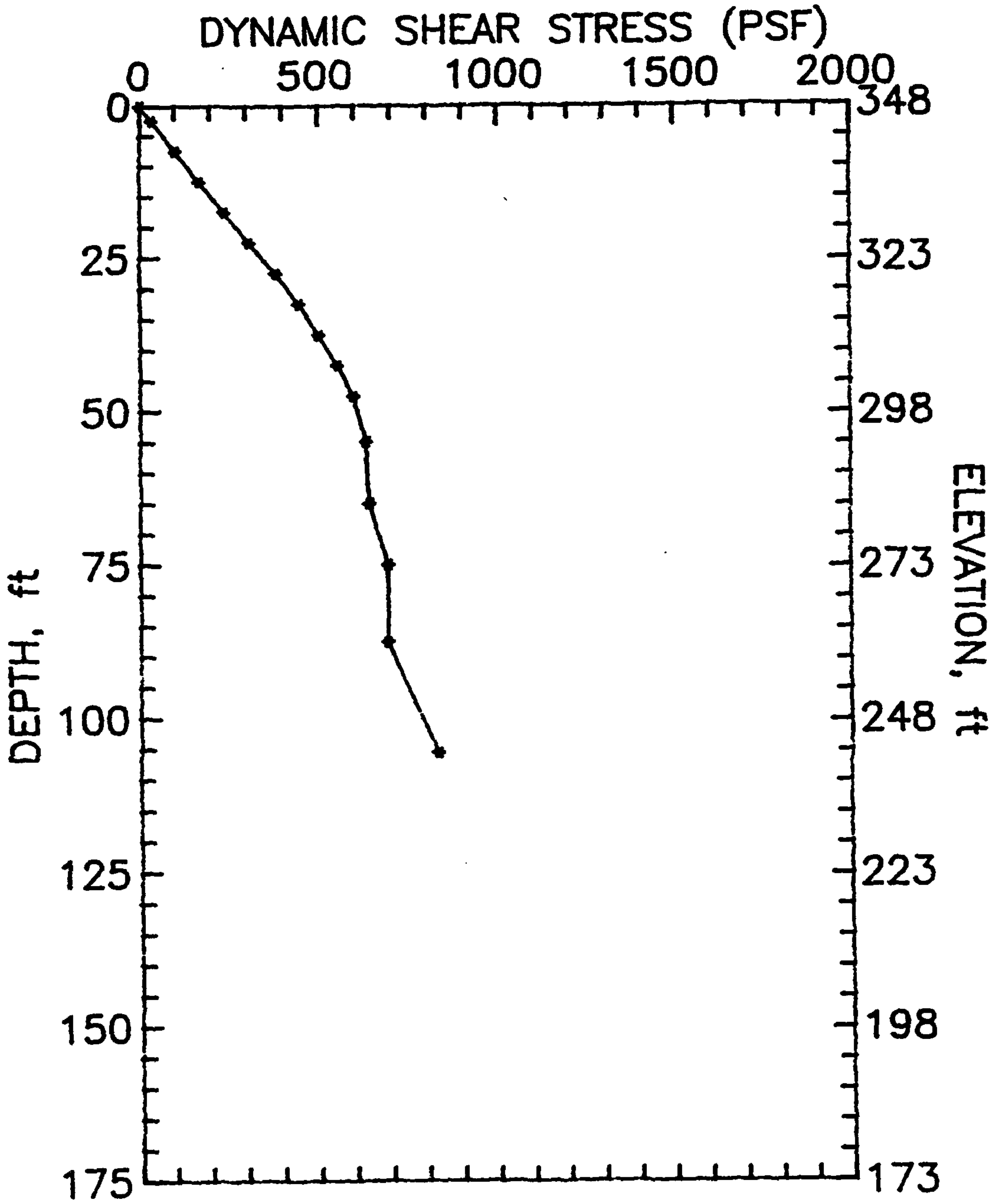


Figure 51. Dynamic shear stress versus depth in Profile 1.

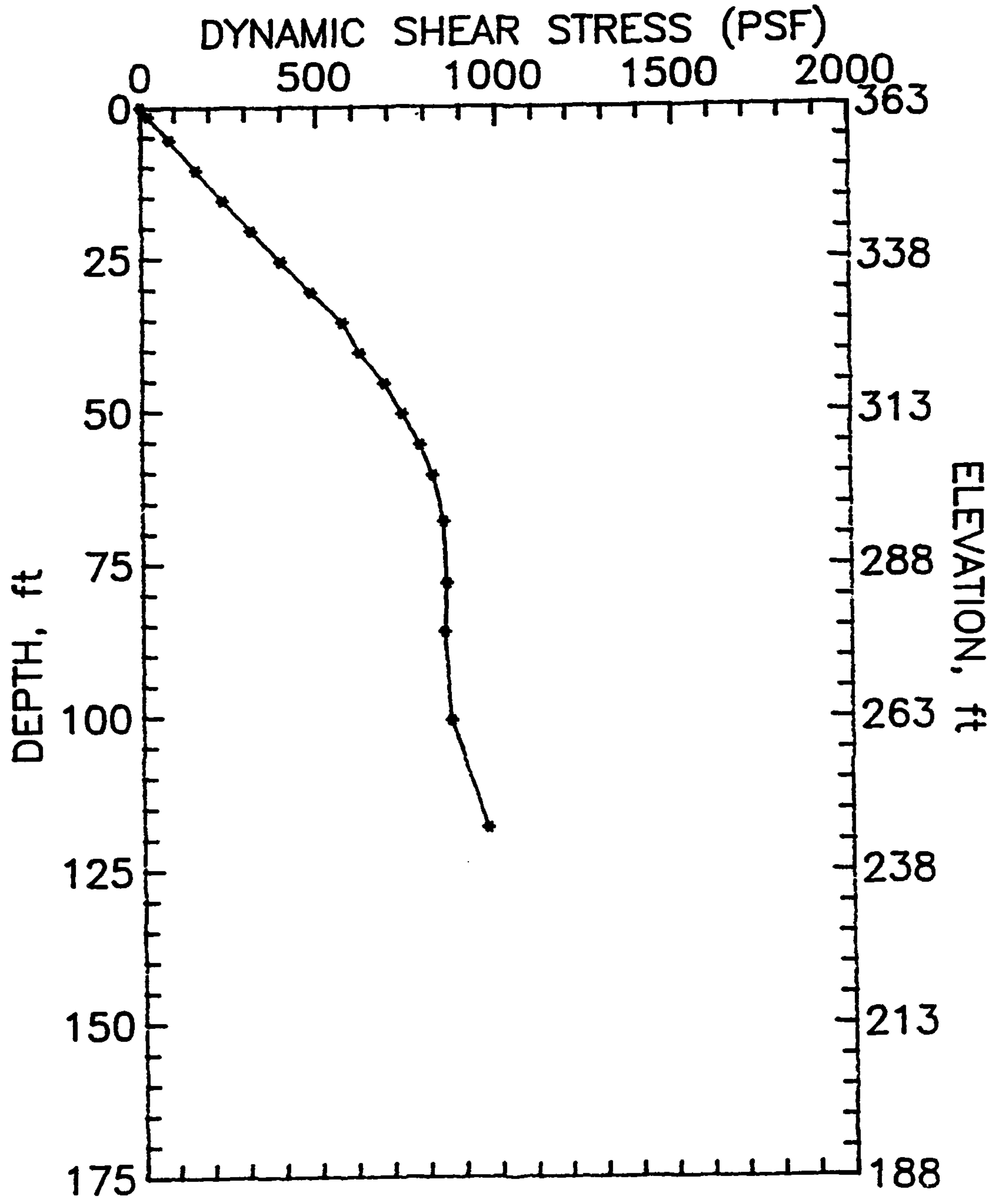


Figure 52. Dynamic shear stress versus depth in Profile 2.

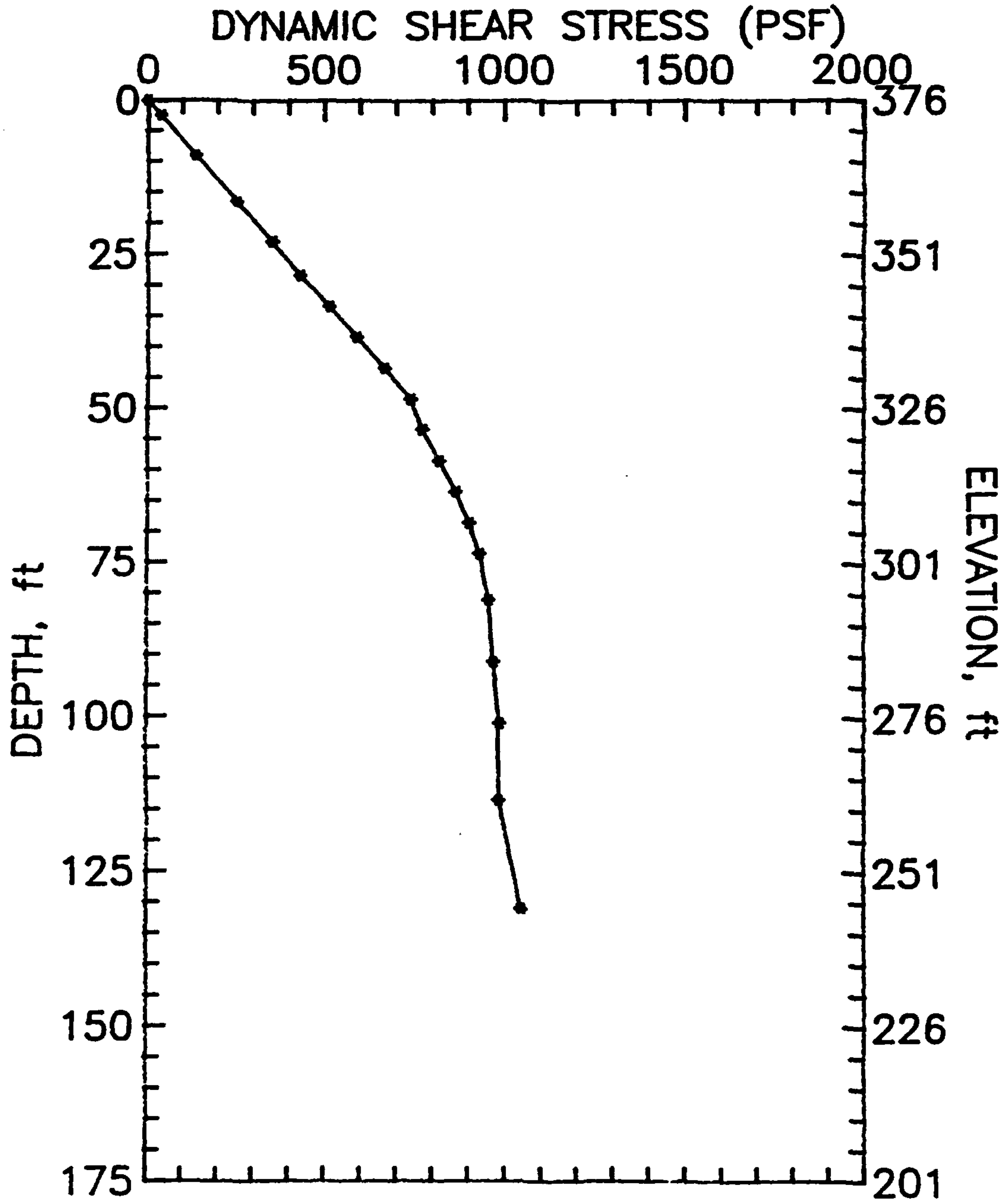


Figure 53. Dynamic shear stress versus depth in Profile 3.

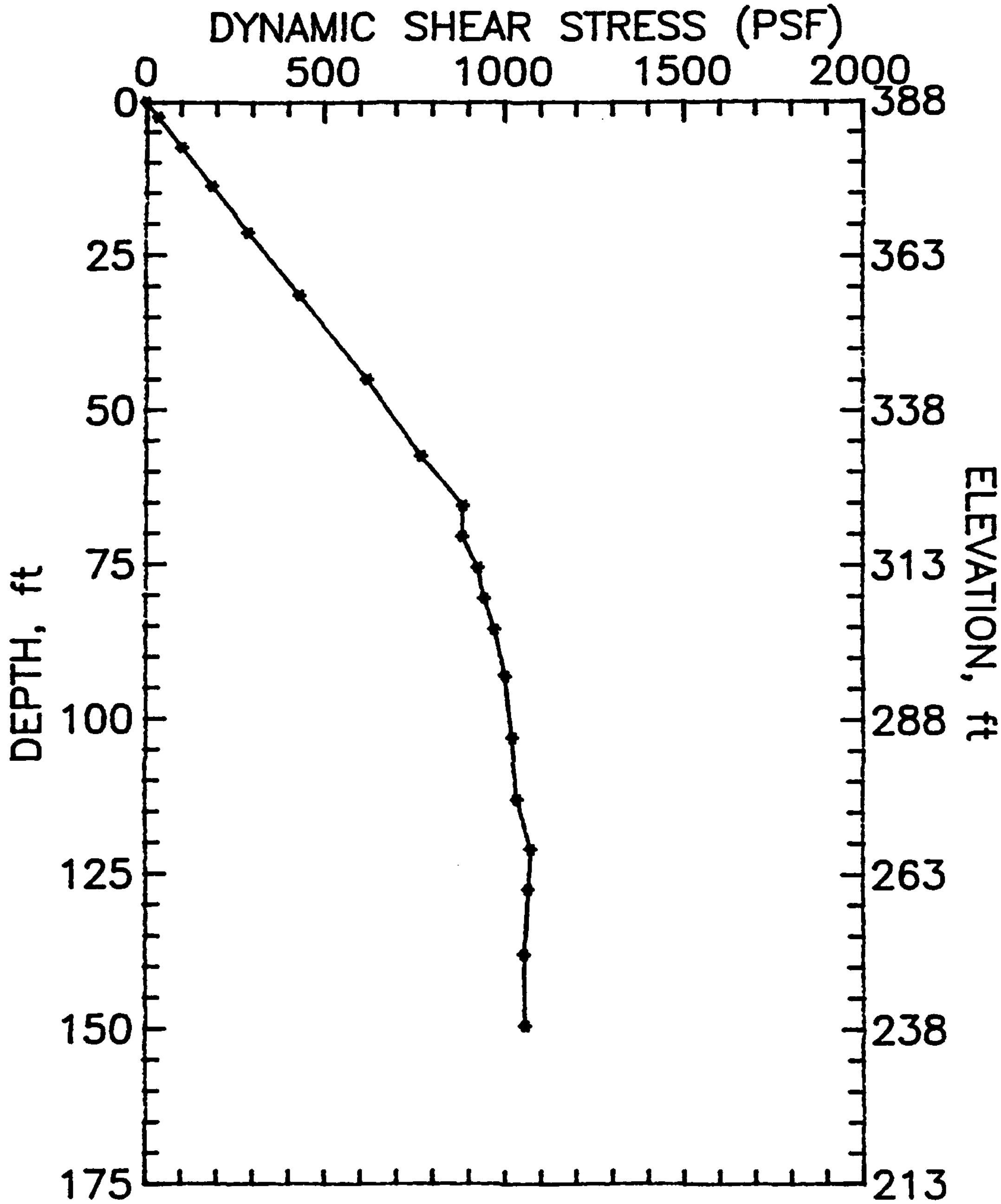


Figure 54. Dynamic shear stress versus depth in Profile 4.

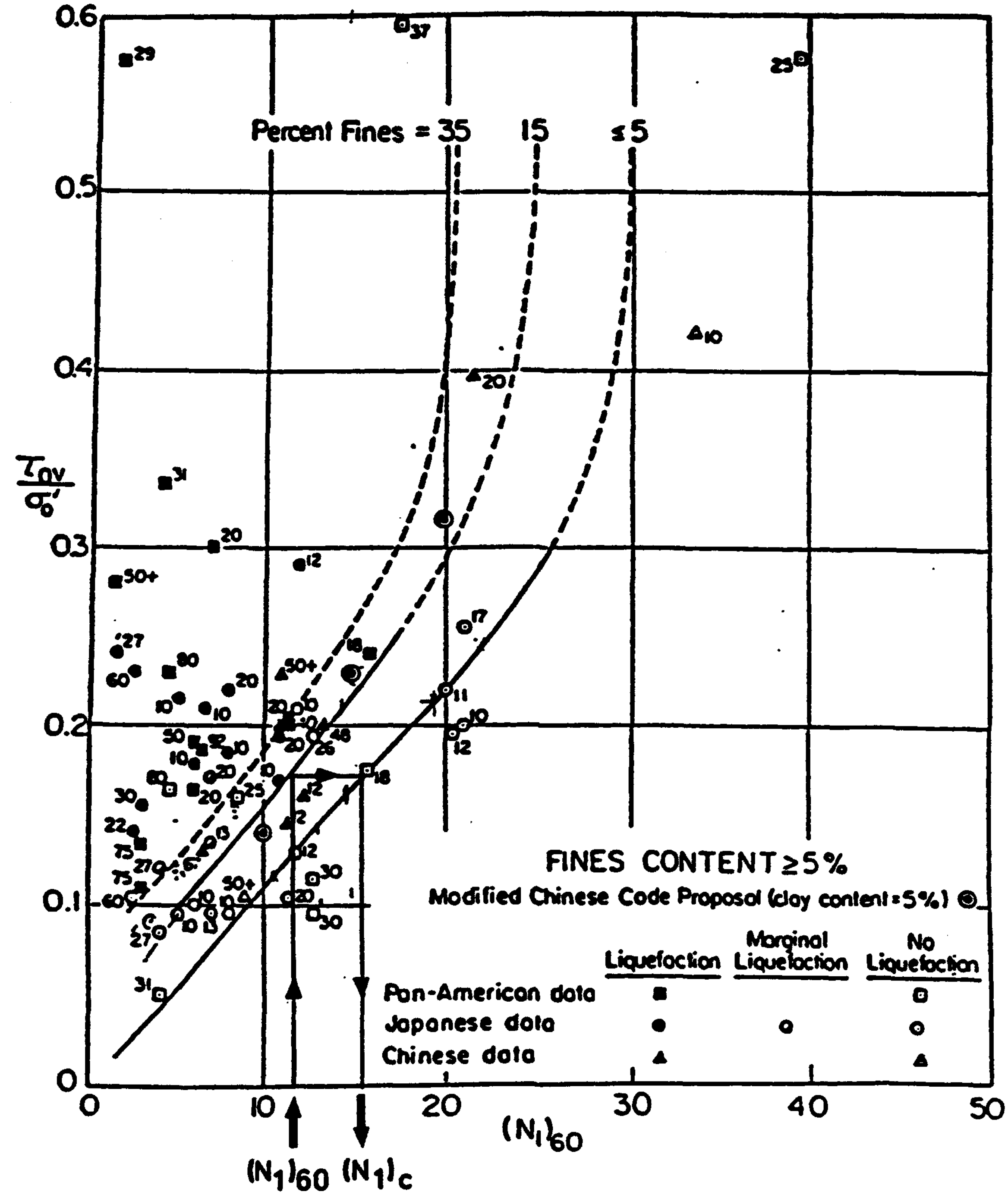


Figure 55. Empirical chart for liquefaction resistance of sands and silty sands for local magnitude 7.5 earthquakes (from Seed, Tokimatsu, Harder, and Chung, 1984).

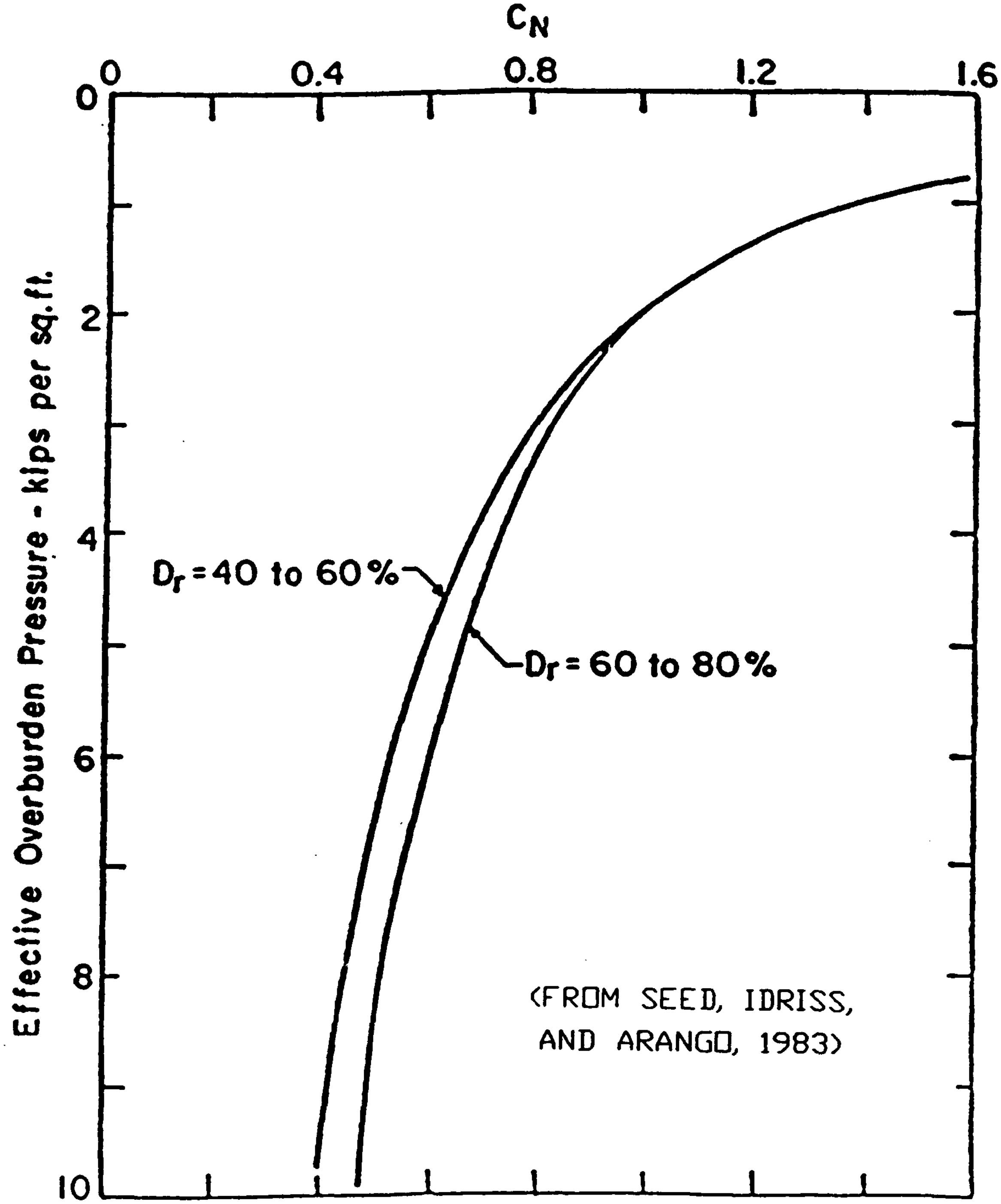


Figure 56. Overburden correction factor, C_N, to correct measured blowcounts to overburden stress conditions of 1 TSF.

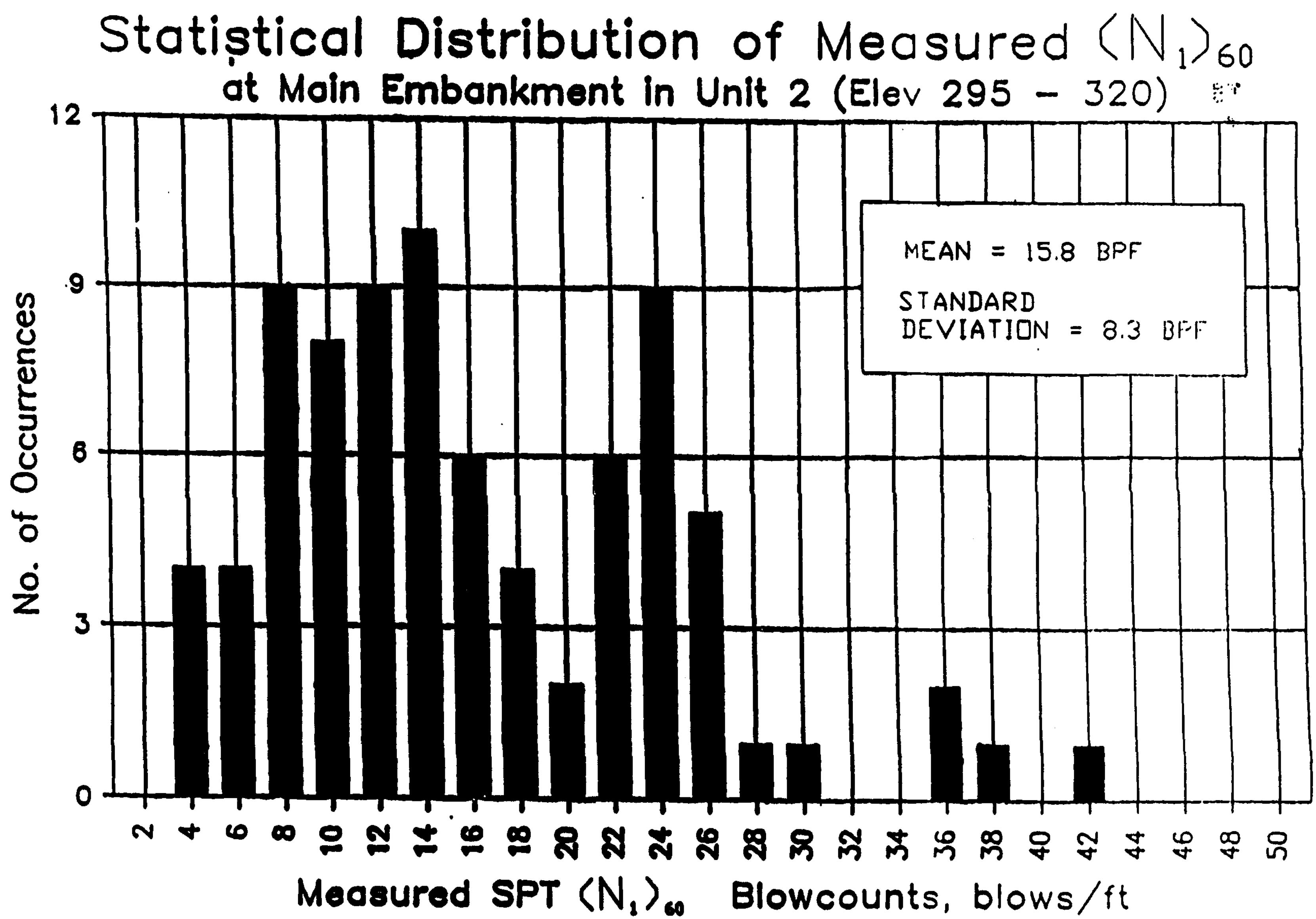


Figure 57. Statistical distribution of the overburden blowcounts, $(N_1)_{60}$, of the main embankment area for Unit 2 between elevations 295 and 320.

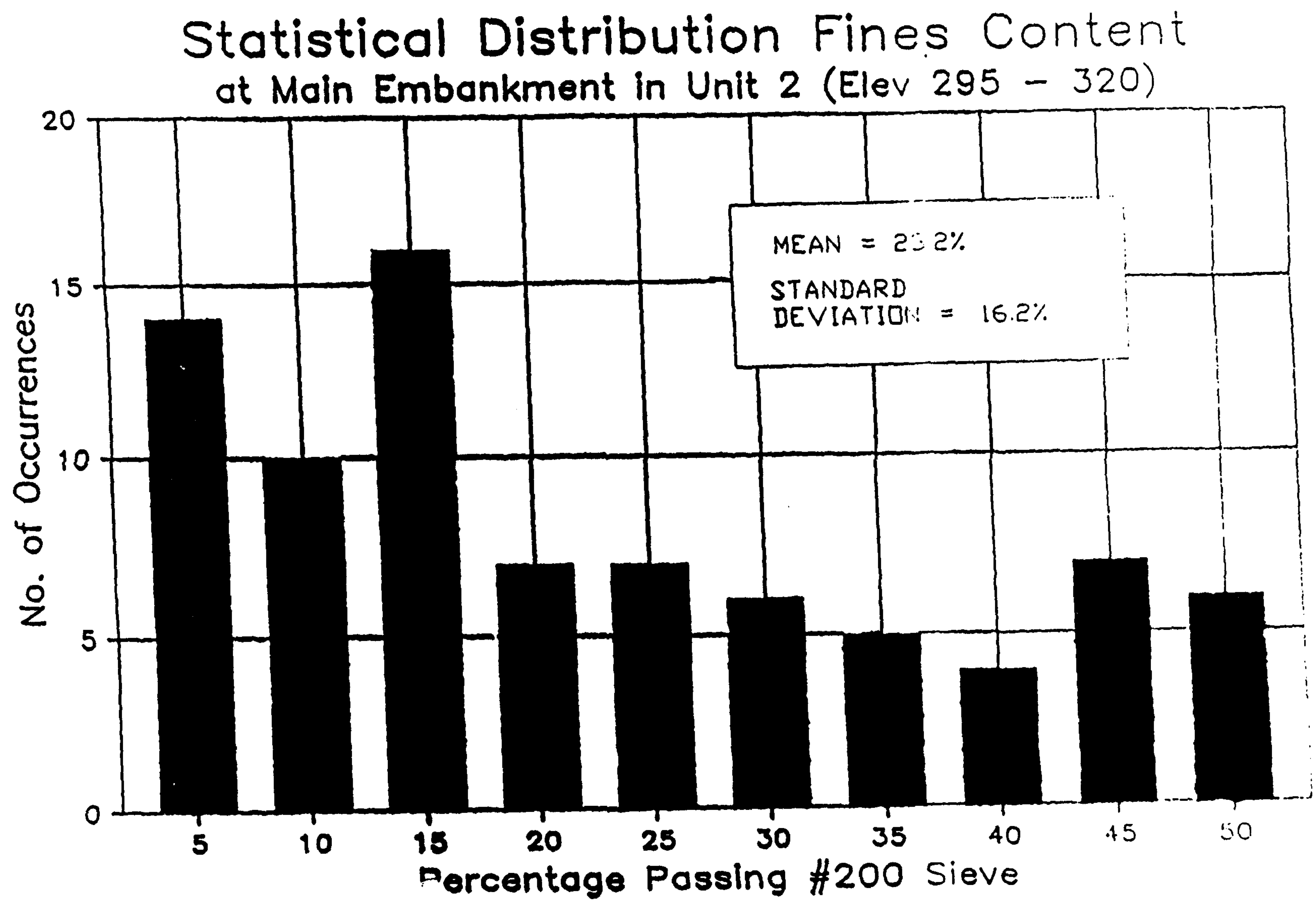


Figure 58. Statistical distribution of the fines content of the main embankment area of Unit 2 between elevation 295 and 320.

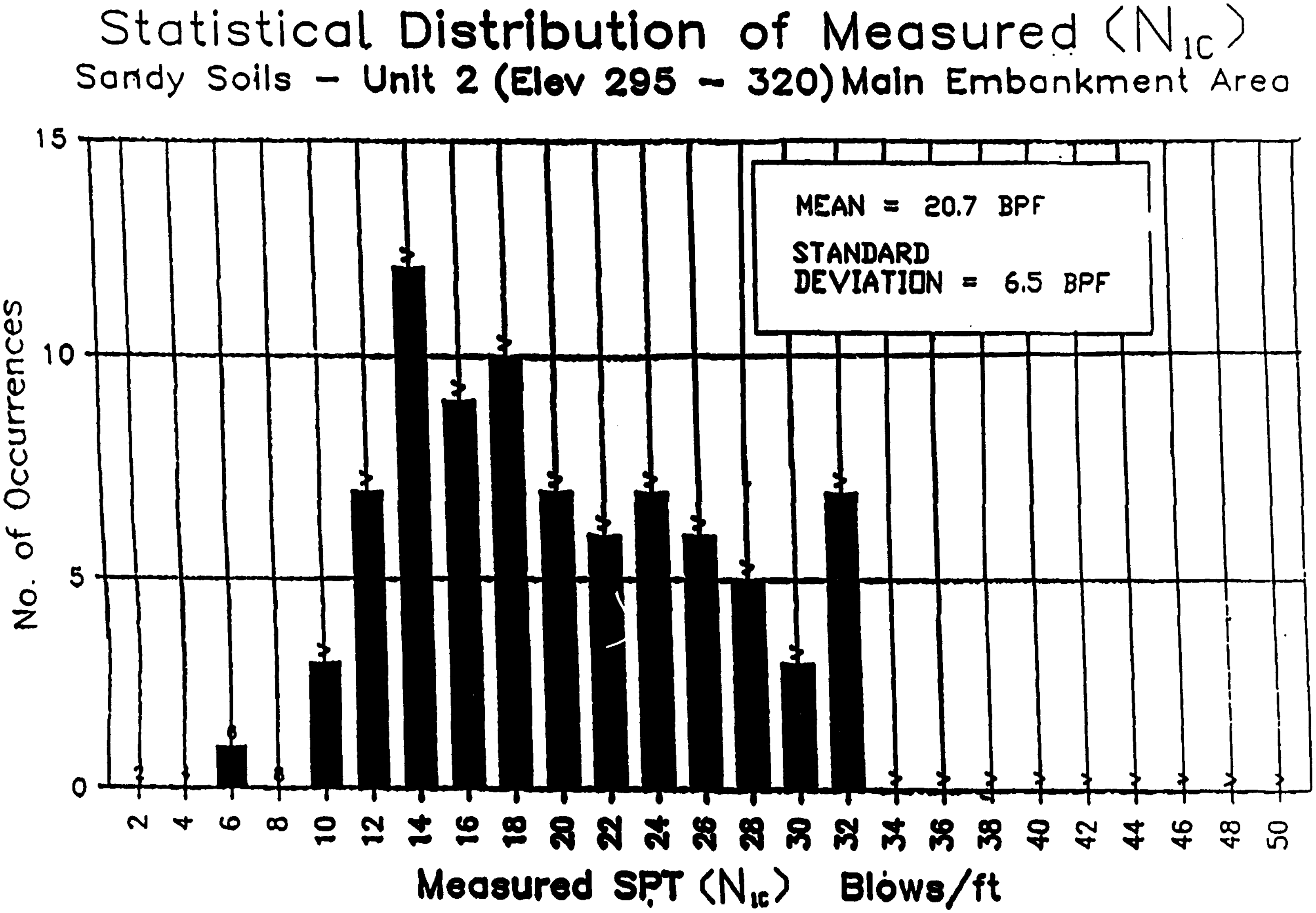


Figure 59. Statistical distribution of the equivalent clean sand blowcounts, N_{1c}, of the main embankment area of Unit 2 between elevations 295 and 320.

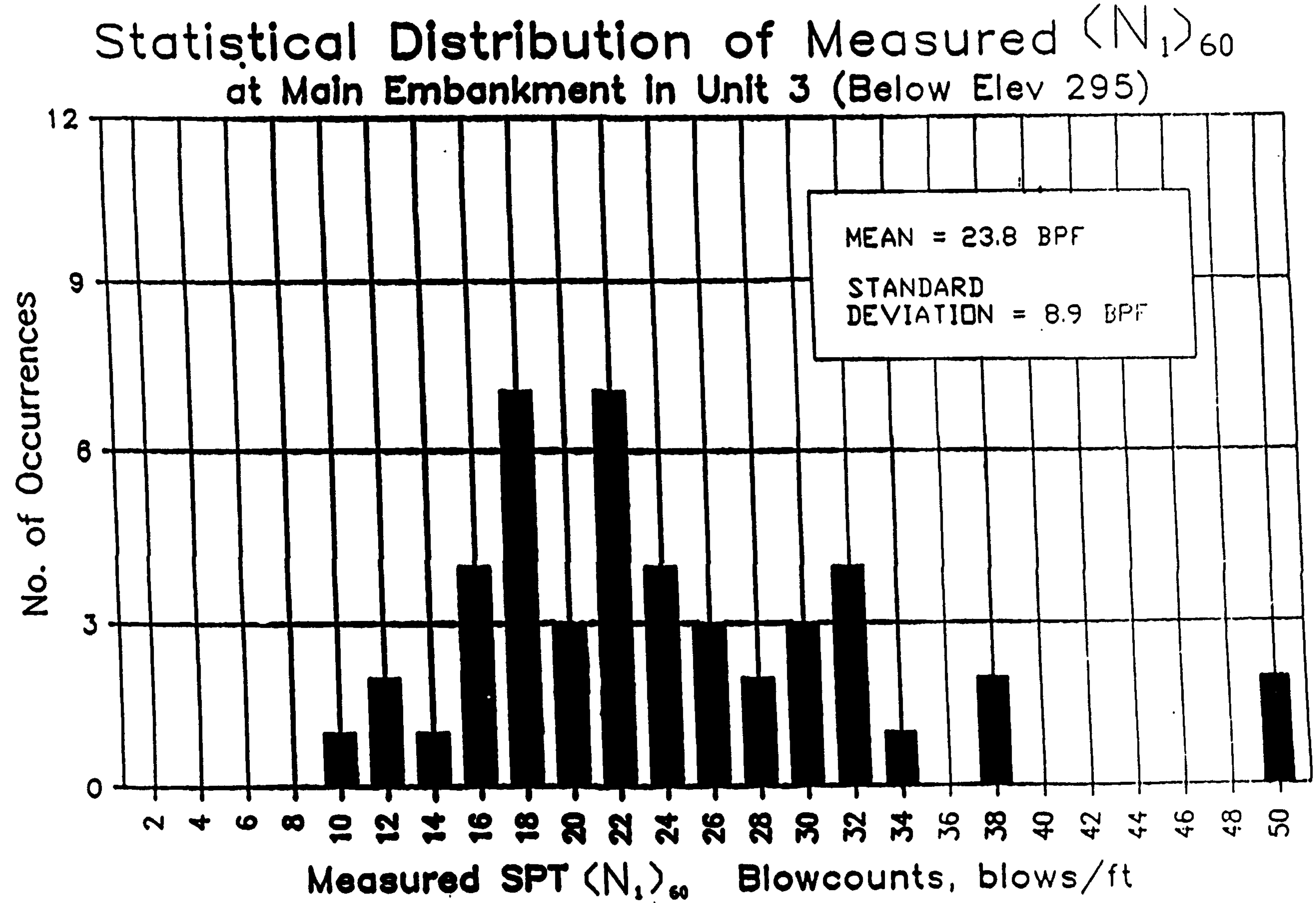


Figure 60. Statistical distribution of the overburden blowcounts, $(N_1)_{60}$, of the main embankment area for Unit 3 below elevation 295.

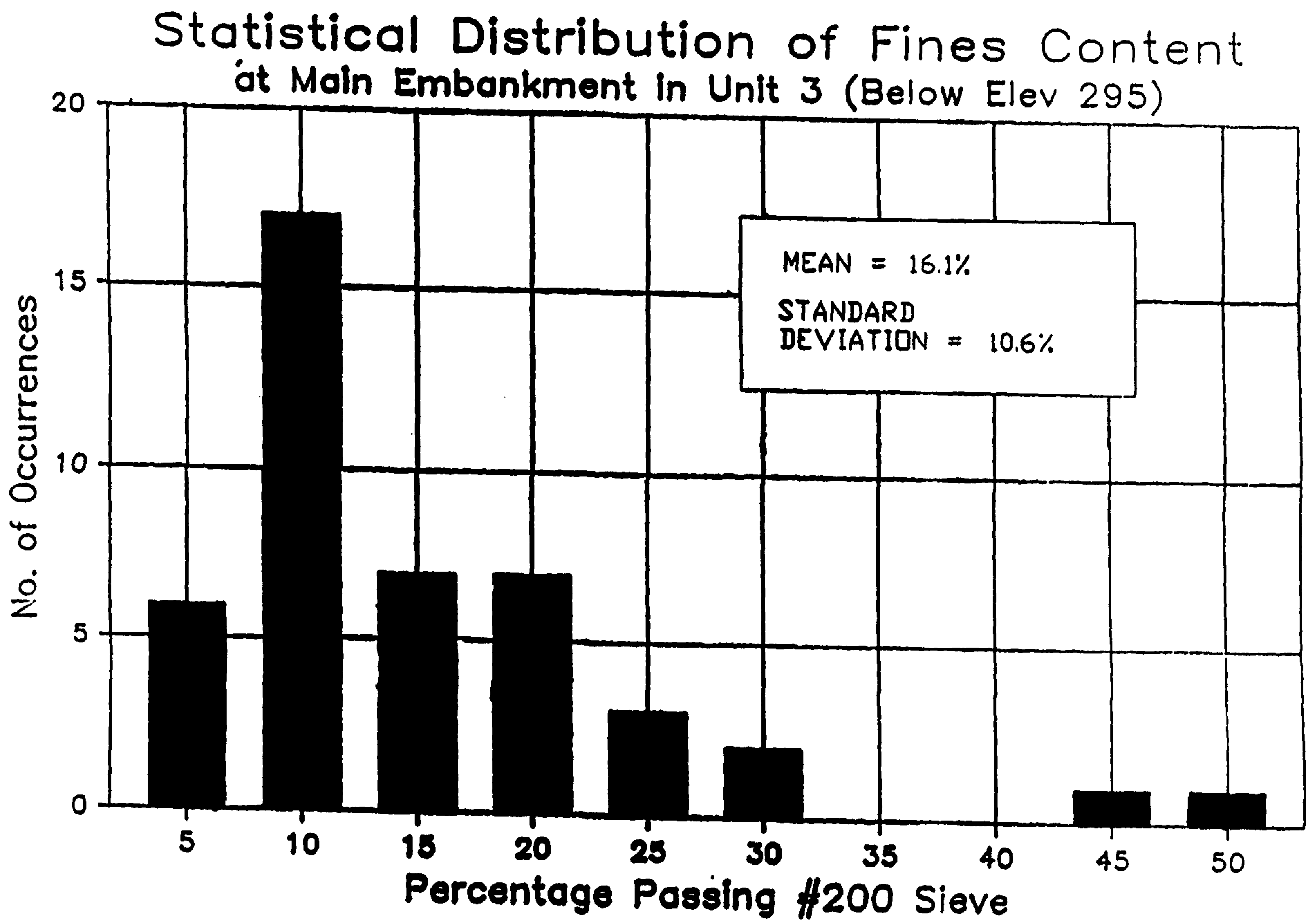


Figure 61. **Statistical distribution of the fines content of the main embankment area of Unit 3 below elevation 295.**

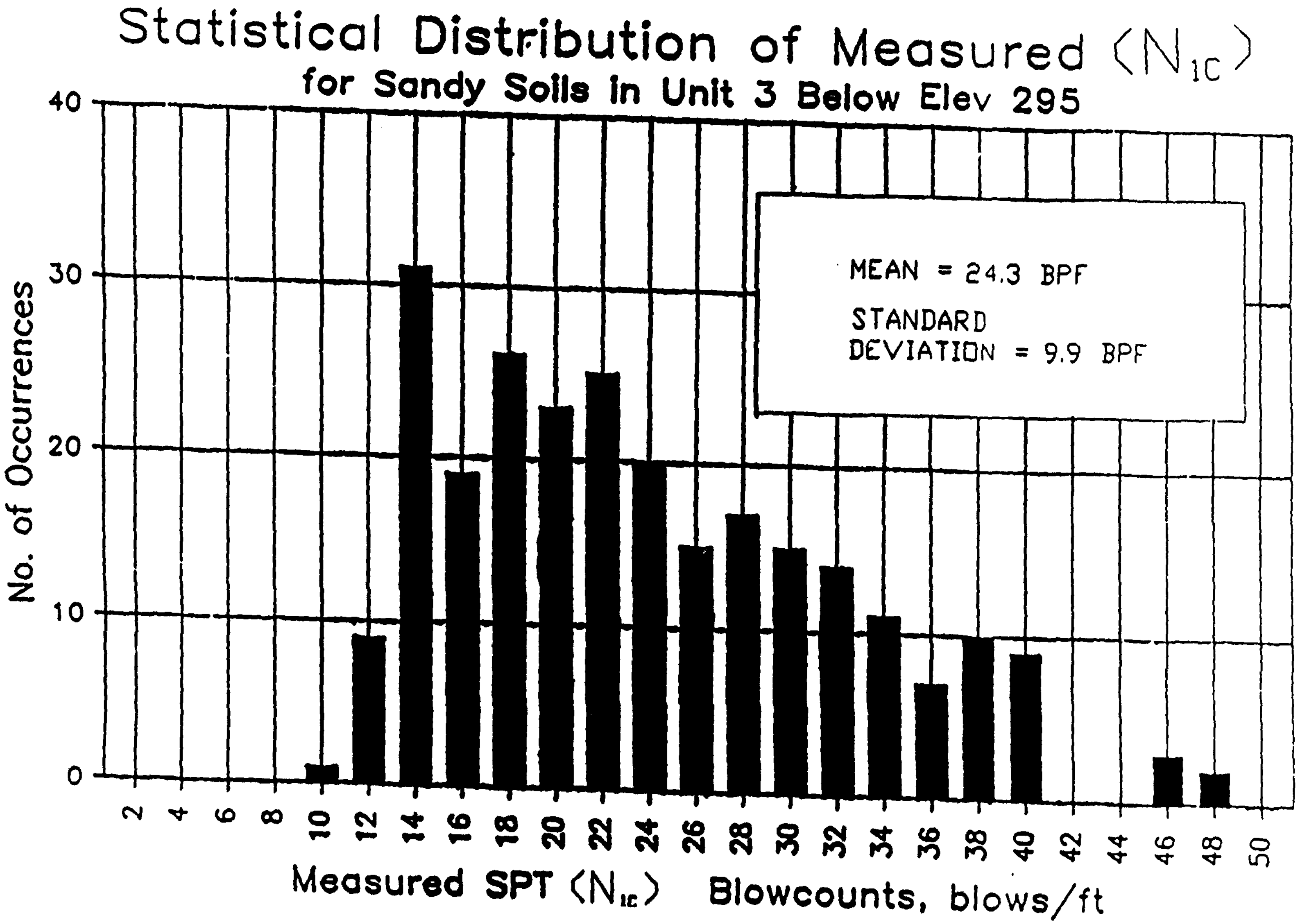


Figure 62. *Statistical distribution* of the equivalent clean sand blowcounts, N_{1c}, of the main embankment area of Unit 3 below elevations 295 and 320.

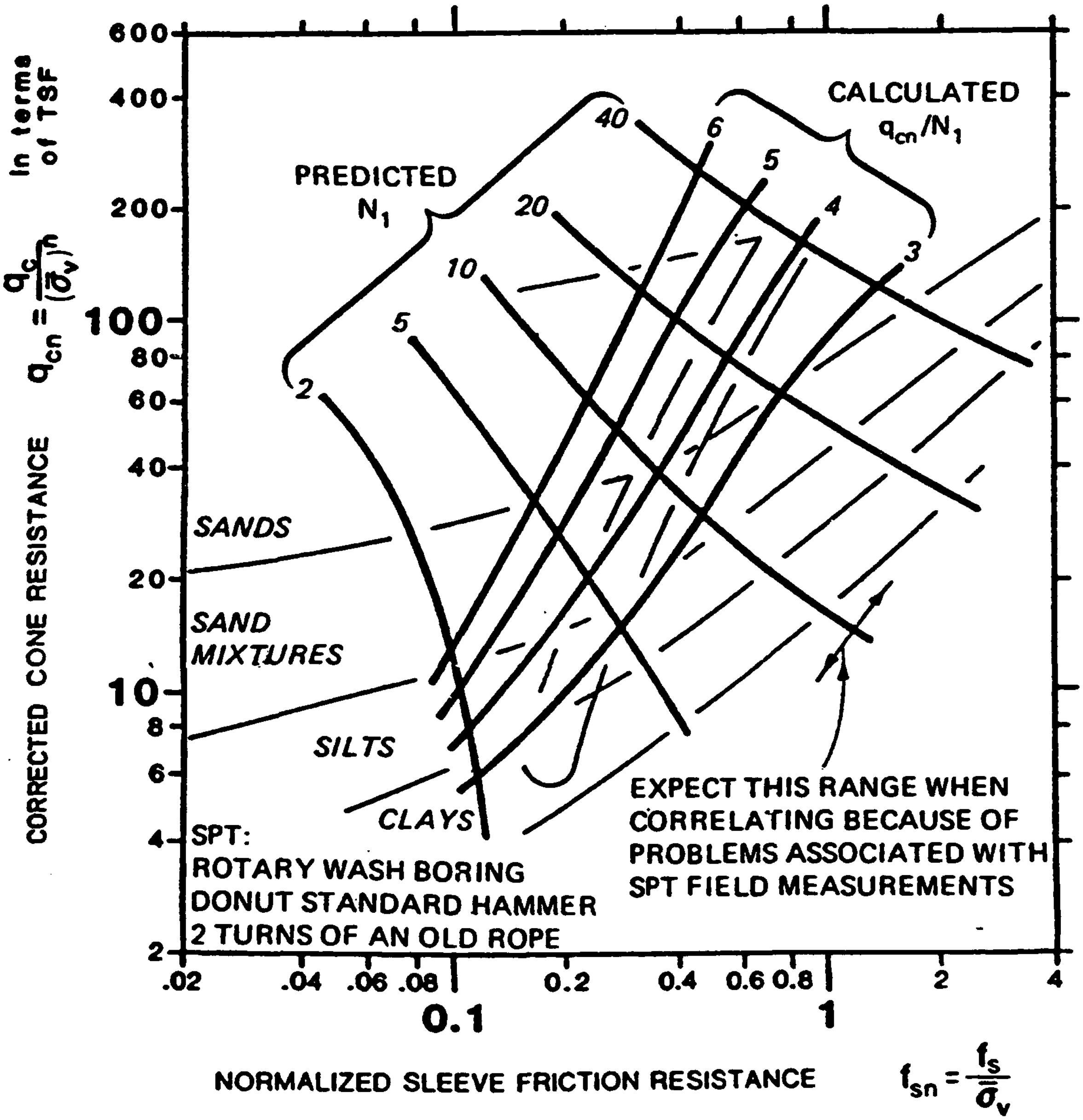


Figure 63. Chart used for CPT prediction of $(N_1)_{60}$.

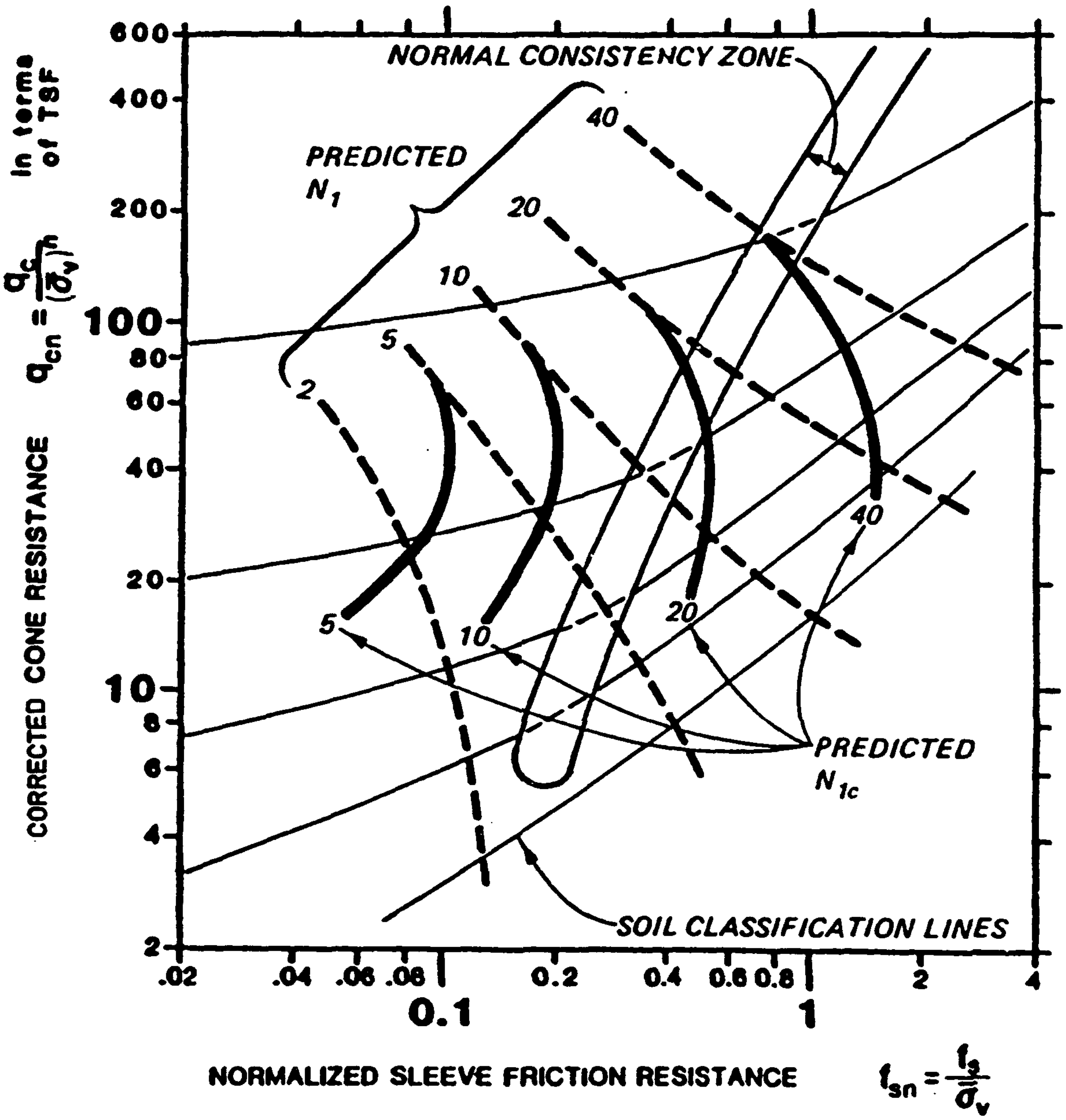


Figure 64. Chart used for CPT prediction of N_{1c}.

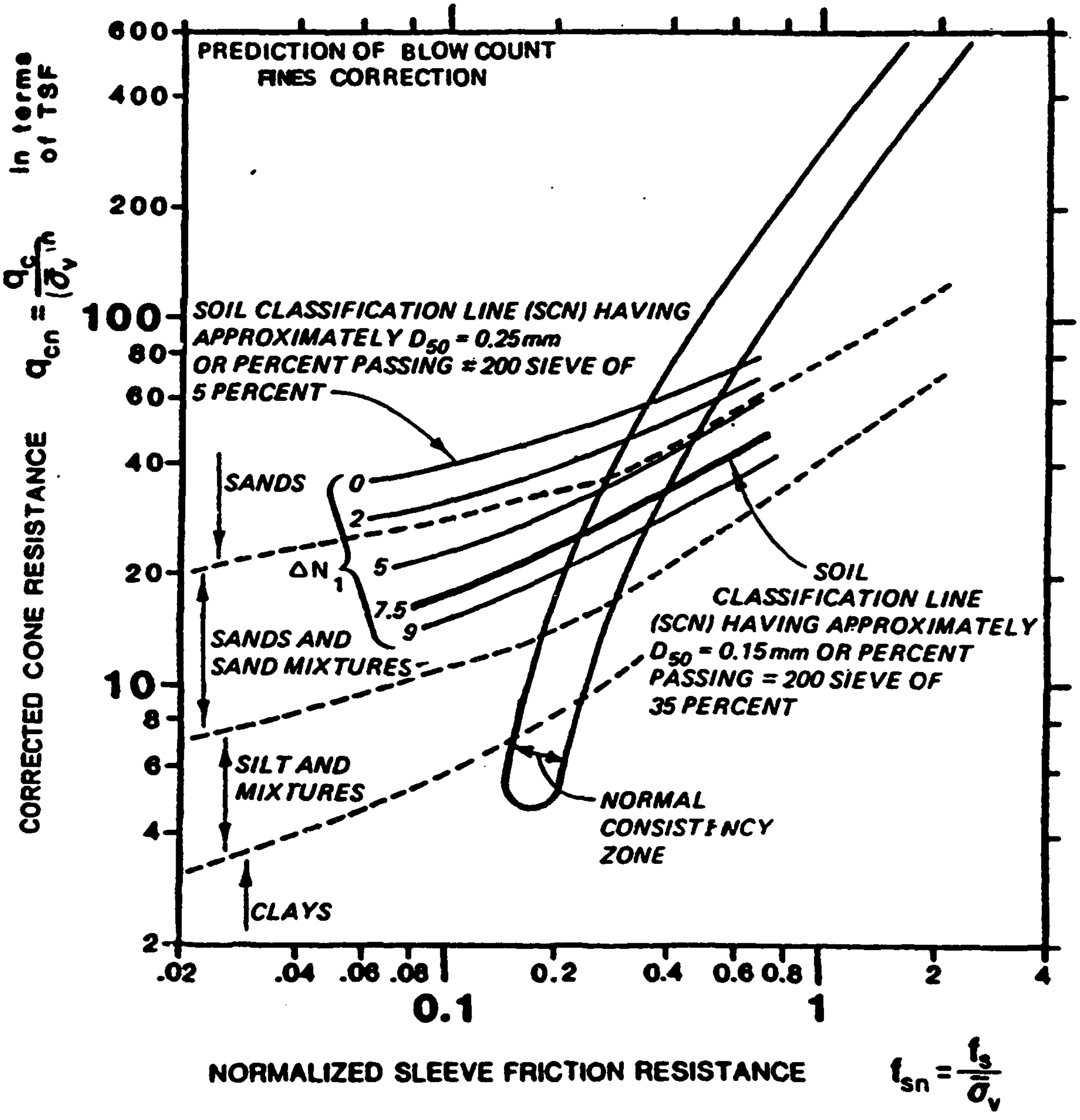


Figure 65. Chart for predicting the CPT fines content.

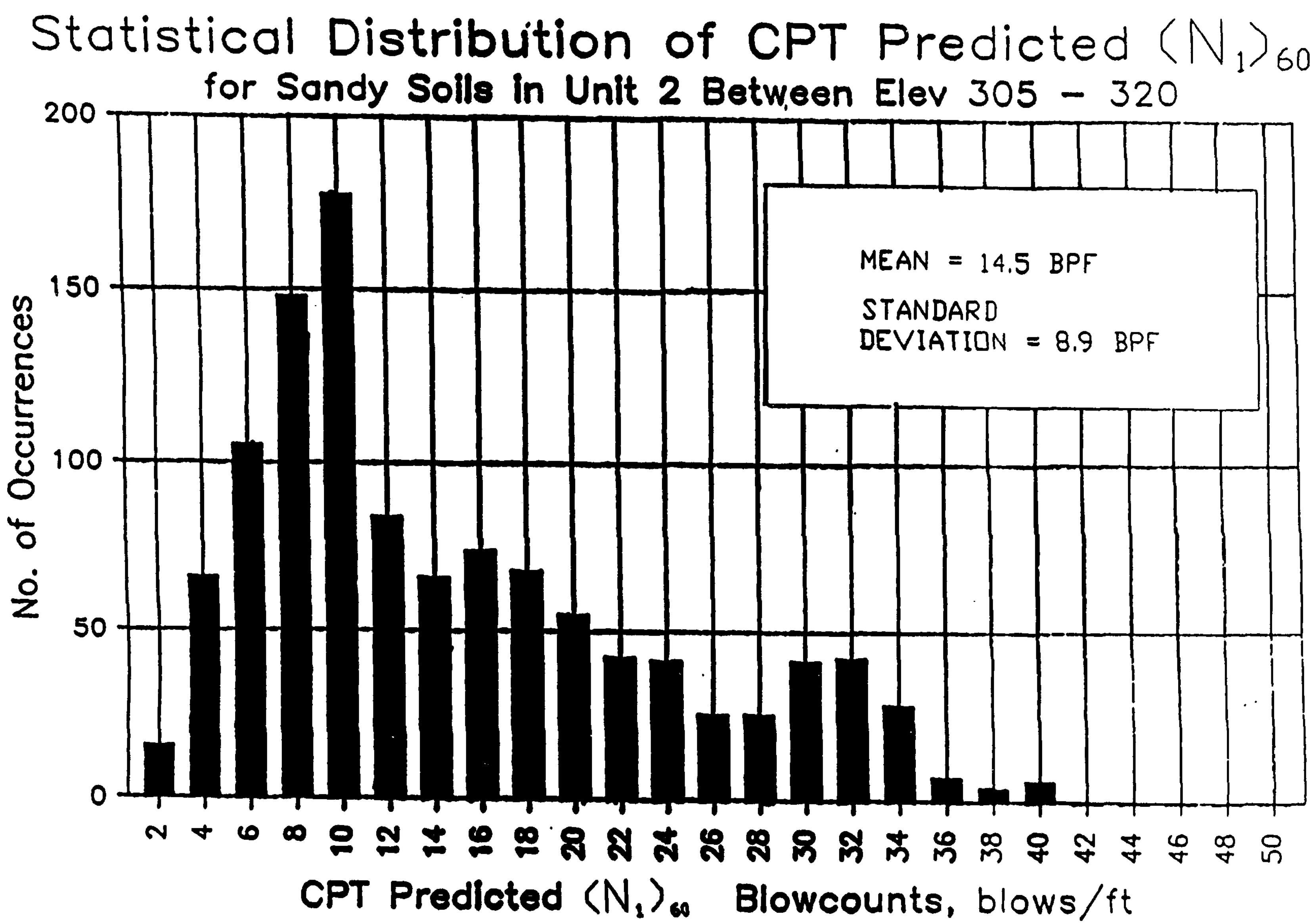


Figure 66. Statistical distribution of CPT predicted $(N_1)_{60}$ blowcounts for sands in Unit 2 (Elevation 305-320).

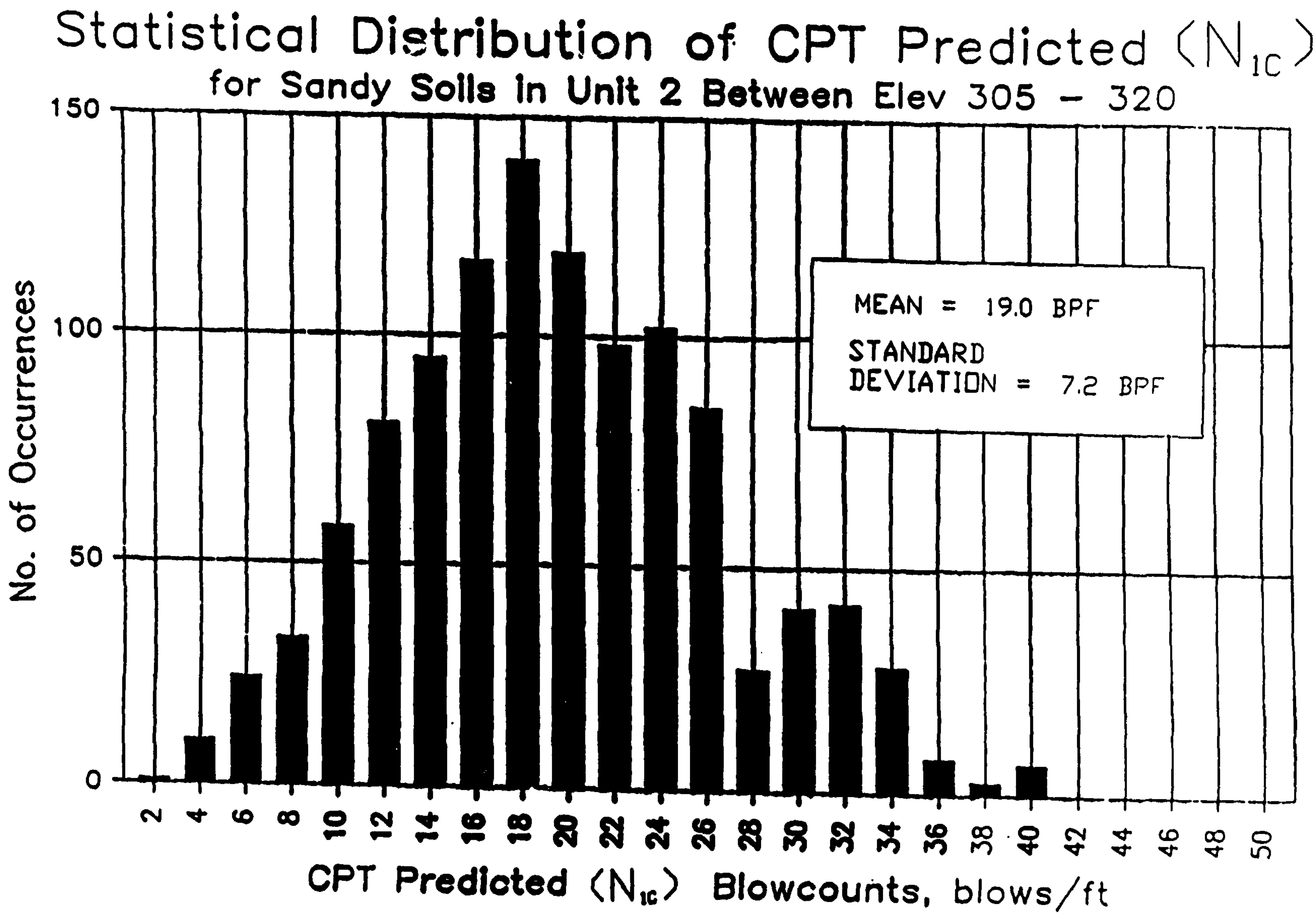


Figure 67. Statistical distribution of CPT predicted N_{1c} blowcounts for sands in Unit 2 (Elevation 305-320).

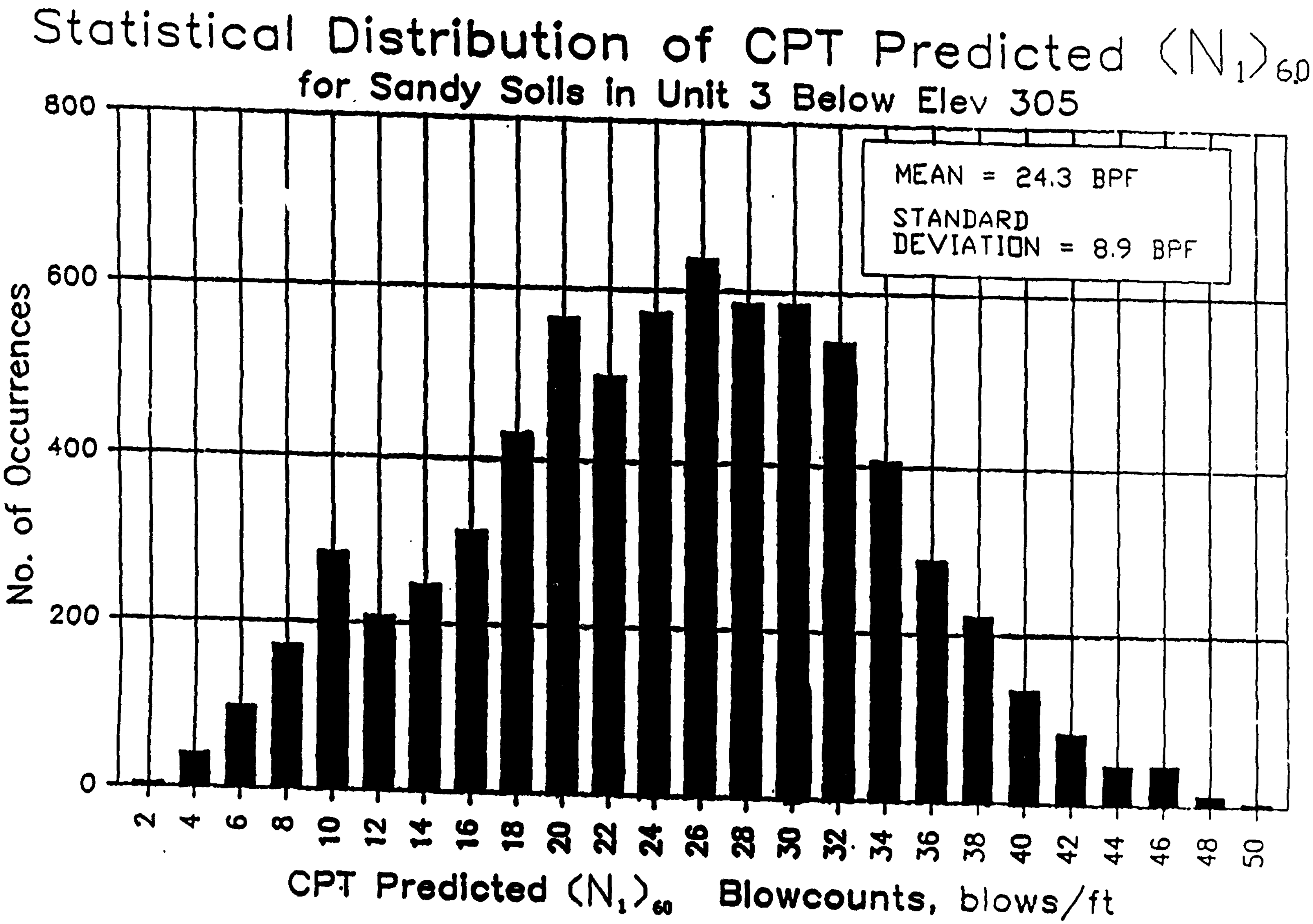


Figure 68. Statistical distribution of CPT predicted $(N_1)_{60}$ blowcounts for sands in Unit 3 (Below elevation 305).

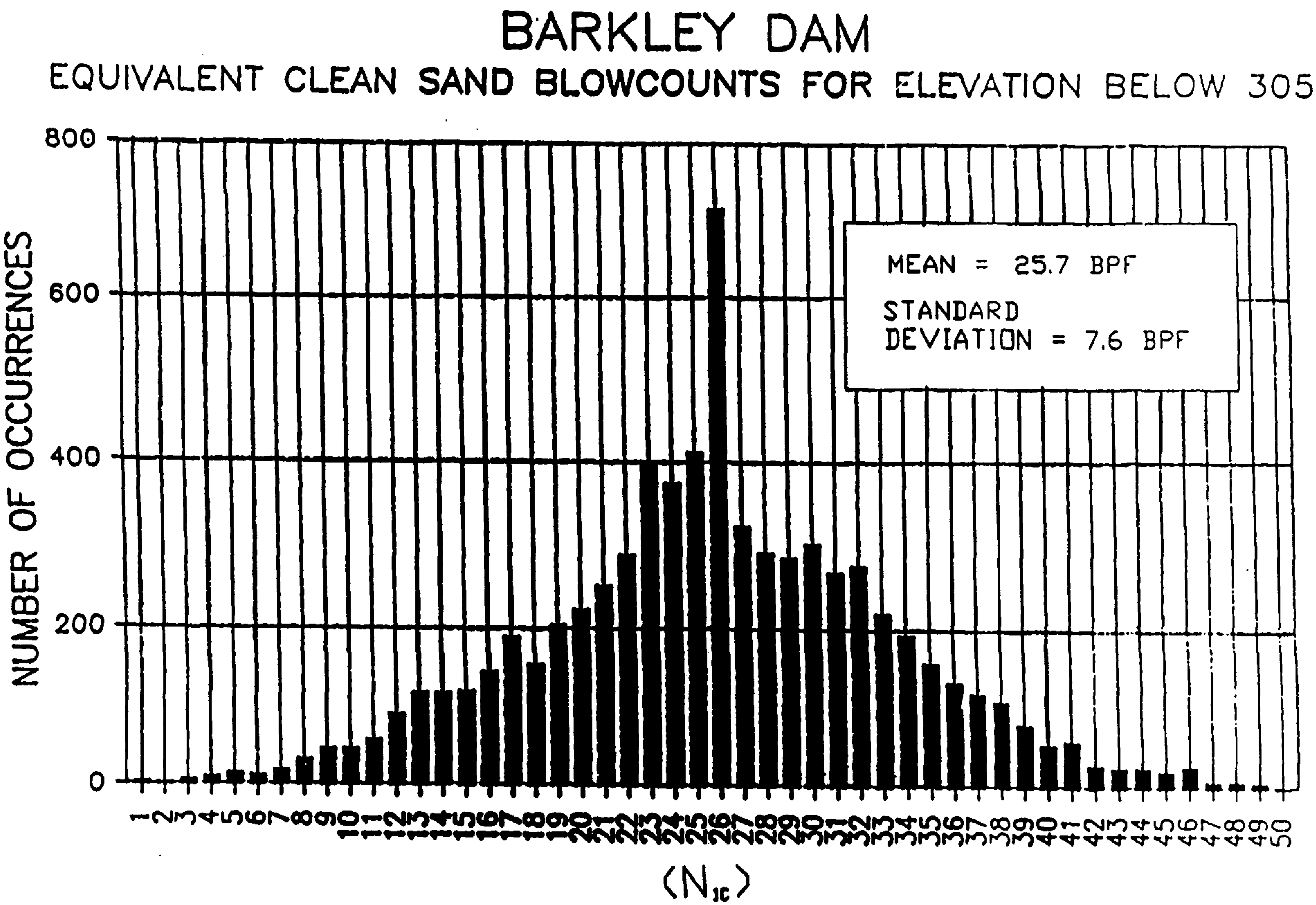


Figure 69. Statistical distribution of CPT predicted N_{1c} blowcounts for sands in Unit 3 (Below elevation 305).

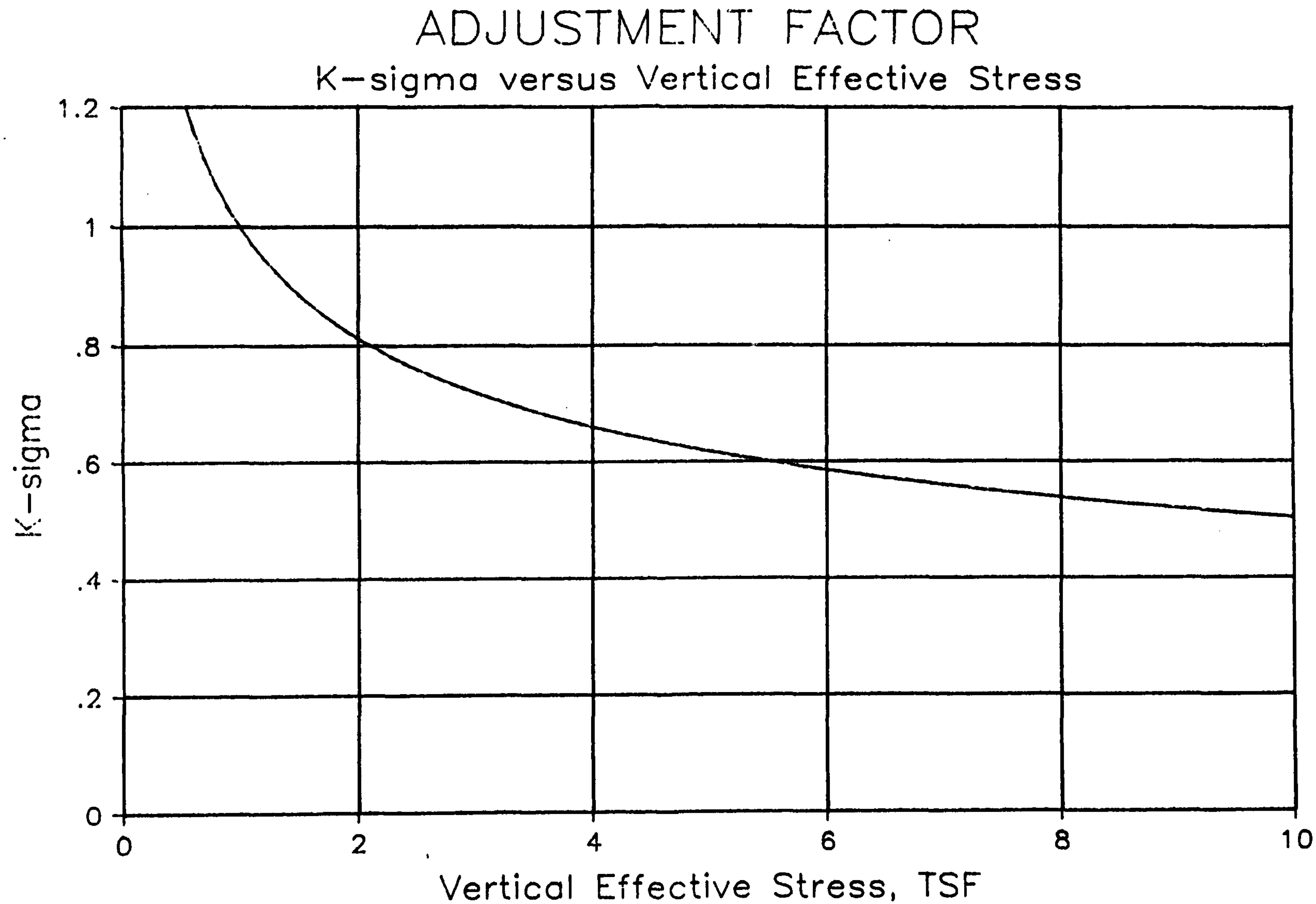


Figure 70. K_r adjustment factors.

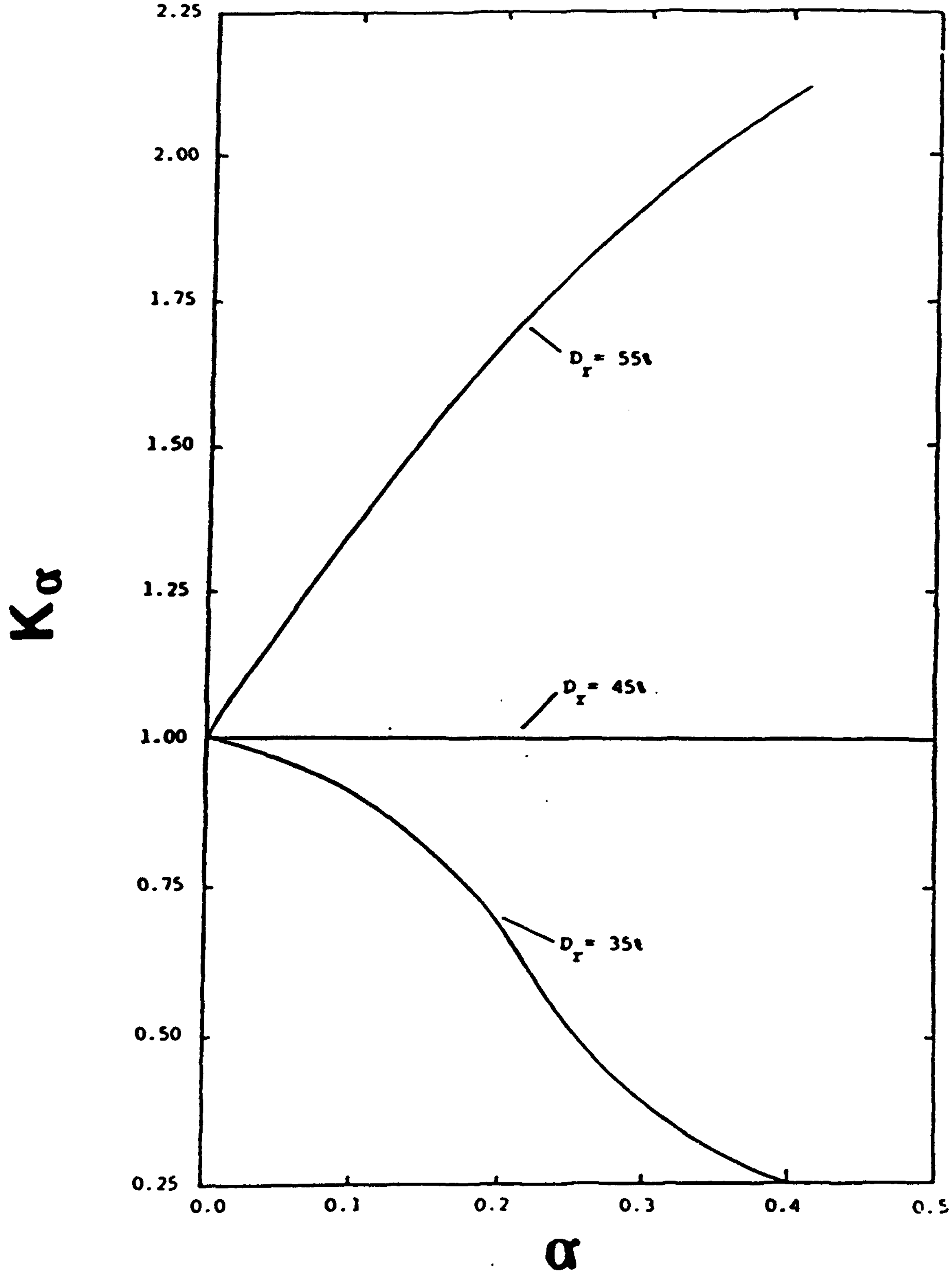


Figure 71. K_α adjustment factors.

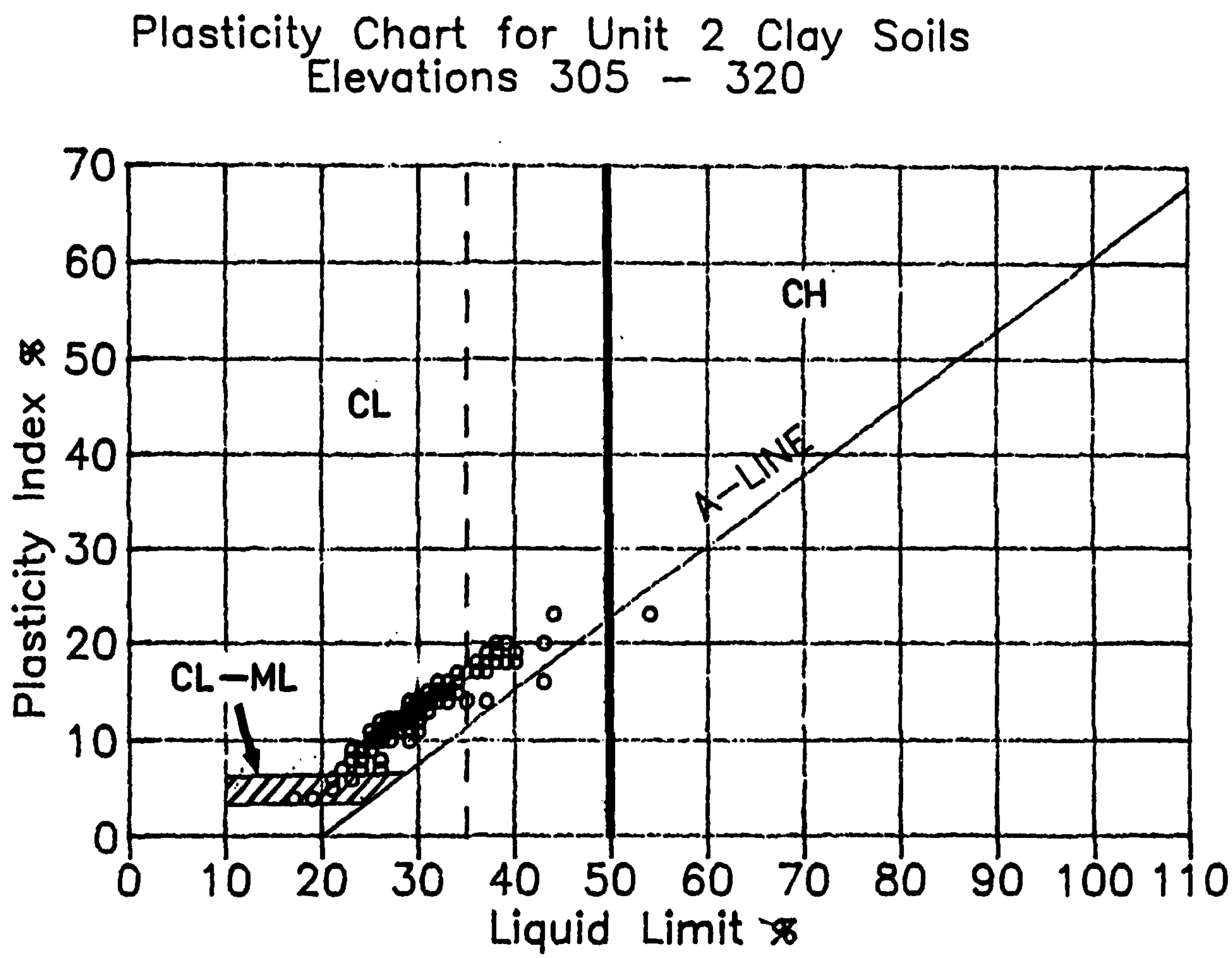


Figure 72. Plasticity chart for Unit 2 clay soils (elevation 305-320).

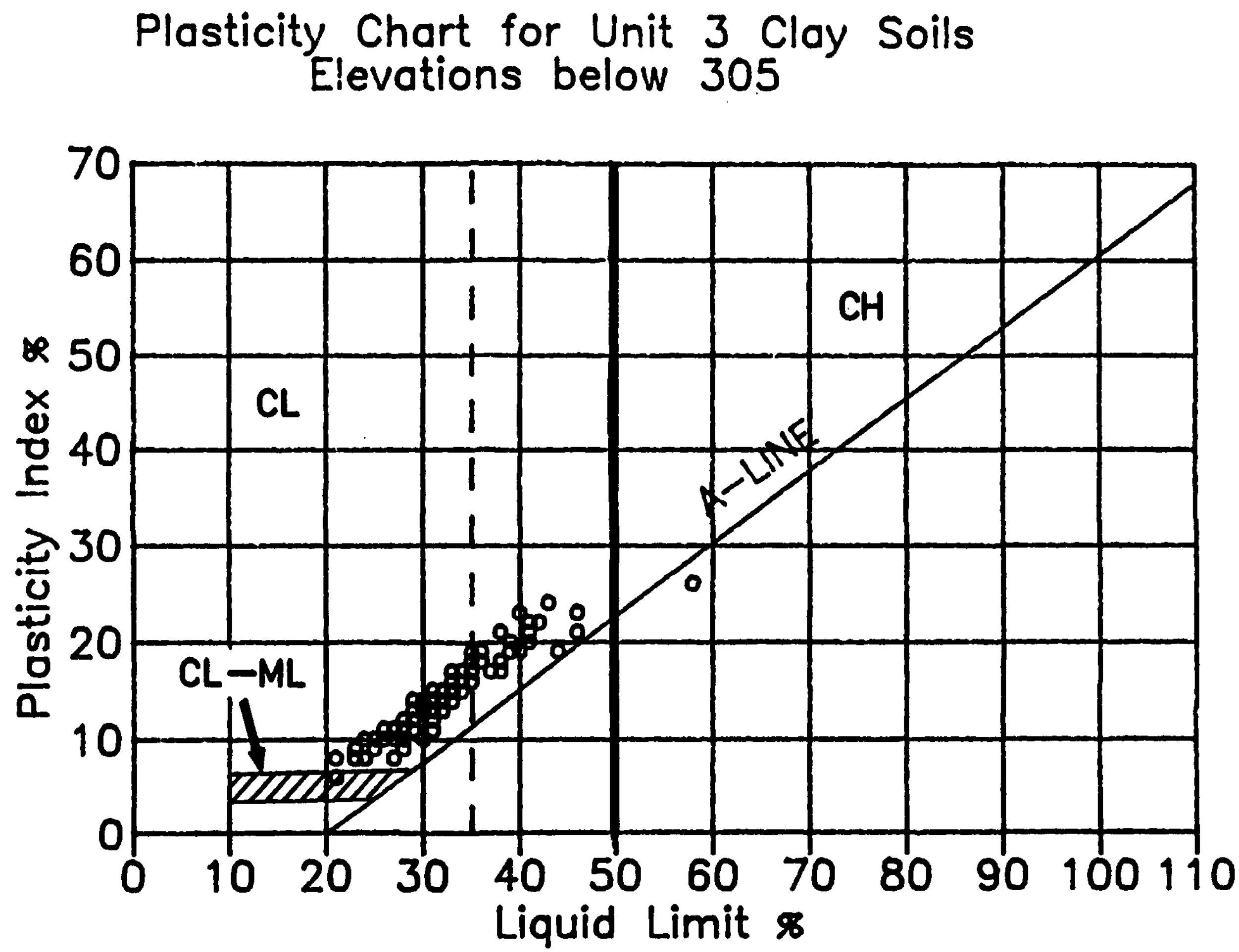


Figure 73. Plasticity chart for Unit 3 clay soils (below elev 305).

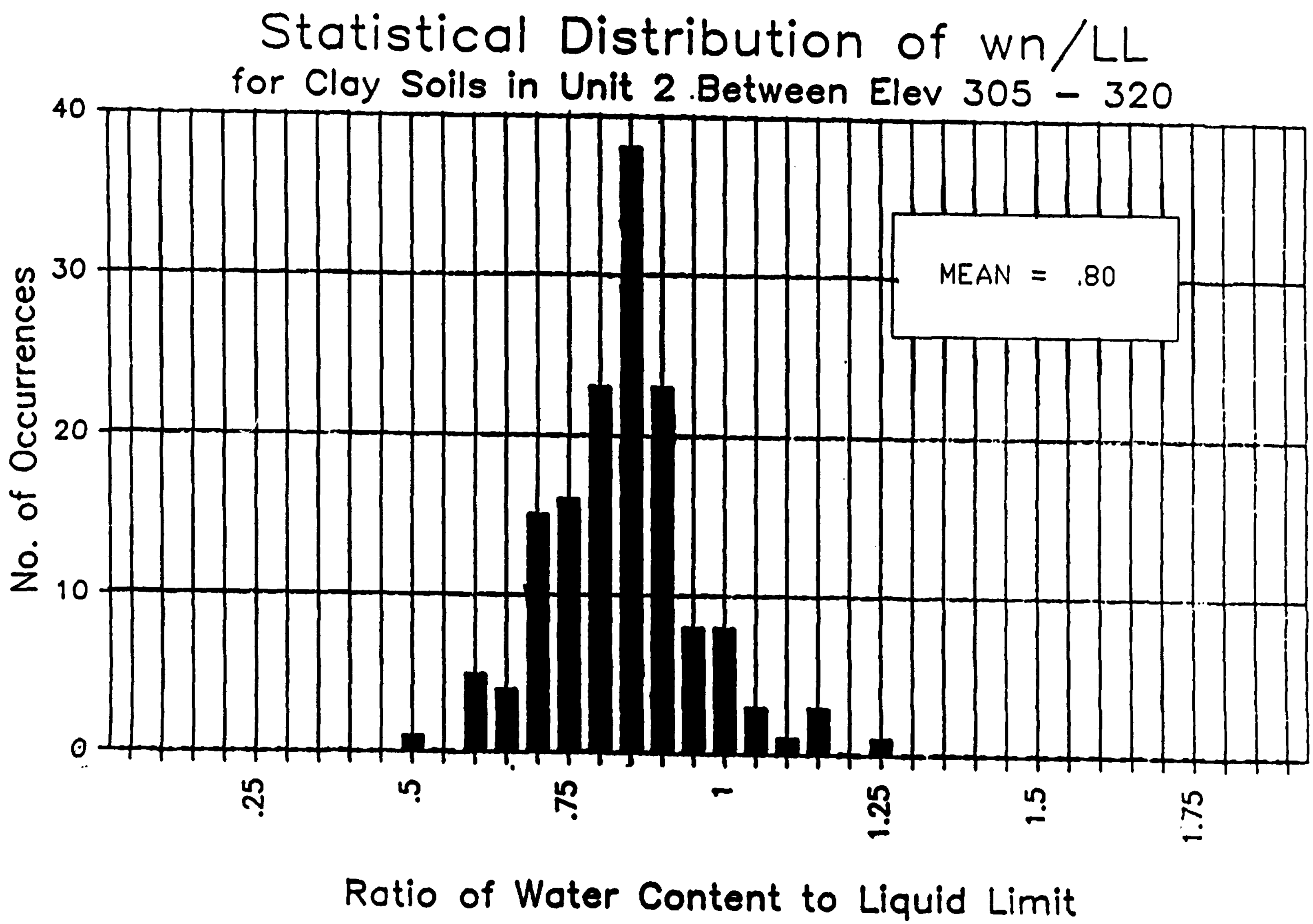


Figure 74. Statistical distribution of the ratio of water content to liquid limit for Unit 2 (elevations 305 - 320).

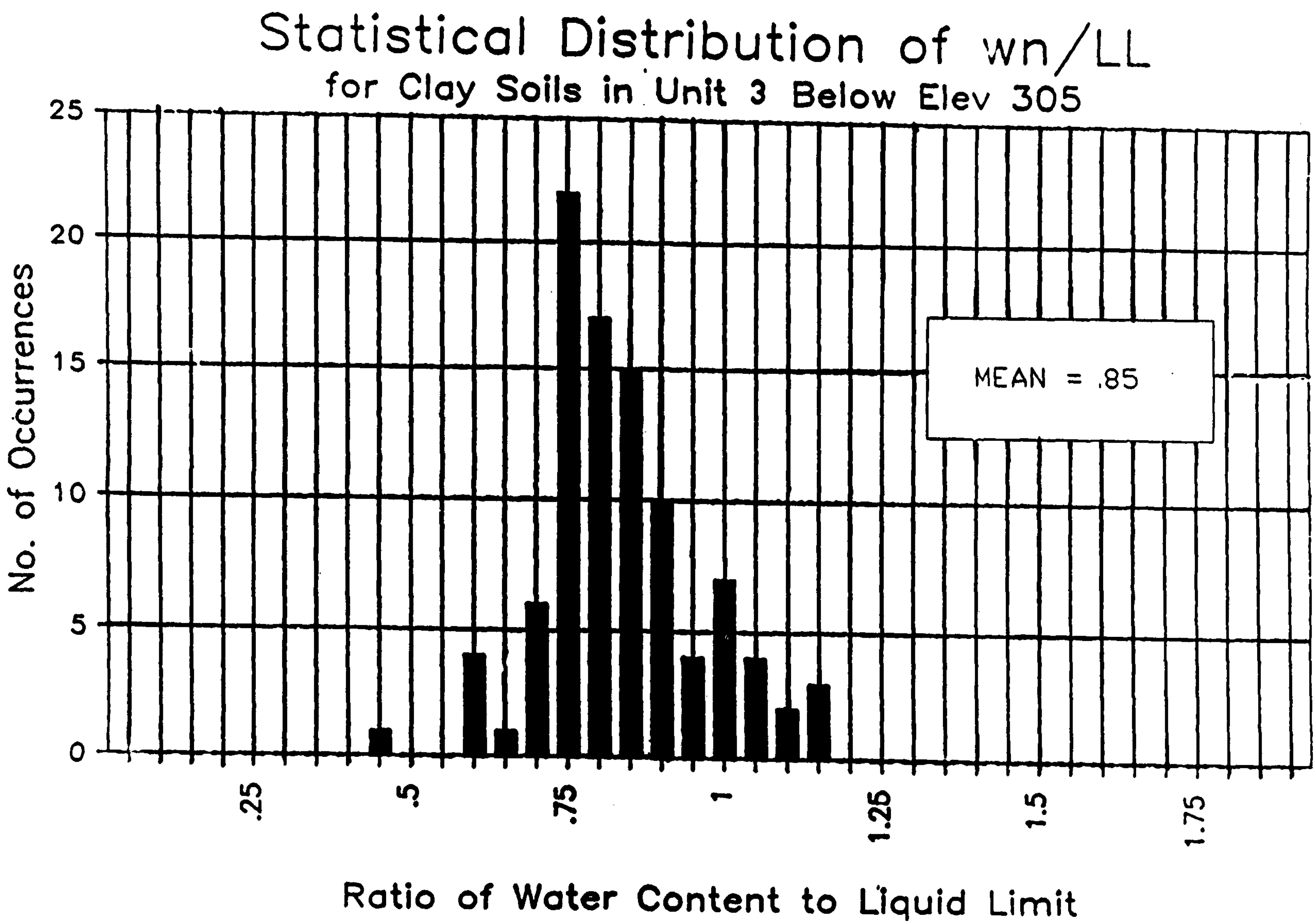


Figure 75. Statistical distribution of the ratio of water content to liquid limit for Unit 3 (elevations below 305).

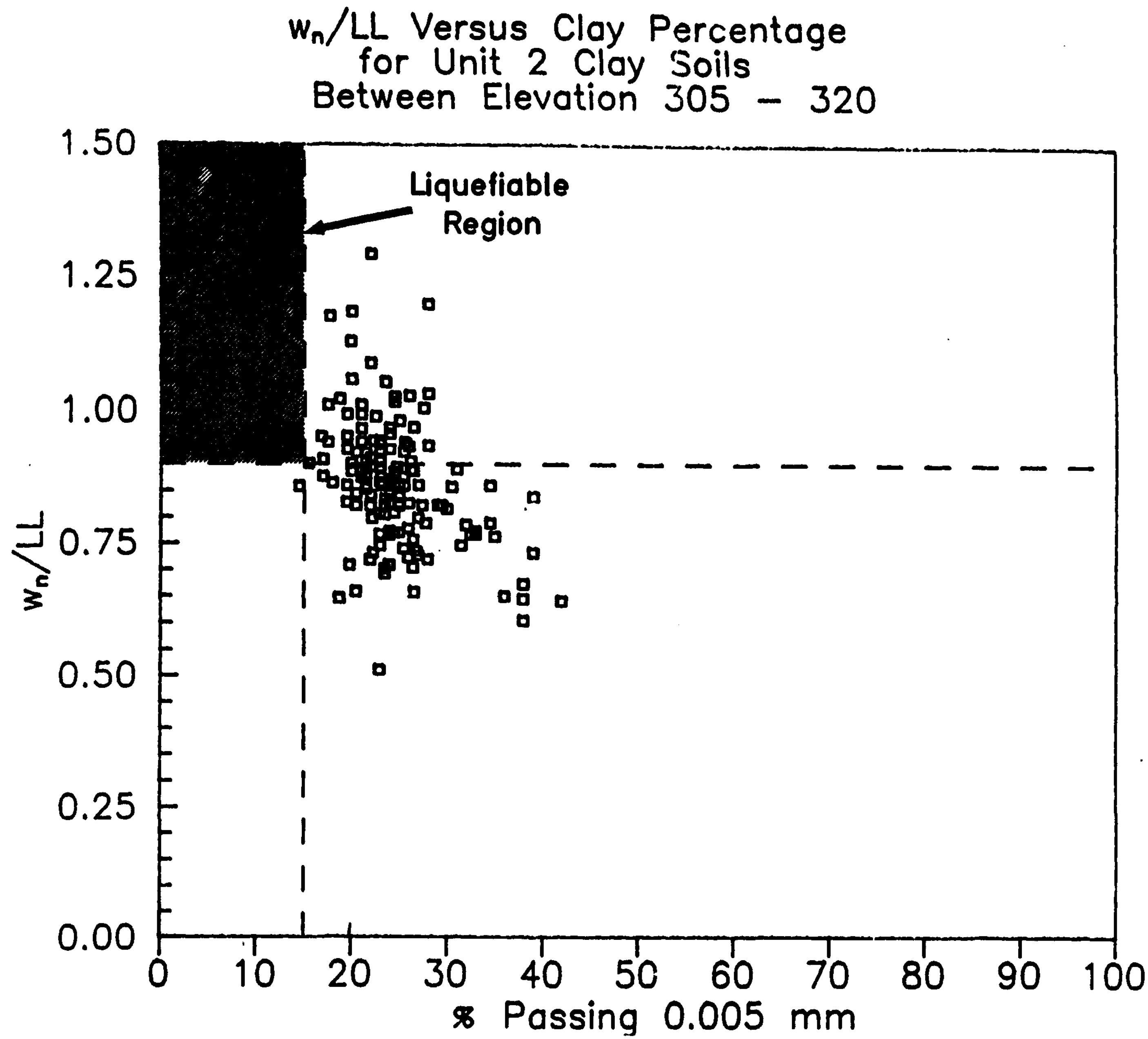


Figure 76. Water content to liquid limit ratio versus percentage finer than 0.005 mm for clay soils of Unit 2 (elev 305 - 320).

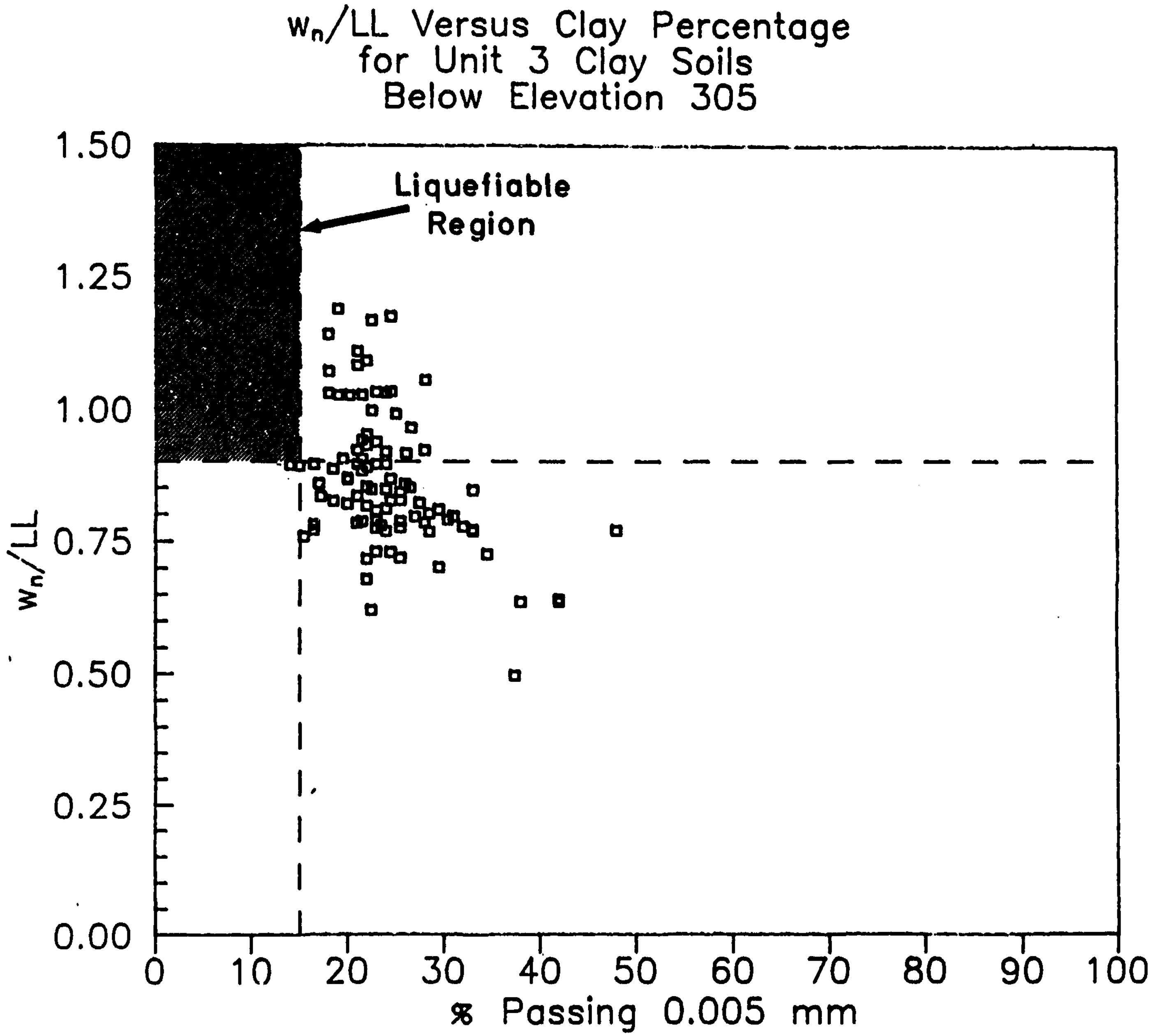


Figure 77. Water content to liquid limit ratio versus percentage finer than 0.005 mm for clay soils of Unit 3 (below elev 305).

Cyclic Strengths Based on CPT Predicted $(N_1)_{60}$ Blowcounts (Mean − 1/2σ) for Sandy Soils

	$(N_1)_c$ − 30%ile	$\left(\tau/\sigma'_v\right)_{\alpha=0,\ \sigma=1\text{ tsf},\ M=8^+}$
Unit 2	15 bpf	0.184
Unit 3a & 3c	22 bpf	0.214

Note: Unit 3b modeled as non-liquefiable.
Unit 3b is designated by ▬.

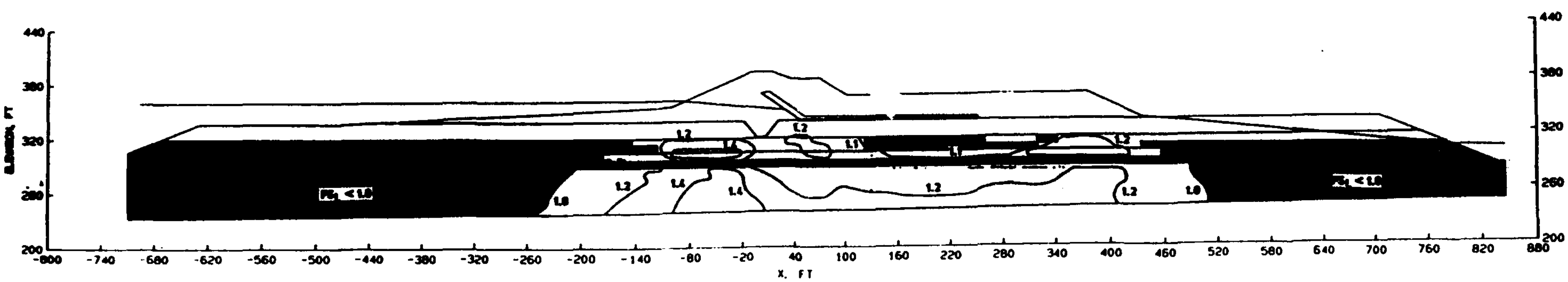


Figure 78. Switchyard Area: Contours of factors of safety against liquefaction, FS_L, in the foundation sands.

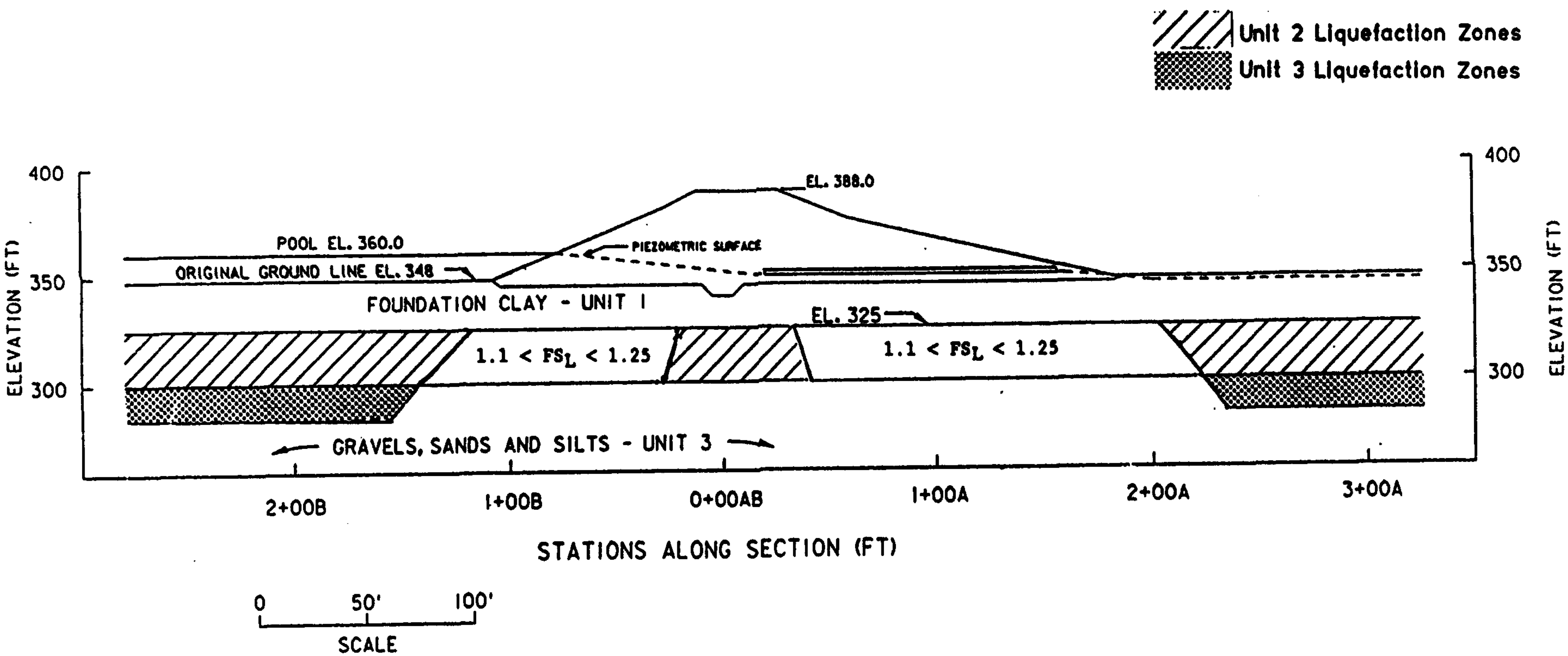


Figure 79. Main Embankment Area: Factors of safety against liquefaction, FS_L, in the foundation sands.

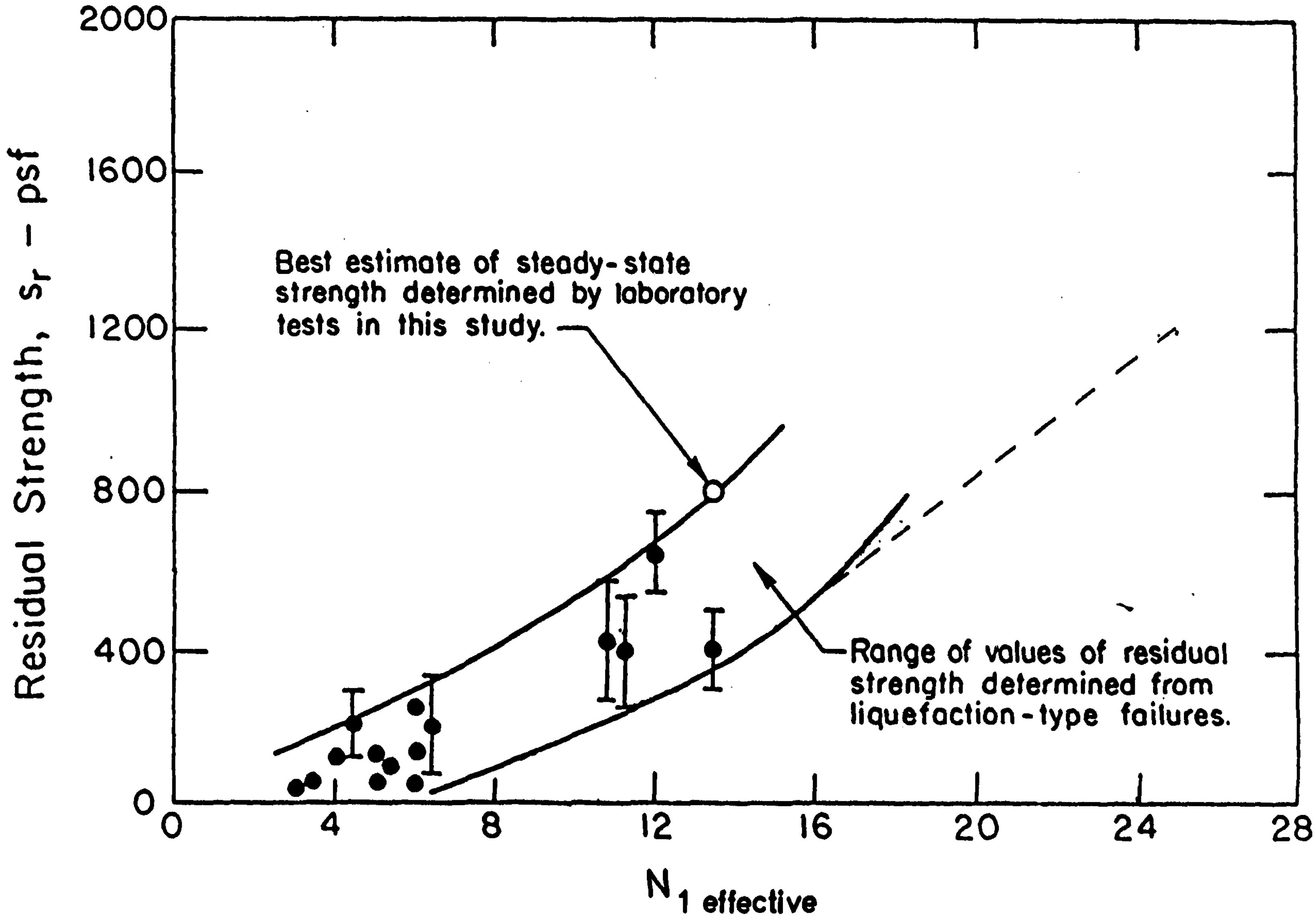


Figure 80. Correlation of residual strengths, S_{ur}, with SPT blowcounts. (After Seed, et al, 1986)

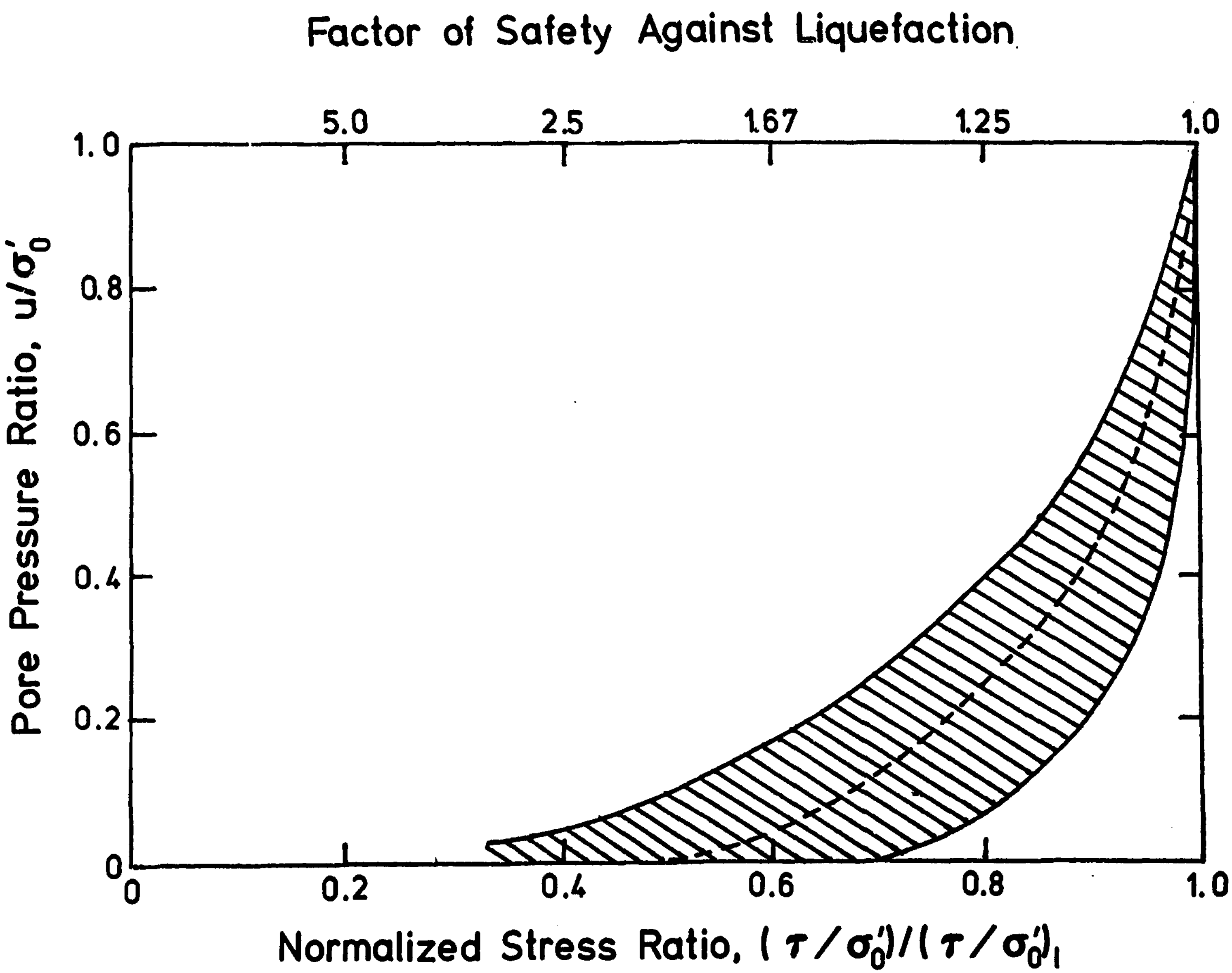


Figure 81. Normalized excess pore pressure ratio verus FS_L.
(After Tokimatsu and Seed, 1983)

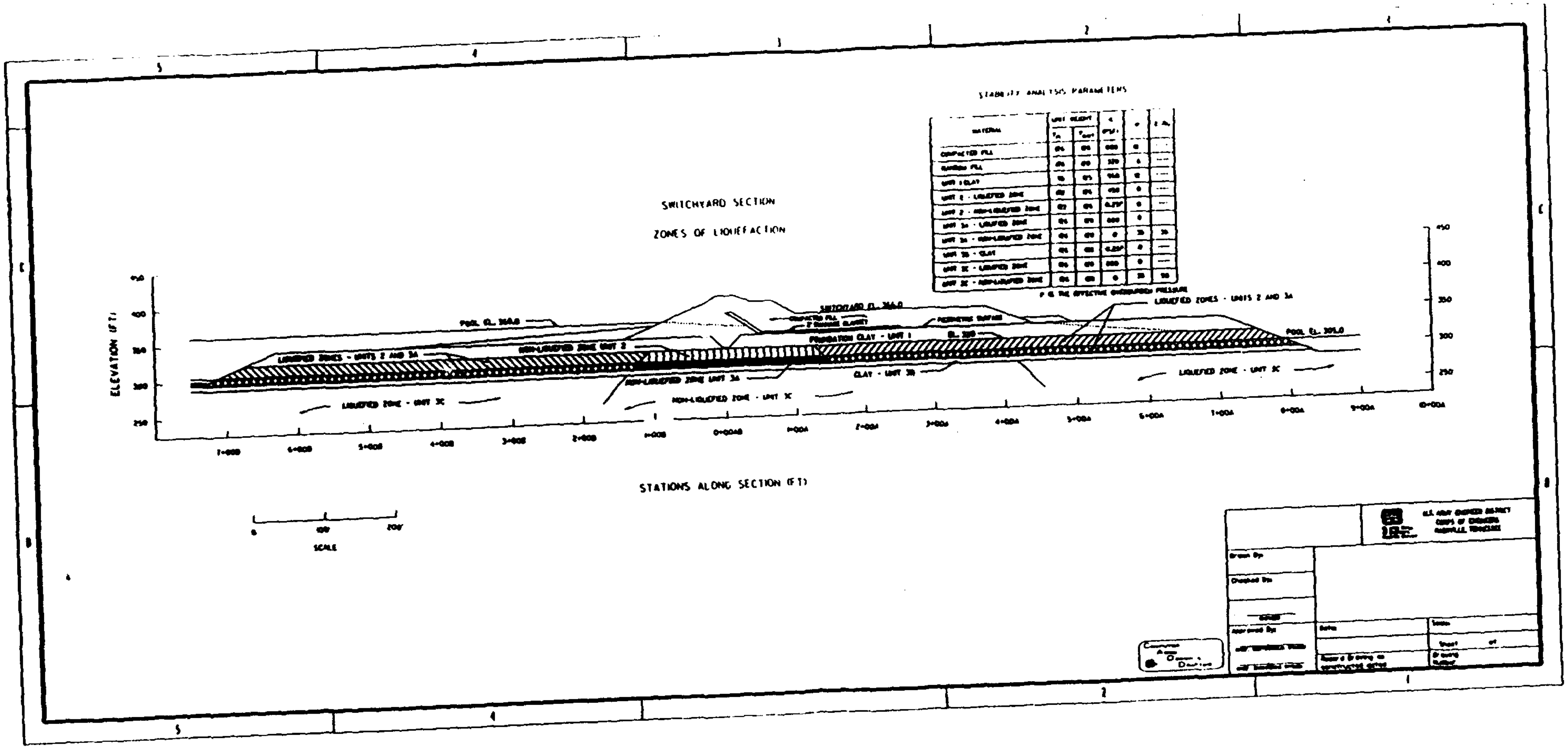
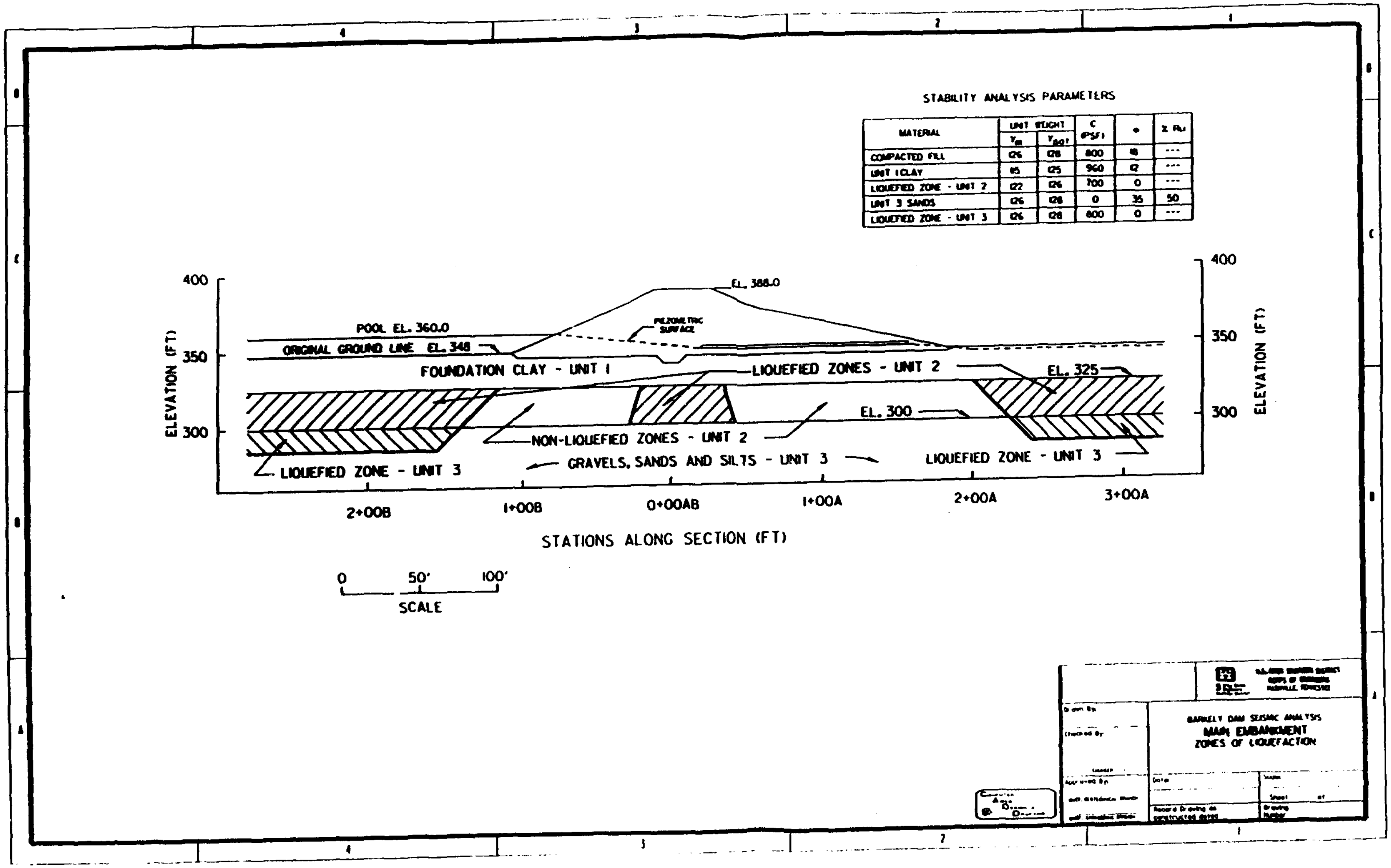


Figure 82. Recommended post-earthquake strengths for switchyard section.



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	---

Figure 83. Recommended post-earthquake strengths for main embankment section.

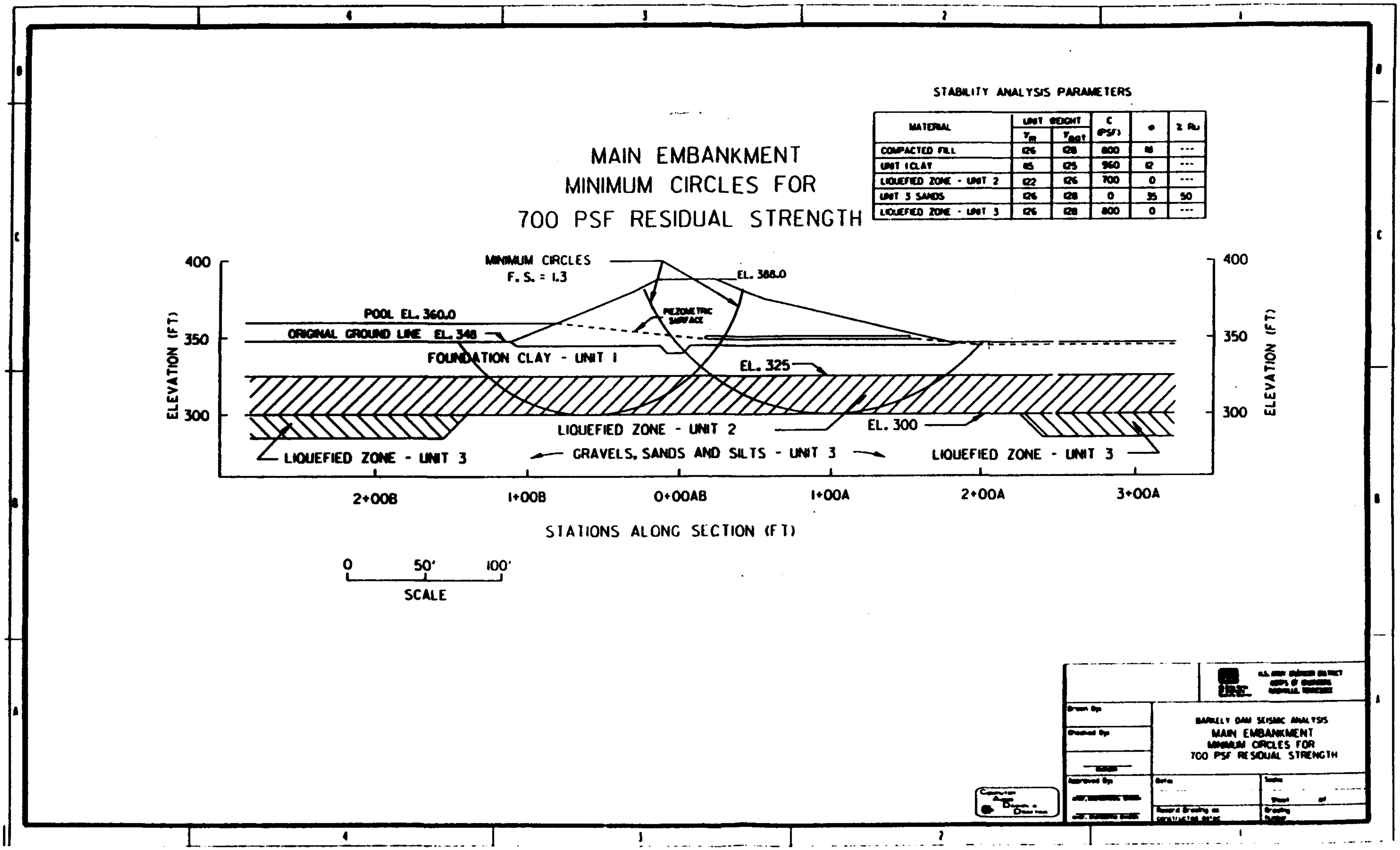


Figure 84 Post-earthquake factors of safety against sliding for main embankment section with residual strength of 700 psf in Unit 2

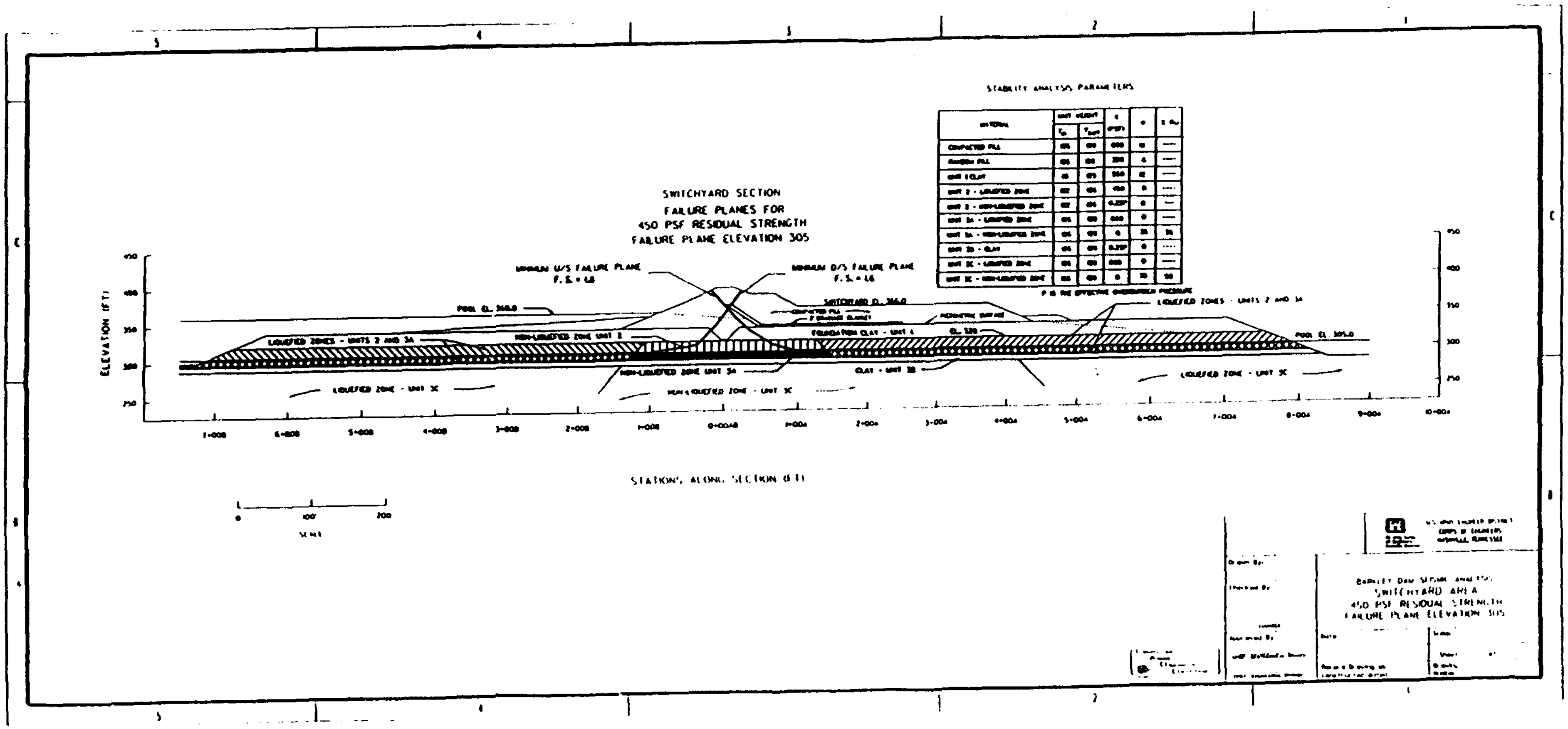


Figure 85. Minimum factors of safety against sliding of upstream and downstream failure wedges tangent to elev 305 of the switchyard section using post-earthquake strength conditions.

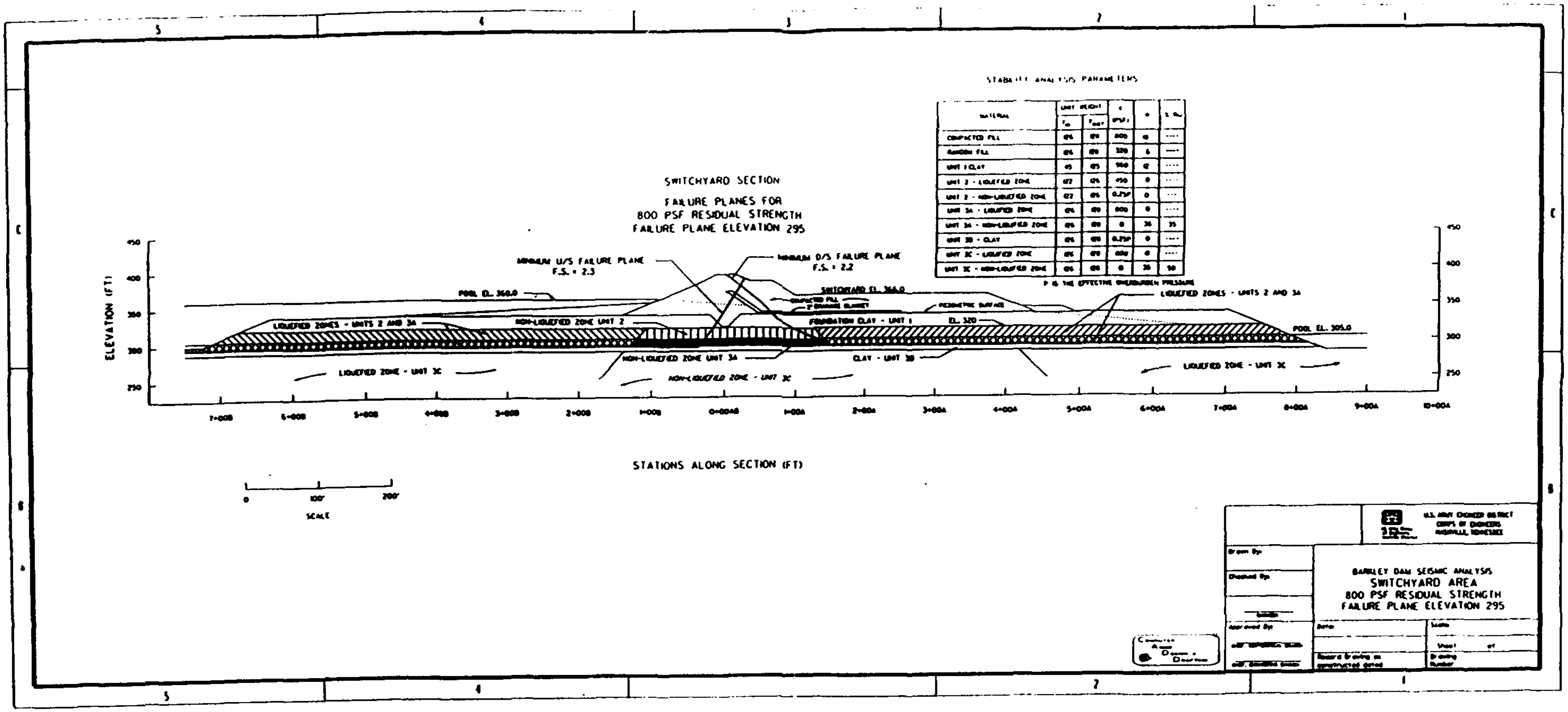


Figure 86. Minimum factors of safety against sliding of upstream and downstream failure wedges tangent to elev 295 of the switchyard section using post-earthquake strength conditions.

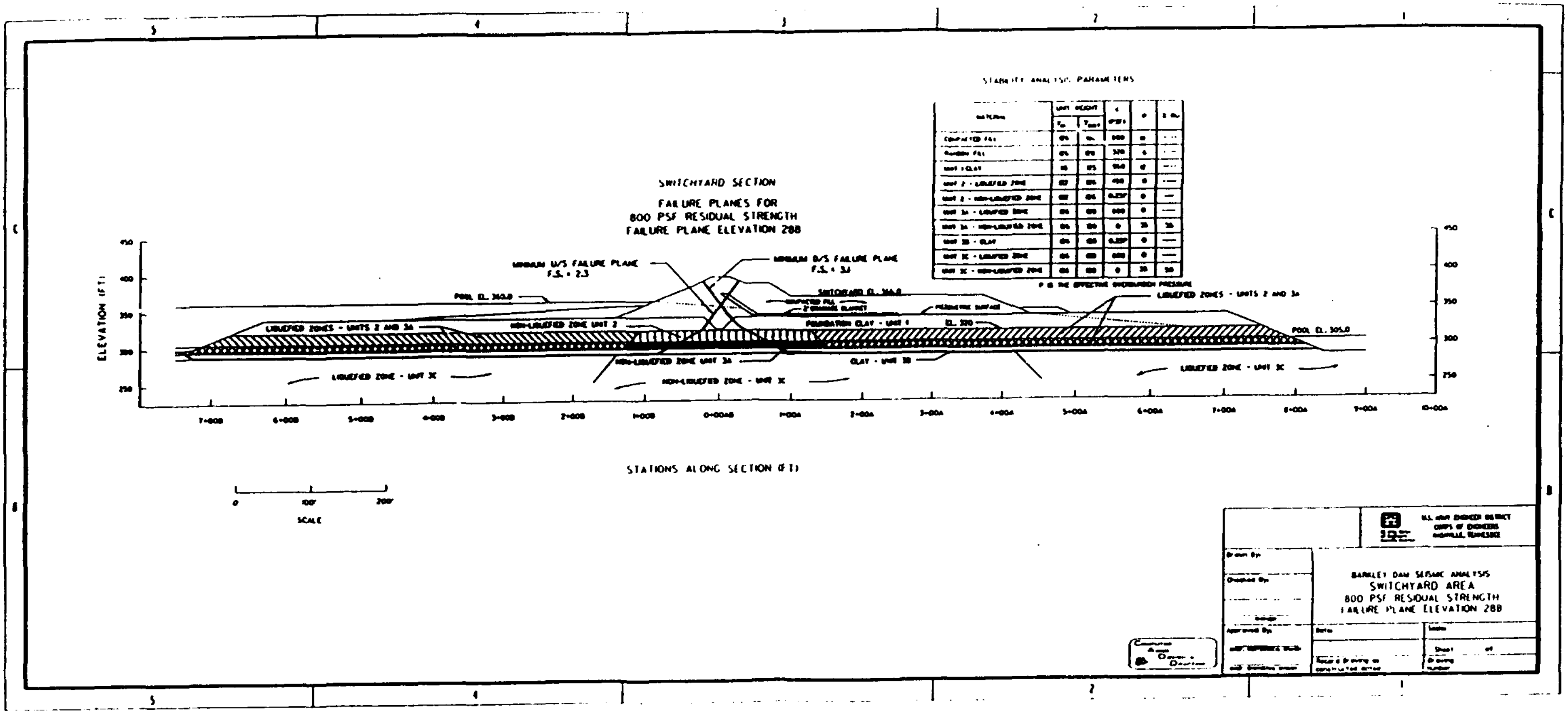


Figure 87. Minimum factors of safety against sliding of upstream and downstream failure wedges tangent to elev 288 of the switchyard section using post-earthquake strength conditions.

APPENDIX A: TECHNICAL ADVISORS' REPORTS

1. This appendix contains the reports of Drs. Alberto Nieto, H. B. Seed, and G. Castro who served as technical advisors to the seismic stability evaluation of the Barkley Project. Their reports were filed with CE-ORN shortly after meetings held to monitor progress and to adjust the technical direction of the investigation.

2. These letter reports are grouped chronologically for each of the three authors. These are in the order of Dr. Castro, Nieto, and Seed.

GEI Consultants, Inc.

1021 Main Street
Winchester, MA 01890-1943
617•721•4000

June 28, 1991
Project 85836

Mr. Paul D. Robinson
Chief, Engineering Division
Department of the Army
Corps of Engineers
P.O. Box 1070
Nashville, TN 37202-1070

Dear Mr. Robinson:

Re: **Seismic Stability Evaluation**
Barkley Dam

We have received Vols. 1, 4, and 5 of the subject seismic evaluation prepared by the Waterways Experiment Station (WES).

The information contained in these reports had been presented in a meeting with the consultants on January 24, 1986 in which it was concluded that no remedial measures are required to enhance the seismic stability of the dam. Our concurrence with this conclusion was documented in our letter of February 24, 1986, in which we presented a brief description of our rationale for assessing the seismic behavior of the dam. We feel that the conclusion that no remedial measures are needed is still valid. The results of the analyses presented in the WES reports led to the same conclusion.

The evaluation of the seismic stability of Barkley Dam constituted a major engineering effort, the results of which are likely to be used by others in future evaluations of other Corps of Engineers' dams as well as of dams of other government and private organizations. Yet we feel that the methodology used could, in other cases, lead to invalid conclusions relative to the seismic safety of the dams. Therefore, we believe it important to present in some detail our disagreements with some of the aspects of the methodology used by WES.

In brief, the methodology in the WES report uses blowcounts, either measured or estimated from cone penetration logs, to determine: (a) pore pressure generation in various zones of the foundation (referred to as liquefaction potential) and (b) post-earthquake strengths of the foundation soils. Both determinations are based on empirical charts. We disagree with the methodology in three general areas, namely (1) on the conclusions drawn from empirical charts based on blowcounts, (2) on the significance of the blowcounts for the particular conditions of the foundation soils at Barkley, and (3) on the use of estimated pore pressure increases in stability evaluations.

1. Empirical Charts

Two empirical charts were used in the WES study: (1) a chart that relates manifestations of pore pressure increases in level ground sand deposits to blowcounts and intensity of earthquake shaking, Fig. 55 of Vol. 1 of the report, and (2) a correlation between blowcounts and residual (post-earthquake) strength, Fig. 80 in Vol. 1 of the report.

"Liquefaction" Chart

In the chart in Fig. 55 of Vol. 1, the cases classified as liquefaction represent instances where an earthquake caused sufficiently high pore pressures for sand boils to develop. Under level ground, high pore pressures are followed by reconsolidation and settlement, but unless heavy structures are present, no shear deformations of significance occur. The relevance of pore pressure increase predictions to the behavior of Barkley Dam is questionable. The use of the blowcount chart to predict pore pressure leads only to the conclusion that the pore pressure will build up. But this result provides no information on the likelihood of a flow (liquefaction) slide due to earthquake shaking or to the deformations that would occur during such an event.

The potential for a liquefaction slide (as in the case of the Lower San Fernando Dam) is present if the value of the driving shear stress exceeds the undrained steady state strength, S_{us}, of the soils in question. The S_{us} values are only a function of the void ratio at which the soil is found *in situ* and not on how much the pore pressure increases when the soil is shaken by an earthquake. If the dam is not subject to a liquefaction slide, i.e., it is inherently stable, seismically induced deformations need to be evaluated. Again, in this case, there is no correlation between these deformations and the pore pressure predictions under level ground.

The above points have been clearly illustrated by the results of laboratory tests (see for example, Castro 1987)[1] and more recently in centrifuge experiments. Model embankments on an instrumented sand foundation have been shaken in the centrifuge. Essentially 100 percent pore pressures were measured throughout the foundations; however, no liquefaction slide occurred, and the permanent deformations of the model embankments ranged from insignificant to potentially damaging to the prototype, even though in all cases the pore pressures were similarly high.

Thus, in summary, pore pressure predictions made on the basis of Fig. 55 in Vol. 1 bear no relationship to the potential for a liquefaction slide or for limited deformations of Barkley Dam as a result of earthquake shaking.

S_{us} Chart

The second empirical chart is presented in Fig. 80 of Vol. 1 of the report and relates blowcounts to the "residual" strength as obtained from analyses of past failures, Seed (1987). In the context of this letter, the terms residual, post-earthquake, and undrained steady state refer to the same strength, S_{us}, which is the strength that would be available to resist an undrained (liquefaction) slide. For contractive soils, it is appropriate to use S_{us} also as the yield strength when estimating potential deformations using a Newmark type of approach. Thus the determination of representative values of S_{us} is crucial to the evaluation of both the seismic stability and the deformations of Barkley Dam and its foundation due to earthquake shaking.

Even though site-specific correlations between S_{us} and blowcount (or cone penetration resistance) have been developed relatively successfully for several sites, e.g., Keller et al. 1987, a universal correlation should be used only as a rough guide and not as a definitive evaluation tool. Two soils with the same S_{us} can have very different blowcounts depending on the degree of drainage that occurs during driving of the spoon and on other factors. For example, in the case of a soil that is highly contractive, its S_{us} value would be much lower than its drained strength, S_d. Thus, if drainage can occur during and after penetration for each blow, the blowcount would be much larger than if the soil permeability and layering are such that no significant drainage occurs in the SPT determination. In practice, the blowcount need not be exclusively a function of drained or undrained strength, but a full range of intermediate cases is possible. For example, the soil may behave undrained during penetration under each blow, but the soil below the tip of the spoon may actually densify as pore pressures caused by the previous blow dissipate prior to the next blow. Thus each successive blow may, in some case, test a soil in a denser state than before the SPT testing. A further complication in the use of the SPT to estimate S_{us} is that the resistance encountered by the spoon is not only a function of steady state strength (drained, undrained, or intermediate, as the case might be) but also

[1]References are listed in an attachment to this letter.

of the corresponding peak strength. Furthermore, there are numerous test errors that occur when blowcounts are measured. The charts in Figs. 55 and 80 are based on blowcounts measured at many sites under a generally unknown degree of care.

Blowcounts determined in the downstream shell of the Lower San Fernando Dam in 1985 were about the same as those determined prior to and soon after the 1971 earthquake (Castro and Keller 1988). The 1985 determinations were made above the water level, while those in 1971 were made below the water level. The fact that a liquefaction failure took place in the upstream shell, indicate that the S_{us} values in the hydraulic shell were about one half to one third of the drained strength. Since differences in saturation of the soil did not result in a measurable difference in blowcounts, one can conclude that in both cases the blowcounts reflect the drained strength rather than the undrained strength, which is the strength relevant to the liquefaction slide. The Lower San Fernando case is by far the best-documented case on which the empirical chart is based. The degree of drainage that may have occurred in the SPT determinations for the other case histories is unknown, even though, in our opinion, a fully undrained SPT test on sand would be rare.

In summary, the empirical correlation in Fig. 80 of the report should be used only as a rough guide for S_{us} values but should not be the basis for decisions relative to the seismic safety of important structures, such as Barkley Dam. Site-specific direct determinations of S_{us} must be used in such an evaluation as discussed later in this letter.

2. Blowcounts and Cone Penetration Resistance at Barkley Dam

The foundation soils have been separated into three major strata, designated as Units 1, 2, and 3 from the ground surface downward.

Unit 1 consists primarily of low to moderate plasticity clays and silty clay found generally above El. 320. Standard penetration test (SPT) blowcounts in Unit 1 were generally in a range of 8 to 20 blows/ foot with a few blowcounts as high as 40 blows/ foot.

Unit 2 consists of a highly stratified sequence of layers of low plasticity silty clay, moderately plastic clays, and layers and lenses of silty fine and fine to medium sand. SPT blowcounts in Unit 2 generally ranged from 3 to 12 blows/foot with a few isolated blowcounts of 1 and 2 blows/foot. Unit 2 is found approximately between El. 305 and 320.

Unit 3 consists of medium to dense silty fine and fine to medium sands near the top grading to gravelly sands near the base. SPT blowcounts in Unit 3 generally range from 20 to 60 blows/foot, with a few isolated higher and lower values.

The soils in Units 1 and 3 are considered to be not susceptible to liquefaction because of their clayey nature and of their high blowcounts, respectively. Unit 2 is the stratum of greater interest in evaluating the seismic stability of the dam because of the low blow-counts measured and the presence of sand layers, and thus subsequent comments relate to this unit.

The Unit 2 soils consist of clay and silty sand layers and are of alluvial origin. The sand layers account for about 5 to 35 percent of the total thickness of Unit 2 soils, as disclosed by an examination of the logs of continuous sample borings of the BEQ series, numbers 7, 16, 20, 22, 26, and 28. The thickness of the sand layers ranges generally from an inch to one foot in thickness. Layers in excess of one foot in thickness but under 2 feet were detected in the area of Sta 5+00B and Sta 34+70L, designated as Test Area A in this letter. This is the area where undisturbed sampling for steady state strength determinations were obtained. In this area, a total of three SPT borings, three undisturbed sample borings, and nine cone penetration soundings were made within an area of about 50 feet by 50 feet. In spite of the close proximity of these borings, it was generally not possible to correlate the sand layers from one boring to the next. This observation is in agreement with a description made by the USCE of an exposure of Unit 2 downstream of the dam along the riverbank, which revealed that the sand layers are often discontinuous and vary substantially in thickness.

Each SPT sample in Unit 2 penetrated generally through both clay and sand layers. The blowcount interpretation thus presented special problems to the WES analysis, which were addressed by defining an "equivalent sand SPT blowcount."

The actual blowcount N was first corrected for confining pressure and energy applied to the spoon, and a value referred to as N_1 was obtained. Then the blowcount N_1 was assumed to be representative of a soil with index properties equal to the average of those determined in the different sections of the SPT sample, except that when recovery was not 100 percent, the lost sample was assumed to be sand (with either 5 or 12 percent fines, depending on the measured blowcount). The average index properties were then used to apply a further correction to the blowcount ranging from 0 for a clean sand to a maximum of 7.5 blows per foot for a sand with 35 percent fines, and the resulting blowcount was designated N_{1c} and defined as the "equivalent sand blowcount" (ESB). The ESB was then used to: (1) predict whether 100 percent pore pressure buildup would occur based on the empirical blowcount chart in Fig. 55 of Vol. 1 of the report and (2) estimate S_{us} values using Fig. 80 of the report.

In order to understand the implications of the WES procedure, let's consider a typical situation in Unit 2 in which an SPT spoon encounters the following sequence of layers:

0 - 6 inches	Sandy Clay, 60% fines
6 - 12 inches	Silty Sand, 20% fines
12 - 18 inches	Sandy Clay, 60% fines

The measured blowcount is 5. Note that the soil stratification of this example is typical of Unit 2 in terms of blowcount, properties of the clay and sands, percentage of sand and clay layers, and thickness of sand layers. The example is, however, a simplification because often the individual layers contain thin streaks of other materials.

After correction for hammer energy and overburden stress (assuming a boring in the area immediately downstream of the switchyard), the blowcount of 5 is reduced to an N_1 value of 3. Using the average percent fines of the spoon of 47 percent, a correction of 7.5 is then applied to N_1, resulting in a value of N_{1c} of 10.5. Entering the chart in Fig. 80, a value of "residual strength" of about 300 psf is obtained.

From many similar computations, WES concluded that their best estimate of S_{us} for the Unit 2 soils was 450 psf in the switchyard area and 700 psf for the main dam. These strength values were then used in stability analyses to determine the potential for a liquefaction slide.

We believe that strengths determined in this manner do not properly represent the strengths of the clay nor of the sand layers applicable to a liquefaction slide analysis.

Test results on the clay, presented in the GEI report of July 1985, indicate average values of LL = 29, PI = 12, and a water content of 24 percent. Note that in testing for plasticity, one cannot avoid mixing the clay with thin partings of sand. The effect of mixing is to lower the liquid limit and plasticity index. Since the water content of the sand layers is similar to the clay in Unit 2, mixing of layers causes an increase in the computed liquidity index. Peak undrained strengths, S_{up}, of the clay measured with a laboratory vane in undisturbed samples from Unit 2 averaged 1,000 psf with a range of 760 to 1,540 psf. The average undrained steady state strength of the clay averaged about 230 psf, and thus a medium sensitivity of about 4 was obtained. Based on the effective vertical stresses where the samples were taken (downstream of the switchyard), the clay, if normally consolidated, would have a peak undrained strength of 300 to 400 psf; thus the Unit 2 clay at the location of sampling is overconsolidated with an OCR about 3.

The strains that are required to reach steady state strength in a clay with the characteristics described above are very large, and thus the strength applicable to an analysis of the potential for a seismically induced liquefaction slide is the peak strength, i.e., about 1,000 psf. Thus the strength estimated from blowcounts using the WES procedure of 300 psf underestimates the applicable strength of the clay by a factor of about 3.

The next question is whether the strength computed using the blowcount WES procedure represents the undrained steady state strength, S_{us}, of the sand. In our opinion, it does not because the blowcounts principally reflect the penetration resistance of the clay and are influenced to only a slight degree by the presence of the sand layers, as discussed below.

Interpretation of CPT Logs

A typical CPT log is shown in Fig. 1. The cone penetration resistance in Unit 2 is characterized by numerous peaks superimposed on a base value of about 5 to 10 tsf. Soil classifications shown in the cone penetration test report indicate that the base value represents the clay while the peaks correspond to the sand layers. I concur with this interpretation for the following reasons:

1. The peak values of point resistance correspond to lows in the friction ratio, see Fig. 1. The peak values and the corresponding friction ratios are plotted in the chart in Fig. 2. They fall in the zone of silty sand to sandy silts, while the base values of cone resistance and friction ratio plot in the zone of clayey silt and clays.

2. The measurements of electrical conductivity indicate lower conductivity at the locations of peaks in the cone penetration plot, see Fig. 3. This observation is in agreement with the fact than sands generally have lower electrical conductivity than clays.

3. Laboratory vane shear strength tests were performed in clay layers from undisturbed samples of Unit 2 (see GEI report of April 17, 1985). The peak undrained strength, S_{up}, ranged from 760 to 1,540 psf with an average of 1,000 psf. The base cone resistance, q_c, from cone soundings located about 5 to 8 feet from the corresponding undisturbed sample boring ranged from 5 to 10 tsf with an average of 7.5 tsf. The average ratio of q_c/S_{up} is thus 7.5/0.5 = 15, which is typical of published data for clays.

Practically all the peaks of the cone resistance in the sand layers are essentially triangular. A typical peak plotted to an expanded scale is shown in Fig. 4. The resistance increases rapidly and approximately linearly as the cone enters the sand layer and then abruptly drops, again about linearly. The shape of the peak suggests that the sand layers are not sufficiently thick for the cone resistance to be representative only of the properties of the sand. Rather, the measured maximum cone resistance in the relatively thin sand layers is strongly influenced by the properties of the clay above and below the sand layer. This observation is in agreement with a Federal Highway Administration report (1978) which states that the minimum layer thickness need to develop the full value of q_c in a layer is equal to 15 times the cone tip diameter. The cone tip used at Barkley for most of the sounding had an area of 15 cm^2 (diameter of 4.4 cm). Thus 15 times the diameter is about equal to 60 cm (2 feet) which corresponds to the maximum sand layer thickness in Unit 2 at Barkley Dam.

The peaks can be analyzed by defining a width of the peak as shown in Fig. 4 as the distance between the beginning and the end of the increased cone resistance. This distance is probably equal to or slightly larger than the thickness of the layer. Cone penetration data from Test Area A are presented in Fig. 5 as a plot of the peak values of cone penetration in Unit 2 versus the width of the peaks. The plot shows that larger peaks correspond to thicker layers with a relatively narrow range for peak resistance for layers thinner than about one foot. This result indicates that the peak cone value for layers thinner than about one foot is primarily a function of the thickness of the layer and the strength of the clay and to a lesser degree on the denseness of the sand. At thicknesses over about one foot, the scatter increases, indicating a larger effect of the density of the sand on the peak cone resistance. Thus the "true" cone penetration resistance in the switchyard area is of about 60 to 100 tsf.

A comparison is presented in Fig. 6 between cone penetration data corresponding to Test Area A, where the undisturbed samples for steady state strength determinations were obtained, with cone data selected to be representative of other areas of the site. This comparison indicates that in Test Area A, the cone penetration resistance is slightly lower than in other areas of the site. Since the penetration resistance in the clay of Unit 2 is about the same for all the cone soundings that were compared, the slightly lower values of peak cone penetration in the Test Area A for the same layer thickness must reflect slightly lower sand densities.

Comparison of CPT and SPT Logs

A comparison of the SPT and CPT data from borings within 5 to 10 feet of each other is shown in Figs. 7 to 10. The SPT and CPT correlate well in Unit 1 and in thick clay zones of Unit 2, indicating a gradual decrease of the strength of the clay with depth within Units 1 and 2. However, the SPT does not reflect the presence of the sand layers in Unit 2 disclosed by the CPT logs. The sand layers are too thin for the SPT to reflect the sand properties. Schmertmann has indicated that for cone friction ratios of 2 to 4 (typical of Unit 2), more than 50 percent of the SPT blowcount resistance is derived from the frictional resistance along the outside of the spoon. Thus, if the tip of the spoon is in a sand layer but a significant length of the spoon length is in clay, the blowcount is mostly due to the clay. Furthermore, similar to the cone penetration resistance, even the tip resistance of the SPT spoon would be influenced strongly by the strength of the clay above and below the sand layers.

The cone penetration log does reflect the presence of the sand layers; however, even the peak penetration resistance developed in each layer underestimates, in most cases, the "true" penetration resistance of the sand. Figures 5 and 6 show that full development of the penetration resistance of the sand requires a layer thickness of about 1.5 feet. Note that the width of the peak plotted in Figs. 5 and 6 is somewhat larger than the actual thickness of the layer.

The data in Fig. 5 indicate that the "true" (fully developed) cone penetration resistance of the sand is about 60 to 100 tsf in Test Area A, which was identified as the weakest area of the Unit 2 soils. Data from other areas, Fig. 6, indicate a "true" penetration resistance of 100 tsf or more. Thus the corresponding "true" SPT values would be about 18 in Test Area A and about at least 25 elsewhere if one uses a ratio of cone to SPT of 4. These N-values compare with values on the order of 5 used in the WES analyses.

The cone data were analyzed by WES to obtain equivalent sand blowcounts to enter the empirical charts of Figs. 55 and 80 of their report. To facilitate computations, the chart in Fig. 55 was combined with a chart for classifying soils from cone point and sleeve resistances. In their report WES recognizes that even the peaks in the cone log may be lower than the true penetration resistance of the sand because the sand layers are not thick enough to develop the true cone resistance of the sand. However, WES notes that there is a compensating error in that the cone soil classification chart will indicate a soil that is finer than it actually is. Thus the correction for fines would be larger, and even though the peak cone resistance is too low, the ESB would be approximately correct. However, no analysis is presented to corroborate the assumption that the compensating errors are of similar magnitude.

An examination of Figs. 5 and 6 reveals that for sand layers causing a spike in the cone record with a width of 1 foot or less, the peak cone resistance would be lower than the true resistance (judged to correspond to the thickest layers) by 20 to 55 tsf in Fig. 5 and by 40 to 80 tsf in Fig. 6.

The maximum correction for fines in the WES method is 7.5. The additional correction due to misclassification of the soil in the sand layers is probably of 4 blows per foot or less, since the actual soil is a silty sand, already the subject of some blowcount correction. A blowcount correction of 4 is roughly equivalent to a difference in cone penetration resistance of about 16, i.e., substantially less than the difference between the measured cone penetration and the true resistance of the sand. Thus the two errors are not compensated in the WES analysis, which leads to severe underestimation of the undrained steady state strength of the sand layers in Unit 2.

In summary, the SPT logs do not even reflect the presence of the sand layers. The CPT logs do indicate the presence of the sand layers but substantially underestimate its true penetration resistance. Thus the use of blowcounts or cone penetration resistance to estimate the properties of the sand layers is inappropriate.

3. Direct Determinations of Undrained Steady State Strength (S_{us})

Determinations of S_{us} were made by GEI by means of tests on undisturbed samples obtained from Unit 2. Tests were performed only on sand layers, and the results are presented in the GEI report dated July 19, 1985, which is included as Appendix D in Vol. 3 of the WES report. A conservative analysis of the test results lead to a recommendation of using a value of S_{us} in the Unit 2 sand layers of 1,100 psf (8 psi) as compared to the WES value of about 450 psf. We believe that the value of 1,100 psf is appropriately conservative for analyzing the seismic stability of Barkey Dam.

Subsequent to our determinations of S_{us} values at Barkley, we performed a re-evaluation of the soils in the hydraulic shell of the Lower San Fernando (LSF) Dam using the same methodology as used for Barkley. The S_{us} determinations at LSF Dam were in agreement with the two key observations of the dam in the 1971 earthquake, namely, there was an upstream liquefaction failure, but no failure occurred in the downstream direction. The results of the investigation were also in agreement with other known characteristics of the failure, such as the fact that it was triggered by the 1971 earthquake but not by earlier events. Thus the methodology used at Barkley for direct determination of S_{us} was corroborated by the most important case history presently available to the profession.

4. Strength Parameters for Stability Analyses

Seismically induced pore pressures were estimated in the WES report using the empirical "liquefaction" chart. For sand layers dense enough so that the estimated pore pressures were less than 100 percent pore pressure, the soil strength was estimated as having been reduced proportional with the pore pressure, e.g., if the seismically induced pore pressure was estimated to be 40 percent, the strength was reduced by 40 percent. We surmise that the intention is to use this reduced strength as representing the undrained strength available for post-earthquake stability.

The undrained strength of the soil is a function of the effective stress during the failure, not the effective stress at other stages of shear, e.g., initial or any intermediate stage prior to reaching failure, such as a reduced effective stress value caused by an instantaneous pore pressure that may develop as a result of seismic shaking. As the dam tends to deform during or after the earthquake, important pore pressure and effective stress changes would take place. For loose sands, the pore pressure may increase further and for dense sands dilation would cause a decrease in pore pressure. Ultimately the available undrained strength will be a function of the effective stress at failure which is a function of the void ratio and not of the initial or other intermediate value of effective stress. In general, the undrained strength that is actually available may be higher or lower than those estimated by WES, and it will be bear no relationship to the WES values. In the case of Barkley Dam, the soils for which WES used this method to estimate strength are dense enough to be dilative, and thus the WES values are conservative. However, there may be cases where the WES method may lead to unconservative results.

We hope that the comments presented in this letter will serve to arrive at a better understanding of the proper methodology for analyzing the seismic safety of embankment dams.

Very truly yours,

GEI CONSULTANTS, INC.

Gonzalo Castro

Gonzalo Castro, Ph.D., P.E.
Principal

GC:ck

Enclosures

cc: E. Pritchett
G. Franklin

REFERENCES

[1] Castro G. (1987). "On the Behavior of Soils During Earthquakes - Liquefaction," *Third International Conference on Soil Dynamics and Earthquake Engineering*, June, Princeton University.

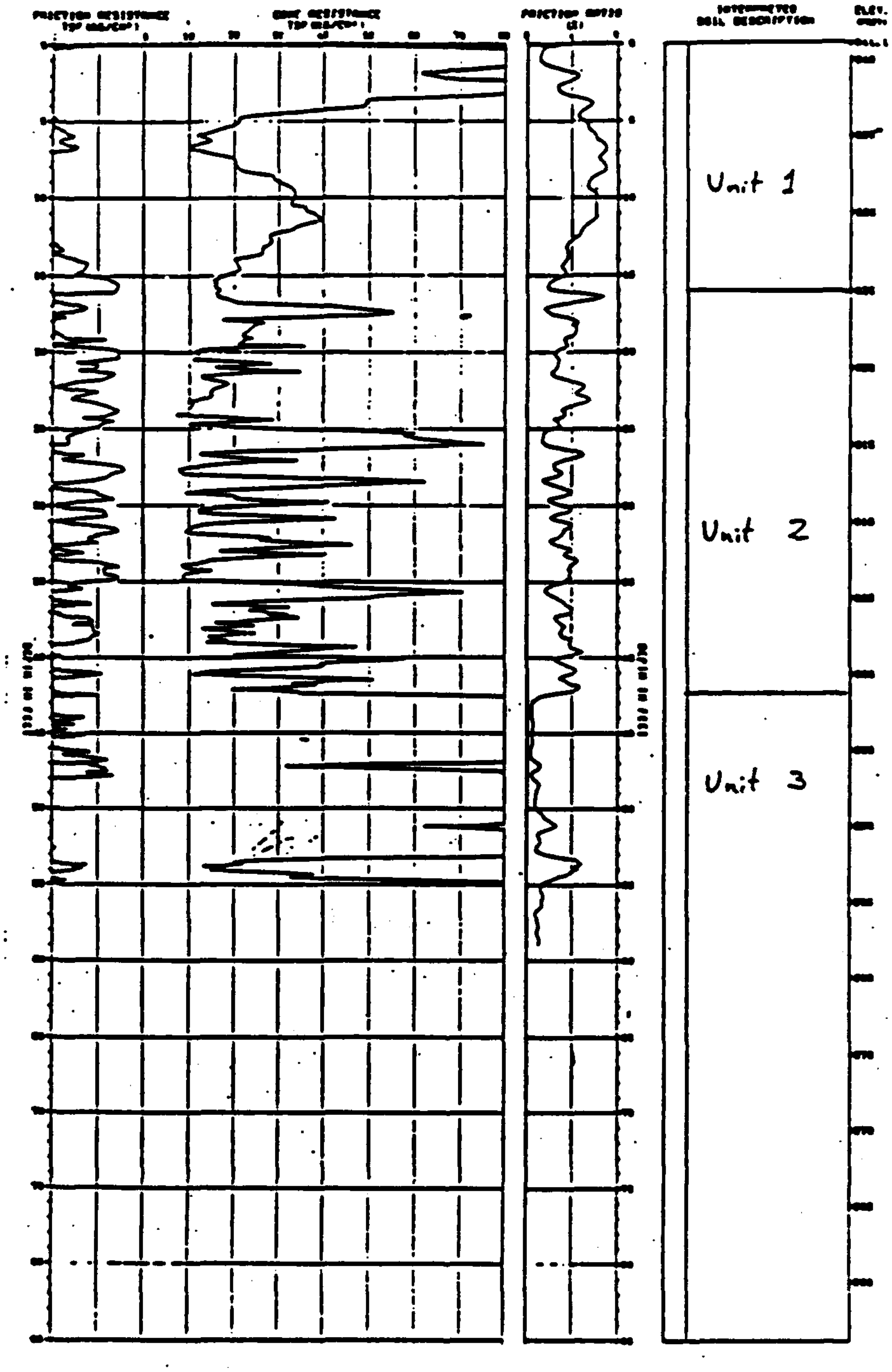
[2] Castro, G. and Keller, T. O. (1988). "Re-evaluation of Lower San Fernando Dam," GEI report to Department of the Army, Corps of Engineers, February.

[3] Federal Highway Administration, Office of Research and Development, (1978), "Guidelines for Cone Penetration Test - Performance and Design, Report FHWA TS-78-209, July.

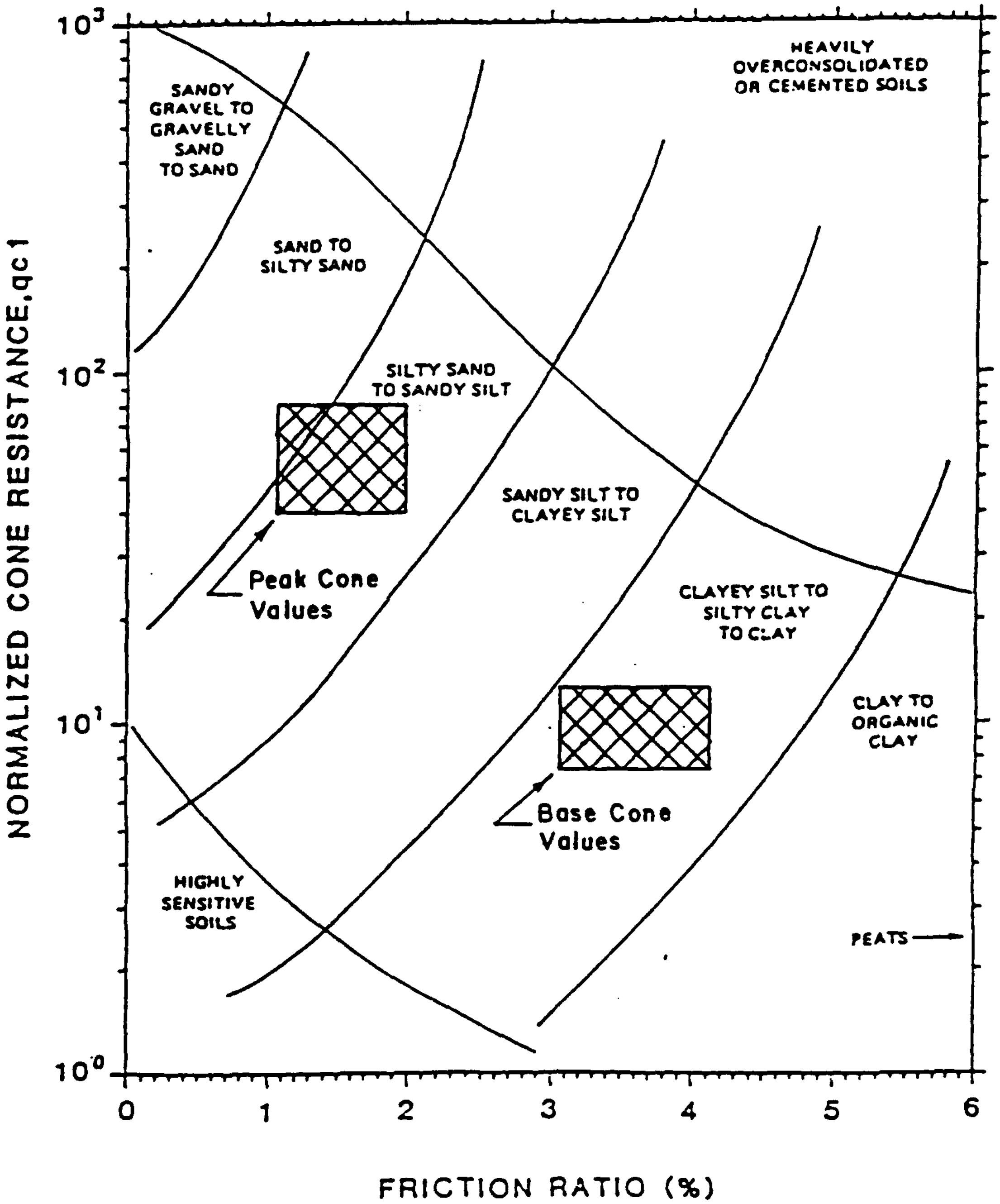
[4] Keller, T. O.; Castro, G., and Rogers, J. H. (1987). "Steel Creek Dam Foundation Densification," *Proceedings of Soil Improvement - A 10-Yr Update*, ASCE, Atlantic City, April.

[5] Schmertmann, John H., (1979), "Statics of SPT," Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT5, Proc. Paper 14573, May, pp. 655-670.

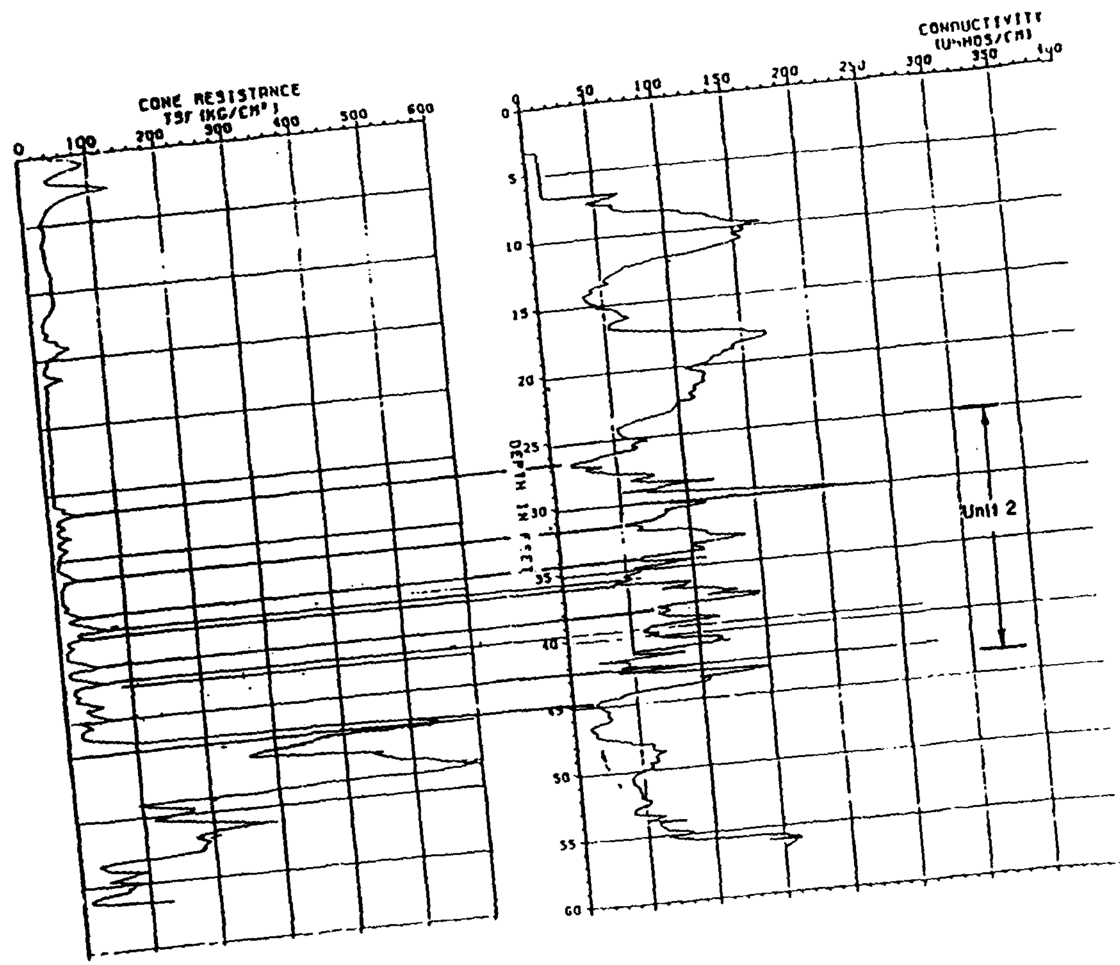
[6] Seed, H. B. (1987), "Design Problems in Soil Liquefaction," ASCE, *Journal of Geotechnical Engineering*, Vol. 113, No. 8, August.



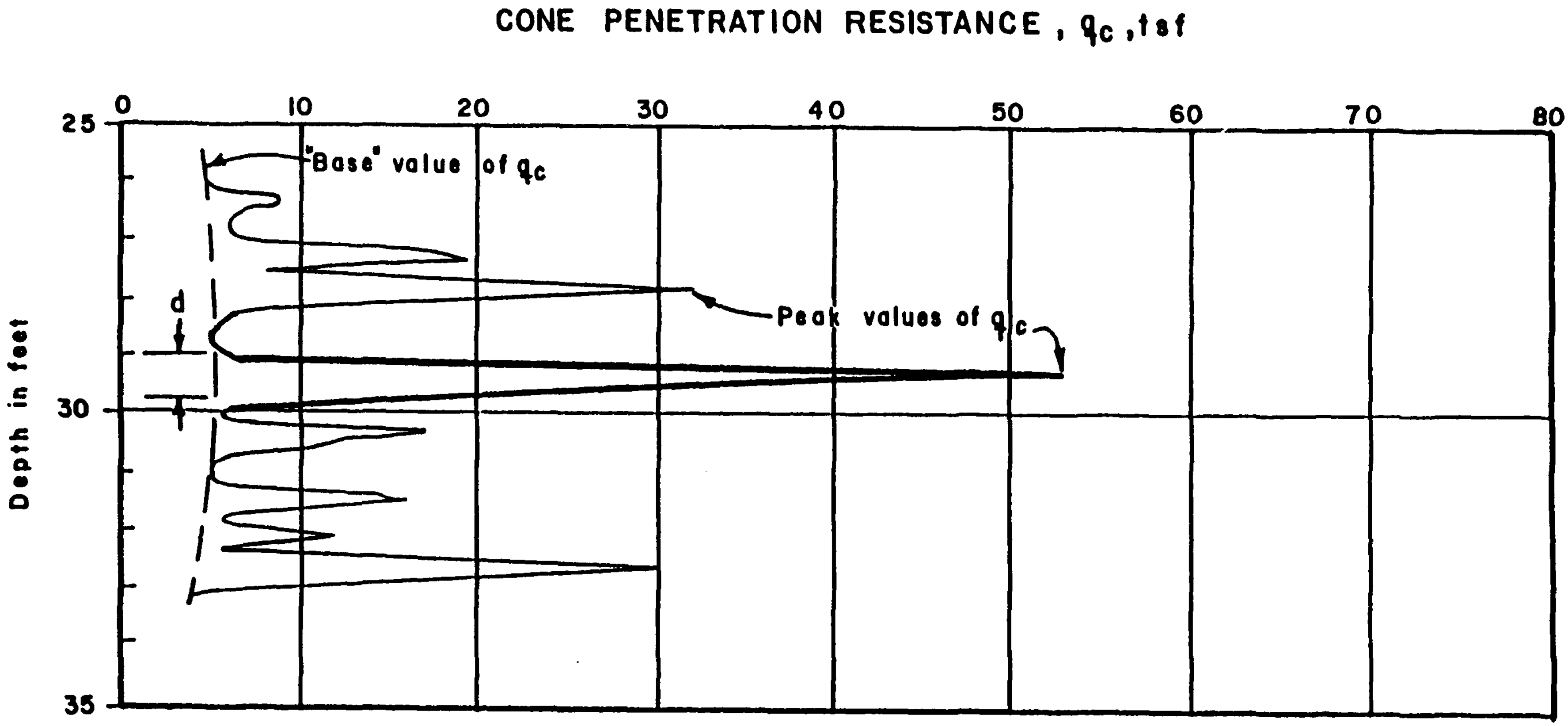
U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	CPT 57
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 1



U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	CONE PENETRATION VALUES UNIT 2 CPT 57
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 2



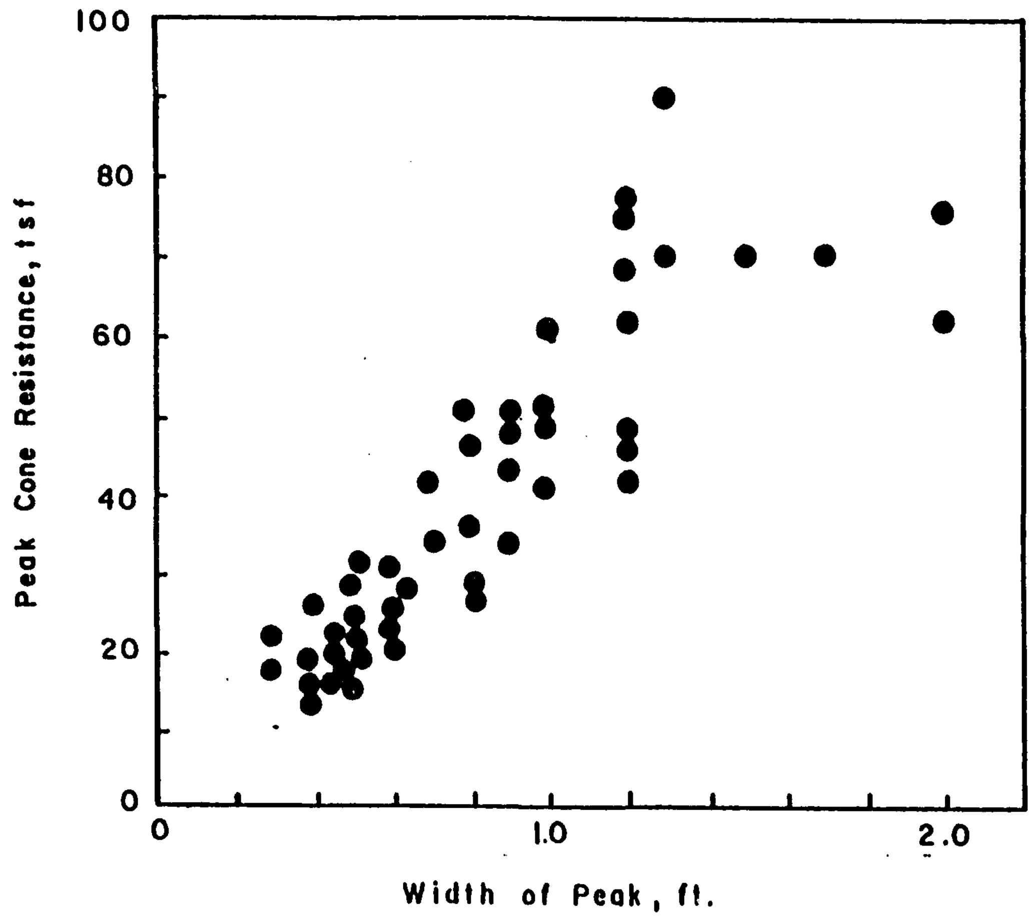
U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	CONDUCTIVITY DATA CPT 54
GEI Consultants, Inc. WINCHESTER · MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 3



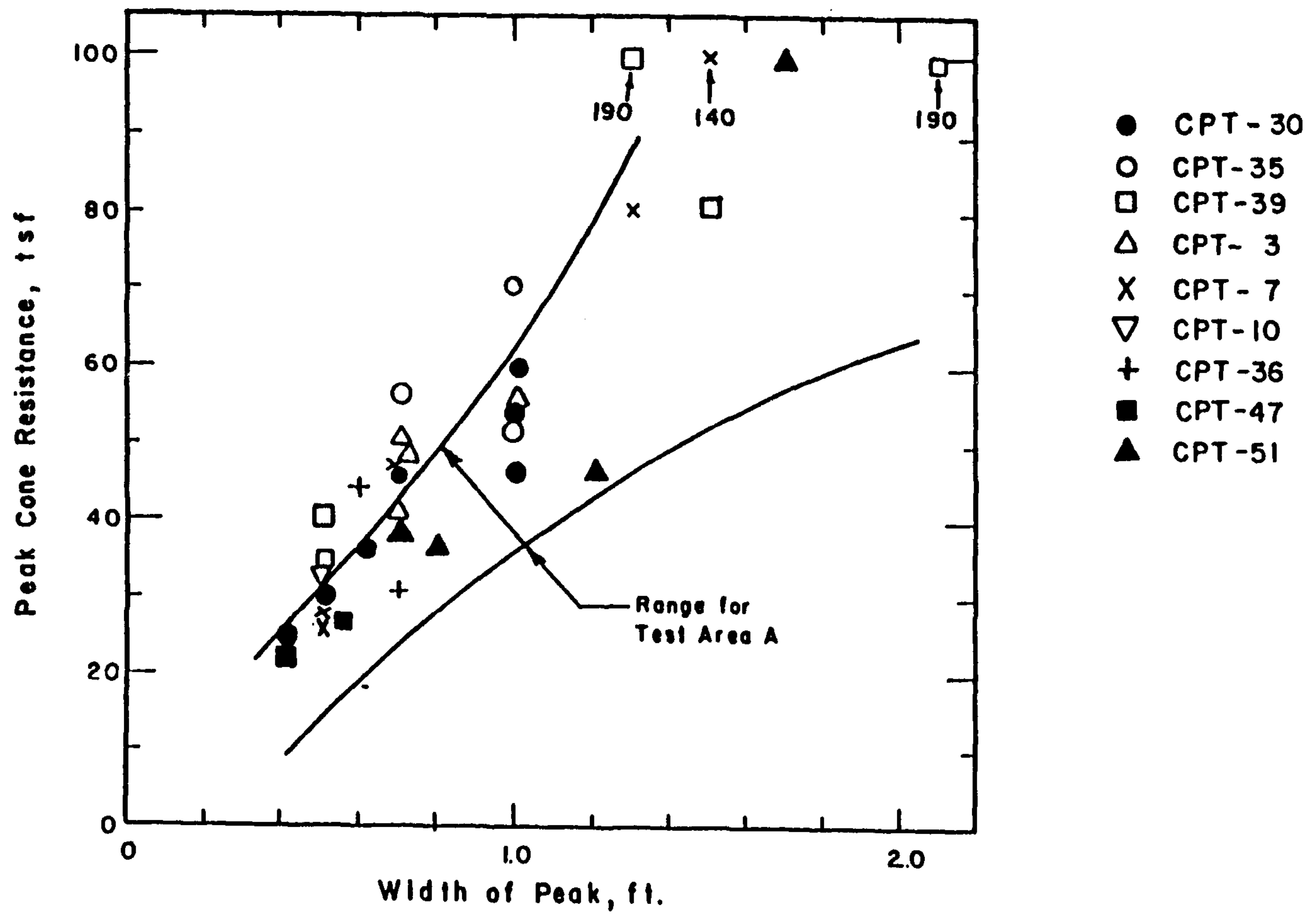
EXPANDED PARTIAL LOG OF CPT 58

d = width of peak

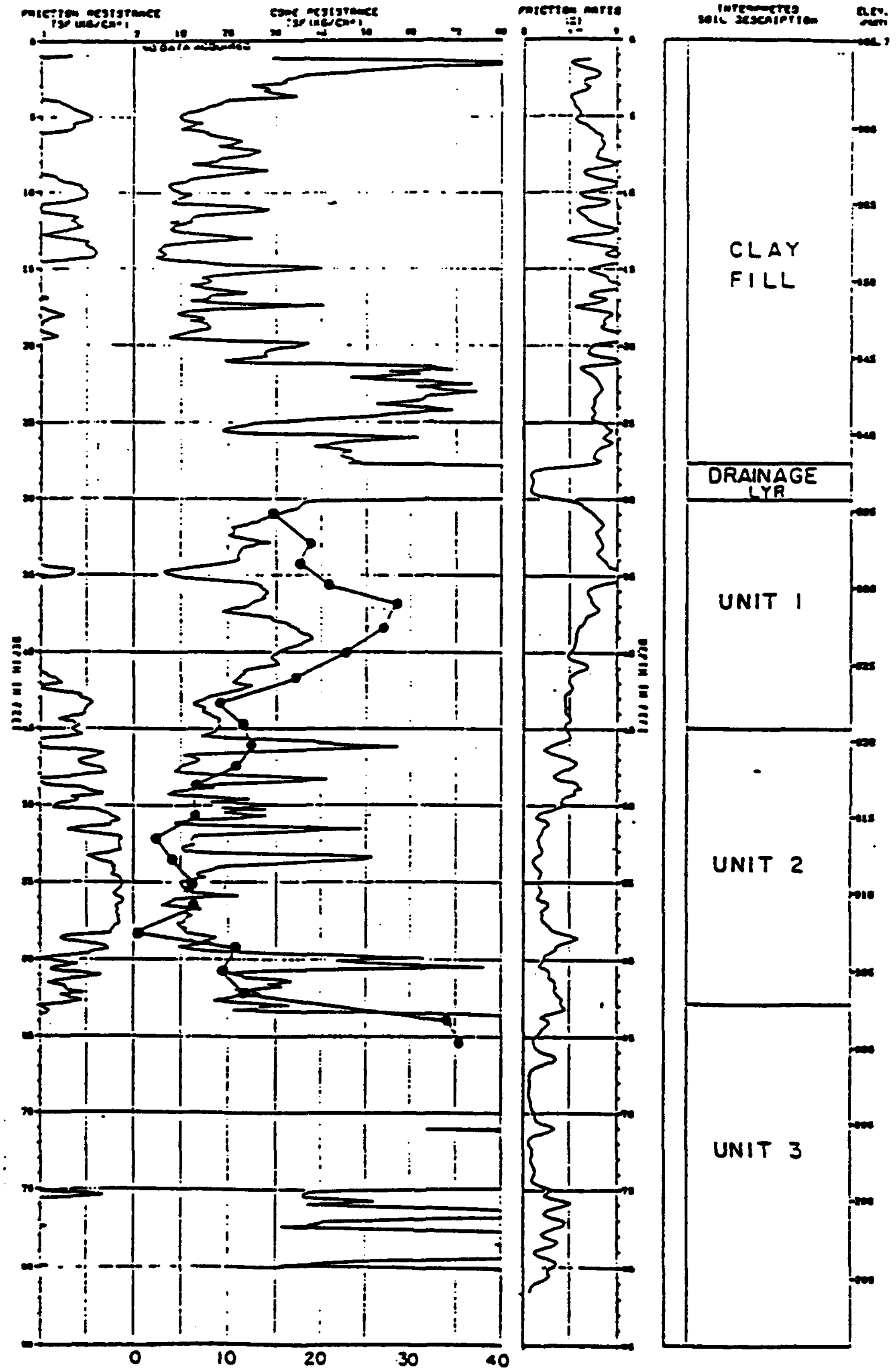
U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	EXPANDED PLOT CPT 58
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 4



U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	PEAK CONE RESISTANCE UNIT 2 SANDS TEST AREA A
GEI Consultants, Inc.	Project 85836	June 28, 1991 Fig. 5

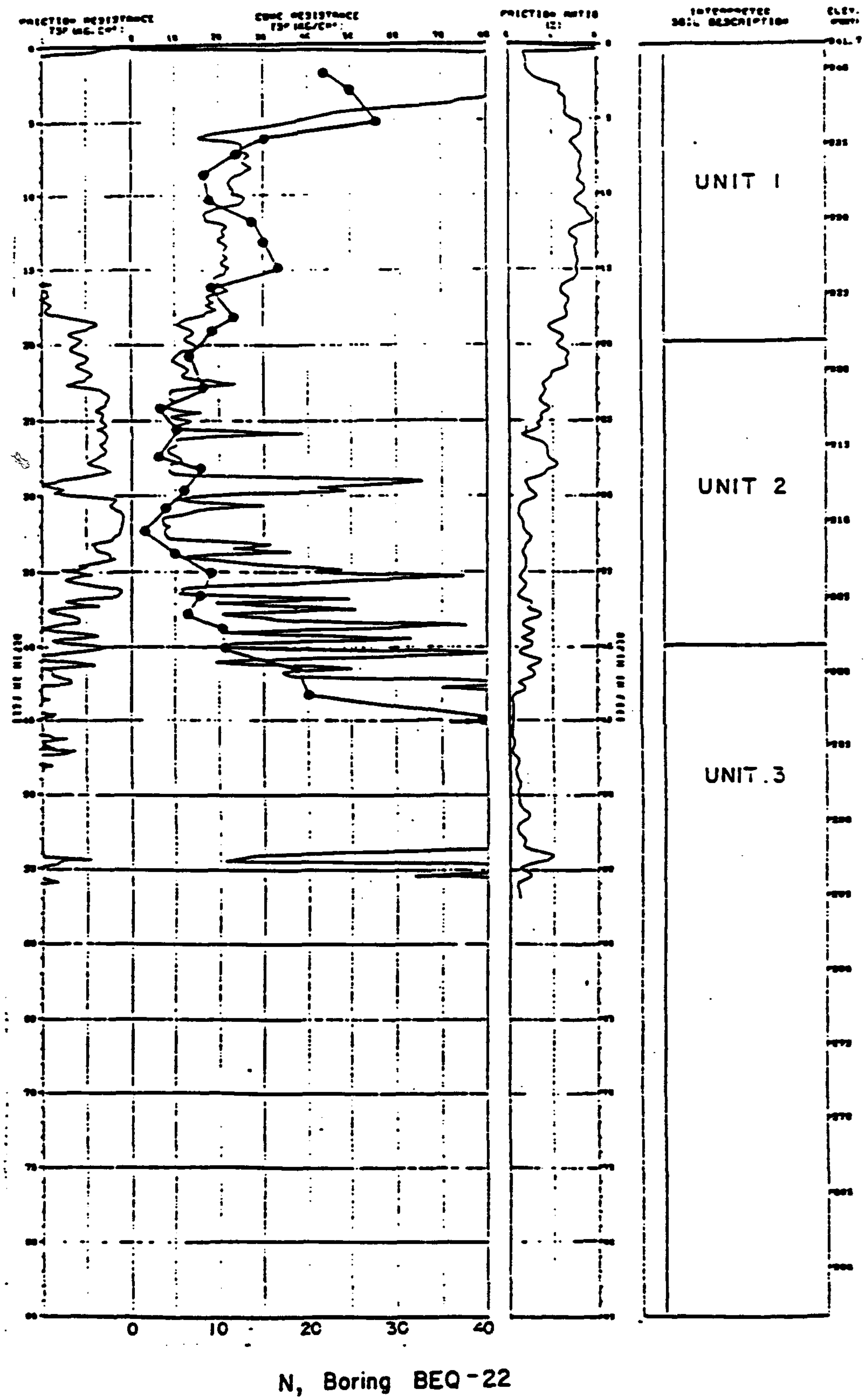


U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	PEAK CONE RESISTANCE UNIT 2 SANDS
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 6

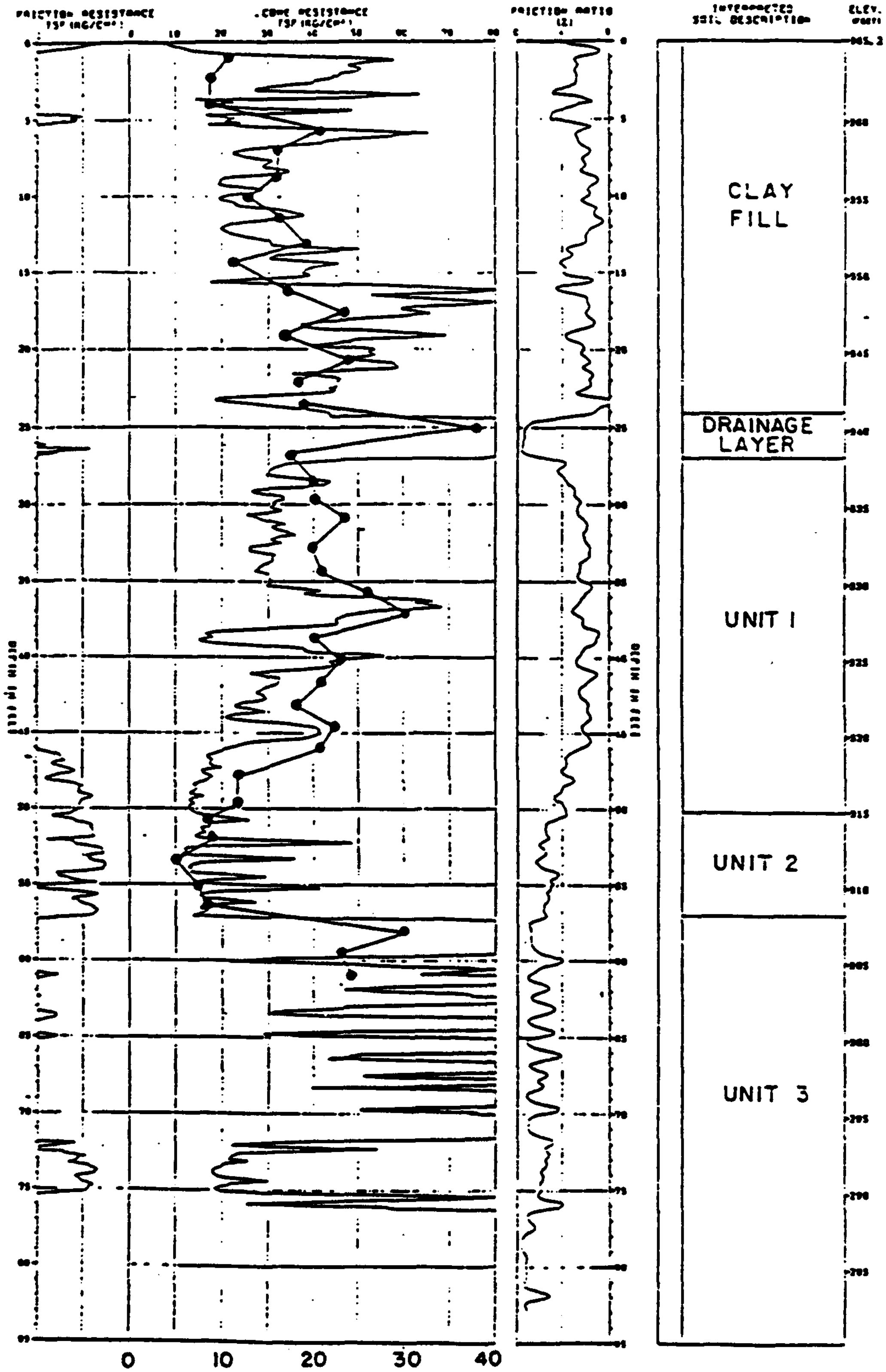


N, Boring BEQ-16

U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	COMPARISON OF CPT 3 WITH BLOWCOUNT DATA
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 7

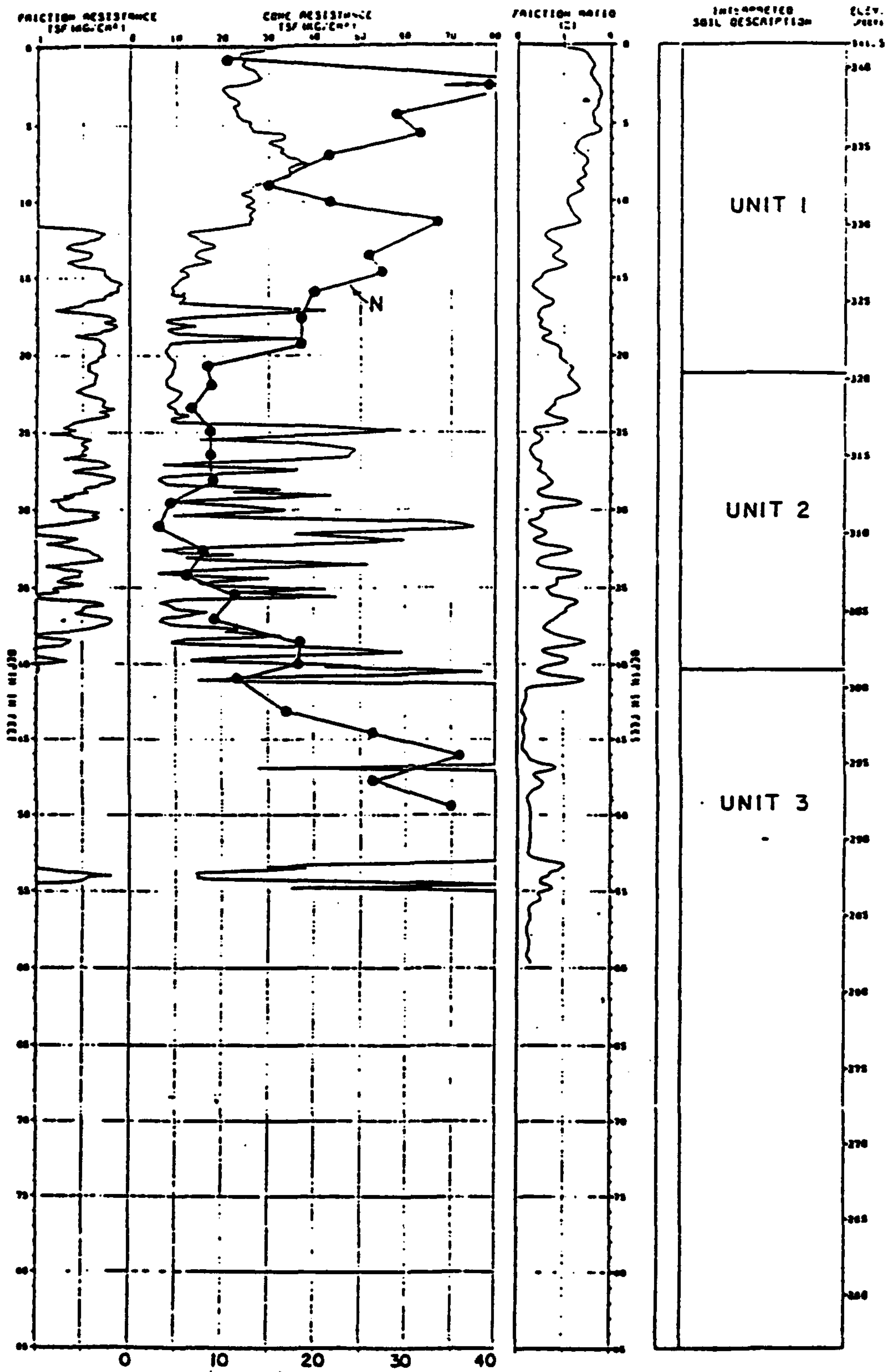


U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	COMPARISON OF CPT 41 WITH BLOWCOUNT DATA
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 8



N, Boring BEQ-20

U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	COMPARISON OF CPT 11 WITH BLOWCOUNT DATA
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 9



N, Boring BEQ-7

U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	COMPARISON OF CPT 25 WITH BLOWCOUNT DATA
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 10

GEI Consultants, Inc.

1021 Main Street
Winchester, MA 01890-1943
617•721•4000

June 28, 1991
Project 85836

Mr. Paul D. Robinson
Chief, Engineering Division
Department of the Army
Corps of Engineers
P.O. Box 1070
Nashville, TN 37202-1070

Dear Mr. Robinson:

Re: **Seismic Stability Evaluation**
Barkley Dam

We have received Vols. 1, 4, and 5 of the subject seismic evaluation prepared by the Waterways Experiment Station (WES).

The information contained in these reports had been presented in a meeting with the consultants on January 24, 1986 in which it was concluded that no remedial measures are required to enhance the seismic stability of the dam. Our concurrence with this conclusion was documented in our letter of February 24, 1986, in which we presented a brief description of our rationale for assessing the seismic behavior of the dam. We feel that the conclusion that no remedial measures are needed is still valid. The results of the analyses presented in the WES reports led to the same conclusion.

The evaluation of the seismic stability of Barkley Dam constituted a major engineering effort, the results of which are likely to be used by others in future evaluations of other Corps of Engineers' dams as well as of dams of other government and private organizations. Yet we feel that the methodology used could, in other cases, lead to invalid conclusions relative to the seismic safety of the dams. Therefore, we believe it important to present in some detail our disagreements with some of the aspects of the methodology used by WES.

In brief, the methodology in the WES report uses blowcounts, either measured or estimated from cone penetration logs, to determine: (a) pore pressure generation in various zones of the foundation (referred to as liquefaction potential) and (b) post-earthquake strengths of the foundation soils. Both determinations are based on empirical charts. We disagree with the methodology in three general areas, namely (1) on the conclusions drawn from empirical charts based on blowcounts, (2) on the significance of the blowcounts for the particular conditions of the foundation soils at Barkley, and (3) on the use of estimated pore pressure increases in stability evaluations.

1. Empirical Charts

Two empirical charts were used in the WES study: (1) a chart that relates manifestations of pore pressure increases in level ground sand deposits to blowcounts and intensity of earthquake shaking, Fig. 55 of Vol. 1 of the report, and (2) a correlation between blowcounts and residual (post-earthquake) strength, Fig. 80 in Vol. 1 of the report.

"Liquefaction" Chart

In the chart in Fig. 55 of Vol. 1, the cases classified as liquefaction represent instances where an earthquake caused sufficiently high pore pressures for sand boils to develop. Under level ground, high pore pressures are followed by reconsolidation and settlement, but unless heavy structures are present, no shear deformations of significance occur. The relevance of pore pressure increase predictions to the behavior of Barkley Dam is questionable. The use of the blowcount chart to predict pore pressure leads only to the conclusion that the pore pressure will build up. But this result provides no information on the likelihood of a flow (liquefaction) slide due to earthquake shaking or to the deformations that would occur during such an event.

The potential for a liquefaction slide (as in the case of the Lower San Fernando Dam) is present if the value of the driving shear stress exceeds the undrained steady state strength, S_{us}, of the soils in question. The S_{us} values are only a function of the void ratio at which the soil is found *in situ* and not on how much the pore pressure increases when the soil is shaken by an earthquake. If the dam is not subject to a liquefaction slide, i.e., it is inherently stable, seismically induced deformations need to be evaluated. Again, in this case, there is no correlation between these deformations and the pore pressure predictions under level ground.

The above points have been clearly illustrated by the results of laboratory tests (see for example, Castro 1987)[1] and more recently in centrifuge experiments. Model embankments on an instrumented sand foundation have been shaken in the centrifuge. Essentially 100 percent pore pressures were measured throughout the foundations; however, no liquefaction slide occurred, and the permanent deformations of the model embankments ranged from insignificant to potentially damaging to the prototype, even though in all cases the pore pressures were similarly high.

Thus, in summary, pore pressure predictions made on the basis of Fig. 55 in Vol. 1 bear no relationship to the potential for a liquefaction slide or for limited deformations of Barkley Dam as a result of earthquake shaking.

S_{us} Chart

The second empirical chart is presented in Fig. 80 of Vol. 1 of the report and relates blowcounts to the "residual" strength as obtained from analyses of past failures, Seed (1987). In the context of this letter, the terms residual, post-earthquake, and undrained steady state refer to the same strength, S_{us}, which is the strength that would be available to resist an undrained (liquefaction) slide. For contractive soils, it is appropriate to use S_{us} also as the yield strength when estimating potential deformations using a Newmark type of approach. Thus the determination of representative values of S_{us} is crucial to the evaluation of both the seismic stability and the deformations of Barkley Dam and its foundation due to earthquake shaking.

Even though site-specific correlations between S_{us} and blowcount (or cone penetration resistance) have been developed relatively successfully for several sites, e.g., Keller et al. 1987, a universal correlation should be used only as a rough guide and not as a definitive evaluation tool. Two soils with the same S_{us} can have very different blowcounts depending on the degree of drainage that occurs during driving of the spoon and on other factors. For example, in the case of a soil that is highly contractive, its S_{us} value would be much lower than its drained strength, S_d. Thus, if drainage can occur during and after penetration for each blow, the blowcount would be much larger than if the soil permeability and layering are such that no significant drainage occurs in the SPT determination. In practice, the blowcount need not be exclusively a function of drained or undrained strength, but a full range of intermediate cases is possible. For example, the soil may behave undrained during penetration under each blow, but the soil below the tip of the spoon may actually densify as pore pressures caused by the previous blow dissipate prior to the next blow. Thus each successive blow may, in some case, test a soil in a denser state than before the SPT testing. A further complication in the use of the SPT to estimate S_{us} is that the resistance encountered by the spoon is not only a function of steady state strength (drained, undrained, or intermediate, as the case might be) but also

[1]References are listed in an attachment to this letter.

of the corresponding peak strength. Furthermore, there are numerous test errors that occur when blowcounts are measured. The charts in Figs. 55 and 80 are based on blowcounts measured at many sites under a generally unknown degree of care.

Blowcounts determined in the downstream shell of the Lower San Fernando Dam in 1985 were about the same as those determined prior to and soon after the 1971 earthquake (Castro and Keller 1988). The 1985 determinations were made above the water level, while those in 1971 were made below the water level. The fact that a liquefaction failure took place in the upstream shell, indicate that the S_{us} values in the hydraulic shell were about one half to one third of the drained strength. Since differences in saturation of the soil did not result in a measurable difference in blowcounts, one can conclude that in both cases the blowcounts reflect the drained strength rather than the undrained strength, which is the strength relevant to the liquefaction slide. The Lower San Fernando case is by far the best-documented case on which the empirical chart is based. The degree of drainage that may have occurred in the SPT determinations for the other case histories is unknown, even though, in our opinion, a fully undrained SPT test on sand would be rare.

In summary, the empirical correlation in Fig. 80 of the report should be used only as a rough guide for S_{us} values but should not be the basis for decisions relative to the seismic safety of important structures, such as Barkley Dam. Site-specific direct determinations of S_{us} must be used in such an evaluation as discussed later in this letter.

2. Blowcounts and Cone Penetration Resistance at Barkley Dam

The foundation soils have been separated into three major strata, designated as Units 1, 2, and 3 from the ground surface downward.

Unit 1 consists primarily of low to moderate plasticity clays and silty clay found generally above El. 320. Standard penetration test (SPT) blowcounts in Unit 1 were generally in a range of 8 to 20 blows/ foot with a few blowcounts as high as 40 blows/ foot.

Unit 2 consists of a highly stratified sequence of layers of low plasticity silty clay, moderately plastic clays, and layers and lenses of silty fine and fine to medium sand. SPT blowcounts in Unit 2 generally ranged from 3 to 12 blows/foot with a few isolated blowcounts of 1 and 2 blows/foot. Unit 2 is found approximately between El. 305 and 320.

Unit 3 consists of medium to dense silty fine and fine to medium sands near the top grading to gravelly sands near the base. SPT blowcounts in Unit 3 generally range from 20 to 60 blows/foot, with a few isolated higher and lower values.

The soils in Units 1 and 3 are considered to be not susceptible to liquefaction because of their clayey nature and of their high blowcounts, respectively. Unit 2 is the stratum of greater interest in evaluating the seismic stability of the dam because of the low blowcounts measured and the presence of sand layers, and thus subsequent comments relate to this unit.

The Unit 2 soils consist of clay and silty sand layers and are of alluvial origin. The sand layers account for about 5 to 35 percent of the total thickness of Unit 2 soils, as disclosed by an examination of the logs of continuous sample borings of the BEQ series, numbers 7, 16, 20, 22, 26, and 28. The thickness of the sand layers ranges generally from an inch to one foot in thickness. Layers in excess of one foot in thickness but under 2 feet were detected in the area of Sta 5+00B and Sta 34+70L, designated as Test Area A in this letter. This is the area where undisturbed sampling for steady state strength determinations were obtained. In this area, a total of three SPT borings, three undisturbed sample borings, and nine cone penetration soundings were made within an area of about 50 feet by 50 feet. In spite of the close proximity of these borings, it was generally not possible to correlate the sand layers from one boring to the next. This observation is in agreement with a description made by the USCE of an exposure of Unit 2 downstream of the dam along the riverbank, which revealed that the sand layers are often discontinuous and vary substantially in thickness.

Each SPT sample in Unit 2 penetrated generally through both clay and sand layers. The blowcount interpretation thus presented special problems to the WES analysis, which were addressed by defining an "equivalent sand SPT blowcount."

The actual blowcount N was first corrected for confining pressure and energy applied to the spoon, and a value referred to as N_1 was obtained. Then the blowcount N_1 was assumed to be representative of a soil with index properties equal to the average of those determined in the different sections of the SPT sample, except that when recovery was not 100 percent, the lost sample was assumed to be sand (with either 5 or 12 percent fines, depending on the measured blowcount). The average index properties were then used to apply a further correction to the blowcount ranging from 0 for a clean sand to a maximum of 7.5 blows per foot for a sand with 35 percent fines, and the resulting blowcount was designated N_{1c} and defined as the "equivalent sand blowcount" (ESB). The ESB was then used to: (1) predict whether 100 percent pore pressure buildup would occur based on the empirical blowcount chart in Fig. 55 of Vol. 1 of the report and (2) estimate S_{us} values using Fig. 80 of the report.

In order to understand the implications of the WES procedure, let's consider a typical situation in Unit 2 in which an SPT spoon encounters the following sequence of layers:

0 - 6 inches	Sandy Clay, 60% fines
6 - 12 inches	Silty Sand, 20% fines
12 - 18 inches	Sandy Clay, 60% fines

The measured blowcount is 5. Note that the soil stratification of this example is typical of Unit 2 in terms of blowcount, properties of the clay and sands, percentage of sand and clay layers, and thickness of sand layers. The example is, however, a simplification because often the individual layers contain thin streaks of other materials.

After correction for hammer energy and overburden stress (assuming a boring in the area immediately downstream of the switchyard), the blowcount of 5 is reduced to an N_1 value of 3. Using the average percent fines of the spoon of 47 percent, a correction of 7.5 is then applied to N_1, resulting in a value of N_{1c} of 10.5. Entering the chart in Fig. 80, a value of "residual strength" of about 300 psf is obtained.

From many similar computations, WES concluded that their best estimate of S_{us} for the Unit 2 soils was 450 psf in the switchyard area and 700 psf for the main dam. These strength values were then used in stability analyses to determine the potential for a liquefaction slide.

We believe that strengths determined in this manner do not properly represent the strengths of the clay nor of the sand layers applicable to a liquefaction slide analysis.

Test results on the clay, presented in the GEI report of July 1985, indicate average values of LL = 29, PI = 12, and a water content of 24 percent. Note that in testing for plasticity, one cannot avoid mixing the clay with thin partings of sand. The effect of mixing is to lower the liquid limit and plasticity index. Since the water content of the sand layers is similar to the clay in Unit 2, mixing of layers causes an increase in the computed liquidity index. Peak undrained strengths, S_{up}, of the clay measured with a laboratory vane in undisturbed samples from Unit 2 averaged 1,000 psf with a range of 760 to 1,540 psf. The average undrained steady state strength of the clay averaged about 230 psf, and thus a medium sensitivity of about 4 was obtained. Based on the effective vertical stresses where the samples were taken (downstream of the switchyard), the clay, if normally consolidated, would have a peak undrained strength of 300 to 400 psf; thus the Unit 2 clay at the location of sampling is overconsolidated with an OCR about 3.

The strains that are required to reach steady state strength in a clay with the characteristics described above are very large, and thus the strength applicable to an analysis of the potential for a seismically induced liquefaction slide is the peak strength, i.e., about 1,000 psf. Thus the strength estimated from blowcounts using the WES procedure of 300 psf underestimates the applicable strength of the clay by a factor of about 3.

The next question is whether the strength computed using the blowcount WES procedure represents the undrained steady state strength, S_{us}, of the sand. In our opinion, it does not because the blowcounts principally reflect the penetration resistance of the clay and are influenced to only a slight degree by the presence of the sand layers, as discussed below.

Interpretation of CPT Logs

A typical CPT log is shown in Fig. 1. The cone penetration resistance in Unit 2 is characterized by numerous peaks superimposed on a base value of about 5 to 10 tsf. Soil classifications shown in the cone penetration test report indicate that the base value represents the clay while the peaks correspond to the sand layers. I concur with this interpretation for the following reasons:

1. The peak values of point resistance correspond to lows in the friction ratio, see Fig. 1. The peak values and the corresponding friction ratios are plotted in the chart in Fig. 2. They fall in the zone of silty sand to sandy silts, while the base values of cone resistance and friction ratio plot in the zone of clayey silt and clays.

2. The measurements of electrical conductivity indicate lower conductivity at the locations of peaks in the cone penetration plot, see Fig. 3. This observation is in agreement with the fact than sands generally have lower electrical conductivity than clays.

3. Laboratory vane shear strength tests were performed in clay layers from undisturbed samples of Unit 2 (see GEI report of April 17, 1985). The peak undrained strength, S_{up}, ranged from 760 to 1,540 psf with an average of 1,000 psf. The base cone resistance, q_c, from cone soundings located about 5 to 8 feet from the corresponding undisturbed sample boring ranged from 5 to 10 tsf with an average of 7.5 tsf. The average ratio of q_c/S_{up} is thus 7.5/0.5 = 15, which is typical of published data for clays.

Practically all the peaks of the cone resistance in the sand layers are essentially triangular. A typical peak plotted to an expanded scale is shown in Fig. 4. The resistance increases rapidly and approximately linearly as the cone enters the sand layer and then abruptly drops, again about linearly. The shape of the peak suggests that the sand layers are not sufficiently thick for the cone resistance to be representative only of the properties of the sand. Rather, the measured maximum cone resistance in the relatively thin sand layers is strongly influenced by the properties of the clay above and below the sand layer. This observation is in agreement with a Federal Highway Administration report (1978) which states that the minimum layer thickness need to develop the full value of q_c in a layer is equal to 15 times the cone tip diameter. The cone tip used at Barkley for most of the sounding had an area of 15 cm^2 (diameter of 4.4 cm). Thus 15 times the diameter is about equal to 60 cm (2 feet) which corresponds to the maximum sand layer thickness in Unit 2 at Barkley Dam.

The peaks can be analyzed by defining a width of the peak as shown in Fig. 4 as the distance between the beginning and the end of the increased cone resistance. This distance is probably equal to or slightly larger than the thickness of the layer. Cone penetration data from Test Area A are presented in Fig. 5 as a plot of the peak values of cone penetration in Unit 2 versus the width of the peaks. The plot shows that larger peaks correspond to thicker layers with a relatively narrow range for peak resistance for layers thinner than about one foot. This result indicates that the peak cone value for layers thinner than about one foot is primarily a function of the thickness of the layer and the strength of the clay and to a lesser degree on the denseness of the sand. At thicknesses over about one foot, the scatter increases, indicating a larger effect of the density of the sand on the peak cone resistance. Thus the "true" cone penetration resistance in the switchyard area is of about 60 to 100 tsf.

A comparison is presented in Fig. 6 between cone penetration data corresponding to Test Area A, where the undisturbed samples for steady state strength determinations were obtained, with cone data selected to be representative of other areas of the site. This comparison indicates that in Test Area A, the cone penetration resistance is slightly lower than in other areas of the site. Since the penetration resistance in the clay of Unit 2 is about the same for all the cone soundings that were compared, the slightly lower values of peak cone penetration in the Test Are A for the same layer thickness must reflect slightly lower sand densities.

Comparison of CPT and SPT Logs

A comparison of the SPT and CPT data from borings within 5 to 10 feet of each other is shown in Figs. 7 to 10. The SPT and CPT correlate well in Unit 1 and in thick clay zones of Unit 2, indicating a gradual decrease of the strength of the clay with depth within Units 1 and 2. However, the SPT does not reflect the presence of the sand layers in Unit 2 disclosed by the CPT logs. The sand layers are too thin for the SPT to reflect the sand properties. Schmertmann has indicated that for cone friction ratios of 2 to 4 (typical of Unit 2), more than 50 percent of the SPT blowcount resistance is derived from the frictional resistance along the outside of the spoon. Thus, if the tip of the spoon is in a sand layer but a significant length of the spoon length is in clay, the blowcount is mostly due to the clay. Furthermore, similar to the cone penetration resistance, even the tip resistance of the SPT spoon would be influenced strongly by the strength of the clay above and below the sand layers.

The cone penetration log does reflect the presence of the sand layers; however, even the peak penetration resistance developed in each layer underestimates, in most cases, the "true" penetration resistance of the sand. Figures 5 and 6 show that full development of the penetration resistance of the sand requires a layer thickness of about 1.5 feet. Note that the width of the peak plotted in Figs. 5 and 6 is somewhat larger than the actual thickness of the layer.

The data in Fig. 5 indicate that the "true" (fully developed) cone penetration resistance of the sand is about 60 to 100 tsf in Test Area A, which was identified as the weakest area of the Unit 2 soils. Data from other areas, Fig. 6, indicate a "true" penetration resistance of 100 tsf or more. Thus the corresponding "true" SPT values would be about 18 in Test Area A and about at least 25 elsewhere if one uses a ratio of cone to SPT of 4. These N-values compare with values on the order of 5 used in the WES analyses.

The cone data were analyzed by WES to obtain equivalent sand blowcounts to enter the empirical charts of Figs. 55 and 80 of their report. To facilitate computations, the chart in Fig. 55 was combined with a chart for classifying soils from cone point and sleeve resistances. In their report WES recognizes that even the peaks in the cone log may be lower than the true penetration resistance of the sand because the sand layers are not thick enough to develop the true cone resistance of the sand. However, WES notes that there is a compensating error in that the cone soil classification chart will indicate a soil that is finer than it actually is. Thus the correction for fines would be larger, and even though the peak cone resistance is too low, the ESB would be approximately correct. However, no analysis is presented to corroborate the assumption that the compensating errors are of similar magnitude.

An examination of Figs. 5 and 6 reveals that for sand layers causing a spike in the cone record with a width of 1 foot or less, the peak cone resistance would be lower than the true resistance (judged to correspond to the thickest layers) by 20 to 55 tsf in Fig. 5 and by 40 to 80 tsf in Fig. 6.

The maximum correction for fines in the WES method is 7.5. The additional correction due to misclassification of the soil in the sand layers is probably of 4 blows per foot or less, since the actual soil is a silty sand, already the subject of some blowcount correction. A blowcount correction of 4 is roughly equivalent to a difference in cone penetration resistance of about 16, i.e., substantially less than the difference between the measured cone penetration and the true resistance of the sand. Thus the two errors are not compensated in the WES analysis, which leads to severe underestimation of the undrained steady state strength of the sand layers in Unit 2.

In summary, the SPT logs do not even reflect the presence of the sand layers. The CPT logs do indicate the presence of the sand layers but substantially underestimate its true penetration resistance. Thus the use of blowcounts or cone penetration resistance to estimate the properties of the sand layers is inappropriate.

3. Direct Determinations of Undrained Steady State Strength (S_{us})

Determinations of S_{us} were made by GEI by means of tests on undisturbed samples obtained from Unit 2. Tests were performed only on sand layers, and the results are presented in the GEI report dated July 19, 1985, which is included as Appendix D in Vol. 3 of the WES report. A conservative analysis of the test results lead to a recommendation of using a value of S_{us} in the Unit 2 sand layers of 1,100 psf (8 psi) as compared to the WES value of about 450 psf. We believe that the value of 1,100 psf is appropriately conservative for analyzing the seismic stability of Barkey Dam.

Subsequent to our determinations of S_{us} values at Barkley, we performed a re-evaluation of the soils in the hydraulic shell of the Lower San Fernando (LSF) Dam using the same methodology as used for Barkley. The S_{us} determinations at LSF Dam were in agreement with the two key observations of the dam in the 1971 earthquake, namely, there was an upstream liquefaction failure, but no failure occurred in the downstream direction. The results of the investigation were also in agreement with other known characteristics of the failure, such as the fact that it was triggered by the 1971 earthquake but not by earlier events. Thus the methodology used at Barkley for direct determination of S_{us} was corroborated by the most important case history presently available to the profession.

4. Strength Parameters for Stability Analyses

Seismically induced pore pressures were estimated in the WES report using the empirical "liquefaction" chart. For sand layers dense enough so that the estimated pore pressures were less than 100 percent pore pressure, the soil strength was estimated as having been reduced proportional with the pore pressure, e.g., if the seismically induced pore pressure was estimated to be 40 percent, the strength was reduced by 40 percent. We surmise that the intention is to use this reduced strength as representing the undrained strength available for post-earthquake stability.

The undrained strength of the soil is a function of the effective stress during the failure, not the effective stress at other stages of shear, e.g., initial or any intermediate stage prior to reaching failure, such as a reduced effective stress value caused by an instantaneous pore pressure that may develop as a result of seismic shaking. As the dam tends to deform during or after the earthquake, important pore pressure and effective stress changes would take place. For loose sands, the pore pressure may increase further and for dense sands dilation would cause a decrease in pore pressure. Ultimately the available undrained strength will be a function of the effective stress at failure which is a function of the void ratio and not of the initial or other intermediate value of effective stress. In general, the undrained strength that is actually available may be higher or lower than those estimated by WES, and it will be bear no relationship to the WES values. In the case of Barkley Dam, the soils for which WES used this method to estimate strength are dense enough to be dilative, and thus the WES values are conservative. However, there may be cases where the WES method may lead to unconservative results.

We hope that the comments presented in this letter will serve to arrive at a better understanding of the proper methodology for analyzing the seismic safety of embankment dams.

Very truly yours,

GEI CONSULTANTS, INC.

Gonzalo Castro

Gonzalo Castro, Ph.D., P.E.
Principal

GC:ck

Enclosures

cc: E. Pritchett
G. Franklin

REFERENCES

[1] Castro G. (1987). "On the Behavior of Soils During Earthquakes - Liquefaction," *Third International Conference on Soil Dynamics and Earthquake Engineering*, June, Princeton University.

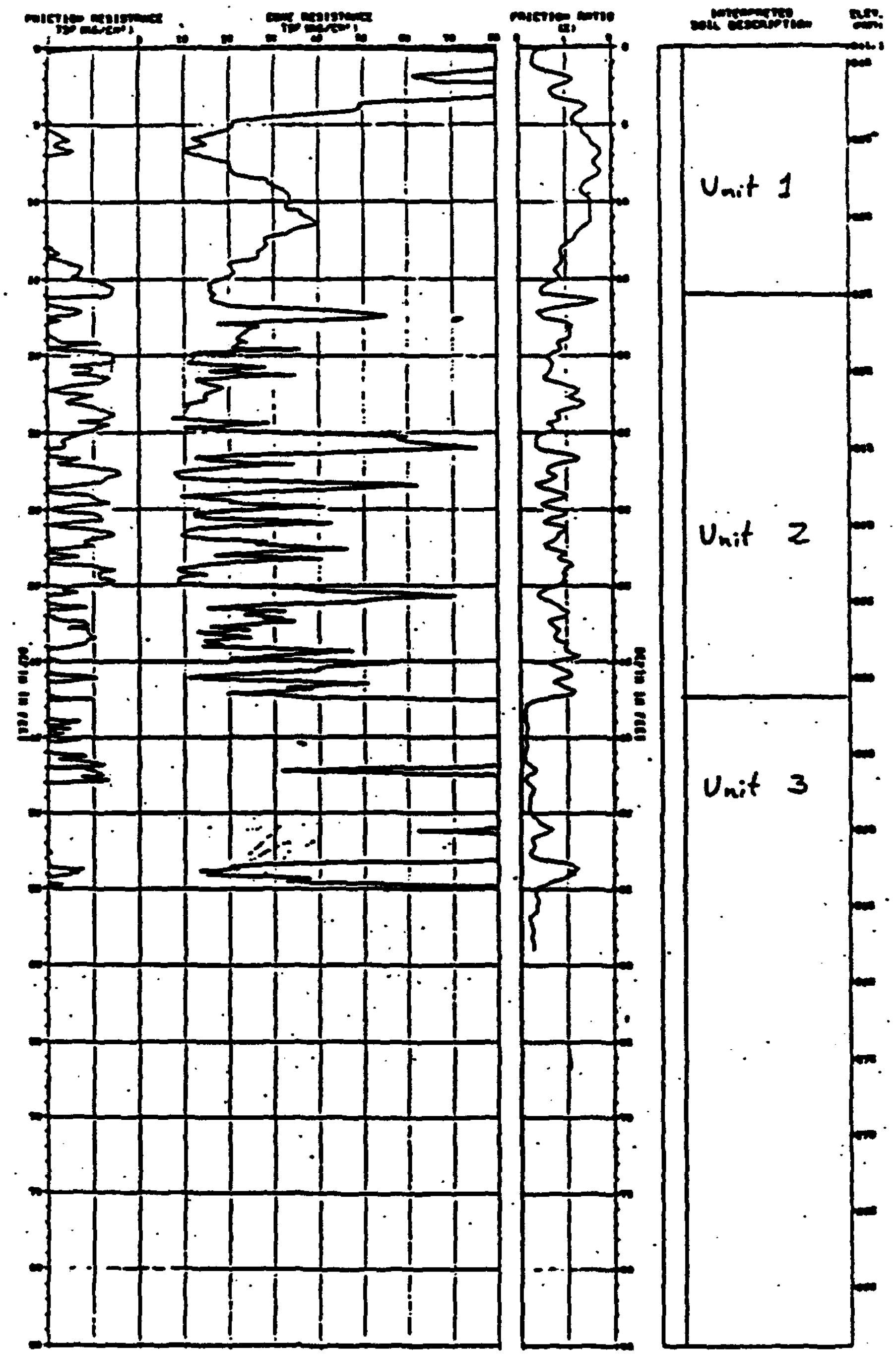
[2] Castro, G. and Keller, T. O. (1988). "Re-evaluation of Lower San Fernando Dam," GEI report to Department of the Army, Corps of Engineers, February.

[3] Federal Highway Administration, Office of Research and Development, (1978), "Guidelines for Cone Penetration Test - Performance and Design, Report FHWA TS-78-209, July.

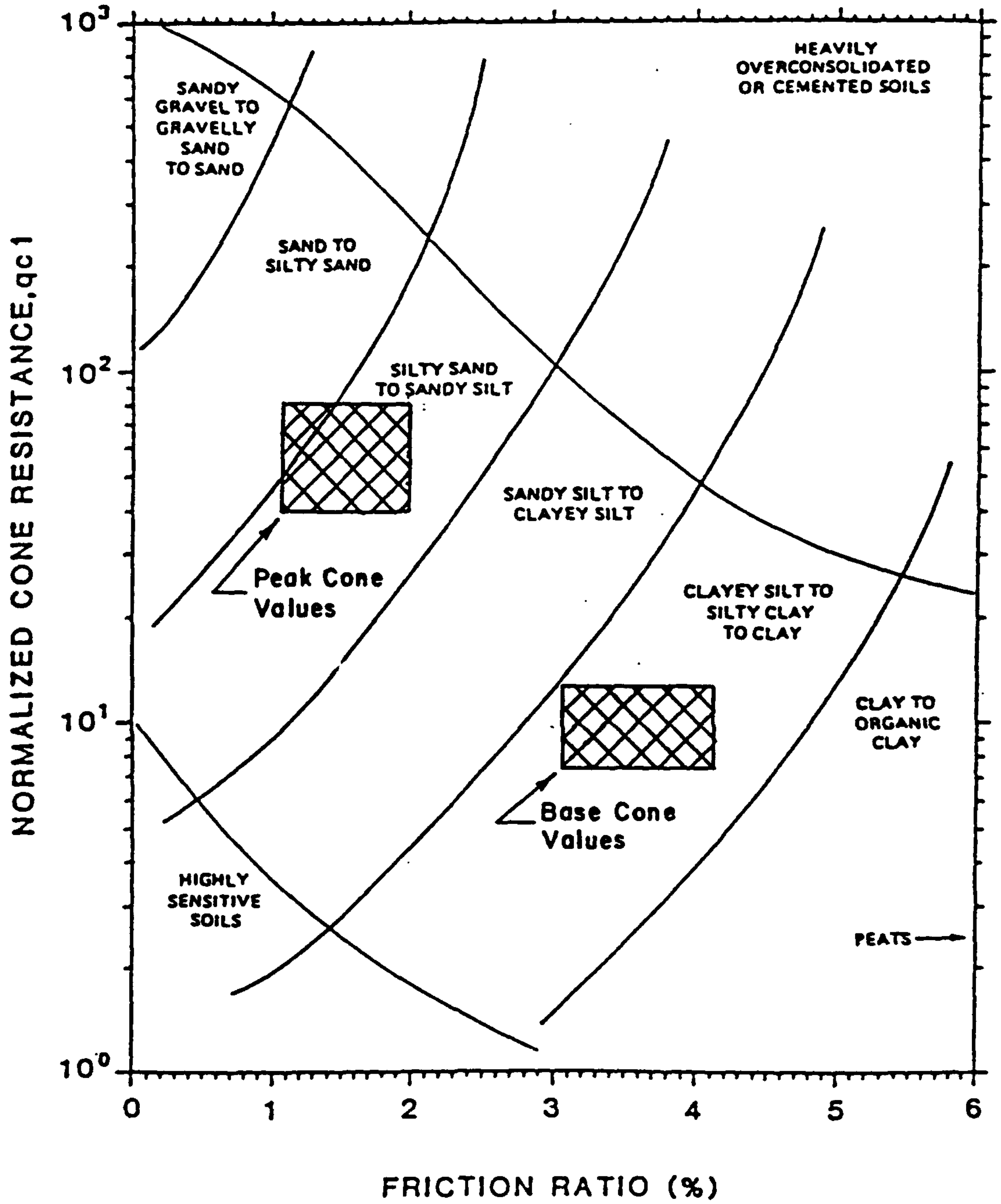
[4] Keller, T. O.; Castro, G., and Rogers, J. H. (1987). "Steel Creek Dam Foundation Densification," *Proceedings of Soil Improvement - A 10-Yr Update*, ASCE, Atlantic City, April.

[5] Schmertmann, John H., (1979), "Statics of SPT," Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT5, Proc. Paper 14573, May, pp. 655-670.

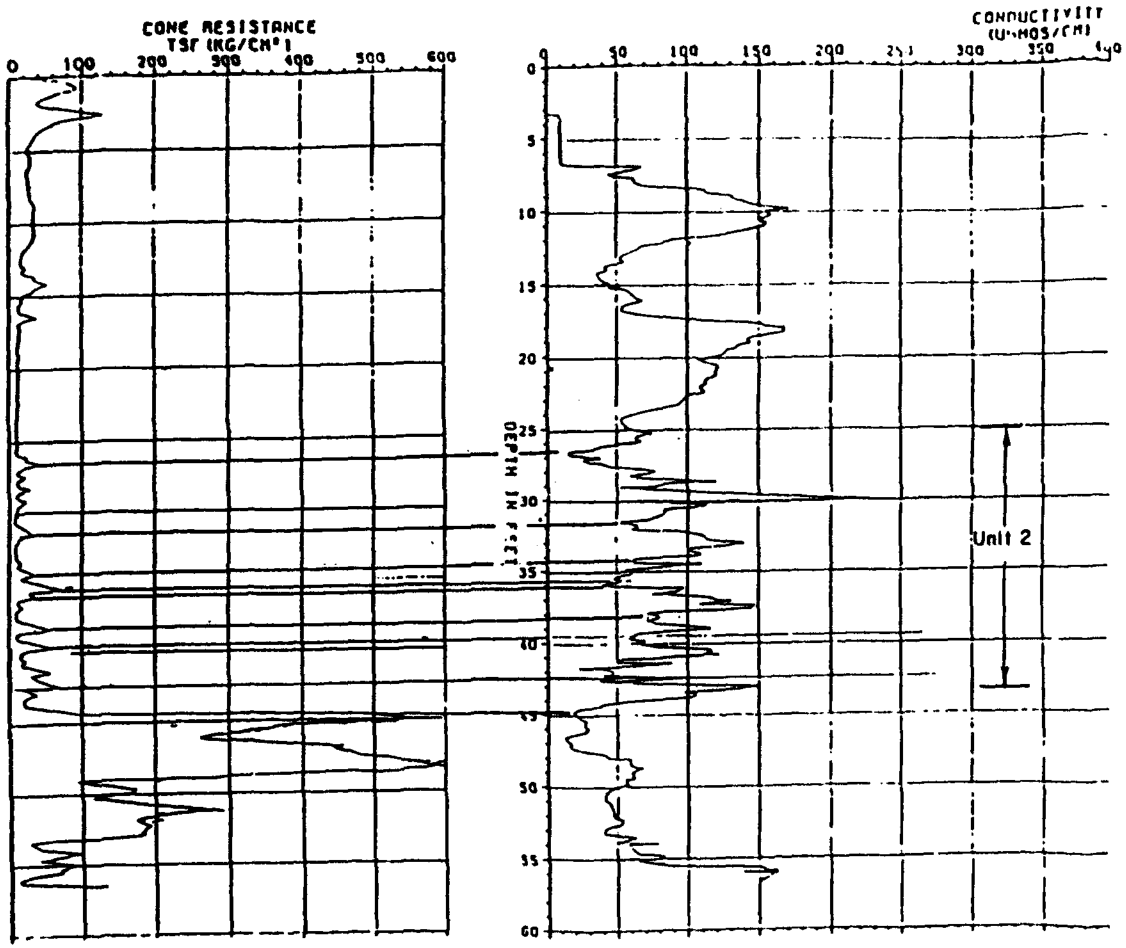
[6] Seed, H. B. (1987), "Design Problems in Soil Liquefaction," ASCE, *Journal of Geotechnical Engineering*, Vol. 113, No. 8, August.



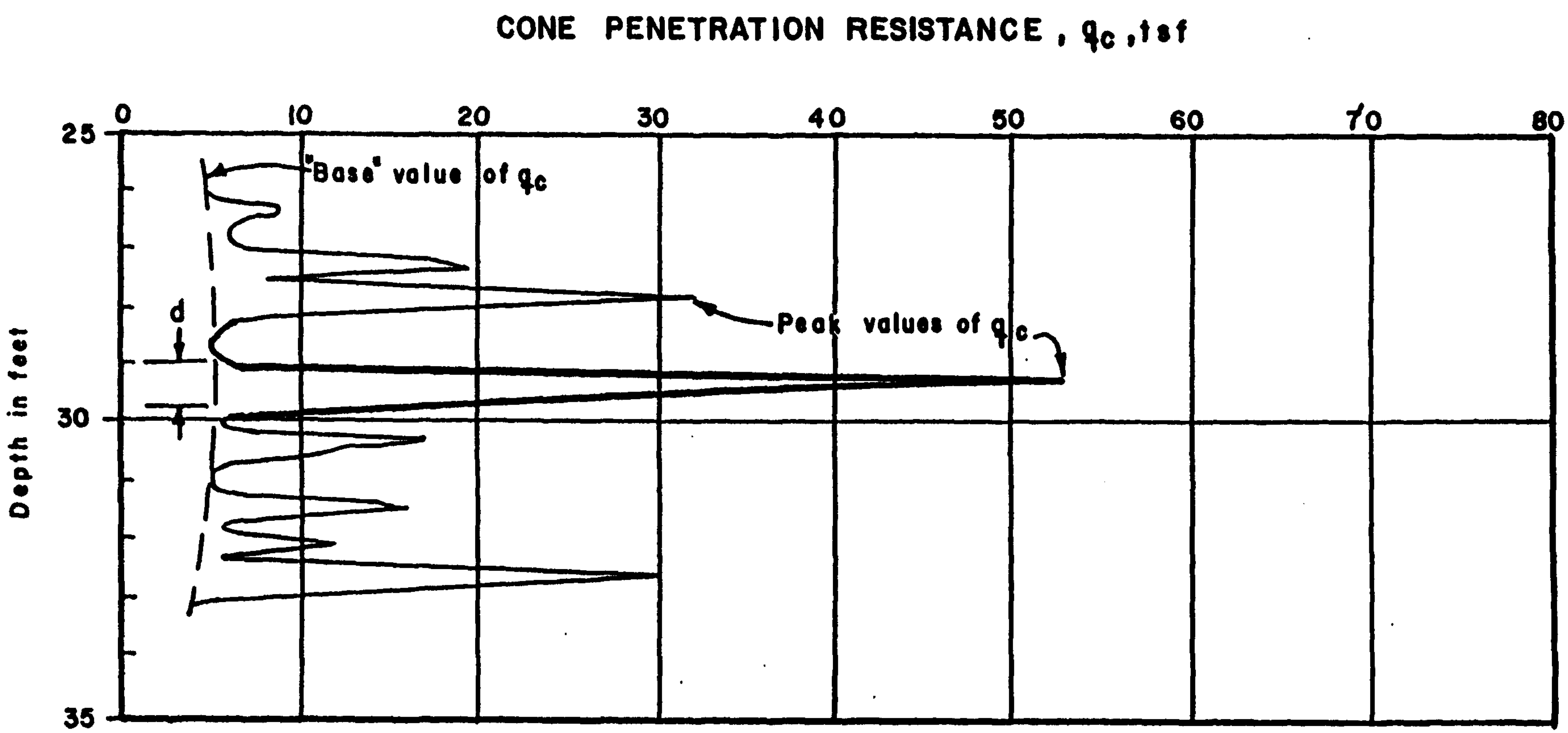
U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	CPT 57
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 1



U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	CONE PENETRATION VALUES UNIT 2 CPT 57
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 2



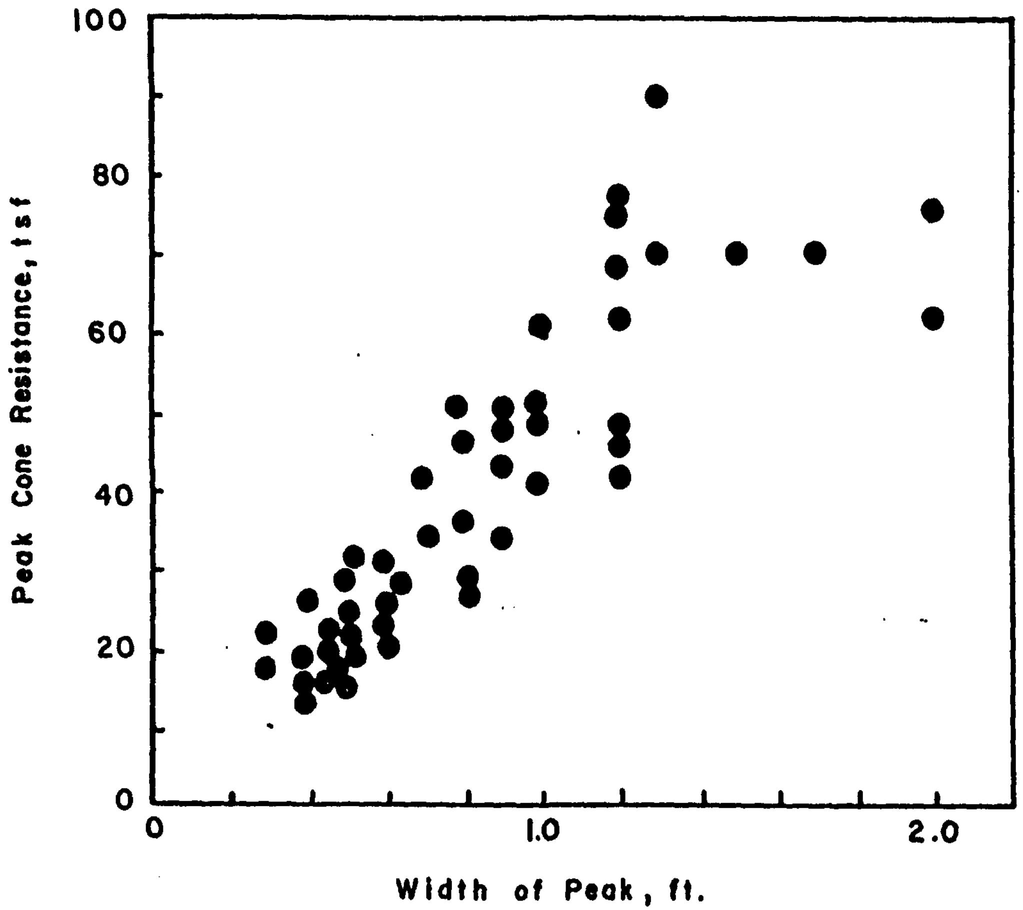
U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	CONDUCTIVITY DATA CPT 54
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 3



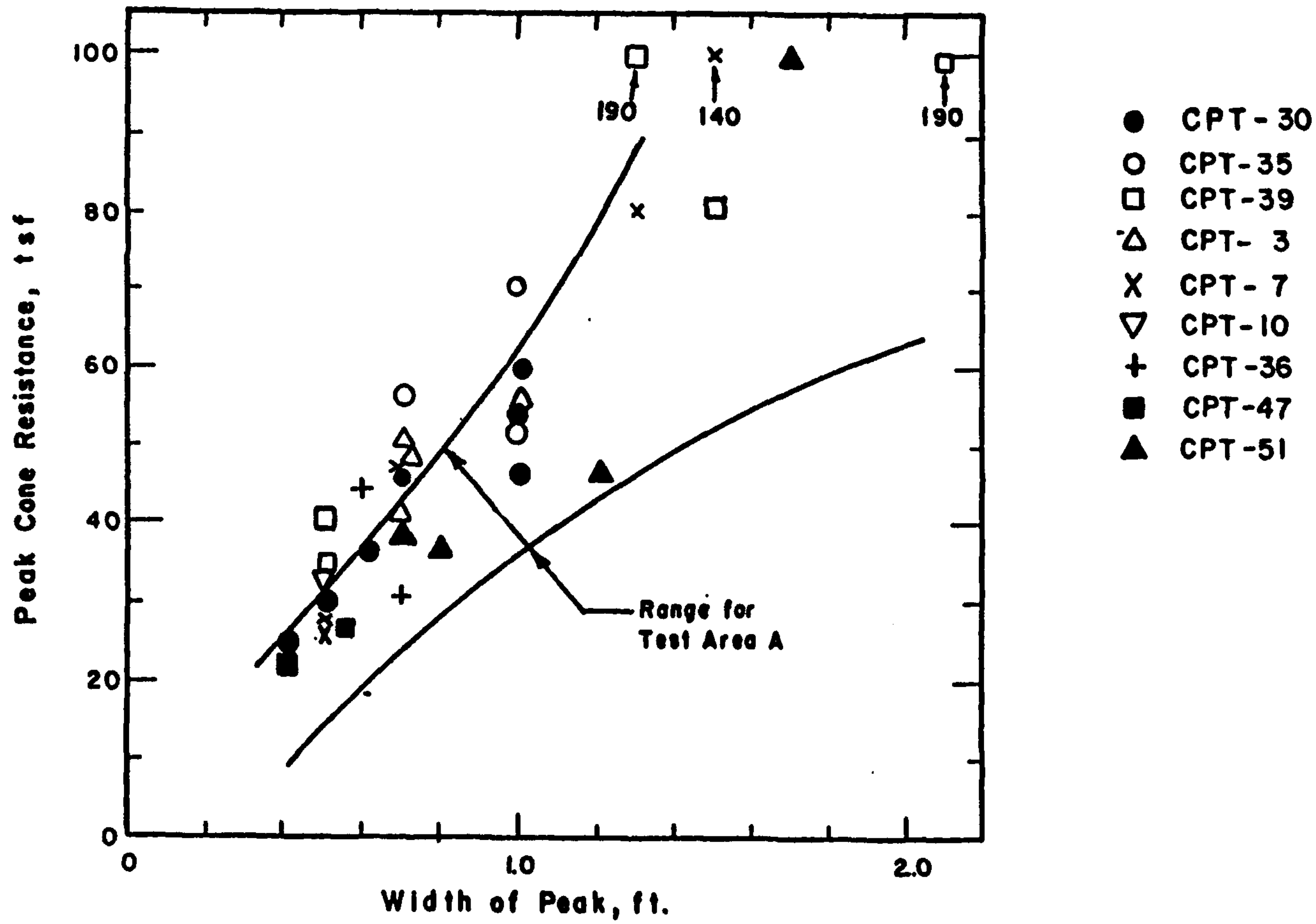
EXPANDED PARTIAL LOG OF CPT 58

d = width of peak

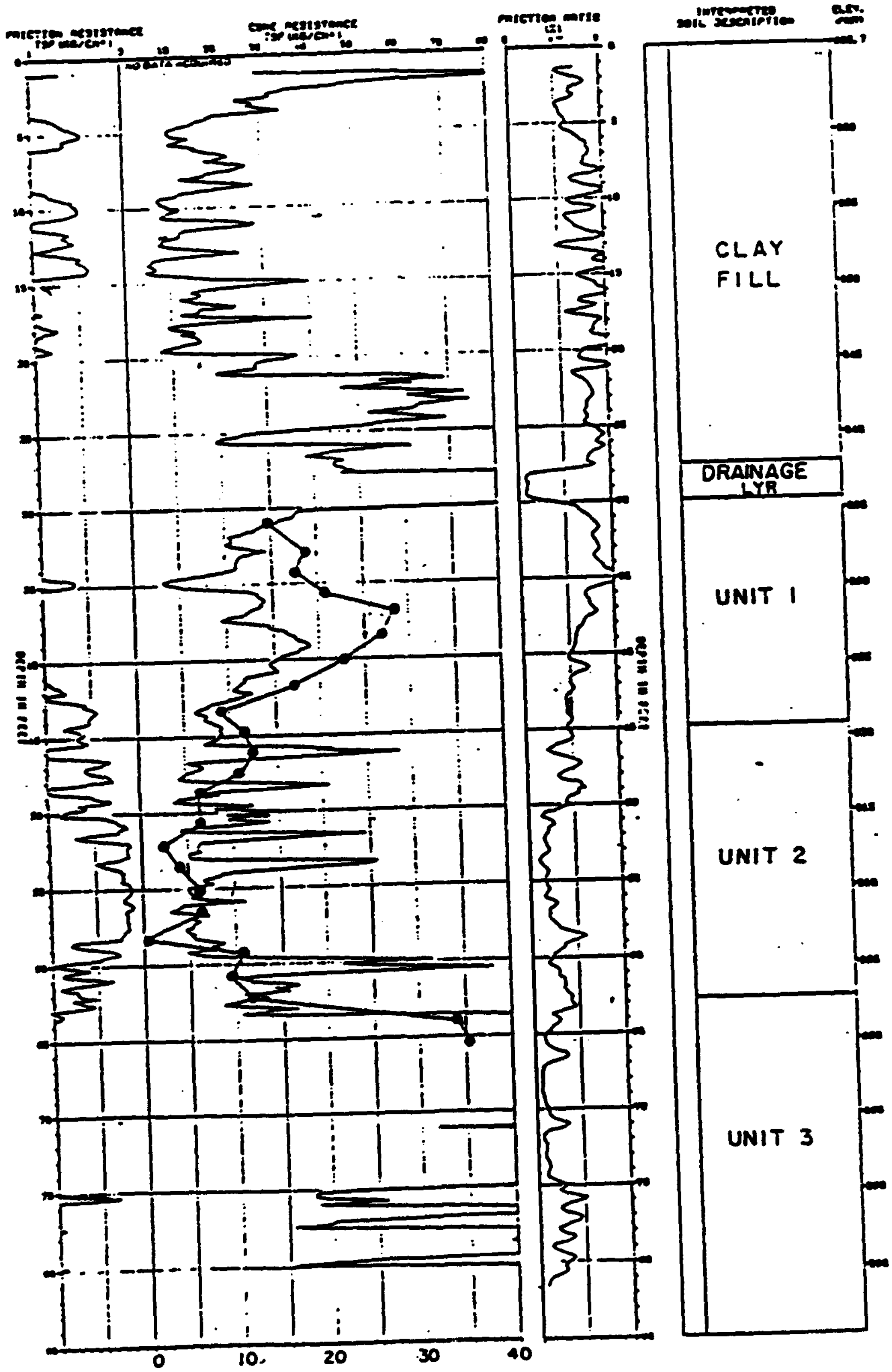
U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	EXPANDED PLOT CPT 58
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 4



U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	PEAK CONE RESISTANCE UNIT 2 SANDS TEST AREA A
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 5

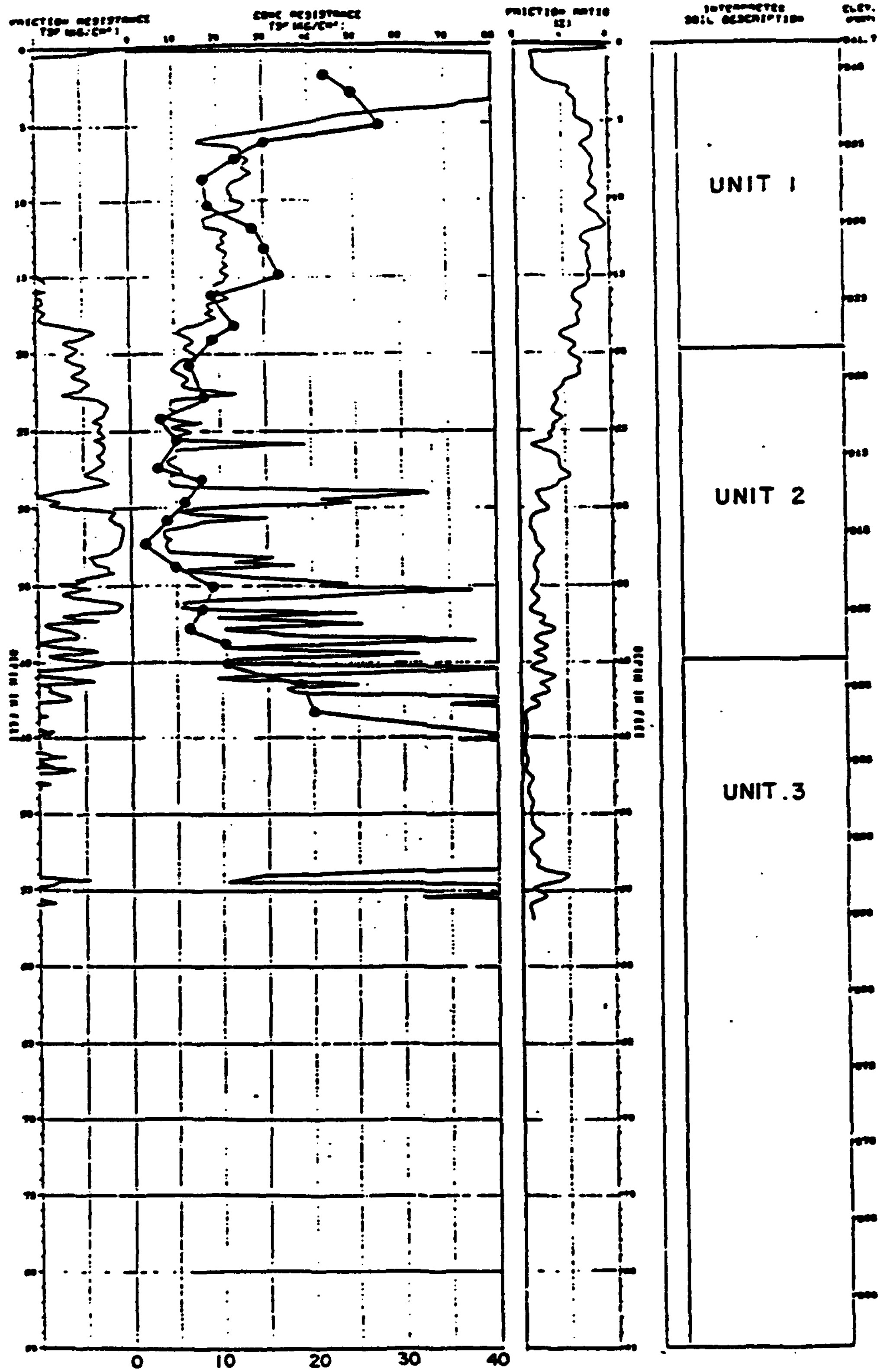


U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	PEAK CONE RESISTANCE UNIT 2 SANDS
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 6



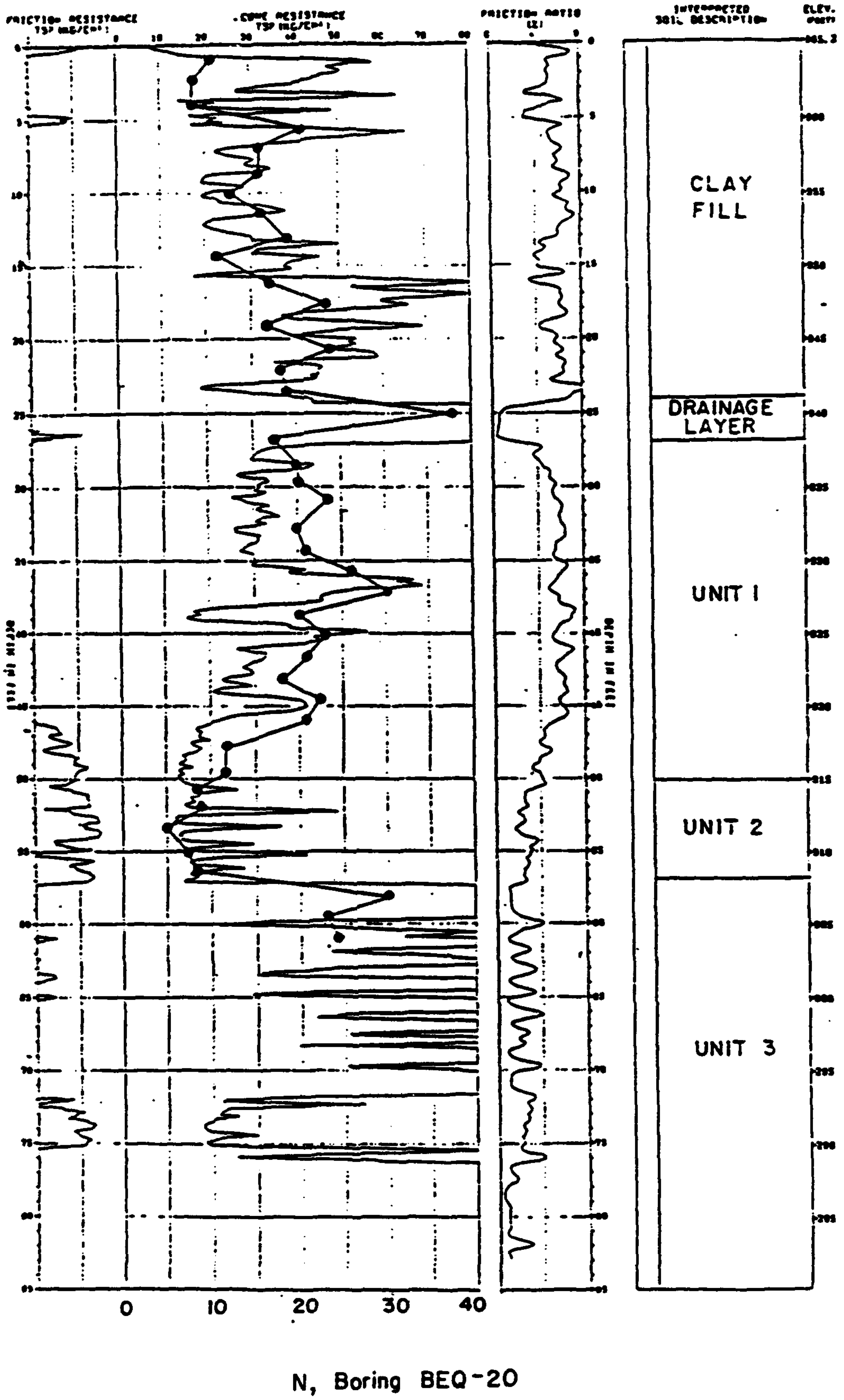
N, Boring BEQ-16

U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	COMPARISON OF CPT 3 WITH BLOWCOUNT DATA
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 7

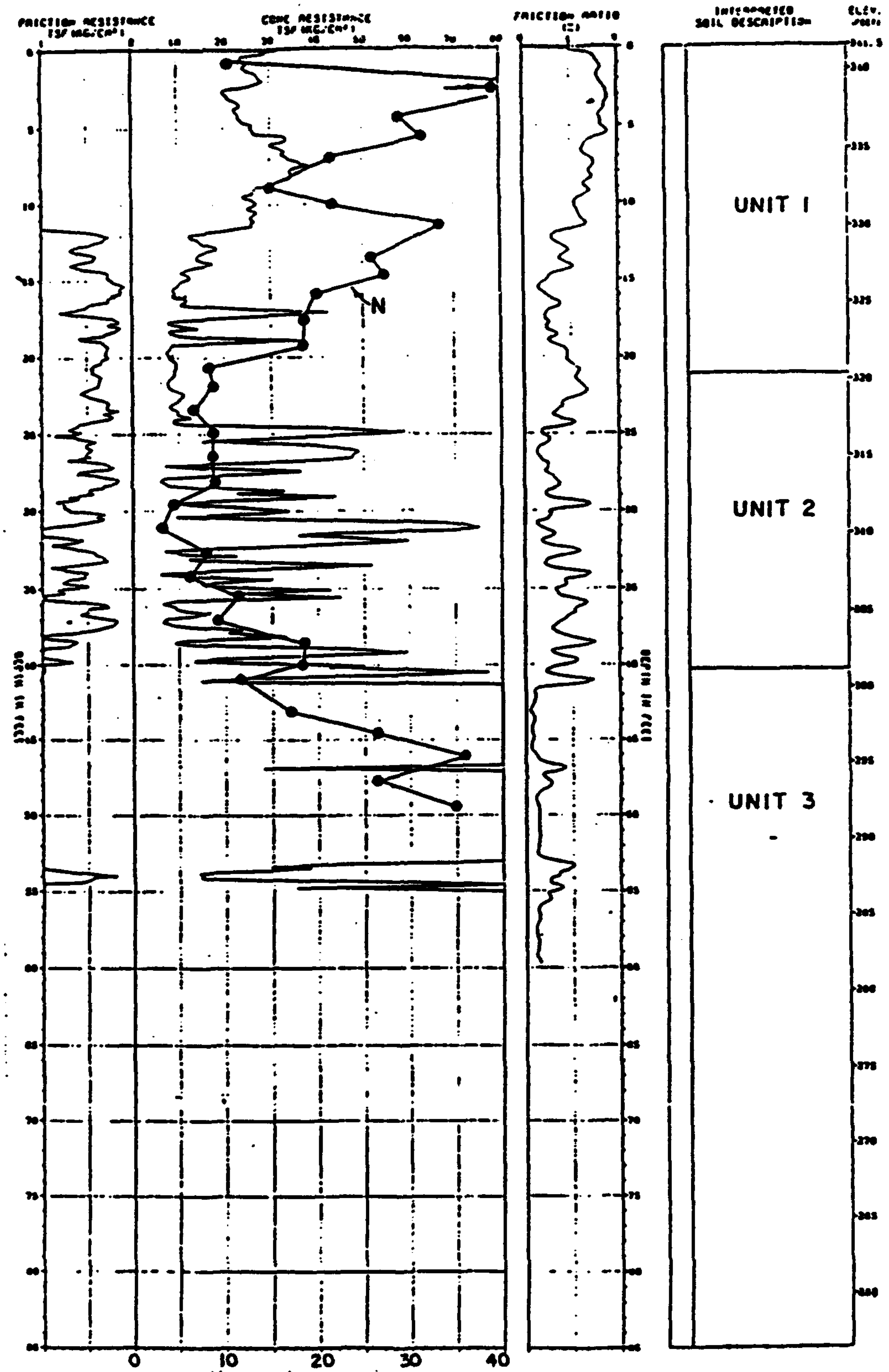


N, Boring BEQ-22

U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	COMPARISON OF CPT 41 WITH BLOWCOUNT DATA
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 8



U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	COMPARISON OF CPT 11 WITH BLOWCOUNT DATA
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 9



N, Boring BEQ-7

U.S. Army Corps of Engineers Nashville, Tennessee	Barkley Dam	COMPARISON OF CPT 25 WITH BLOWCOUNT DATA
GEI Consultants, Inc. WINCHESTER • MASSACHUSETTS	Project 85836	June 28, 1991 Fig. 10

GEOTECHNICAL ENGINEERS INC.

1017 MAIN STREET WINCHESTER MASSACHUSETTS 01890 (617) 729-1625

PRINCIPALS
RONALD C. HIRSCHFELD, P.E.
STEVE J. POULOS, P.E.
DANIEL P. LA GATTA, P.E.
RICHARD F. MURDOCK, P.E.
GONZALO CASTRO, P.E.
ASSOCIATES
JOHN W. FRANCE, P.E.
FRANCIS D. LEATHERS, P.E.

February 24, 1986
Project 85836

Mr. Paul Bluhm
U.S. Corps of Engineers
Nashville District
P.O. Box 1070
Nashville, TN 37202

Subject: Barkley Dam
Paducah, Kentucky

Dear Mr. Bluhm:

The purpose of this letter is to comment on the results of the seismic stability evaluation of Barkley Dam discussed during the meeting of January 24, 1986.

The conclusion was reached at the meeting that no remedial measures are required to enhance the seismic stability of Barkley Dam. I concur with this conclusion based on the following:

1. The stability analyses performed by the COE indicate factors of safety of at least 1.8, when using the value of undrained steady state shear strength of 8 psi (1150 psf) determined by GEI for the sand layers in the critical stratified stratum in the foundation (Unit 2 soils). Under these circumstances one can conclude that a flow slide type of failure is not possible.

2. The deformation analysis performed by COE indicates that permanent deformations induced by the design earthquake will be small with resulting tolerable displacements, probably not exceeding one foot.

3. The analysis of the cone penetrometer data, presented in GEI's letter of October 3, 1985, indicates that the values of cone penetration resistance measured in the sand layers in Unit 2 significantly underestimate the penetration resistance of the sand because of the influence of the clay above and below the sand layers. The cone data indicate that the true penetration resistance of the sand is probably in excess of 100 tsf, which would correspond to a medium dense to dense sand.

4. The low blowcounts measured in the stratified clay and sand (Unit 2) had been in the past the main basis for concern over the liquefaction potential of the sand and the seismic stability of the dam. The analysis of the cone penetration data by GEI has shown that the low blowcounts do not reflect the properties of the sand layers but are rather mostly a function of the strength of the clay, and thus the blowcounts cannot be used as a basis for assessing the liquefaction potential of the sand.

I understand that the COE will prepare a report to summarize the investigation of the seismic stability of Barkley Dam. The following comments on the studies performed to date are offered for your consideration in preparing your report.

During the January 24, 1986 meeting, WES presented an analysis of the standard penetration (SPT) and cone penetration data. The analysis by WES was based on an empirical chart by Seed and Idriss that correlates blowcounts in sands with their observed behavior during earthquakes for level ground conditions. The blowcounts used in the analyses of the foundation soils at Barkley Dam were the actual measured blowcounts and also blowcounts computed from the cone penetration resistance. The analyses indicate that high pore pressures would develop during the earthquake in certain areas of the Unit 2 foundation sands, specifically under the switch yard, in the free field upstream and downstream of the dam, and in scattered locations elsewhere. I believe that this conclusion is not warranted, since the measured blowcounts and cone penetration resistances significantly underestimate the resistance to seismic loading of the sand layers in the Unit 2 soils, as explained above and in more detail in GEI's letter of October 3, 1985. It should be noted that even if the high pore pressures predicted by the WES analyses were to develop, the dam would remain stable and the earthquake induced deformations would be small, as shown by the stability and deformation analyses presented by the Corps at the January 24 meeting.

I agree with the general methodology used in the stability analyses performed to determine the potential for a flow slide failure (post earthquake stability). As strength parameters for the Unit 2 soils, I recommend an undrained strength represented by a conservative c value of 8 psi (1150 psf) throughout the layer. The strength parameters used for the clay cap (c = 960 psf, $\varphi = 12°$) and the clayey embankment material (c = 800 psf, $\varphi = 18°$) are somewhat conservative. Since the results of the analyses using the above parameters would appear to indicate a high factor of safety, further refinement in selecting strength parameters is not warranted.

The deformation analysis presented in the January 24 meeting was performed using state-of-the-art techniques. I concur with the yield strength parameters used for the embankment material and for the clay cap in the foundation. However, for the Unit 2 soils, I recommend a yield strength (c) of 1150 psf, i.e., the undrained steady state strength. This yield strength should be used under all sections of the dam as well as beyond the toes of the dam. The computed yield accelerations using the recommended yield strength should not be substantially different from the yield accelerations presented in the meeting, which were based on friction angles of 10° and 17° for different zones of the Unit 2 soils. Thus the computed displacements will also be small for the recommended strength parameter.

I will be pleased to provide any additional information you may need.

Very truly yours,

GEOTECHNICAL ENGINEERS INC.

Gonzalo Castro

Gonzalo Castro, P.E.
Principal

GC:ck

Φ GEOTECHNICAL ENGINEERS INC.

1017 MAIN STREET · WINCHESTER · MASSACHUSETTS 01890 (617) 729-1625

PRINCIPALS
DANIEL P. LA GATTA
STEVE J. POULOS
RONALD C. HIRSCHFELD
RICHARD F. MURDOCK
GONZALO CASTRO

January 4, 1984
Project 79615
File 2.0

Mr. Frank B. Couch
Nashville District
U. S. Army Corps of Engineers
P. O. Box 1070
Nashville, TN 37202

Subject: Barkley Dam Seismic Study
Contract DACW 62-84-M-0504

Dear Mr. Couch:

The purpose of this letter is to present comments made by the undersigned during the meeting of December 19 and 20, 1983 on the subject project. A copy of the meeting agenda and the list of attendees is attached for your reference. Personnel from the Nashville District and from the Waterways Experiment Station presented information, obtained subsequent to our previous meeting of November 13, 1981, in the areas of characterization of the foundation soil, blowcount analyses, and analyses of post-earthquake stability and potential modes of failure.

Characterization of Foundation Soils

An exposure of the stratified silty sands, clayey sands and clays was examined by Corps' personnel on the river bank about 1.5 miles downstream of the dam. Mapping of the various layers indicated that the sandy layers were generally continuous and that one layer extended for distances of at least 100 ft. The layers were close to horizontal and not uniform in thickness and the stratification was intense with layer thicknesses on the order of one foot. The observations made in piezometers located in the stratified soils indicate a rapid response to fluctuations in the tail water level to substantial distances from the river, thus indicating hydraulic continuity of the more pervious layers.

A detailed description of split-spoon samples provided further details on the stratification of the soils and on the type of soil associated with each blowcount.

The investigations on the characterization of the foundation soils lead to the conclusion that the seismic analyses should be based on the assumption that there is continuity of the sand layers. I believe that it is unlikely that this conclusion will be changed by any further efforts in defining the stratigraphy of the foundation soils. Thus I recommend that the trenching of the foundation soils suggested in our meeting not be pursued.

Blowcount Analysis

A blowcount analysis was performed in accordance with the state of the art. The analysis utilized the detailed description of the split-spoon samples to identify the type of soil to which each blowcount was associated. I agree with the assumptions and the methodology of the analysis. The only remaining question is whether a lower groundwater level than used by WES may be appropriate for the analysis near the river channel. A review of piezometer data and possibly installation of additional piezometers is required.

The blowcount analysis has identified the silty sand layers as the soils that are most susceptible to liquefaction. Further refinement of the blowcount analysis is not warranted with the possible exception of a revised analysis near the river channel if it is determined that a lower groundwater level is realistic.

The blowcount analysis is based on empirical charts derived from observations of the behavior of sand deposits with essentially level ground, during past earthquakes. A site is classified as having "liquefied" based on observations that include evidences of high pore pressures, such as sand volcanoes, settlements of buildings of varying degrees, and cracks due to lateral spreading in gently sloping ground. Some cases include the extreme behavior observed in Niigata where the soil truly liquefied in the sense of deforming (flowing) with very small resistance as the buildings sank until apparently a condition of about full flotation was reached. In other cases, it is not known whether the soil could have liquefied and flowed since under level ground and with no building loads there are no shear stresses that can cause the flow. For the cases of limited settlements it is clear that the soil could not flow. Thus the cases classified as liquefaction in the empirical charts may correspond to either liquefaction or to cyclic mobility as defined in

Seed, 1979[1], and in Castro, 1975[2]. Thus the low factors of safety obtained from the blowcount analyses do not necessarily mean that a flow-slide type of failure (similar to the Lower San Fernando Dam) is possible.

Stability Analysis

Stability analyses were performed to evaluate the post earthquake stability of the dam and its foundation and the possible configuration of the dam after the failure when the analysis indicated that a failure was possible. Assumptions relative to shear strength parameters designated as classes 1, 2, and 3 were varied within a wide range, particularly for the zones in the foundation (silty sands) suspect of having a liquefaction potential. I believe that the actual strength parameters will probably fall within the assumed range. Class 1 and 2 assumptions lead to the conclusion that there is post-earthquake instability and thus that a major slide is possible. Various mechanisms of failure were investigated for predicting whether the remnant dam and debris will allow release of water from the reservoir after a failure. In my opinion, a condition in which a major slide is possible is not acceptable since the prediction of the configuration of the failed dam is not reliable with the present state of the art. On the other hand, Class 3 assumptions lead to the conclusion that the dam is stable and thus, even though some deformations would develop during the earthquake, a flow-slide type of failure is not possible. This condition would be considered acceptable because the freeboard of about 30 ft is large as compared with the height of the dam of about 40 ft to 50 ft in the zone in question.

Since the post-earthquake stability is the key issue on assessing the safety of the dam, I believe that future efforts should be directed to a determination of the post-earthquake (undrained steady state) shear strength of the silty sands in the foundation soils. The strength of these soils is the most critical assumption in the analyses. A determination of this strength cannot be made from the data obtained to date, and therefore, I believe that an adequate assessment of the safety of the dam under the design earthquake cannot be made on the basis of the available data.

The determination of undrained steady-state strength (S_{us}) is similar to the determination of other strength

---

[1]Seed, H. B., "Soil Liquefaction and Cyclic Mobility Evaluation For Level Ground During Earthquakes," Journal of the Geotechnical Division, ASCE, Feb. 1979.

[2]Castro, G., "Liquefaction and Cyclic Mobility of Saturated Sands," Journal of the Geotechnical Division, ASCE, June 1975.

parameters in soil mechanics, except that special precautions and procedures are required because the value of S_{us} is extremely sensitive to changes in density. In addition to using careful sampling and handling techniques and monitoring of samples for volume changes, a method is required to correct measured strengths for the volume changes that will unavoidably occur. These procedures have been used already in many projects and there should be no particular difficulty in applying them to the silty sands in the foundation of Barkley Dam. I will be pleased to discuss with you a detailed sampling and testing program for determining the undrained steady-state shear strength of the silty sands.

I will be pleased to answer any questions you might have concerning this letter.

Sincerely yours,

GEOTECHNICAL ENGINEERS INC.

Gonzalo Castro
Principal

GC:ms

Enclosures

cc: Prof. H. B. Seed
Prof. A. Nieto

BARKLEY SEISMIC STUDY MEETING

19 Dec 83

List of Attendees

Name	Office
James E. Paris	ORNED-G
Harry B. Seed	Technical Advisor
Gonzalo Castro	Technical Advisor
Alberto S. Nieto	Technical Advisor
Lee A. Knuppel	ORDED-G
Ben Couch	ORNED-G
Dave Hammer	ORDED-G
Marvin Simmons	ORNED-G
Charles Canning	ORDED-G
Richard S. Olsen	WES-GH
Mary Ellen Hynes-Griffin	WES-GH
A. G. Franklin	WES-GH
Roger A. Brown	SADEN-F
Ben C. Foreman	SASEN-FS
Earl L. Spearman	TVA-EN DES
R. Joe Hunt	TVA-EN DES
Sam Stone	TVA-EN DES
Paul E. Booth	ORNED-G
Paul Fisher	OCE
Ed Pritchett	OCE
Paul F. Bluhm	ORNED-G

BARKLEY SEISMIC MEETING - AGENDA

19 DEC 1983 - Room A-640

1000 - 1030	Introduction - Summary of Previous Work	- Bluhm
1030 - 1100	Correlation of Borings Downstream Riverbank Exposure	- Simmons
1100 - 1200	SPT Analysis	- Hynes-Griffin
1200 - 1300	LUNCH	
1300 - 1415	Stability Analysis	- Olsen
1415 - 1430	BREAK	
1430 - 1545	Stability Analysis (cont.)	- Olsen
1545 - 1600	Conclusion	- Bluhm

20 DEC 1983 - Room A-440

0800 - 1000	Questions and Discussion
1000 - 1100	Meeting of Consultants
1100 - 1200	Evaluations from Consultants

GEOTECHNICAL ENGINEERS INC.

1017 MAIN STREET · WINCHESTER · MASSACHUSETTS 01890 (617) 729-1625

PRINCIPALS
DANIEL P. LA GATTA
STEVE J. POULOS
RONALD C. HIRSCHFELD
RICHARD F. MURDOCK
GONZALO CASTRO

December 23, 1981
Project 79615

Mr. Frank B. Couch
Nashville District
U.S. Army Corps of Engineers
P.O. Box 1070
Nashville, TN 37202

Subject: Barkley Dam
Seismic Stability Studies

Dear Mr. Couch:

Please find enclosed the consultants report for our meeting of November 13 on the subject project.

I will be pleased to answer any question you might have.

Very truly yours,

GEOTECHNICAL ENGINEERS INC.

Gonzalo Castro
Gonzalo Castro
Principal

GC:ck

Enclosure

CONSULTANTS' REPORT

BARKLEY DAM SEISMIC STUDIES

November 13, 1981

The consultants for the Barkley Dam Seismic Stability Evaluation met in Nashville on November 13 with representatives of the Waterways Experiment Station, the Nashville District, the Ohio River Divison, and the Office of the Chief of Engineers of the U.S. Corps of Engineers. A list of attendees and an agenda for the meeting are attached to this report.

Mr. Parish described a sand boil observed at the toe of the dam while removing vegetation during routine maintenance operations. A 10-ft-thick layer of gravelly sand was placed over the boil and adjacent areas. Borings performed later in the boil area indicated the presence of a silty sand layer within the upper clay stratum.

Dr. Franklin and Ms. Hynes-Griffin presented the result of one-dimensional wave propagation analyses at the centerline of the dam and at a point in the free field downstream of the dam. Modified Taft and Santa Barbara records with a peak acceleration of 0.24 g were utilized. They were assumed to represent probable ground motions on firm ground in the Barkley Dam area. The records were deconvoluted through a firm ground profile (all sand) to bedrock and then applied to the Barkley Dam profile at bedrock for both dam centerline and free field locations. Peak accelerations obtained at the surface were 0.10 g at the top of the dam and 0.17 g at the free field. Earthquake shear stresses computed from these analyses were compared with the results of isotropic cyclic triaxial tests for the appropriate type of soil, and factors of safety were computed. At the centerline of the dam, with an assumed water table at El 360, the computed factors of safety exceeded 1.6 at all depths. For the free field, factors of safety lower than one were obtained at elevations of 300 to 330 and 250 to 260. An analysis at the free field with a superimposed 15 ft berm indicated factors of safety above one with the exception of one value of 0.98 at El 315. One-dimensional wave propagations analyses for the free field were also performed assuming an hydraulic downgradient in the foundation soil and an input motion computed from the Santa Barbara record for a rock outcrop. Both of these assumptions raised the factors of safety. Shear stresses computed for the bedrock outcrop motion as input and no downgradient resulted in factors of safety larger than one except for a value of 0.94 at El 315.

Ms. Hynes-Griffin also presented an analysis based on blowcounts utilizing H. B. Seed's empirical chart for liquefaction. The empirical chart was utilized assuming an earthquake duration corresponding to a magnitude 7.5 event and with corrections applied for confining pressure. Different empirical criteria were utilized for clean and for silty sands. The range of factors of safety obtained at a given elevation was generally very large because of the large scatter in blowcount data. The computed factors of safety ranged between 0.5 and 6.3.

A discussion among the consultants relative to the status of the seismic stability investigation of Barkley Dam led to the conclusion that at this time the consultants could not either endorse the seismic safety of the dam nor justify a decision regarding the necessity for remedial measures. Consideration was given to the following findings of the investigation.

1. The foundation soils at Barkley were formed in a very complex depositional environment resulting in intensely stratified soils consisting of clean sands, silty sands and clays. Because of the complexity of the stratification, attempts to date to establish continuity of individual layers from boring to boring have proven unsuccessful. Only broad stratigraphic units have been found to be reasonably continuous.

2. The potential seismic behavior of these soils is different as shown by the results of laboratory triaxial tests, both with monotonic and cyclic loading.

3. The significance of blowcount data relative to liquefaction depends on the type of soil with which the blowcounts are associated.

4. The field investigation to date has concentrated in the area of Sta 64+00, and as indicated by the nearby sand boil area, soil conditions can vary significantly along the dam.

On the basis of these considerations, the following additional field investigations are recommended:

1. Study all available information to attempt to characterize in greater detail the soil conditions at the site. Laboratory descriptions should be completed for all available samples to better establish the type of soil in which blowcounts have been measured.

2. Perform additional SPT borings to about El 270, spaced about 300 ft apart along the crest and along the downstream toe. The borings should include SPT sampling every 2.5 ft. The full length of the soil samples should be described in the field. Alternatively, the sample description could be made in the laboratory but only if the full length of the sample were retained and sent to the laboratory. A 2.5-ft SPT spacing will provide sampling of 60% of the soil profile when using the standard spoon which is driven 18 in. A larger spoon, which allows sampling of 24 in. soil, would be preferable to effectively sample 80% of the soil profile.

3. If any boring shows blowcounts in clean sands of less than 5 or weight of rods (WOR) for silty sands, a verification of the accuracy of these blowcounts should be made by reviewing the boring procedures utilized and by possibly performing an adjacent boring to the depth in question. If the low blowcounts are confirmed, additional borings should be made at a distance of about 50 ft, measured along the dam, at both sides of the boring showing the low blowcounts. A total of ten borings should be budgeted for this purpose.

4. A visual inspection should be made to identify areas where a minimum amount of excavation could expose the type of deposits that constitute the foundation of the dam. A possible such area could exist along the river banks downstream of the dam. An exposure of the soil deposits would allow a visual examination of the stratigraphy of these soils over relatively large horizontal distances. Such an examination will greatly assist in the interpretation of stratigraphy from the borings at the dam site.

5. Consideration should be given to using downhole resistivity logging in the borings if it appears that it could provide economically a better resolution of the soil layering as compared with sample descriptions.

6. Consideration should also be given to the use of high resolution reflection profiling, possibly upstream of the dam, for obtaining information on the horizontal extent of soil layers.

Leland T. Long
Alberto Nieto
H. B. Seed
G. Castro

Attachment

ALBERTO S. NIETO, Consultant

Engineering Geology and Applied Rock Mechanics

March 25, 1986

Mr. E. C. Moore, Chief
Engineering Division
Nashville District, Corps of Engineers
P.O. Box 1070
Nashville, TN 37202

REF: Dynamic Stability Studies for Barkley Dam
AN-ORNED-861

Dear Mr. Moore;

Further to the ongoing studies on the seismic stability of Barkley Dam, I have the following comments:

1. Continuity of sands vs. percentage of sands in a potential failure plane.

Summary: A geologic detail thus far not considered in the geotechnical characterization of Unit 2 satisfactorily explains: a) The relatively small proportions of sands penetrated by the boreholes in Unit 2 (U2). b) The good hydraulic communication between these sands. c) The very poor correlation between sands parallel to the dam axis and the somewhat better correlation perpendicular to the dam axis. d) The low piezometric head drops perpendicular to the dam axis. e) The variation of piezometric heads parallel to the dam axis.

The geologic detail is a system of small creeks that drain the valley sides and the floodplain of the Cumberland River. These creeks have deposited a series of sand channels that are connected with each other and to the river but make up only a relatively small portion of U2. The limited amount of sands actually present would make the assumption of "full continuity of sands" in U2 used in the stability analysis rather conservative. This high degree of built-in conservatism in the analysis needs to be considered in the evaluation of the dynamic stability of Barkley Dam.

Discussion: The question of what percentage of a potential failure surface at the Barkley site would go through liquefied sands and what percentage through non-liquified clays, silty clays and clayey silts has largely remained unanswered. Because of this uncertainty and because of the rapid response of U2 sands to variations of Lower Resevoir (LR) levels, the conservative assumption has been made in all the stability analyses that the sands are pervasive throughout the site, i.e., that a potential failure surface would occur essentially in sand throughout.

Tel: 217-352-696

The results of the analyses would be conservative (or overconservative) to the extent that the sand percentage in the potential plane departs form that assumption. It is then important to take a more rigorous look at the evidence for "continuity".

Several lines of indirect evidence suggests that the U2 sands are relatively thin bodies (generally not exceeding 2 ft), pressumably lenticular, and that they make up a limited portion (20-30%) of that unit either on a vertical or a horizontal direction. This indirect evidence includes:

a) Rate of pore pressure dissipation as measured by the U-probe (Report by Ardman & Associates).

b) Description of disturbed and undisturbed samples (ORNED and WES staff).

c) Description of U2-equivalent exposure downstream from dam (ORNED staff and A. S. Nieto).

d) Profiling by CPT and electrical conductivity (Geoelectronics and Earth Technology Corporation).

e) X-ray imaging of samples for residual strength tests (Report by G. Castro).

f) General agreement of sand percentages in vertical and horizontal planes (Presentation of Jan. 17, 1986 by R. Olsen).

In spite of all this evidence, the rapid response of U2 sands to LR fluctuations, as far as 3,000 ft away from the reservoir edge, has been interpreted as evidence of the "continuity of sand layers".

My own thinking has been that some of the sand lenses are probably elongated parallel to the river and several of them may connect to create a continuous sand body several hundred feet long (parallel to the river) and a few hundred feet wide. Thus I have not explained satisfactorily the rapid piezometric response 3,000 ft away from LR.

In the following paragraphs I will describe a geologic model for U2 which reconciles the evidence from items a) through f) above with the rapid piezometric response. First, a distinction should be made between "continuity" and "areal extent". The rapid piezometric response establishes continuity but not areal extent, in the same sense that wires conducting electric current have continuity but are not areally extensive (large sheets). Consideration of heretofore overlooked geologic features allows constructing a geologic model in which the sand bodies are a relatively small fraction of U2 in the vertical and horizontal direction but have a high degree of continuity.

The geologic features are the small creeks that drain the vall sides and flood plain of the Cumberland River itself. These creeks have deposited a <u>well integrated</u> system of sands bodies in the Cumberland flood plain each one of which is the small scale equivalent of the entire alluvial fill deposited by the Cumberland River. These sand bodies are called "channel" or "substratum materials" but in the field they are not actually channel shaped. When plotted in cross-sections, however, the vertical exaggeration renders them channel shaped. They are, in fact, thin bodies of sand (up to a few feet thick), up to a few hundred feet wide and up to several thousand feet long. The main stream of each of these mini-drainage systems empties into the Cumberland so that it and the entire system are in direct hydraulic connection with the river. Fig. 1 shows the present configuration of the creek drainage systems in the general area of the Barkley site. The map was constructed from the geologic quadrangle sheets for Birmingham and Grand Rivers, Ky (USGS, 1964, 1966). Note that the river whips around from the southeast so that the river channel has favored the present-day location on the west side of the plain. Also note that the creek drainage probably was modified during construction of Highway 62 and the Illinois Central railroad to the north and south of Barkley Dam respectively. The drainage systems to the north of the highway and to the south of the railroad have been less tampered with and show that they can cover the entire flood plain. Note further that several of the creeks have long runs, of up to a mile, parallel to the river.

One characteristic of streams that drain flat areas - such as the Cumberland flood plain - is that every so often the streams abandon entire sections of their courses in response to increased flows (particularly wet years) or to slight variations in topography such as those that may be created by the depositon of clays, silty clays and clayey silts either by the overbank (flood) stages of the Cumberland or during a lake stage of the flood plain. (There is abundant evidence that lake stages occurred in many of the tributaries of the Ohio when this river deposited large amounts of outwash at their mouths periodically damming them. Alluvial stages occurred when the outwash dams were breached and the lakes emptied.) It is then reasonable to believe that within the fine-grained deposits of U2 are a series of well integrated channel deposits of mostly fine and medium sands with some occasional coarse sands and gravels. On a given horizontal plane these alluvial sand bodies cover the entire flood plain and are a well integrated drainage system. However, they occupy but a small portion of the total flood plain area.

Shown in the sketch of Fig. 2 as dashed lines are three possible previous positions of the creek flowing out of Gulley A (see Fig. 1 as well). Also shown are some hypothetical positions of tributaries for the position of the creek farthest downstream. Early in the development of U2 the stream probably flowed at right angles toward the Cumberland (position 1); as the levee and the near bank sediments built up, the creek curved downstream and met the river progressively farther downstream (positions 2 and 3). Thus each position shown is of a different age and occurs in a different horizontal plane.

Let us assume that the combined widths of tributaries and the main creek along the axis of the dam are about 25% of the width of the Cumberland flood plain. Let us assume further that during the deposition of U2 the creek had an opportunity to occupy every position between the three positions shown. It can be shown that if the channels are not very deep the amount of channel sand encountered by a vertical borehole anywhere along the axis of the dam or just downstream from it would be about 25%. Fig. 3 shows a sequence of ten clay layers and nine maincreek positions. The channel sands can be viewed as established after the deposition of each clay layer. One can see that boreholes drilled in such a sequence would penetrate a limited amount of sand; correlations between boreholes would be impossible except for some correlations perpendicular to the line of cross section; and as seen below, all the sand bodies would be in good communication.

It can be shown that not only all the channels of one position of the drainage system would be in good communication but that probably channels from different positions are also hydraulically connected. Fig. 4 shows three layers of fine-grained materials and two positions of a creek and tributaries. It should be kept in mind that there is a great deal of vertical exaggeration in this and the previous diagram. Note that all the channels are connected although the main creeks never intercept. This happens whenever the more recent main creek channel cuts through the youngest clay layer and comes in contact with any of the channels of the previous system.

2. SPT values as predictors of liquefaction potential.

I concur, in view of the evidence presented by Dr. Castro for the Barkley site and by other workers in general, that N values are not valid to evaluate the liquefaction potential of thinly stratified deposits. N values are strongly influenced by the clays layers above and below the thin sand layers and reflect more the consistency of the clays than the relative density of the sand. The Standard Penetration Test is too coarse a test for the thin sand layers and does not reflect the rapid changes of resistance of these thinly layered deposits. I also agree that the thin sands of U2 generally show higher resistance to cone penetration than the clays. The CPT values, however, are again a function of the sand thickness so if they are used for analysis they need to be corrected for thickness.

3. Residual strength values for deformation analysis.

I believe it is more prudent to obtain values of residual strength from a plot of CPT results vs. backcalculated residual values than from lab tests. Obviously, the CPT values for Barkley would have to be corrected for sand thickness. The laboratory values should be considered less reliable because they are arrived at by very large corrections for sample disturbace - in some cases nearly of one order of magnitude. One would also wonder whether the method to obtain in situ relative densities requires reevaluation in view of the large gaps x-rayed in the samples for residual strength. Further, as pointed out by Dr.

Seed, the liquefied sands in the field may settle creating increases in water content at the top of a sand layer that are not measured in the lab tests.

4. Deformation required to mobilize residual shear strength.

The shear strains required to mobilize residual strength have been estimated at about 20%. If a section of about 40ft within U2 is considered susceptible to liquefaction but if only 25% of that section is sand likely to liquefy, the total surface displacements require to mobilize residual strength should be in order of a few feet only.

I will be pleased to dicuss with you any additional aspects of these issues.

Sincerely,

Alberto S. Nieto

ASN/man

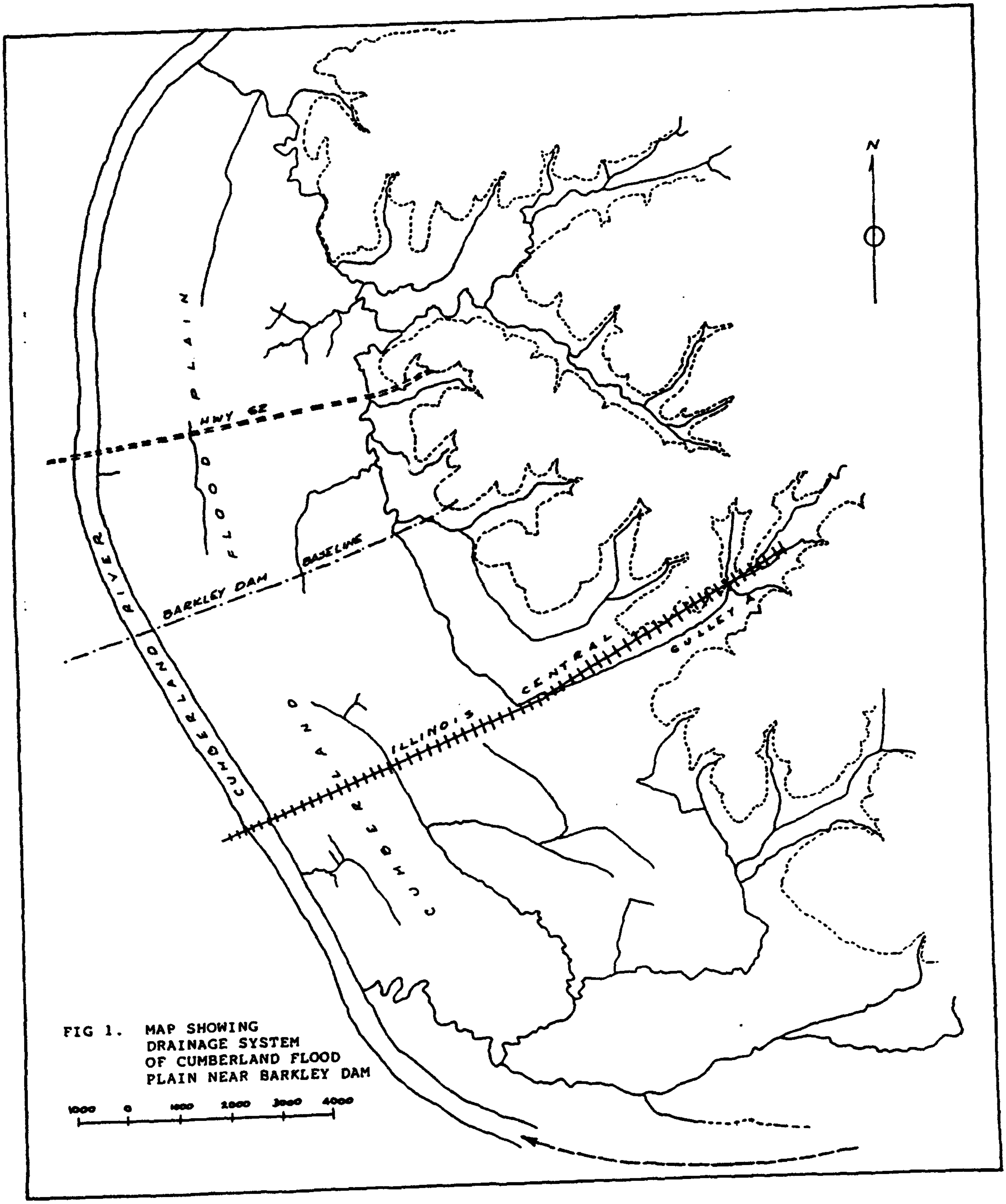


FIG 1. MAP SHOWING DRAINAGE SYSTEM OF CUMBERLAND FLOOD PLAIN NEAR BARKLEY DAM

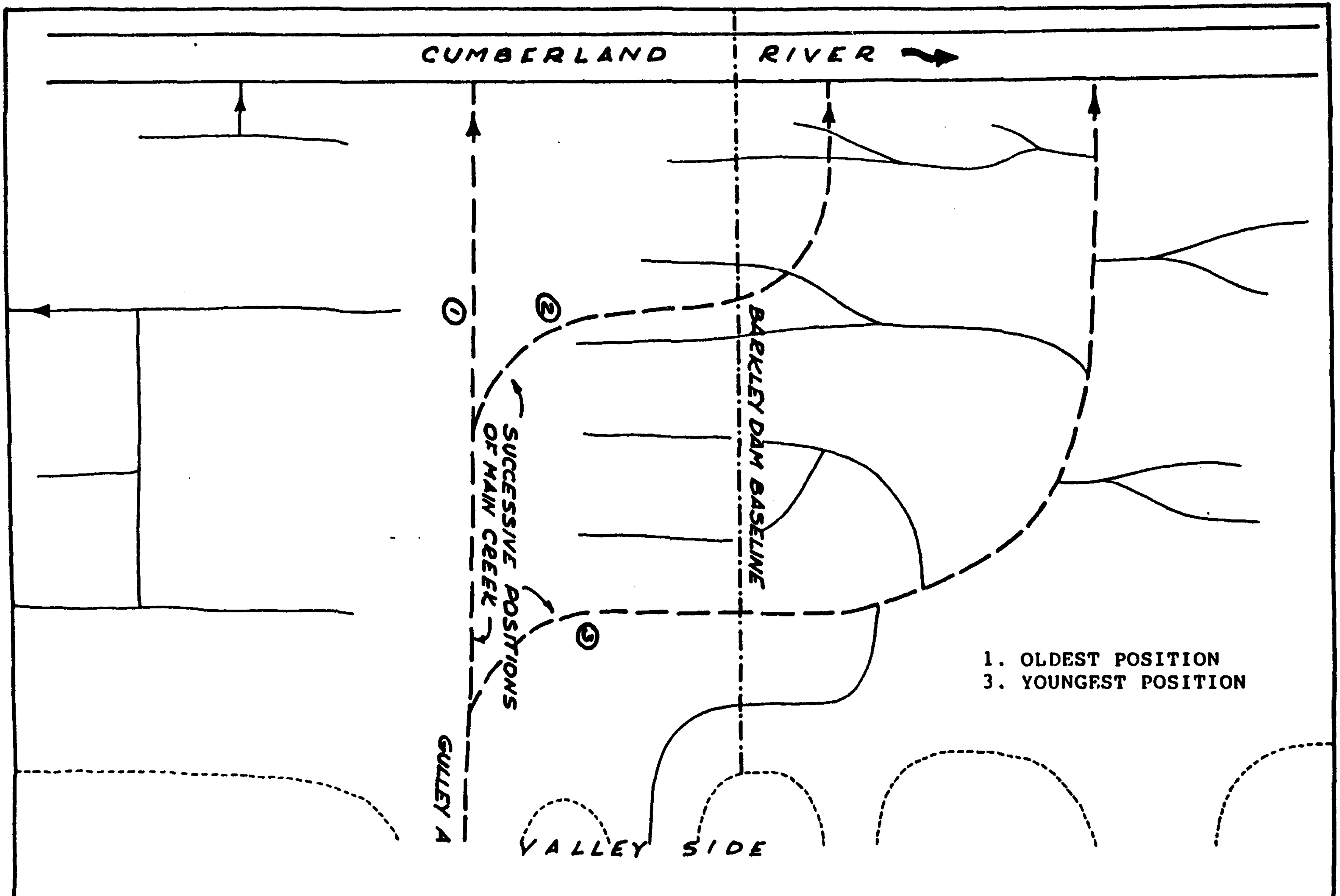


FIG 2. SKETCH OF PROBABLE DRAINAGE SYSTEM NEAR BARKLEY DAM

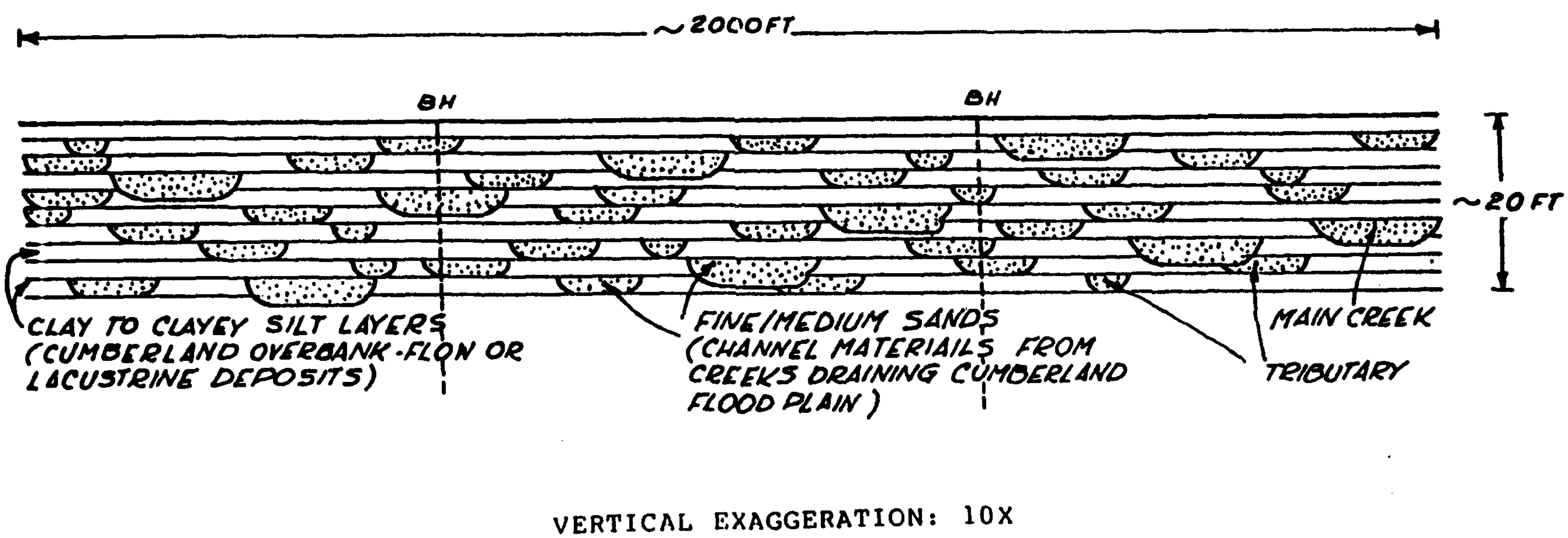


FIG. 3. SKETCH OF HYPOTHETICAL CROSS SECTION THROUGH PART OF U2 PARALLEL TO DAM AXIS. NOTE VERTICAL EXAGGERATION.

AL :RTO S. NIETO, Consultant
Engineering Geology and Applied Rock Mechanics

January 16. 1984

Mr. E. C. Moore, Chief
Engineering Division
Nashville District, Corps of Engineers
P.O. Box 1070
Nashville, Tennessee 37202

REF: Dynamic Stability Studies for Barkley Dam
AN-ORNED-841

Dear Mr. Moore:

This letter contains a summary of the findings on our latest effort to geotechnically characterize the Barkley site. A hydrogeologic model has been developed based on recent field observations at a river bank exposure about 1.5 miles downstream from the dam and on the study of piezometric responses. Whereas the soil units that comprise the model are the same as described in the past, the hydrological picture that has emerged is new and should prove useful in providing basis for the assumptions used in the stability analyses, and guidelines for remedial work. I acknowledge the help of Messrs. Paul Bluhm, who provided the piezometric information and useful insights, and of Marvin Simmons and Joseph Melnyk, who described the downstream exposure.

The assertions that follow will be further substantiated in a supplementary letter to be submitted in the next few weeks.

<u>Conclusions and Recommendations</u>

1) From boring samples, the Barkley site can be characterized in terms of three soil units of contrasting properties. Unit 1, from the surface to about 10-15ft of depth, consists mostly of low permeability material (fat and lean clays). Unit 2, from 10-15ft to approximately 50-60ft, is the criti-

1114 Newbury Road, Champaign, IL 61820 *Tel: 217-352-6965*

cal unit in these studies; it is a complex, highly stratified sequence of soil layers and lenses which range widely in grain size, thickness and areal extent. The grain-size end members are clays and relatively clean, fine- to medium-grained sands but intermediate mixtures (silty and clayey sands, clayey silts, silty clays, etc.) are also present. The thickness of the individual layers and lenses varies from a fraction of an inch to 2-3ft. Correlation of individual layers on the basis of subsurface samples has been unsuccessful, thus the areal extent of Unit 2 layers has been a pressing question in the Barkley studies. Unit 3, from about 50-60ft to about 120ft of depth, consists of more coarsely grained soils, from fine-grained sands and some silts near the top of the unit to conglomeratic sands near the base. Unit 3 is underlain by karstic limestone.

2) At our suggestion, field observations where made by the geological staff at a exposure of Unit 2 along the left river bank, about 1.5 miles downstream from the dam. The length of the most persistent sand layer is at least 100ft. Other sand bodies are more lenticular and extend only from a few feet to a few tens of feet. However, it can be seen that because of their relief two or more individual sands lenses come in touch with one another resulting in composite sand bodies of greater areal persistence. This coalescence of indiviual sand bodies should also take place in the direction parallel to the dam axis (not seen in the exposure). Thus, even if one allows for sand bodies elongated in the direction of river flow, the maximum areal continuity of the sand bodies that can be safely inferred from this expossure should be several times the largest dimension observed (100ft). The thicker clay layers (2-3ft) observed appear rather continous

3) A detailed study was undertaken on the piezometric responses to variations of reservoir heads for the 1981-1983 period. Approximately 10ft of upper-reservoir head are lost as water flows through Unit 1 under more or less normal operating conditions. The piezometers in Unit 2 (WES-1 through 6 and PB-10, 11, 12 and 21) respond slowly (lag time is approximately one month) to head variations in the upper reservoir. On the other hand, the piezometers in Unit 2 respond rapidly (lag time may be 1 or 2 days) to head variations in the lower reservoir even though some of these piezometers are more than 6,000ft away from the limits of the lower reservoir. There is a difference of more than one order of magnitude between the vertical and horizontal hydraulic gradients measured by the piezometers in Unit 2. The maximum vertical gradient measured in 1981 by the WES piezometers was approximately 0.25; at that time the horizontal gradient was 0.01 (PB piezometers).

4) On the basis on the preceding observations the following model is postulated: a) upper-reservoir water loses a substancial amount of head as it flows through Unit 1 which acts as a natural blanket. b) It is also probable that upper-reservoir water may ingress Unit 2 upstream from the dam near the right valley side, in an area where borrow operations stripped the natural blanket. Within Unit 2, most of the water flows horizontally along the more permeable sand bodies with very small head losses. A certain amount of flow takes place across the lower permeability layers (clayey silts, silty clays, clays) again with considerable head losses. Thus, in a given vertical plane of Unit 2 the upper sand layers display higher heads. This, in turn, results in the measurement of a downstream gradient

in Unit 2 and the interpretation of "downward seepage" in that unit. c) The more permeable bodies (cleaner sands) are interconnected over distances of at least a few thousand feet. Silty and clayey sands are probably equally interconnected. d) When bound by areally extensive clay layers, the sandy bodies behave as confined aquifers. e) The sand bodies in Unit 2 and the sands of Unit 3 are probably not connected; this point, however, requires further study.

5) The proposed model has the following implications: a) It explains the piezometric data and the occurrence of the boil at the toe of the dam. b) It indicates that continuity of liquefiable layers should be assumed for purposes of analysis. c) It strongly suggests the feasibility of reducing the liquefaction potential of Unit 2 by reducing water pressures under the dam and downstream from the toe. d) It also suggests, that despite the care exercised, some of the N values of Unit 2 might have been affected by the unique hydrogeologic conditions. The presence of alternating clay layers could allow the development of suction as the sampling tool was withdrawn. This could happen even if the sampling tool was withdrawn relatively slowly and the borehole was full of mud. Because they possess relatively high pressures (heads are only a few feet from the surface), the sand layers would behave as confined aquifers: water and sand from the walls and bottom of the freshly sampled section would flow into the borehole. This flow could create a bulb of loosened sands and disturbed clays that could extend below the bottom of the borehole so that the next section sample would be affected.

6) I offer the following comments regarding the analyses performed by WES: a) Given the importance of N values in

the evaluation of the seismic safety of Barkley dam, it would be desirable to run a series of SPT's on a limited number of borings after the water level has been depressed to elevation 320. Particular care would be exercised in the withdrawal of the sampling tool. This would evaluate the possiblity of sample disturbance and, more importantly, the effect of depressing the water level on the N values of the complex sedimentary sequence of Unit 2. b) The results of the large-deformation analysis should be used as additional evidence that loss of containment of the reservoir is likely and that remedial action needs to be undertaken at Barkley. Despite the apparent reasonableness of this yet-untested method of analysis and of the range of undrained residual strength values, its results ought not be relied upon for not undertaking remedial work. This opinion is based not on a knowledge of soil dynamics but on my own experience and that of others in predicting the static behavior of soil and rock slopes under much simpler conditions of geometry, geology and loading.

7) If the hydrogeological model proposed is correct, then a lowering of the water level under the dam and/or downstream from the toe is feasible and would result in an increased resistence to liquefaction of the Unit 2 sediments. Lowering of the water table to at least preconstruction levels (elevation 320) could be achieved by means of a sheetpile cutoff near the upstream toe, vertical drainage wells near the downstream toe, or long-range horizontal drains (currently attainable to at least 4,000-ft lengths). This remedial action could be used alone or in conjuction with a berm if such is considered necessary.

8) In order to evaluate the points developed in this letter,

I recommend that the water level be lowered to elevation 320 in a small area near the toe by means of three or four pumping wells. A limited number of boreholes (two or three), with SPT samples, should be put down and piezometric and hydraulic conductivity measurements should be made over the small area. The main goals of the study are: a) To confirm and refine the hydrogeologic model (flow quantities, flow directions, vertical communication between Units 2 and 3, etc.) so as to evaluate the most efficient method of lowering the water level. b) To evaluate the effect of the lowered water level on N values and c) To provide design parameters for dewatering in case a trench for sampling is deemed necessary.

I will be glad to discuss further any of the issues in this letter, in particular the details of the pumping program.

Respectfully submitted,

Alberto S. Nieto

Alberto S. Nieto

ASN/lbn

cc. Mr. Frank B. Couch
Mr. M. Simmons
Prof. H.B. Seed
Dr. G. Castro

A.. .ERTO S. NIETO, Consultan
Engineering Geology and Applied Rock Mechanics
AN-ORNED-821

January 12, 1982

Mr. Frank B. Couch, Jr.
Authorized Representative
ORNED-G
Nashville District
U. S. Army Corps of Engineers
P. O. Box 1070
Nashville, TN 37202

CC: Mr. Marvin D. Simmons, Chief Geologist, ORNED-G
Dr. Gonzalo Castro
Professor L. T. Long
Professor H. B. Seed

Dear Mr. Couch:

RE: Barkley Dam Seismic Stability Studies, DACW62-79-C-0194,
AN-ORNED-821

At the request of Mr. Marvin Simmons I am submitting the information gathered in connection with high-resolution seismic reflection as well as my opinions on the continuing effort to characterize the Barkley site geomechanically. This is written without the benefit of a discussion with Professor Seed but by copy of this letter I invite his comments.

INDICATIONS

1. Experts in the high-resolution seismic method agree that, with present technology, it is doubtful the method can outline the geometry of thin (6 in to 1 ft) critical layers at the Barkley site in the interval of interest (0-50 ft). Professor Long and I feel, at this point, that the method could only be used to map the areal extent of thicker layers (2 to 3 ft) if the expense is justified.

2. If SPT borings on 300-ft centers determine that loose sands are responsible for low blowcounts, at least a few loose sand sections are anticipated in many of the borings. Because of the relief characteristic of these alluvial deposits, the possible interpretations regarding continuity will be numerous and require closely spaced drilling. Correlations of sand sections between widely spaced borings (even as little as 50 ft apart) are, of necessity, conservative. I propose that the 10 borings budgeted to establish lateral continuity of the loose sand sections be drilled around one typical boring on a high density pattern (as close as 10 to 15 feet) rather than dispersed throughout the site on 50 ft centers. Correlations based on visual descriptions, downhole logs, and pumping or slug tests should be attempted. The results of this effort then can be used to decide whether to proceed with more high-density exploration or begin considering appropriate remedial work.

1114 Newbury Road, Champaign, IL 61820 Tel: 217-352-696

HIGH-RESOLUTION SEISMIC REFLECTION METHOD

Pursuant to the recommendation that consideration be given to high-resolution seismic reflection methods to characterize the shallow soil section under the dam, I have had discussions with some experts in high-resolution seismic methods regarding the limitations of the technique. These persons are Dr. John Farr, Western Geophysical Corp. of America, Houston; Professor Edward White, Colorado School of Mines, Golden; and Mr. Frank Rusky, U. S. Bureau of Mines, Denver. Further, Professor Long and I have discussed the problem as well as the paper by L. E. Parkinson on high-resolution equipment, and reevaluated the recommendation.

Our problem can be outlined as one of finding a geophysical method which: 1) can resolve the geometry of sand lenses with minimal thicknesses of 6 in to 1 ft throughout a 50-ft section, 2) is field proven, and 3) is relatively inexpensive (inexpensive being arbitrarily defined as less than $50,000.00). The consensus is that, at the present state of the art, high-resolution seismic reflection is not likely to meet any of the above requirements.

The resolution attained with available devices is directly proportional to signal frequency. However, penetration of the seismic signals is drastically reduced as frequency increases. The persons consulted agree that sand beds or lenses several inches thick interspersed in clay can be discerned with underwater devices (pingers, sparkers) that can generate frequencies in the kHtz range but that penetration is not likely to exceed 10 ft. On the other hand, penetrations up to 100-150 ft can be obtained with underwater devices (tuned down boomers) which can generate frequencies in the range of hundreds of Htz;

here, resolution is reduced to beds or lenses 2 to 3 thick. Thus, at the Barkley site, the proposed method could indicate the presence of relatively thick layers for the interval of interest (50 ft) but could not prove that thinner layers or lenses are absent.

Interestingly enough, all three persons suggested that slight variations in stratigraphy for the interval of interest might be detected by an overland method which involves using very small blastcaps as a seismic source, detonating these very small charges to generate a spectrum rich in high frequencies inside a borehole below the water table, and picking up reflections with a hydrophone, also below the water table, in a borehole 10 to 15 ft apart. This method has been recently used with some success in soft rocks at some nuclear power sites. The shallow drilling required for a complete coverage of the area immediately downstream from the dam would be extensive, the amount of data to be processed would be very large, and the total cost is estimated to be well over $100,000.00. Further, equipment and personnel are not easily available. Oil-exploration firms could be retained; but not for periods of less than a month, at a cost between $200,000.00 and $300,000.00. More importantly, none of the experts could give assurance that the resolution requirement could be met.

It would appear that "resolutions of perhaps 1 foot in the first 200 ft or so" mentioned in the paper by Parkinson may not always be achieved. Figs. 14 and 15 in that paper are high-resolution seismic cross sections which in fact show that resolution of beds, at our maximum depth of interest, is not better than 1 ft.

I have discussed most of the above mentioned assertions with Professor Long, and he believes that they are probably accurate. Furthermore, he believes, in closer examination, that the likely presence of shallow, strong reflective horizons and of laterally truncated beds at our site would mask the finer features of the seismic profiles and make the interpretation of thin beds difficult. Therefore, Professor Long and I feel that with the present technology, the high resolution seismic method could be used to demonstrate the presence and lateral extent of thicker beds (over 2 ft) but should not be relied upon to determine the presence, frequency and extent of thinner beds for the interval of interest.

FURTHER EXPLORATION AT BARKLEY DAMSITE

The following paragraphs contain some of my thoughts about the geologic factors that may control the interpretation of results of the exploration program recommended by the Board of Consultants, and eventually the course of remedial action to be undertaken.

As indicated in my letter of June 28, 1980 to Mr. Marvin Simmons, Chief Geologist, I believed that the deposits in the first 50 ft of section at the Barkley site could have been largely formed either by lacustrine (lake) or by overbank (fluvial) processes. I favored the lacustrine origin because it fitted better the recent history of the area and our observations of the samples of BEQ and DS borehole series, and because such origin was a more conservative assumption. The latter means: assuming a lacustrine origin implies extensive continuity of alternating layers of sand and clay--an unfavorable feature to the dynamic stability of the embankment. Thus, I recommended d detailed correlations be made using the available samples. Mr. Joseph Melnyk, Staff Geologist, tried unsuccessfully to establish these correlations, even for boreholes only a few tens of feet apart. In general, it could not be demonstrated that a sand section (say 1 or 2 feet thick) in one borehole was or was not connected to one of several others recovered in the samples of the adjacent borehole, nor that the former was or was not connected to sections that had not been sampled in the second borehole. Mr. Melnyk also noticed that, although some sections showed rapid alternations of clay and sand (sections that had been interpreted by me as varved sediments, these alternations occurred at different depths in the boreholes and could not be correlated. On the basis of this work and on my own detailed observations of samples from borehole DS-1, I abandoned the lacustrine model in favor of a fluvial model--a model which, incidentally, had been favored from the beginning by Mr. Simmons and his coworkers.

The materials in the top 50 ft of section appear to be a combination of overbank and channel deposits. The first 6-8 ft are fat clays most likely laid down by the present river as overbank deposits during relatively very high but very infrequent flood stages. As regards the deeper materials down to 50 ft, thick and more coarsely-grained beds (sands, silty sands, and silts) are probably pointbar (channel) deposits laid down by the old Cumberland River; the thinner layers of any type of material (fine or coarse) are probably overbank (floodplain) deposits laid down on top of preexisting deposits during flood stages of the old river.

The topography over which these thinner, overbank beds were laid down can be rather irregular and is typically composed of a series of elongated ridges or levees and swales which give the floodplain substantial local relief (15 ft of relief and 10% slopes are typical for some rivers of comparable size). Because of this, individual layers may be thick in the swales and may thin out or pinch out toward the ridges. Large floods will deposit relatively coarser materials (fine sands, silty sands, and silts) over the entire floodplain, whereas smaller but more frequent floods will deposit the same materials only in the swales. Finer materials (clayey silts and lean clays) are deposited over the entire floodplain except during the smallest floods. As swales are gradually filled the relief is diminished and the lateral continuity of all the overbank deposits increases. The geometry of these alluvial deposits can be further complicated when the river breaks across the pointbar surfaces and occupies one of the swales, eroding it first and subsequently filling it with coarsely-grained channel deposits.

The preceding geologic description is of a general nature and could not be used as a predictive tool in assessing stability or corrective design. However, it emphasizes that sand lenses and channels of limited width but elongated parallel to the river, as well as fairly continuous layers, should be expected at this site.

The boring program recommended by the Board of Consultants on November 13, 1981 can now be viewed within the geologic scenario just described. The principal objective of the SPT borings on 300-ft centers is to provide the basis for a more detailed and complete description of the stratigraphic column throughout the site, and more importantly, to verify if the low blow-counts are indeed associated with loose sands. If properly executed this should be a relatively straightforward task. If low blow-counts are related to clean or silty sands, the dynamic stability of the dam will depend on the areal extent and frequency of these loose sand bodies. Ten additional boreholes on 50 ft centers have been recommended to establish lateral continuity of these sands. I anticipate a great deal of difficulty in meeting this second goal of the boring program.

If in the first stage of the boring program low N values are found to be related to clean or silty sands, it can be expected, on the basis of already existing N value profiles and samples (BEQ, DS, BD series), that at least a few loose sand intervals will be found within say a 10-ft interval in the first 50 ft of section. The possible interpretations regarding continuity can be several given the depositional complexity and the probable relief of these sand bodies. Fig. 1, 2, and 3 show a simple example of this problem for a total distance of 100 ft (parallel to axis of dam) with boreholes 25 ft apart. Fig. 1 shows the sand intervals as sampled, and Fig. 2 and 3 show a conservative and an "unconservative" interpretation respectively. The strength of the clay "bridges" (Fig. 3) depends on the distance, <u>a</u>, between loose sands and can be considerable enough even for values of <u>a</u> as small as 5 ft to render the embankment stable against downstream displacement if the sands lose all their strength during a seismic event. However, in the absence of reliable correlations one is forced to conservatively assume continuity (Fig. 2), and the wider the spacing of borings the more potentially conservative the interpretation.

Sand sections thicker than about 2 ft probably can be mapped out in the subsurface by high-resolution seismic reflection, if the expense could be justified, or more likely by downhole logs. Thicker sections have very distinctive downhole log signatures and probably can be traced over distances of 50 ft or more. For thinner beds (6 in to 1 ft) the problem of continuity may be impossible to solve by those methods. Here, I suggest that we consider *pumping tests or slug tests* (injection at constant head) and piezometric observations in closely spaced (10-15 ft) boreholes. Sections can be isolated and hydraulic response to pumping or to slugging at several distances can be observed provided that the permeability of these sand sections is high enough. However, here again, as with other exploration methods, positive responses could be useful in establishing continuity but negative responses could not be completely relied upon.

I do not believe that any of the main points discussed in this document are significantly at variance with the recommendations of Nov. 13, 1981 issued by the Board of Consultants.

Respectfully submitted

Alberto S. Nieto

Alberto S. Nieto

ASN: tap

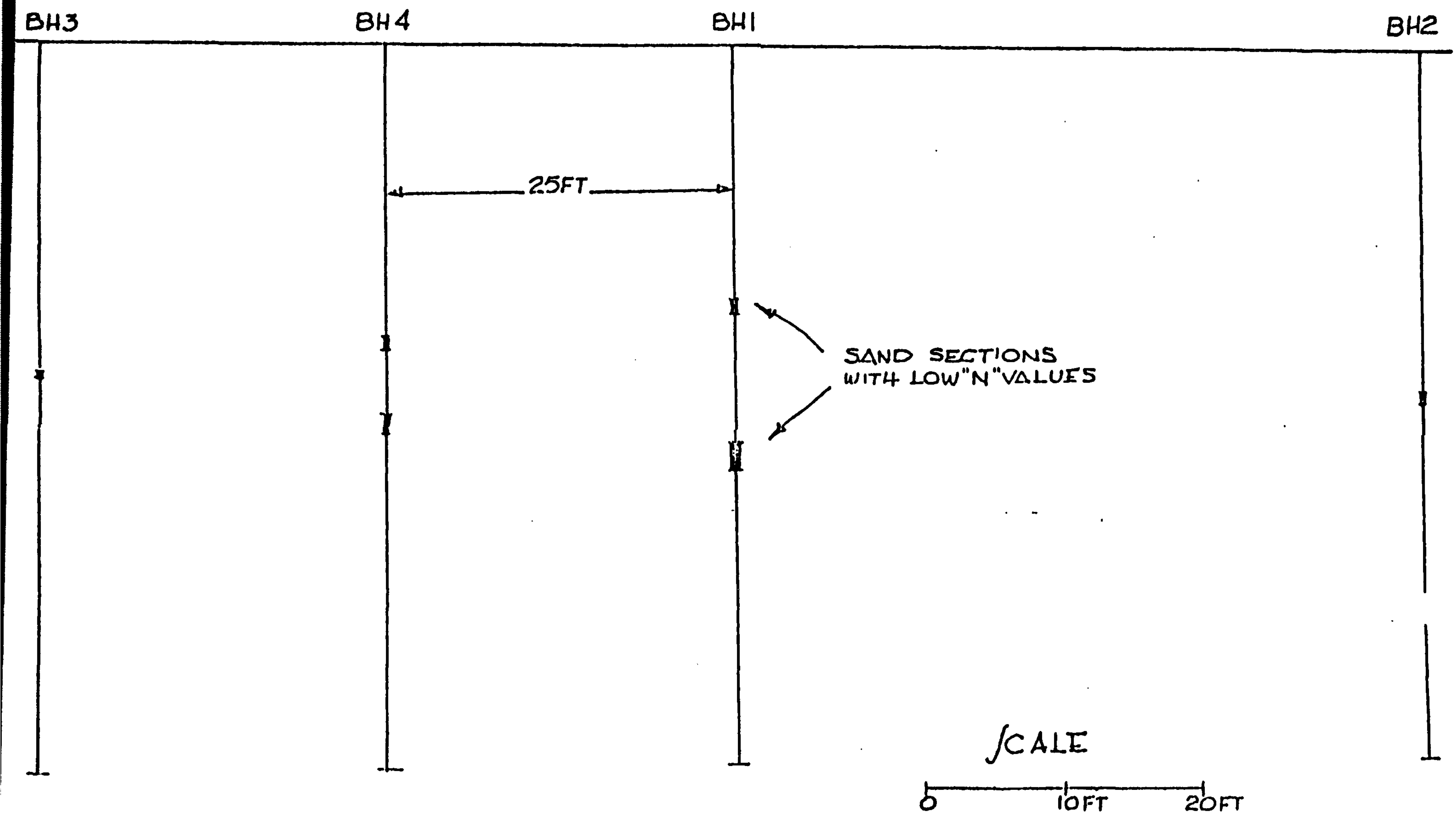


Figure 1. Hypothetical cross-section showing boreholes advanced around BH1 to establish extent of loose sand layers in a section approximately 10 ft thick (remaining sections not shown)

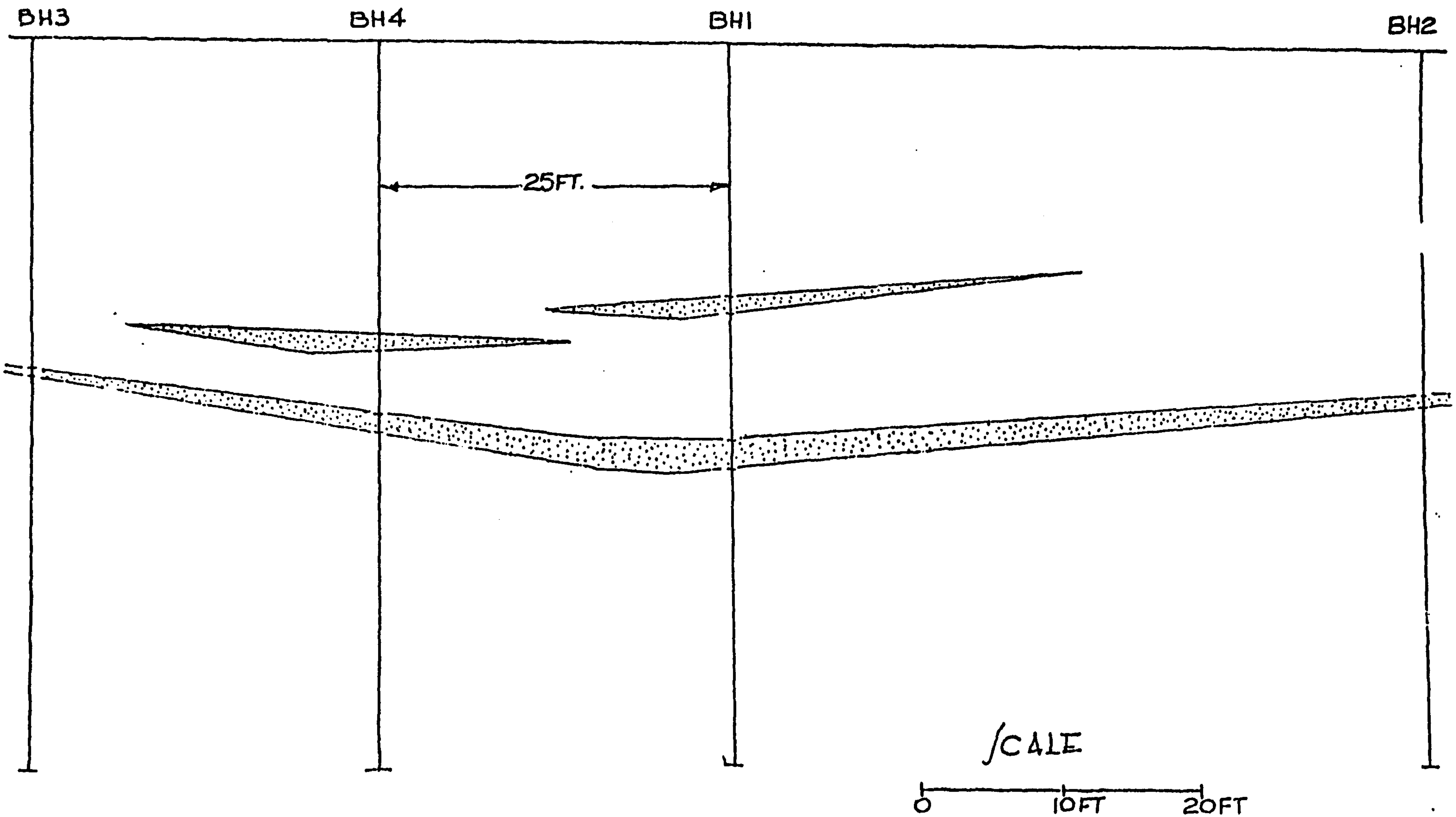


Figure 2. Conservative interpretation of boring data shown in Fig. 1.

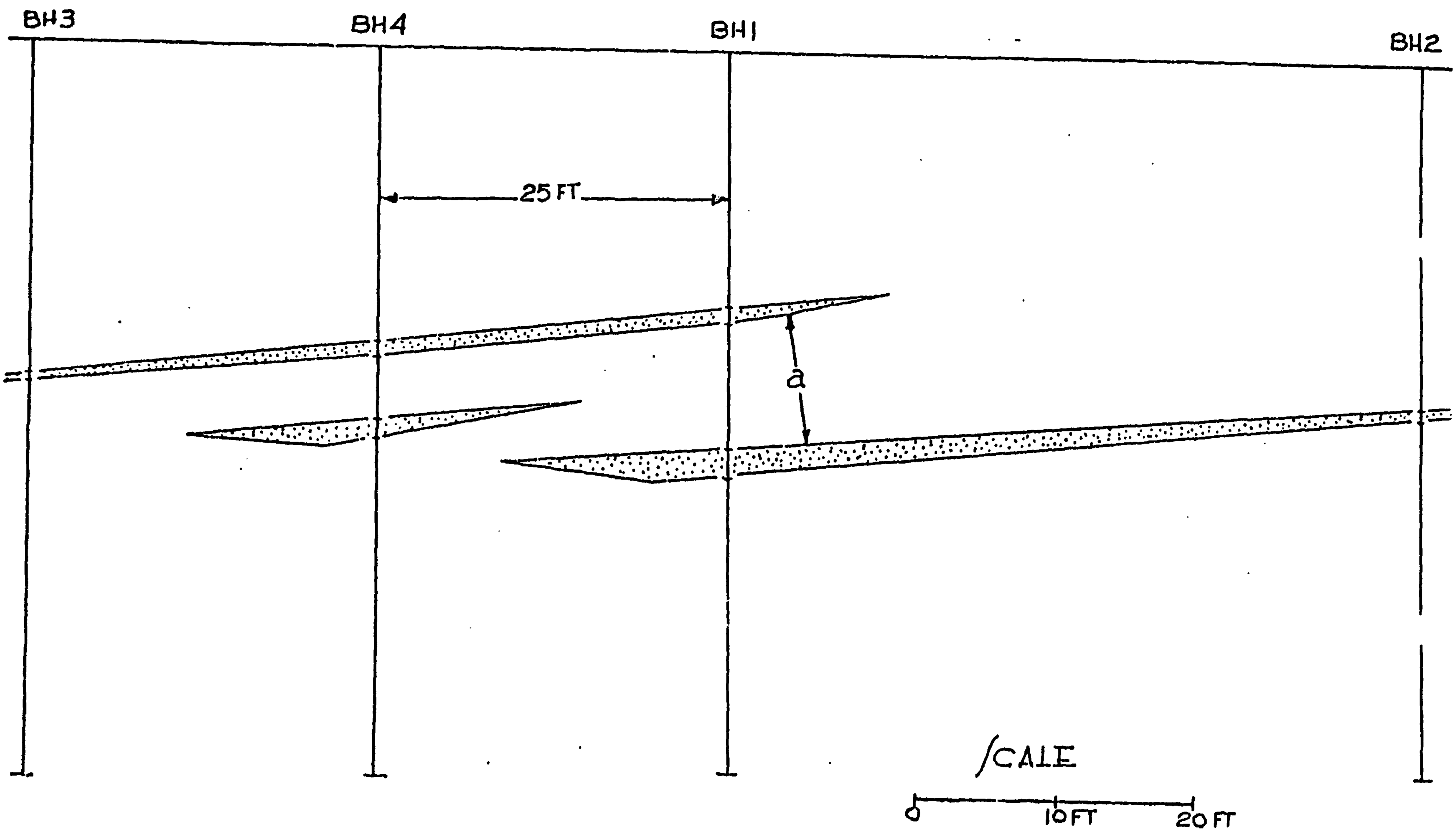


Figure 3. "Unconservative" interpretation of boring data shown in Fig. 1.

H. Bolton Seed, Inc.
623 Crossridge Terrace. Orinda. California 94563
(415) 254-3036

February 3, 1986

Mr. E. C. Moore, Chief,
Engineering Division
Nashville District, Corps of Engineers
P. O. Box 1070
Nashville, Tennessee 37202

Dear Mr. Moore,

On January 24, 1986, I participated in a meeting in your offices in Nashville where presentations were made by members of your staff and the Waterways Experiment Station staff concerning the studies which have been made during the past year to evaluate the seismic stability of Barkley Dam. The presentations were informative and insightful and led to the conclusion that the dam embankments would safely withstand the effects of the Safety Evaluation Earthquake for this project.

Based on my review of the information presented at the meeting I have the following comments and conclusions:

1. The foundation soils for the switchyard section of the embankment, which was the area under discussion at this meeting, are extremely variable and while they consist mostly of clay, it is prudent to consider that continuity may exist between the sand layers.

2. For the type of site conditions existing at Barkley Dam, I do not believe that currently-available analytical procedures provide a reliable basis for evaluation of

seismically-induced embankment deformations under conditions where liquefaction (that is, a pore pressure ratio close to 100%) develops during the period of earthquake shaking. Using the usual methodology for evaluating the possibility of such liquefaction occurring, this would be the case if the computed factor of safety against liquefaction were significantly less than 1.0. If the computed factor of safety is close to 1.0, this means that a condition of r_u = 100% is attained just at the end of earthquake shaking and deformations occurring after this time will be due to static stresses only. Evaluating deformations following such liquefaction under the effects of static stresses only is, I believe, within the capabilities of current technology. Thus it is extremely important to make an evaluation of the factors of safety against liquefaction, defined as r_u = 100%, in the seismic safety evaluation for the Barkley Dam embankments.

3. Because of the fact that the sand seems to exist in thin layers or seams with thicknesses of 6 to 12 inches, it is extremely difficult to evaluate the SPT-resistance of the sand by direct testing methods. For this reason I believe it is better at this site to investigate the penetration resistance by CPT methods and then use a liquefaction potential evaluation procedure, similar to that described by Mr. Olsen of WES, which is calibrated to produce results

which agree well with the large body of existing field performance data for liquefaction evaluations in terms of SPT N_1-values, rather than use the SPT data directly.

4. For the conditions existing at Barkley Dam I would suggest that liquefaction evaluations of the type described in 2 and 3 above be made for the following conditions:

   (a) Use the 35-percentile CPT values of the sand seams for liquefaction evaluation studies.

   (b) To allow for the fact that the full penetration resistance may not be measured when thin layers are underlain by softer clay layers, increase the measured values of cone tip resistance by 10%.

   (c) To compensate the fact that there is some uncertainty concerning the continuity of the sand layers, increase the measured cone tip resistance values by a further 10%.

   (d) Using the above values compute the factor of safety against liquefaction, taking into account the computed initial static stresses in the foundation soils by including the effects of the factor K_α when the equivalent N_1-value for clean sand is greater than 10 and for silty sand is greater than 7.5. Include the effects of the K_σ correction factor in all evaluations.

(e) If the factor of safety against liquefaction is found to be close to 1.0, conclude that a condition of r_u = 100% will develop at the conclusion of the earthquake shaking and then evaluate the consequences of such an occurrence under the effects of the applied static stresses only. My preliminary evaluation of the conditions at the site indicates that this will be the prevailing condition but it requires more detailed confirmation.

(f) For liquefied zones (r_u = 100%) assign the soil a residual strength (s_r) based on its equivalent N_1-value and the correlation I have previously proposed between s_r and N_1. If the computed factor of safety against sliding is less than 1.0, consider the situation potentially unstable. If the computed factor of safety against sliding is greater than 1.0, consider that the sand develops its residual strength at a shear strain of about 25% and hence assess the probable deformation of the embankment on this basis.

My own preliminary evaluation of the seismic stability of the cross-section of the embankment in the switchyard area indicates that

(a) The factor of safety against liquefaction (r_u = 100%) is close to unity, and

(b) The resulting deformation of the embankment following liquefaction will be only about 1 or 2 ft.

If these results are confirmed by more detailed studies than I have been able to make in the limited time available, then I would conclude that the embankment cross-sections have an adequate level of seismic stability and that no remedial measures are necessary to ensure their satisfactory performance for the Safety Evaluation Earthquakes used in this evaluation.

Sincerely yours,

H. Bolton Seed

H. Bolton Seed

HBS/nh

H. Bolton Seed, Inc.

623 CROSSRIDGE TERRACE. ORINDA. CALIFORNIA 94563 (415) 254-3036

March 26, 1984

E. C. Moore, Chief
Engineering Division
U.S. Army Corps of Engineers
Nashville District
P. O. Box 1070
Nashville, Tennessee 37202

Dear Mr. Moore,

In your letter of March 2 you requested that I clarify the statement in item 4 of my letter of December 28, 1983 concerning the Barkley Dam Seismic Study.

I regret that I was not precise in my wording of this statement which was really intended to state the following:

4. The possibility that a hazardous condition may exist with regard to the seismic safety of the slopes of a section of Barkley Dam could be temporarily alleviated by a small lowering of the maximum reservoir elevation if this is deemed necessary and appropriate.

I hope this provides the clarification you were seeking. I am enclosing a revised copy of page 2 with the revised statement for substitution in my report.

Sincerely yours,

H. Bolton Seed

H. Bolton Seed

HBS/nh

Enclosure

H. Bolton Seed, Inc.

623 Crossridge Terrace, Orinda, California 94563 (415) 254-3036

December 28, 1983

Mr. E. C. Moore, Chief
Engineering Division
Nashville District, Corps of Engineers
P.O. Box 1070
Nashville, Tennessee 37202

Dear Mr. Moore,

Further to the discussions at our meeting on the Barkley Dam Seismic Study on December 19 and 20, I am summarizing below my conclusions concerning this project:

1. I believe there is sufficient evidence to conclude that the continuity of potentially liquefiable layers in the foundation should be assumed for analysis purposes and that further investigations concerning this question are unnecessary.

2. The evaluation of the residual strength of a soil is a significant part of any seismic stability study. A potentially valuable approach to this determination of residual strength is through the determination of steady-state strengths by means of $\overline{R}$ tests. However before strengths determined by this approach are used as a basis for actual design, the appropriateness of their use needs to be validated by a good field study.

   There are several reasons for cautiousness in the use of this approach:

   (a) The general principle that all concepts used to determine the field behavior of soils need to be validated by field experience before they can be used with confidence.

   (b) It appears that the best field evidence available concerning the use of steady-state strengths to evaluate the residual strength of liquefied sand is that provided from the observed failure of the upstream slope of the lower San Fernando Dam in 1971. The available data concerning this failure can be interpreted to show either that the residual strength is appropriately predicted by steady-state strength evaluations or that it is not, depending on how the data is used and evaluated. I believe the question can only be resolved by a new investigation specifically conducted to address this question and that such a study should be undertaken in the near future.

(c) Attractive as the steady-state strength concept may be, arguments have been advanced by Corps of Engineers personnel that the residual strength of soils in the field may well be less than that determined in a laboratory triaxial compression test due to limitations of the laboratory test procedure.

Until these matters are clarified by a good field validation of the steady-state strength approach, it is not prudent to be relying on this concept for seismic safety evaluations.

3. I believe that there is an excellent chance that a new study of the lower San Fernando Dam will be made during the next two years and this could greatly clarify the significance of the steady-state strength. The Barkley Dam project could be a major beneficiary of such a study and it may well be justified to defer a decision on such a marginal case as this until the San Fernando Dam study is completed.

4. The hazard presently indicated with regard to the seismic safety of Barkley Dam could be temporarily mitigated by a small lowering of the maximum reservoir elevation if this is deemed necessary and appropriate.

5. I consider that the best data presently available for evaluating the seismic safety of Barkley Dam is the SPT data expressed in terms of N_1 values. The engineering studies of these tests have been carefully performed and interpreted. A preliminary assessment of this data indicates that:

upstream slope of the lower San Fernando Dam in 1971. The available data concerning this failure can be interpreted to show either that the residual strength is appropriately predicted by steady-state strength evaluations or that it is not, depending on how the data is used and evaluated. I believe the question can only be resolved by a new investigation specifically conducted to address this question and that such a study should be undertaken in the near future.

(c) Attractive as the steady-state strength concept may be, arguments have been advanced by Corps of Engineers personnel that the residual strength of soils in the field may well be less than that determined in a laboratory triaxial compression test due to limitations of the laboratory test procedure.

Until these matters are clarified by a good field validation of the steady-state strength approach, it is not prudent to be relying on this concept for seismic safety evaluations.

3. I believe that there is an excellent chance that a new study of the lower San Fernando Dam will be made during the next two years and this could greatly clarify the significance of the steady-state strength. The Barkley Dam project could be a major beneficiary of such a study and it may well be justified to defer a decision on such a marginal case as this until the San Fernando Dam study is completed.

4. The possibility that a hazardous condition may exist with regard to the seismic safety of the slopes of a section of Barkley Dam could be temporarily alleviated by a small lowering of the maximum reservoir elevation if this is deemed necessary and appropriate.

5. I consider that the best data presently available for evaluating the seismic safety of Barkley Dam is the SPT data expressed in terms of N_1 values. The engineering studies of these tests have been carefully performed and interpreted. A preliminary assessment of this data indicates that:

(a) For the clean sands in the foundation soils N_1 seems to be generally greater than 12 and for the silty sands N_1 seems to be generally greater than about 9. For these conditions I consider it appropriate to include the correction factor K_α (or R_α) in the liquefaction analysis procedure.

(b) A reasonably representative value of the effective N_1 for the foundation sands, silty sands and silts appears to be about 17.5. The analysis indicates that liquefaction would not occur under the main dam for this condition and would be limited to zones beyond the toes of the embankment.

Thus there is reason to believe that catastrophic failure of the main embankment would not occur regardless of the residual strength values of the soils. To confirm this preliminary interpretation, the analysis of stability in terms of N_1 values should be refined, finalised and clearly presented.

6. An analysis of the seismic stability of conditions near Station 38 + 00, near the switchyard area should also be made in terms of the N_1 values and the results reviewed to better assess the potential for seismic instability in this zone.

7. In view of the importance of the analysis discussed in 5 and 6 above for this dam, it would seem desirable to refine the analysis by making

   (a) site specific evaluations of the initial static stress conditions in the foundation soils

   and (b) determination of the cyclic stresses induced by the design earthquake motions using 2-D analyses rather than 1-D approximations.

8. It is pushing the present state of the art enormously to attempt to asses residual strengths and very large-scale deformations for an embankment dam on such a complex foundation condition. If it proves necessary to do this

without further testing, a conservative selection of residual strength values would be appropriate. The parameters designated as Class II residual strengths are reasonable estimates but I do not consider them to be conservative choices for such complex conditions.

9. Better data on steady-state strengths of the Barkley Dam foundation soils could be obtained by $\overline{R}$ tests on better quality and more representative samples. Consideration should be given to obtaining high-quality samples from a test-pit excavation in a representative area downstream of the main embankment. Undisturbed samples could then be hand-trimmed from the exposed sides and in-situ densities could be determined directly to ensure that laboratory tests are performed on samples of the same density. Characterization of the zones from which samples are taken so they can be related to other zones of the foundation soils is important if this procedure is followed.

10. It is possible that in spite of the best efforts of all concerned, none of the additional studies that might be performed will resolve the uncertainties concerning the seismic stability of Barkley Dam. Thus it would seem desirable at this time to initiate a study of the costs associated with possible remedial measures.

    However, I believe that some studies are still necessary and some additional investigations can still be helpful and should be performed before a final decision is taken on the need for remedial work.

I trust these observations will be helpful in your further work on this project.

Sincerely yours,

H. Bolton Seed

H. Bolton Seed

H. Bolton Seed, Inc.
138 Whitethorne Drive • Moraga, California 94556

September 2, 1980

Colonel Lee W. Tucker
U. S. Army Corps of Engineers
Nashville Engineer District
P. O. Box 1070
Nashville, Tennessee 37202

Dear Colonel Tucker,

A meeting of the Board of Consultants for the Barkley Dam Seismic Studies was held on August 27, 1980 at the Barkley Dam site to review previous recommendations, discuss new information and approaches and make new recommendations as may seem appropriate concerning the design earthquake to be used in the seismic analysis studies. The meeting began with an inspection of the present condition of the dam in order to provide the Board members with a more detailed knowledge of the overall lay-out and geologic environment and this was followed by detailed presentations concerning the possible design earthquake by Dr. E. L. Krinitzksy (Waterways Experiment Station), Dr. L. T. Long (Board Member and Professor at Georgia Institute of Technology) and Dr. Otto W. Nuttli (Consultant to WES and Professor at St. Louis University).

Following a discussion of the different points of view presented by these gentlemen, the Consulting Board (L. T. Long, Albert S. Nieto and myself) supplemented for this special discussion by Dr. Otto Nuttli met in Executive Session to make its recommendations for appropriate seismic parameters to be used in evaluating the seismic safety of Barkley Dam. Board member Gonzalo Castro could not be present for this meeting. As a result of its deliberations the Board made the following recommendations to the Nashville District concerning the seismic studies for Barkley Dam:

1. The dam should be analyzed initially for seismic safety using a test earthquake producing ground motions believed to be representative of the 80 percentile values which may develop from the maximum credible earthquake to which the dam may be subjected.

2. For the above earthquake, the ground motions should be expressed in terms of the maximum acceleration, maximum velocity and duration of ground shaking which are expected to develop at the ground surface in the free field some short distance from the downstream toe of the dam.

3. The ground motions selected for the analysis discussed above should have the following characteristics

   (a) Maximum acceleration (at about 2 Hz): 0.24g.

   (b) Maximum velocity (at about 1 Hz): 50 cm/sec.

   (c) Duration of shaking in excess of 0.05g: 25 seconds.

The Board also discussed possible acceleration histories having the above characteristics, which might be used for seismic stability evaluations of the dam and its formation. Informal recommendations were made to engineers of the Nashville District and the Waterways Experiment Station concerning the next steps to be made in the seismic analysis studies, with the suggestion that the results of simplified analyses be made for the recommended test earthquake motions and the results of these studies reviewed before any attempt is made to perform either elaborate analytical or experimental studies. It was generally agreed that this procedure would be followed.

The primary objective of this meeting was to establish appropriate ground motions for use in the seismic evaluation studies and the Board considers that this goal was satisfactorily achieved. We will be pleased to review the results of further analyses using the recommended test earthquake motions as they become available.

A copy of the meeting agenda and a list of participants in the meeting is attached.

Sincerely yours,

H. Bolton Seed

H. Bolton Seed, Chairman

HBS/nh

Enclosures

cc: Members, Consulting Board on Barkley Dam Seismic Studies

BARKLEY SEISMIC MEETING 8-27-80

James E. Paris --------------	ORNED-G
Frank B. Couch, Jr. ----------	ORNED-G
Marvin Simmons ---------------	ORNED-G
Joe Melnyk ------------------	ORNED-G
E.L. Krinitzsky --------------	WES
Leland Timothy Long ----------	Georgia Tech
Otto W. Nuttli ---------------	St. Louis University
W.F. Marcuson III ------------	WES
Harry B. Seed ----------------	University of California, Berkley
Mary E. Hynes-Griffin --------	WES
A.G. Franklin ----------------	WES
David P. Hammer --------------	ORD
A.S. Nieto ------------------	University of Illinois, Urbana
Ralph R.W. Beene ------------	DAEN-CWE-S
Wayne E. McIntosh ------------	DAEN-CWE-SG
Robert J. Smith --------------	DAEN-CWE-DS
Thurman Gaddie ---------------	ORDED-T
Russ Fondelier ---------------	ORDED-G
Euclid Moore -----------------	ORNED
Bruce Dunn ------------------	ORNOP-H BAR/P
Milton Myers -----------------	LMVED-G
Todd H. Riddle ---------------	LMVED-G
David E. Wright --------------	SWDED-G
Lawson E. Jackson ------------	SWDED-G
Ronald G. Welbern ------------	ORNOP-R BAR/R
William T. Brown -------------	ORNOP-R BAR/R
Thomas A. Ramey --------------	ORNOP-R BAR/R
John East --------------------	ORNOP-H BAR/P
Gary W. Duncan ---------------	ORNED-G

AGENDA

BARKLEY DAM BOARD OF
CONSULTANTS MEETING
27 August 1980

26 August 1980

Travel to project site; overnight stay at Kentucky Dam Village State Park Lodge.

27 August 1980

8:00 - 10:00	Barkley Dam Site Visit - ORN
10:00 - 10:30	Assemble and opening remarks - ORN, ORD, OCE, Board, WES.
10:30 - 12:00	Discussion at Design Earthquake - WES
	1. Kriniskey - Design Eq.
	2. Long - Consultant
	3. Nuttli- WES Consultant
12:00 - 1:00	Lunch
1:00 - 3:00	Discussion of Design Earthquake
3:00 - 4:00	Summary and Conclusions

28 August 1980

Return Trip

H. Bolton Seed, Inc.
138 Whitethorne Drive • Moraga, California 94556

February 11, 1980

Colonel Robert Tener
U. S. Army Corps of Engineers
Nashville Engineer District
P. O. Box 1070
Nashville, Tennessee 37202

Rest 18 Feb

Dear Colonel Tener,

On behalf of the Board of Consultants for the Barkley Dam Seismic Studies, I am enclosing a final copy of our report on the Board Meeting held on January 17.

Sincerely yours,

H. Bolton Seed

H. Bolton Seed

HBS/nh

Enclosure

cc: Members, Consulting Board

REPORT OF BOARD OF CONSULTANTS

BARKLEY DAM SEISMIC STUDIES

January 17, 1980

The Board of Consultants for the Barkley Dam Seismic Stability Evaluation met in Vicksburg on January 17. Board members had been briefed on the studies conducted by the staff of the Waterways Experiment Station at a meeting in Nashville on November 19, 1979 and provided with a draft copy of the results of the studies for review prior to the January meeting.

During the morning, the Waterways Experiment Station staff responded to questions raised by the Board concerning the studies and then conducted Board members to the Soil Mechanics Laboratory to examine samples of foundation soils from the dam. (Board members Castro and Nieto had also spent a few hours examining undisturbed samples obtained during the foundation investigation on the afternoon of January 16, 1980).

Based on the information provided at the briefing on November 19, the information contained in the WES draft report, responses to questions addressed to individual members on the WES staff between November 19 and January 17, the information gained from examination of the soil samples, and the responses of the WES staff to questions raised by Board members at its January 17 meeting, the following report was prepared.

1. <u>Overall Summary of Seismic Stability Investigations Conducted to Date</u>

The studies conducted to date have involved detailed investigations of the characteristics of the soil comprising the dam and its foundation, the assessment of the characteristics of the most severe earthquake motions to which the dam may be subjected, analytical studies of the response of the dam and foundation soils to the maximum earthquake motions, and evaluation of the probable dam performance during the maximum anticipated earthquake using (1) empirical evaluation procedures and (2) a combination of the computed response of the dam and foundation soils with the measured properties of the soils under simulated earthquake loading conditions.

Somewhat simplified analyses were made for this initial investigation, an approach which is entirely consistent with good engineering practice in the conduct of complicated studies of this type, since they often lead to clear conclusions with a minimum of investigative costs. Based on the studies conducted, it was concluded that "there is considerable doubt about the containment of the reservoir and it is impossible to guarantee that the reservoir would be contained" if the maximum earthquake should occur. Accordingly, it was recommended "that action be taken to improve the earthquake safety of the dam reservoir system."

The Board agrees that such action may ultimately be required, but it also believes that before this conclusion can be considered warranted, more elaborate analyses and studies than those so far conducted should be performed (a) because there is some small possibility that by eliminating conservatism from the studies conducted to date, the dam and its foundation may be found to have adequate stability against earthquake effects and (b) because the large costs likely to be involved in providing remedial treatment to protect the dam against the earthquake effects (should this be found to be necessary) should not be expended until they are found to be absolutely necessary based on the best investigative procedures which can be brought to bear on the subject. The Board believes that there is a sufficient element of doubt concerning the question of the seismic inadequacy of the dam at the present time to justify the conduct of more detailed analyses, while at the same time recognizing that the poor quality of the foundation soils at the Barkley Dam site may not change the conclusions already derived from the initial investigation. Whatever the outcome of the additional studies, however, the Board believes that they will be useful in evaluating the type and extent of possible remedial measures and thus would have value from this point of view regardless of their influence on conclusions and recommendations based on the studies already conducted.

2. Significant Engineering Geological Features

The most significant engineering geologic feature of the soil profile at the Barkley site regarding liquefaction-induced failure, is the presence of lacustrine deposits in the upper 65 ft of the section. Three lines of evidence converge to indicate a lacustrine origin for these soils. First, knowledge of regional geologic history indicates that during the last glacial period (Wisconsinan), almost all tributaries of the Ohio River were dammed at their mouths by outwash levees deposited by a sediment-laden Ohio River. Strong evidence exists for the presence of more than 70 ft of lacustrine deposits in the adjacent Tennessee River. Second, the surface geology map for the Barkley Dam area displays conspicuous lake-related ridges that outline the periphery of the ancient lake. Last, observation of several feet of undisturbed samples shows a clearly rhythmic deposition of clayey beds and more coarsely-grained beds. Quite significantly, some of the coarser beds appear to be, at visual inspection, medium-grained sands with little silt (probably less than 10-15%) and negligible amounts of clay (a few percent or less). One such layer, about 8-in.-thick in Boring DS2, seems to occur more or less pervasively at the site and to be centered around 20 feet of depth. This layer is vertically bounded by clayey beds.

The engineering significance of these lacustrine beds centers about the thickness and areal extent of potentially liquefiable horizons. Lacustrine materials are well known for the relatively large areal extent of these beds with contrasting mechanical properties. Thus, if liquefaction occurred in one of the relatively cleaner sand layers, such an event would probably involve several hundreds of feet, or even thousands of feet, if several adjacent units would coalesce. More importantly, the presence of clayey units above and below the liquefied layer would preclude rapid pore water pressure dissipation. This last characteristic would lead to a loss of shear strength (along a relatively thin zone of material) for a protracted length of time. The end result would be a translational failure of the embankment on the liquefied layer.

Therefore, it is the opinion of the Board that further studies are required to geomechanically characterize the subsoil at Barkley in the light of a lacustrine model. These studies would include: a correlation of conspicuous silty sand layers throughout depths ranging from 20 to 60 feet using the existing samples and samples from additional borings, and an attempt to observe the section in question in the field, either at the Barkley site or in the adjacent valley, downstream from Kentucky Dam.

3. Seismic Design Criteria

The Board of Consultants concurs with the definitions of the seismic zones and the determinations of the maximum earthquakes within each zone. The best available information was used in this phase of the analysis, and procedures were consistent with contemporary and acceptable engineering practice. However, the Board believes that the predicted peak bedrock motions at the site of the Alben Barkley Dam are probably greater than the peak motions that might actually occur and are hence more severe than necessary for the stability analysis. The Board suggests that a reconsideration of the motions for the maximum earthquake could lead to a reduction in the magnitude of some of the ground motion parameters. Such a reconsideration would incorporate recent developments which may lead to a more deterministic approach and would avoid statistical uncertainties which stem from extrapolation beyond the limits of available data. For the $m_b = 7.5$ event at 118 km, the acceleration may be excessive and a reduction up to, perhaps, 30 percent might prove appropriate. In contrast, the duration chosen (10.0 seconds of acceleration greater than 0.05g) is shorter than might be expected for a $m_b = 7.5$ event). Determination of the appropriate duration (or equivalently the amplitude envelope of the bedrock motions) may require additional analysis of existing data. Data pertaining to motions from m_b = 7.5 earthquakes at distances near 118 km are sparse and special consideration may be needed to prevent contamination of the analysis by inappropriate data.

Two possible approaches to the reevaluation of the peak bedrock motions evolved during discussions of the choice of the maximum earthquake. The first approach would be to reevaluate the statistical base so as to determine whether the motions refer to soil or hard rock. If the motions are more appropriate for soil, then the rock motions should be determined by propagating these surface motions down to rock. The resulting rock motions should then be used as bedrock input for a dynamic analysis of the foundation soil-dam system. The second approach would be to replace those portions of the statistical analysis that require extrapolation beyond the data base with semi-deterministic evaluations based on observed attenuation rates for acceleration for different magnitude events. The use of a linear extrapolation in statistical data from VII to IX (MM) may be questionable because the linearity of the intensity scale has not been demonstrated near IX (MM) and because acceleration data for intensity IX (MM) and higher are sparse. In a semi-deterministic evaluation, the source spectral characteristics may help to place upper bounds on extrapolations.

The Board recognizes that many of the changes in parameters that could be considered may reveal compensating effects and eventually lead to only a slight net change in severity of the safety analysis earthquake. Also the reevaluation may reveal that distinct sets of bedrock motions may be appropriate for distant and near-by events. However, the general opinion of the board is that a reevaluation of the bedrock motions for the safety analysis earthquake would result in motions having less severe effects on the dam and its foundation than the motions used in the studies conducted to date.

4. Analytical Studies and Determination of Soil Properties

The analyses of liquefaction presented in the report consisted of the following:

a) Determination of the degree of dilativeness and contractiveness of undisturbed samples by means of $\bar{R}$ tests.

b) One-dimensional wave propagation analysis for determining earthquake shear stresses on horizontal planes.

c) Isotropic cyclic triaxial tests on undisturbed samples.

d) A comparison of the shear stresses obtained in b) with the resistance to cyclic loading obtained in c), which indicated factors of safety less than one for extensive zones in the foundation of the dam.

The Board members feel that the liquefaction analysis presented in the report represents a reasonable but conservative approach. Since the results indicate an apparent unsafe condition, it is appropriate to perform a more detailed analysis. Such an analysis should include the following:

a) A two-dimensional analysis of both static and earthquake stresses using appropriate elastic and viscoelastic procedures.

b) Performance of anisotropic cyclic triaxial tests on undisturbed specimens. Consolidation stresses and cyclic shear stresses should cover the range of stresses obtained in step a).

c) Determination of the steady or critical state of the sands by means of the following tests:

- $\bar{R}$ tests on undisturbed samples. In order to enable the determination of the steady state, it is necessary to use consolidation stresses that are high enough to result in a contractive behavior. Tests in which sands exhibit a dilative behavior do not generally result in a steady state.

• $\bar{R}$ tests on remolded sand specimens. The sand to be tested should be a mixture of similar samples of the most prevalent type of sand, which appears to be a silty sand. By preparing specimens of the same sand at different void ratios and consolidating them to different pressures, it is possible to determine the steady state over a wide range of void ratios. The slope of this steady-state line can then be used in the interpretation of the results of the $\bar{R}$ tests on the undisturbed samples.

d) The modulus of the surficial clayey soils will probably have an important influence on the dynamic behavior of the foundation-dam system. Thus it is recommended that resonant column tests be performed on undisturbed samples of this soil to determine their modulus degradation with strain.

The procedure previously utilized for the handling of the undisturbed samples involved drainage followed by freezing of the tube samples before testing. Given the nature of the soils involved, it is felt that even after drainage the samples remained essentially saturated with the possible exception of the few samples, if any, that consisted only of clean sands. Freezing of saturated samples by placing the full tube in contact with a cold environment could have resulted in disturbance of the soil as the expansion of water upon freezing found no avenue of escape, thus allowing expansion of the voids within the soil mass. It is felt that future soil samples can and should be handled without freezing. It is however essential that a control of the volume of the samples be kept throughout all the steps involved in transporting, cutting, extruding, and setting the specimen in the triaxial cell.

The examination of the samples by the Board members confirmed their intense stratification. It is not generally possible to examine in detail a specimen before testing. Thus, at the end of each triaxial test performed, the specimen should be removed from the cell as intact as possible, and a longitudinal slice should be cut, described and photographed. The zone where deformations concentrated should be described. Grain size tests should not be performed in a mixture of the full specimen but in the soil from the zone where failure occurred. Only in this manner can one judge the significance of each test performed.

Gonzalo Castro
Leland T. Long
Alberto Nieto
H. Bolton Seed, Chairman

Waterways Experiment Station Cataloging-in-Publication Data

Wahl, Ronald E.
Seismic stability evaluation of Alben Barkley Lock and Dam Project. Volume 1, Summary report / by Ronald E. Wahl and Paul F. Bluhm ; prepared for US Army Engineer District, Nashville.
331 p. : ill. ; 28 cm. -- (Technical report ; GL-86-7 vol. 1)
Includes bibliographical references.
1. Dams -- Kentucky -- Foundations -- Testing. 2. Alben Barkley Lock and Dam (Kent.) 3. Earthquake hazard analysis -- Kentucky -- Alben Barkley Lock and Dam. 4. Soil liquefaction. I. Bluhm, Paul F. II. United States. Army. Corps of Engineers. Nashville District. III. U.S. Army Engineer Waterways Experiment Station. IV. Title. V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; GL-86-7 vol. 1.
TA7 W34 no.GL-86-7 vol.1