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STATE-OF-THE-ART FOR ASSESSING EARTHQUAKE HAZARDS IN THE UNITED STATES

Report 22

MAPPING THE EXTENT AND THICKNESS OF LIQUEFIABLE SOIL LAYERS AT ENGINEERING SITES

by

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This study addresses the problem of mapping the subsurface extent and thickness of potentially liquefiable layers in a dam foundation. Accurate delineation of such materials is essential to evaluation of dam safety in seismic regions. Existing techniques for mapping liquefiable sediments are evaluated. Economics, limitations, and usefulness of each method in delineating soil layering are outlined. Case histories are examined which demonstrate the usefulness of each technique under field conditions.

For most sediments, the cone penetrometer was found to be the most useful and economical technique for subsurface delineation of liquefiable soil layers. In all materials the cone was able to penetrate (including some gravel deposits), it gave high resolution and allowed identification of soil layers down to a few inches thick. In sediments too coarse for the cone to penetrate, the Becker drill may be a useful and economical reconnaissance tool. In gravelly and/or cobbly material, soil layers a foot or two in thickness are

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distinguishable from Becker blowcount logs, and retrieved samples. Standard Penetration Testing has been widely used, and is an excellent technique for use in conjunction with CPT. When only SPT is used for profiling, delineation of layers is dependent on test spacing. The SPT may be used in most soil types, including some gravels. Continuous sampling by thin-walled tube is expensive, but provides complete definition of the soil profile. Tube sampling is not possible in materials containing more than about 20 percent gravel. The Central Mining and Equipment (CME) continuous sampler can retrieve core-samples up to 5 ft long that are very useful in soil profiling, but they are more disturbed than tube samples. The CME device is cheaper than piston tube sampling, and may be performed in any material which is augerable. However, samples taken by the CME sampler in loose gravels may be mixed and disturbed. Seismic velocity techniques and geophysical borehole logging are able to indicate a coarse profile of subsurface sediments, but are not very useful in detailed mapping of thin layers.

To gain understanding concerning the minimum thickness of liquefiable layer which would be of concern to dam stability (and, thus, must be detected in subsurface investigation), experts in earthquake engineering were surveyed. A 1-ft criterion was found to be a general practical limit, but continuous thinner layers that might be coincident with a developing failure plane must also be delineated for critical structures. A parametric slope stability study was conducted to quantify the effect of length of liquefiable layer in an embankment. This analysis showed that the length of liquefied layer critical to the stability of an embankment dam increase with depth beneath the base of the dam. Results of this analysis, or a stability analysis of a dam in question can also be used as a general guide in selection of initial test and/or borehole spacings.

Several field studies are reviewed to compare the usefulness, advantages, and disadvantages of the several exploratory techniques considered herein.

PREFACE

This report was prepared by Mr. Scott L. Hardman and Dr. T. Leslie Youd, Brigham Young University, Provo, Utah, under Contract No. DACW39-85-M-1504. It is part of ongoing work at the US Army Engineer Waterways Experiment Station (WES) in the Civil Works Investigation Study, "Methodologies for Selecting Design Earthquakes," sponsored by the Office, Chief of Engineers (OCE), US Army. Technical Monitor for OCE was Mr. Paul R. Fisher.

Preparation of this report was under the direct supervision of Dr. E. L. Krinitzsky, Engineering Geology and Rock Mechanics Division (EGRMD), Geotechnical Laboratory (GL), WES, and the general supervision of Dr. D. C. Banks, Chief, EGRMD, and Dr. W. F. Marcuson III, Chief, GL.

The authors are appreciative of those who provided information, opinions, and comments. In particular, the statements of Dr. K. Ishihara, University of Tokyo, Dr. G. Castro, Geotechnical Engineers Inc., Dr. H. B. Seed, University of California, Berkeley, CA., Dr. L. Von Thun, US Bureau of Reclamation, Dr. W. F. Marcuson, (GL), WES, and Dr. J. M. Ferritto, US Naval Civil Engineering Lab were especially interesting and helpful. Among the many others who provided information for this study are Messrs. Russ Carter and Earl Mikula, U.S. Bureau of Reclamation, Mr. Charles Riggs, Central Mine Equipment Co., Dr. K. H. Stokoe, University of Texas, Austin, Mr. M. J. Bennett, US Geological Survey, and Mr. R. D. Andrus, Brigham Young University. Mr. R. M. Pyke, TAGA Engineering Software, donated the slope stability program used in this project. Messrs. James Geyer and Reed Murray provided help in drafting figures. Constructive comments by anonymous Geotechnical Laboratory reviewers were helpful in improving this report.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

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I. INTRODUCTION

The failure or near failure of several earth dams in the past few decades has heightened concern for dam safety in the United States and abroad. One area of major concern is the possibility of earthquake-induced liquefaction of sediment layers in a dam foundation, causing deformation and possible failure of the overlying structure. This report addresses the problem of mapping the subsurface extent and thickness of potentially liquefiable soil layers. Such information is essential to the evaluation of dam safety where liquefaction is a possible cause of failure.

Methods are available which allow economical and accurate profiling of sediments containing little or no gravel-size material. The methods include cone penetration tests, conventional drilling using split-spoon and thin walled samplers, and various geophysical methods. In some instances trenching may be required to examine these materials. Greater difficulty is encountered in delineating layers in materials containing large amounts of gravel, cobbles, or boulders. The following statement from a study of the Ririe Dam, Idaho, by Sykora, et al. (1986) indicates the difficulty of probing coarse materials:

> One of the problems [in the Ririe Dam investigation] was determining the location, characteristics, and lateral extent of materials susceptible to the development of high excess pore water pressures and possible liquefaction. Due to the large particle sizes contained in the alluvium and throughout rock-fill portions of the dam, drilling and sampling of these materials are very difficult and cumbersome. . . . All material types within the alluvial deposit necessarily have to be sampled and if lenses are susceptible to liquefaction, the lateral extent of liquefiable zones must be identified. [However, sediments at Ririe Dam] contain primarily cobble-sized material which can not be sampled or tested in situ with techniques traditionally used in geotechnical or liquefaction evaluation projects.

A few techniques, such as the Becker Hammer Drill and the large-diameter cone penetrometer, have been employed successfully in dense deposits of gravel and cobbles. Guidelines for the use of these and other investigative techniques in earth dam exploration are proposed herein.

II. GEOLOGIC CONSIDERATIONS IN SUBSURFACE INVESTIGATION

In a project where location of liquefiable soil layers is of primary concern, general geologic conditions at the site should be determined before detailed exploration begins. A preliminary geologic study will indicate the types of material most likely to be encountered, the expected complexity of subsurface layers, and the likelihood of liquefiable soil deposits existing at the site. These factors influence the selection of investigative techniques for the project, as well as initial spacing of boreholes or soundings. Geologic considerations which have a bearing on subsurface investigation of liquefiable materials are discussed below.

ENVIRONMENT OF DEPOSITION

Studies of liquefaction failures resulting from past earthquakes show that the occurrence of liquefaction is limited to specific geologic settings. The depositional origin of a soil layer determines the grain size distribution, looseness of the material, and packing arrangement of the soil grains. Subsequent geologic processes acting on the deposit control the degree of cementation, consolidation, and drainage restrictions in the layer. All of these properties or conditions are related to the susceptibility of a soil mass to liquefaction.

Table 1, compiled by Youd and Perkins (1978) from experience with past earthquakes, summarizes the expected performance of various types and ages of sedimentary deposits during strong seismic shaking. The table shows that

Type of deposit (1)	General distribution of cohesionless sediments in deposits (2)	Likelihood that Cohesionless Sediments, When Saturated, Would Be Susceptible to Liquefaction (by Age of Deposit)			
		<500 yr (3)	Holocene (4)	Pleistocene (5)	Pre-pleistocene (6)
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	—	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Esturine	Locally variable	High	Moderate	Low	Very low
Beach					
High wave energy	Widespread	Moderate	Low	Very low	Very low
Low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
(c) Artificial					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

Table 1. Estimated Susceptibility of Sedimentary Deposits to Liquefaction during Strong Seismic Shaking (from Youd and Perkins, 1978).

recently deposited soils are the most susceptible to liquefaction and that resistance to liquefaction generally increases with age. Soils deposited in a fluvial environment are most commonly disturbed by liquefaction. Deltaic deposits and saturated colluvial and eolian sand deposits have been commonly affected as well. Cases of liquefaction in alluvial-fan and -plain, beach, terrace, playa and estuarine deposits have also been reported, but are not as common as in the deposits listed above. Glacial till, residual soils, and clay-rich sediments are generally immune to liquefaction. Varying degrees of sorting, amount of compaction during sedimentation, and grain-size class for each type of deposition are factors which influence this hierarchy of relative liquefaction susceptibility.

Age of the deposit is an important factor, with increase in age generally decreasing the susceptibility of a soil to liquefaction. Most liquefaction failures have occurred in Holocene (10,000 years or younger) material, while relatively few Pleistocene deposits have exhibited this behavior. Even within the Holocene, liquefaction susceptibility has apparently diminished with age. Sediments deposited less than 500 years ago may be considered most susceptible to liquefaction (Youd and Hoose, 1977). Increase in cementation and compaction of a soil mass with time are primarily responsible for this behavior. Vulnerability of material to liquefaction is generally decreased with depth. Most instances of earthquake-induced strength loss have occurred in the upper 30 feet or so of the soil profile, with relatively few cases reported at depths greater than 100 feet (Seed, 1973). This is so because of the strengthening effect of high overburden pressures and a general increase in sediment compactness and age with depth. The water table must be at a depth such that susceptible materials are saturated, else liquefaction becomes very unlikely.

Because fluvial and deltaic deposits are highly susceptible to liquefaction and are frequently sites of extensive development, they pose the greatest hazard of damage to constructed works. Shallow, saturated deposits of loose material encountered in these geologic environments should be viewed with suspicion.

In a site investigation, the areal extent of all potentially damanging liquefiable layers must be delineated. An understanding of the depositional environment is necessary so that boreholes or soundings may be spaced at intervals which will preclude the possibility of missing a critical soil deposit. For example, channel fill deposits generally occur as long, narrow lenses. To locate these lenses, tests should be closely spaced perpendicular to the direction of flow. Loess deposits are usually extensive and uniform, so larger test intervals would be appropriate.

Geologic processes are extremely complex, and wide variations occur in dimensions of sedimentary layers, even for a given depositional environment. It is therefore difficult to quantify test spacings based solely on the type of deposit under investigation. Specific geologic studies and local experience may be the best guide in specifying initial spacings for the investigation. If these spacings are inadequate in delineating the subsurface stratigraphy of the site, additional tests will need to be conducted. Successive halving of the initial intervals until the desired resolution is attained may be the best approach in this situation. The dimensions of the structure to be constructed as well as the width or length of a liquefied layer that would be critical to the structure are factors to be considered in laying out test spacings. The latter factors are considered in a later chapter.

LIQUEFACTION SUSCEPTIBILITY AND GRAIN-SIZE CHARACTERISTICS

A wide range of materials, from cohesionless fines to gravelly material, have experienced liquefaction in the past. Selection of exploratory methods at a site will be governed to a large extent by the grain size of materials in the subsurface. The methods selected must be capable not only of detecting and delineating liquefiable soil layers but of penetrating surrounding materials as well. Penetration may be difficult in soil layers containing large amounts of gravel, cobbles, or boulders.

Generally, assessment of liquefaction hazard requires more detailed subsurface investigation than investigations for other engineering purposes, both for delineation of liquefialble layers and for evaluation of soil properties related to liquefaction susceptibility of those layers. Preliminary identification of possibly susceptible material is a integral part of subsurface mapping; this preliminary assessment is needed to identify the critical layers to be mapped. An understanding of soil properties affecting liquefaction susceptibility is helpful in this regard.

It is not the purpose of this report to review in detail procedures for evaluating liquefaction susceptibility, nor do personnel responsible for delineating potentially liquefiable soil layers need to be experts in susceptibility evaluation. Nevertheless, understanding of some basic soil properties affecting susceptibility is essential to those delineating possibly liquefiable layers. Liquefiable soils generally have low penetration resistance and low clay content, both of which can be identified on a preliminary basis either from CPT or SPT tests and visual observation of soil samples. Review papers such as Seed et al. (1983) and National Research Coucil (1985) provide the essentials needed by engineering geologists and geotechnical engineers to delineate liquefiable layers in nongravelly soils.

In the past, the feeling has been prevalent that soil deposits containing significant amounts of gravel are immune to liquefaction. This was due in part to errors in laboratory tests associated with membrane penetration, resulting in strength values which were artificially high. In recent years, several instances of the liquefaction of gravelly soils have been identified, and concern about the hazard has heightened. Lateral spreading caused by liquefaction in gravelly alluvial-fan deposits during the 1983 Borah Peak, Idaho earthquake has been studied by Andrus et al. (1986). Coulter and Migliaccio (1966) report that a major landslide appears to have taken place in a sandy gravel during the 1964 earthquake. During the Tangshan earthquake in 1976, a large slide occurred in a sand-gravel blanket on the upstream slope of Baihe Dam near Beijing, China. Wang (1984) has attributed the slide to liquefaction-induced failure of the protective blanket, which contained 50-60 % gravel. Techniques for evaluating liquefaction susceptibility of gravelly materials are not as well developed as for gravel-free soil, but are just as important as delineation of the finer grained layers.

INFLUENCE OF OVERLYING LIQUEFACTION-RESISTANT STRATUM

In some instances liquefaction may occur at depth but cause no disruption to the ground or damage to structures located on or within the ground. Ishihara (1985) has studied the influence of an overlying layer of liquefaction-resistant material and assembled several case histories that give guidance on the matter. Data assembled from sites in Japan are collected in figure 1. These represent cases of liquefaction or non-liquefaction under maximum accelerations of about 0.2 g, with the underlying sand material having a SPT blowcount of 10 or less. There is no evidence of surface damage in these cases where the overlying stratum has a thickness of 3.0 m or greater.

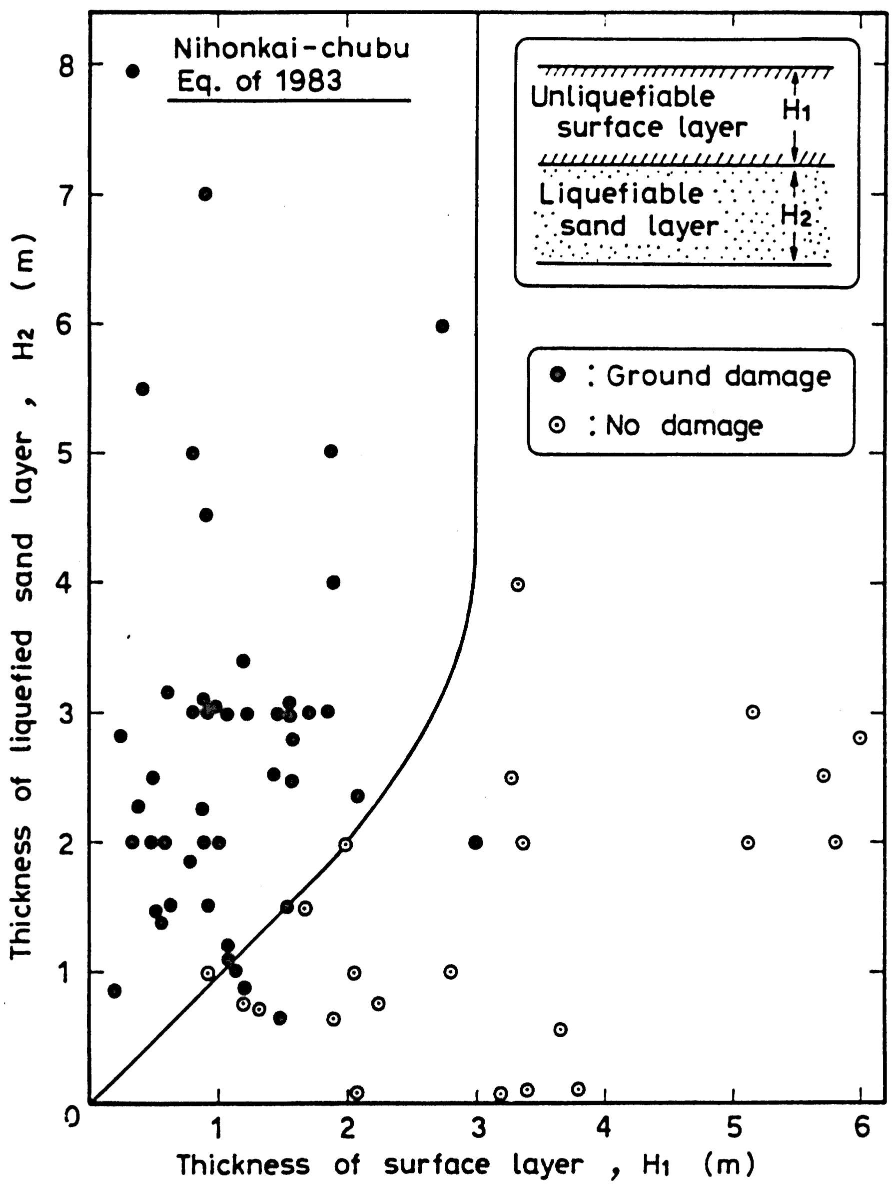


Figure 1. Conditions of Subsurface Soil Stratification Discriminating between Occurrence and Non-occurrence of Ground Rupturing Due to Liquefaction (from Ishihara, 1985).

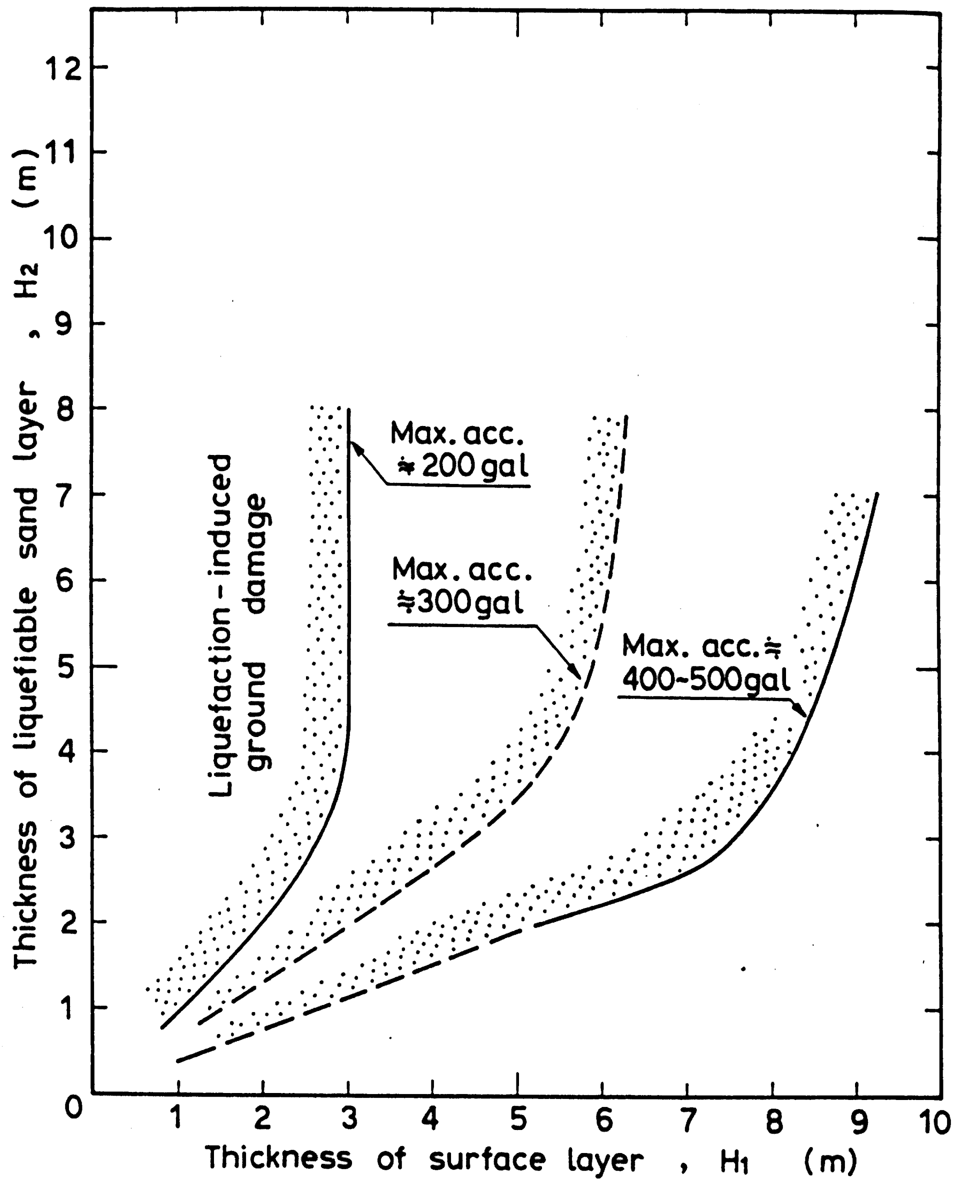


Figure 2. Proposed Boundary Curves for Site Identification of Liquefaction-Induced Damage (from Ishihara, 1985).

If the thickness of the liquefiable sand layer is smaller than 3.0 m, the thickness of the surface layer required to prevent damage would be correspondingly reduced. These conditions are represented by the smoothed boundary line in the figure.

A similar study conducted by Gao et al. (1983) for sites near the epicenter of the 1976 Tangshan, China earthquake showed markedly different results. Based on these site data, the thickness of surface layer required to avoid damage due to liquefaction was much larger. The extreme intensity of earthquake shaking in the area studied and correspondingly high ground accelerations (estimated at 0.40 to 0.50 g) are responsible for this inconsistency. Consequently, Ishihara has proposed a series of boundary curves for site identification of liquefaction-induced damage. These curves, shown in figure 2, may be useful for identifying sites where the potential for ground damage exists, given a certain intensity of earthquake shaking. Further verification is needed before these relationships can be generally recommended, however.

The curves presented by Ishihara and by Gao et al. provide some guidence for depths to hazardous liquefiable layers under level or near level ground conditions. Most of their information was developed from such level sites where applied shear stresses were small. Thus, their conclusions are not directly applicable to sloping sites or sites with imposed loads creating high levels of shear stress such as beneath and embankment dam. with level or nearly level ground.

MINIMUM ALLOWABLE THICKNESS AND EXTENT OF LIQUEFIABLE SOIL LAYER

Liquefaction of a Sand Seam in an Embankment Foundation.

Under some conditions, earthquake-induced liquefaction of a thin, continuous sand seam may result in failure of natural or constructed embankments. For example, it has been suggested that many of the numerous landslides in Alaska during the earthquake of 1964 were caused by the liquefaction of sand seams or thin sand layers underlying otherwise stable masses of soil (Seed, 1973). An understanding of the mechanics and characteristics of this type of slide is necessary in evaluating the stability of an earth dam which might be affected by the same process.

Liquefaction susceptibility of a sand seam is influenced by the initial density, orientation and thickness of sand seam, by the magnitude of the cyclic stresses induced in it by the earthquake, duration of shaking, and by the rate of dissipation of pore-water pressures in the sand seam (Seed, 1973). The conditions involved in this type of failure are shown in figure 3, which assumes no drainage from the sand seam during pore pressure buildup. Variations in the shear stress ratio for static conditions are shown along the plane X-X. Since the high stress ratio near the toe of the slope tends to increase resistance to liquefaction, the number of cycles required to cause liquefaction, N_f, is increased in that area.

The development of sliding under these conditions will likely occur in a progressive manner. Liquefaction may develop first in the sand seam well behind a slope and extend toward the face of the slope as earthquake shaking continues. A nonliquefied section under the slope will act to prevent sliding in the early stages. As the liquefied section extends, however, horizontal forces might become large enough to overcome the resistance provided, and

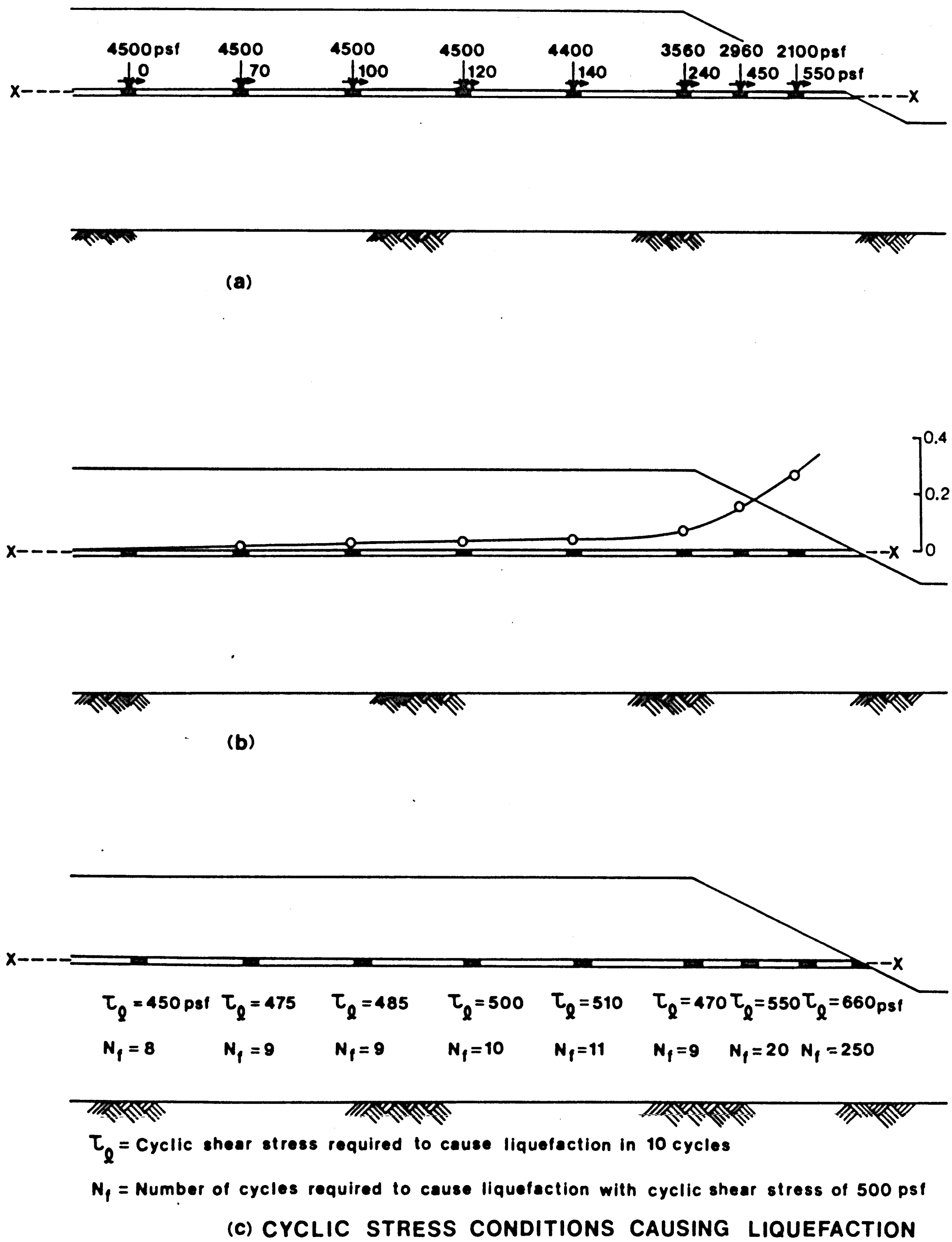


Figure 3. Development of Liquefaction in Sand Seam Underlying Earth Bank. (a) Vertical Normal and Horizontal Shear Stresses on Plane XX; (b) Variation of Cyclic Stress Ratio Along Plane XX; (c) Cyclic Stress Conditions Causing Liquefaction (from Seed, 1973).

deformation of the embankment will occur. Permanent deformations or a flow slide would be the result, depending on the initial void ratio of the sand, as well as slope of the surface and sand layer. The extent of the slide mass is also determined by the slope configuration and depth of the sand seam.

Sand lenses may act in a similar manner during earthquake shaking. Liquefaction of sand lenses embedded in clay deposits develops progressively from the outer edges toward the center and may occur considerably faster than in continuous sand seams or layers. If a sufficient number of discontinuous lenses are present beneath a natural slope or embankment, failure may occur progressively in a manner similar to that described above. In such a case, the failure surface would necessarily pass through the clay or other nonliquefied material between the individual lenses. This mechanism has been proposed for the disastrous Turnagain Heights landslide during the 1964 Alaska earthquake (Seed, 1973).

Because liquefaction of sand lenses can have such an adverse effect on the stability of sloping ground or an embankment, delineation of these leneses is an important part of safety evaluations for such sites or structures.

Expert Opinions on the minimum thickness of critical sand lenses:

No widely accepted criterion has been established with respect to the minimum thickness of hazardous liquefiable layers. In order to gain insight concerning the minimum thickness of liquefiable soil layer which would be a potential problem in a dam foundation, a survey was conducted in which experts on soil liquefaction were asked the following auestions:

1. What, in your opinion, is the thinnest liquefiable layer that one needs to be concerned about in the foundation of an embankment dam? Is a one-foot minimum thickness an acceptable criterion?

2. Would this minimum thickness be affected by the slope of the soil layer? By its depth in the soil profile? By other factors?

The following quotes are excerpts from some of the responses which we received.

Dr. **Kenji Ishihara**, University of Tokyo:

The thickness of the liquefiable sand layer required to get by the danger of dam collapse would depend upon the permeability of the overlying materials. Following liquefaction, water tends to migrate upward and the sand tends to settle down, and if the permeability of the overlying material is small, water is entrapped within a thin layer atop the sand layer. This thin layer consisting of extremely loose sand full of water appears to act as a lubricating sheet with disastrous consequence. There are some cases where seemingly liquefaction induced failure took place in some dams, while no diagnostic evidence was observed on the surface. I came across this type of failure recently in Chile in the area affected by the March 1985 earthquake. The homogenous earth dam was 15 meters high and suffered about two meters of settlement with large strips of cracks developing on the slope. As far as inspected through the cracks and from a trench excavated over the slope, the soils are cohesive fine-grained silt with inclusions of gravel. At the toe of the slope, there was a thin layer of silty sand which appears to have been responsible for triggering the slide.

The other factor to be considered is the continuity of the liquefiable sand layer on the horizontal plane. If the sand layer is continuous horizontally and overlaid by an impervious overlying layer, one-foot criterion would be a good round number.

I think the slope of the interface between the sand layer and overlying material is a factor influencing the stability, because the slip can be easily motivated over such an interface. As far as the depth is concerned, the sand layer at the elevation of the dam's toe is the one to be considered most seriously. In the case of a dam impounding water, there is generally a hydraulic gradient directed downstream. If the sand layer near the bottom of the dam is subjected to such a hydraulic gradient, the danger of liquefaction-induced failure seems more serious.

Dr. **Gonzalo Castro**, Geotechnical Engineers, Inc.:

I will attempt in this letter to answer the questions raised . . . concerning the minimum thickness of a soil layer that one should be concerned about in analyzing liquefaction potential. The question can be answered only in connection with the particular mechanism of liquefaction one is concerned about.

A true liquefaction failure such as a flow slide (e.g., Lower San Fernando Dam) or bearing capacity failure (e.g., Niigata) involves a mechanism of strength loss (not just softening) leading to loss of stability. In such cases, the location and thickness of the layer would be of concern if it is such that a loss in strength of the layer leads to a possible mechanism of failure. In the case of a slope stability problem, a thin layer that daylights downslope could be critical. In the case of a potential bearing capacity failure, the mechanism of the failure requires that the loss in strength affects a larger volume of soil. When the liquefaction potential of a sand layer is in question, I recommend that a stability analysis be performed assigning a low strength to the sand layer. The strength assigned to the layer should be compatible with whatever information is available (e.g., SPT, cone data, laboratory tests) and need not be zero. If the analysis indicated that the configuration in question is stable, then the layer in question is not of concern relative to true liquefaction.

Other phenomena, also referred to by some authors as liquefaction, involve ground settlements, slumping of slopes, and sand boils. One cannot establish a minimum thickness for a sand layer that would be of concern for all phenomena. If one is studying ground settlements, one could conclude that a sand would decrease in volume by say 5% as a result of an earthquake. If the allowable settlements are say 0.05 feet, then one foot would be the minimum thickness one should be concerned about. For slumping of slopes, one has to relate predicted shear strains in a layer to overall movements. The minimum thickness of the layer would then be a function of predicted strains and allowable movements.

An important factor that enters into the question of the layer thickness relates to the geologic origin of the soils. Recent alluvial deposits have been shown to be most prone to liquefaction failures. Such deposits are generally very heterogenous in composition and density. It should be noted that this is true even if the blowcounts are consistently low or high because blowcounts reflect roughly average conditions, not only within the 18 inch drive but also of looser or softer soils that may exist below the range of depth penetrated by the spoon. A practical consequence of the heterogeneity of alluvial deposits is that it is likely that in a thin layer there would be areas which are dense and not susceptible to liquefaction and thus prevent a liquefaction failure through the layer. In a thick (several feet) layer, it is more likely that there would be continuity of loose zones along the layer which could thus result in a potential liquefaction.

Dr. **H. Bolton Seed**, University of California, Berkeley:

The thinnest liquefiable layer to be concerned about in a soil deposit depends on the probability of continuity of the layer and on the nature of the overlying and underlying layers. If the layer is sandwiched between impervious layers and if it is close to an exposed boundary, then a layer less than one foot thick could well be a critical zone of weakness along which sliding could take place. If the slip surface is not horizontal then a one foot layer may be of minor importance.

The minimum thickness of a critical layer will depend on:

1. the slope of the layer,
2. the nature of the overlying and underlying layers,

3. the continuity of the layer,
4. the geometry of the potential slip surface,
5. the configuration of the embankment relative to the layer, and
6. the depth of the layer,

among other factors.

I am not aware of any published guidelines concerning the minimum thickness and extent of liquefiable layers which should cause concern. However I note that the question is addressed regularly on an individual basis by many engineers involved with the seismic stability of dams, with each forming his own conclusions on the matter.

Dr. **Larry Von Thun,** U.S. Bureau of Reclamation:

The criteria [for minimum allowable thickness of a liquefiable layer] is not thickness alone but also three other factors: continuity, in-situ void ratio, and regularity. The only way thickness alone would control is if the shear deformation across the layer disrupted the continuity of the layer so the slide plane was taking place in a new material (such a thickness may be on the order of 1/8-inch). However, taking into account geologic deposition irregularity at a particular site and the scale of a specific civil engineering work the realistic limit thickness may vary significantly. One-foot is not a good criteria.

The minimum thickness is affected by the factors noted above [slope of the soil layer and its depth in the profile]. Slope is not a factor in whether or not the material liquefies, but it is very much a factor in whether or not a flow slide occurs.

Dr. **William F. Marcuson,** III, U.S. Army Corps of Engineers:

In reference to your question [regarding minimum allowable thickness of a liquefiable layer], I think that any layer that is continuous should be of concern. Thus, I guess, if a layer is one inch thick and is continuous under the entire cross section or continuous over a portion of the dam that could fail, it could cause you problems. If you go back to Fort Peck Dam you will note that bentonite seams (some as thin as one inch) caused excessive straining which led to liquefaction in the foundation sands (Middlebrooks, 1942).

There is no doubt that the soil slope and also the depth of the soil would have an effect. Another factor to consider would be an impermeable layer on both sides of the material which would prevent any drainage.

Mr. **John M. Ferritto,** Naval Civil Engineering Laboratory:

This letter is written in response to your letter posing questions on liquefaction of dam foundations. In the central region of a dam the high vertical stresses imposed by a dam's weight tend to reduce the stress ratio; however, at the edges of the dam where the initial static shear stress is high, special concern may be warranted. Yoshimi and Oh-Oka (Japan Soils and Foundations Vol. 15, September 1975) investigated the effect of stress reversals on liquefaction in regions of initial static shear.

A one-foot criteria may be a practical and economic limitation of current sampling procedures. Also of significance is the spatial extent of the one foot layer. Is it an isolated lens or a continuous layer? Anything that thin may tend to be broken and intermittent.

Dr. **T. L. Youd**, Brigham Young University:

In the past few years, I have been directly involved in about 20 subsurface investigations at sites of liquefaction and ground failure and have read reports from several other investigators. Ground failures at these sites have included flow failures, lateral spreads, ground oscillation, loss of bearing strength, and ground settlement. In each of the subsurface investigations, the granular layer identified as the one that liquefied and deformed was thicker than one foot, and in almost all instances the layer was several feet or more thick. The major zone of deformation, however, could have passed through a thinner zone within these layers. The latter detail is difficult to detect from the results of standard drilling and penetration investigations.

It also seems to me that if the ground failure is the consequence of liquefaction and limited flow or cyclic mobility rather than complete loss of strength or flow failure, it would be unlikely that deformations detrimental to most structures would develop in a layer less than about one foot thick. In such materials, dilative tendencies within the soil would force shear deformations to be distributed across a zone rather than occurring within a single thin shear plane. As a consequence, the amount of deformation would be a function of the thickness of the layer. Thus the thinner the layer, the smaller the ground displacement.

On the other hand, if large loss of strength occurs, then flow failure could well develop in a layer less than one foot thick. In that instance, the amount of shear deformation would be controlled by the shear forces acting on the layer and the length or continuity of the layer and its orientation with respect to a developing failure surface. For any given sedimentary depositional environment, the length of a continuous layer generally decreases with the thickness of the layer so that the thicker the layer, the more likely it would be continuous and subject to failure.

My opinion and experience is that few significant ground failures are likely to be generated by liquefaction of granular layers less than one foot thick unless the sediment is very loose. For thin, loose layers, continuity and orientation of the layer with respect to a developing failure plane become the controlling factors. Because the probability of long unfavorably oriented layers also decreases with decreased thickness of the layer in most geologic environments, it would be an unusual situation for layers less than one foot thick to have sufficient continuity to pose a major liquefaction hazard to a large structure such as an earth dam. Nevertheless, for critical structures, knowledgeable engineers and engineering geologists should evaluate the continuity, orientation, and liquefaction susceptibility of all granular layers more than a few inches thick..

From the variety of responses received, it was concluded that no single answer exists for the question of the minimum allowable thickness and extent of liquefiable soil layer in a dam foundation. The variables involved are many, and geotechnical and geometric aspects of the particular dam under investigation must be considered. Factors which influence the stability of a dam founded on potentially liquefiable sediments are:

1. Continuity and depth of the liquefiable layer.
2. Orientation of the layer with respect to a potential failure surface.
3. Looseness of materials in the layer.
4. Grain-size distribution of the soils.
5. Permeability of surrounding materials.
6. The embankment configuration.
7. Geometry of the potential slip surface
8. Mechanism of failure.

The experts agree that any liquefiable layer which is continuous for a substantial length beneath an embankment dam is a potential hazard to the stability of that structure under seismic shaking. Thus, in an earth dam foundation investigation, all detectable liquefiable layers more than an inch or so thick should be identified on the log for each bore-hole or sounding and comparissons made between adjacent test sites to establish the lateral extent and orientation of these potentially unstable sediments. The methods of exploration selected will determine the detail with which this objective may be achieved.

Results From Parametric Slope Stability Study.

To quantify the effect of length of a liquefied sand layer in the foundation of an earth dam, a parametric slope stability study was conducted. A microcomputer version of the slope stability program TSTAB which incorporates Spencer's method of slices was used. The purpose of the study was to establish guidelines for the minimum length of sand layer which, if completely liquefied, would be critical to the stability of an embankment.

Figure 4 shows the conditions assigned to the study model. Stability of this configuration was investigated assuming circular arc failure for varying heights (H) and slopes of the embankment, depths of the horizontal liquefiable layer (D), and strengths of the foundation material surrounding the sand layer. The strength of the foundation soil was assumded to increase with depth as a linear function of the effective overburden pressure. A circular arc analysis was chosen rather than a sliding wedge analysis because the parameter to be determined is the shortest liquefied layer that is critical to stabliity of the embankment. For this condition, most of the failure surface was inclined through unliquefied material and thus is reasonably approximated by a circular arc. Stability was found to be relatively insensitive to the strength parameters and unit weight assigned to the embankment, so these variables were held constant. It was assumed that any liquefied material in the dam foundation would reach complete strength loss at the conclusion of earthquake shaking. Consequently, no earthquake acceleration forces were included in the model.

The static stability of a given configuration with out soil strength loss was determined first. To do this, the program was required to select a failure surface tangent to a horizontal line at depth D. The program then

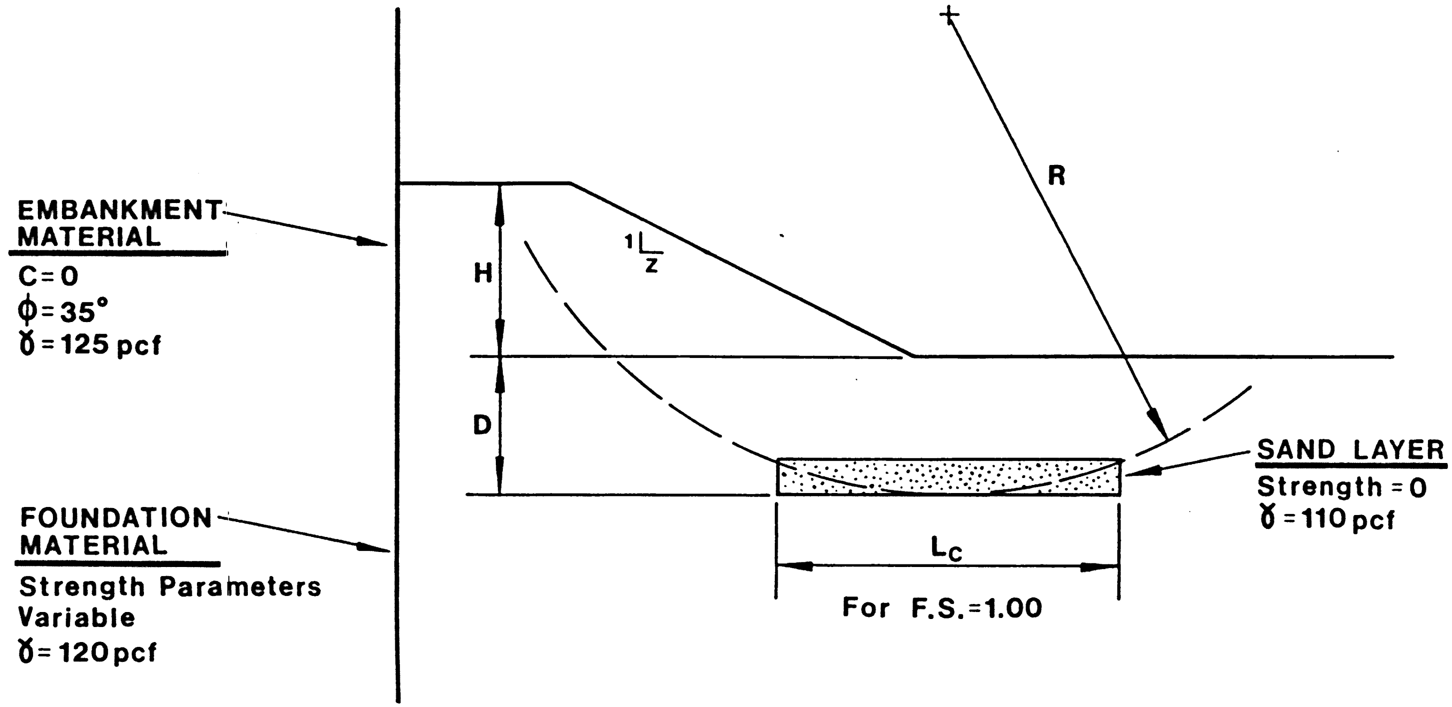


Figure 4. Conditions Modelled in Parametric Slope Stability Study, with Liquefiable Soil Layer in Embankment Foundation.

performed a search routine to locate the slip circle with the smallest factor of safety for this condition. Once the static factor of safety had been calculated, a horizontal layer of liquefied sand of length L was introduced in the model at depth D. A shear strength of zero was assigned to the liquefied layer, representing a worst-case condition. In actual fact, the entire layer may not experience total strength loss. The length L was varied until a factor of safety of 1.00 was obtained. This length was considered the critical length of sand layer, L_c, for the case in point. The analysis was then repeated for various depths, D.

The data obtained for embankment slopes of 1.5:1, 2:1, and 3:1 are shown in figures 5, 6, and 7, respectively. Strength characteristics of the foundation material, which may be considered as S_u/p' or tan o, are plotted on the ordinate. Jamiolkowski et al. (1985) suggest that the following formula applies to many clay materials:

$$S_u/p' = (0.23 \pm 0.04)\ (OCR)^{0.8}$$

This equation yields approximate strength ratios of 0.23, 0.40, and 0.70 for materials with overconsolidation ratio of 1, 2, and 4, respectively. For dense granular materials, use of tan o to characterize strength of the foundation materials is appropriate. The horizontal axis of figures 5-7 represents the dimensionless ratio L_c/D, and curves for constant D/H are plotted.

The following example shows how these curves are used. For a strength ratio characterizing foundation sediments of 0.50, an embankment height (H) of 100 feet, and a sand layer at a 20-foot depth, what is the critical length of liquefied material to reduce F.S. to 1.0? For the above conditions, D/H =

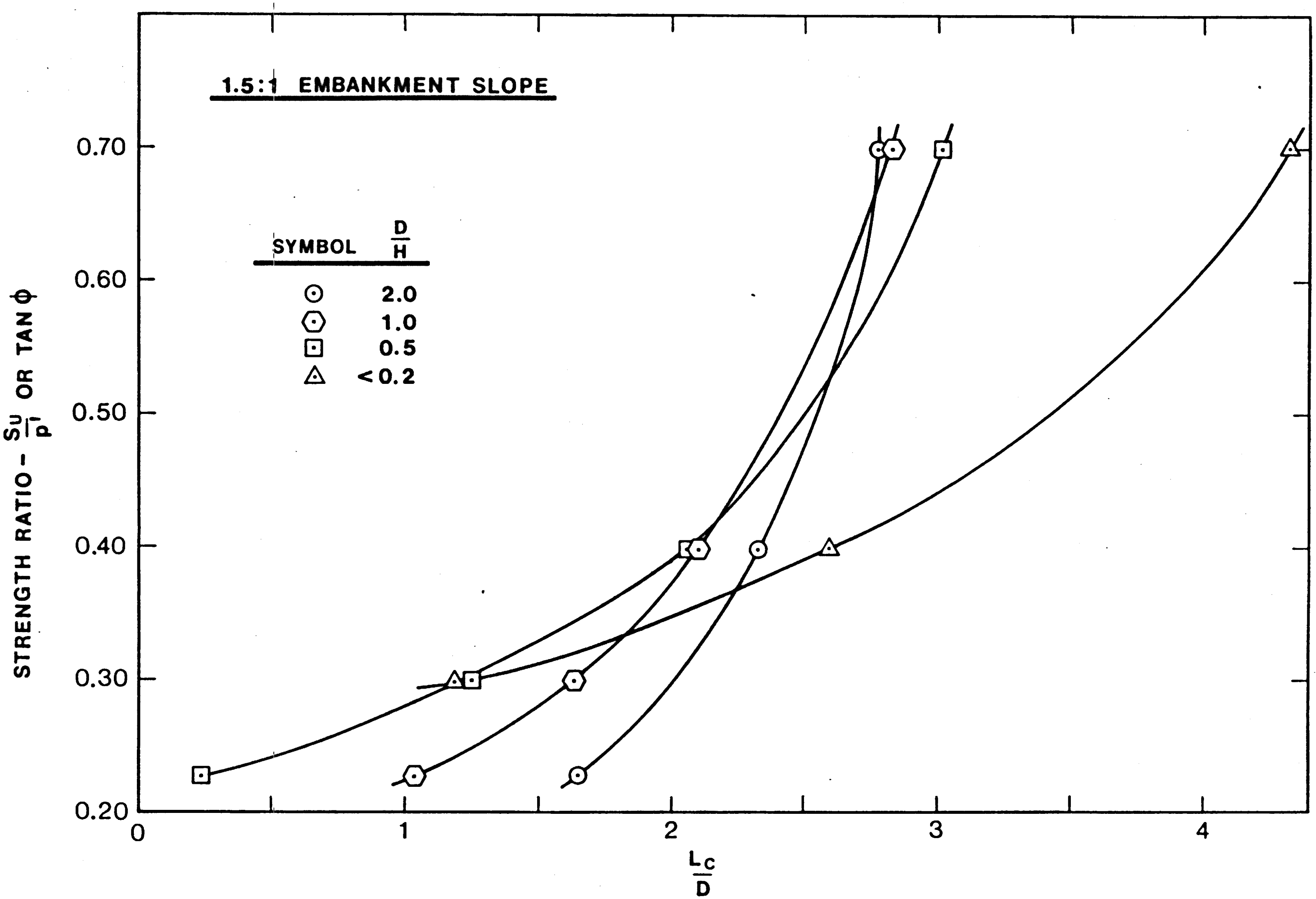


Figure 5. Critical Length of Liquefiable Layer (L_c) for Varying Depth of Layer, Height of Embankment, and Strength of Foundation Material; 1.5:1 Embankment Slope.

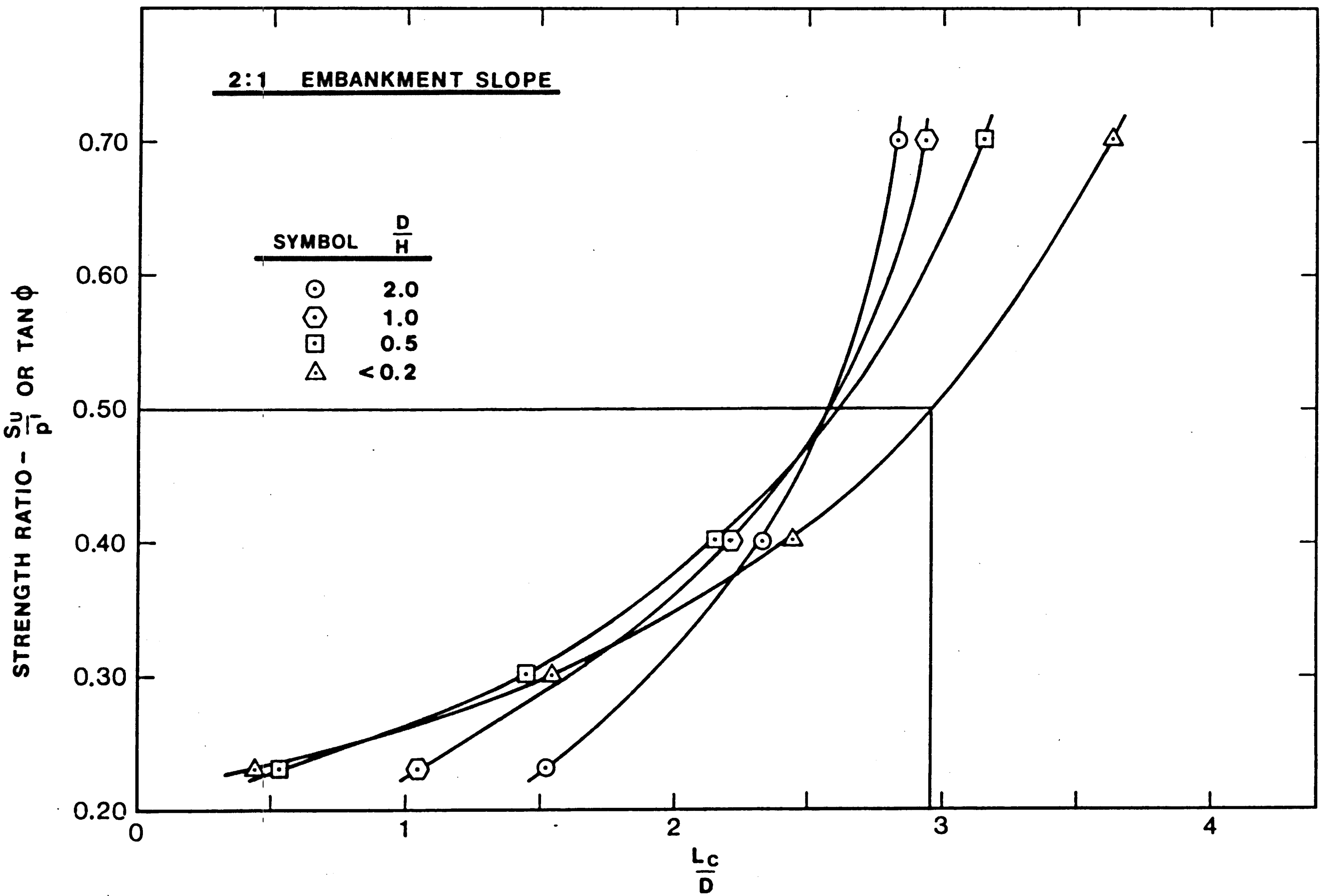


Figure 6. Critical Length of Liquefiable Layer (L_c) for Varying Depth of Layer, Height of Embankment, and Strength of Foundation Material; 2:1 Embankment Slope.

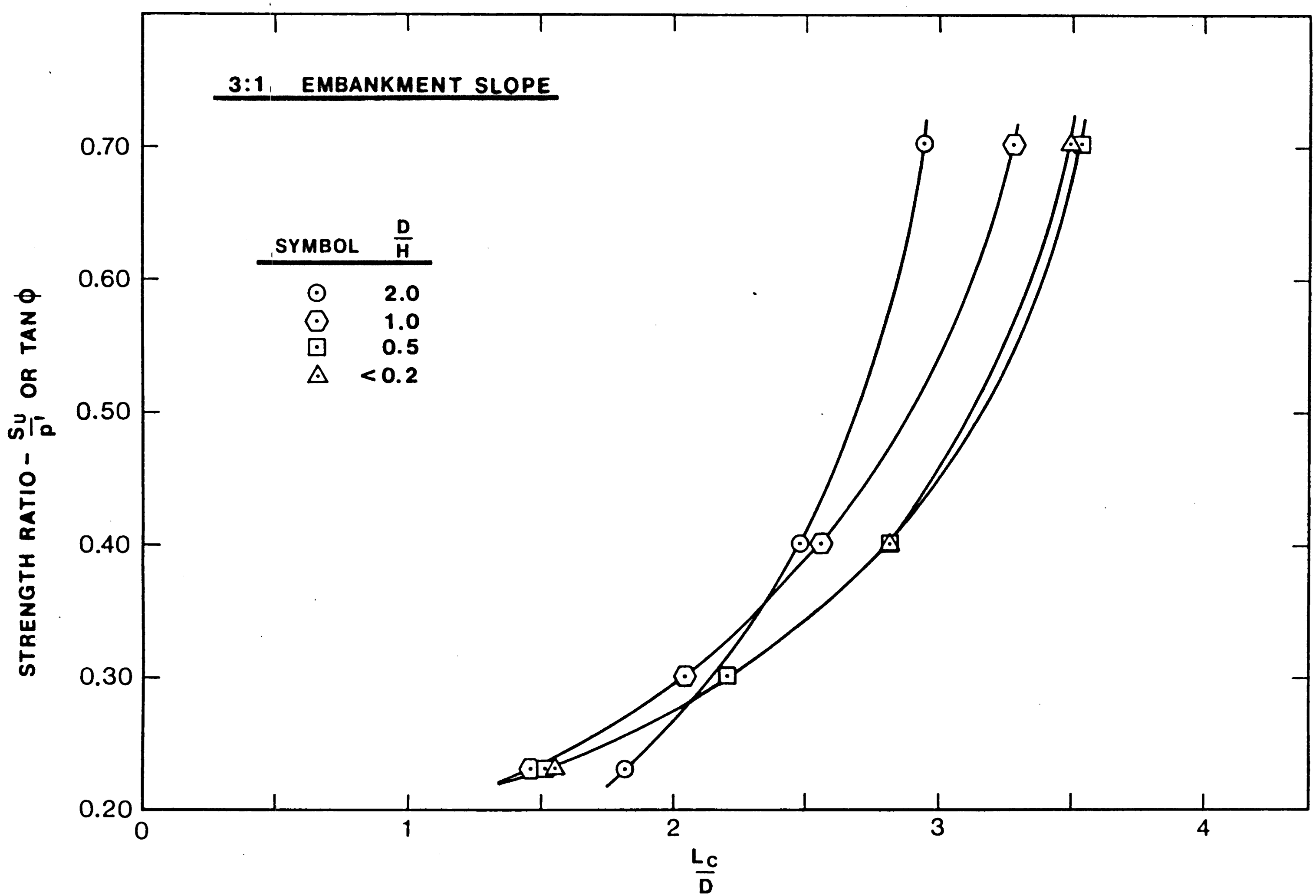


Figure 7. Critical Length of Liquefiable Layer (L_c) for Varying Depth of Layer, Height of Embankment, and Strength of Foundation Material; 3:1 Embankment Slope.

0.2. From the appropriate D/H curve of figure 6, the L_C/D ratio is about 3.0. Thus, the critical length of a liquefied sand layer for these conditions is (20)(3) = 60 feet. Under the assumed conditions, a liquefiable soil layer less than 60 feet in length will not lead to failure of the embankment.

In all cases, the critical length of liquefiable layer increases with depth. Consequently, tests to locate shallow layers need to be spaced more closely than those to locate deeper liquefiable sediments. To demonstrate this fact and show how the curves in Figures 5, 6,and 7 can be used, consider the following example. For an embankment 100 ft high with side slopes of 2 to 1 located on foundation material with a strength ratio of 0.5, what would be the critical length of liquefiable layers at depths of 20, 50, 100, and 200 ft? Using the curves in Fig. 6, critical lengths of 60, 130, 260, and 520 ft are calculated for the given depths, respectively. Hole spacings and depths should be conservatively selected to be sure all layers with these critical lengths are interseted. Spacings of one-half to one-third the critical length should be adequate. For this example such spacings would be about 20, 50, 100, and 200 ft, respecively, which is about the same spacing as the depth to the layer to be detected.

This analysis gives general guidance and may be applicable to many cases with strength and slope conditions similar to those in the model. The results of this parametric study will not be applicable to all cases. For other embankment and/or foundation configurations, particularly large, complex earth dams, the critical length of liquefiable soil layer should be determined. This may be done in a manner similar to the procedure described above, with a low strength assigned to potentially liquefiable foundation units.

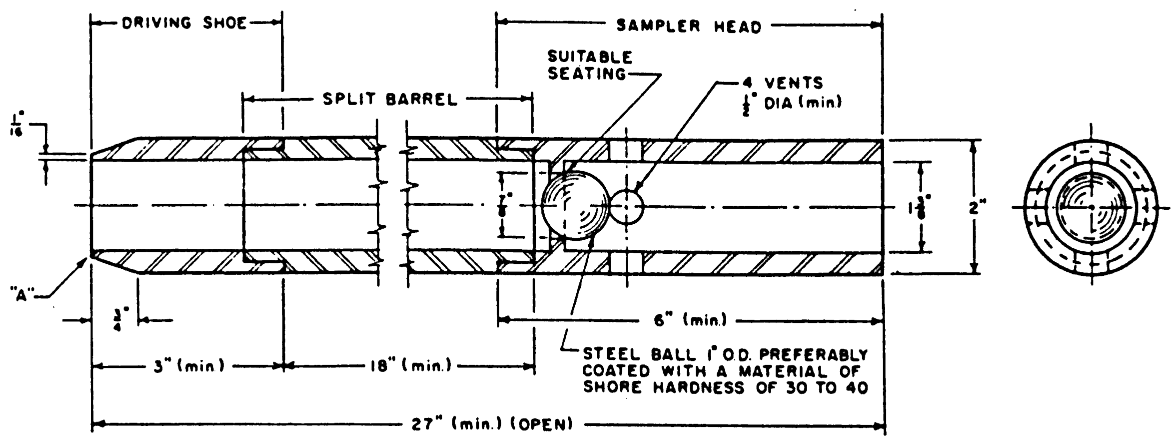
III. TECHNIQUES FOR SUBSURFACE EXPLORATION

STANDARD PENETRATION TEST

General Description.

In the past, the Standard Penetration Test (SPT) has been the most widely used method for investigating liquefiable soil deposits in the U.S. and abroad largely because a large body of SPT blowcount data is available and useful correlations have been developed with liquefaction susceptibility. Equipment for SPT is relatively economical and readily available. Samples taken by SPT are useful for profiling of subsurface sediments, however the precision with which layer boundaries can be delineated decreases with test interval spacing. Because of wide acceptance and advantages of the SPT,, it is likely that this method will see much future use.

Procedures and specifications for the SPT are given by the American Society for Testing and Materials (1985). Figure 8 is a diagram of the ASTM standard split-spoon sampler which is commonly used in SPT testing. Some other configurations of the sampler are also used, the most common being an ASTM standard sampler with the liners removed giving a core barrel diameter of 1.5 in rather than 1.375 in. A 140-lb hammer dropped a distance of 30 inches is used to drive the SPT sampler. Again there are some variances in hammer configuration and hoisting and release mechanisms for dropping the hammer that influence test results. Seed et al. (1984) have evaluated various modifications to the SPT test equipment and procedure and have suggested factors correct for nonstandard methods. In conducting the test, the number of blows are recorded for each 6-inch advance of the sampler over an 18-inch drive interval. The Standard Penetration Resistance, N, is the number of blows required to drive the sampler through the last 12 inches of penetration.



NOTE 1—Split barrel may be 1½ in. inside diameter provided it contains a liner of 16-gage wall thickness.
NOTE 2—Core retainers in the driving shoe to prevent loss of sample are permitted.
NOTE 3—The corners at *A* may be slightly rounded.

Figure 8. Split Barrel Sampler Assembly for Use in Standard Penetration Testing, ASTM Specifications (from ASTM D-1586-85).

A disturbed sample of soil is taken with each drive of the sampling apparatus unless the sample is lost during recovery of the sampler, which, unfortunately is too common of an occurrence. Grain size analysis and Atterberg Limit tests are commonly conducted on the sampled material which provide important information for soil identification and classification. More complex tests cannot be performed because of the degree of disturbance caused by driving the sampler through the soil layer. Samples taken by SPT are generally high-quality enough that details of sedimentary structures can be observed.

Despite problems with standardization (DeMello, 1971; Kovacs et al., 1984), the SPT has been used successfully for many years. In view of the likely continued widespread use of SPT to assess soil properties, the need for more accurate future uses of SPT is evident. A standard procedure for the performance of SPT has been proposed by Seed et al. (1984). Use of this procedure (table 2) to achieve standardized results is recommended until an international standard is adopted. Some minor modifications to the procedures listed in table 2 have been suggested by some knowledgeable drillers, one modification in particular is the reduction of blowcount rate from 30 to 40 blows per minute to 20 to 30 blows per minute to allow for ease of operation and to be sure the hammer has come to rest before lifting for the next blow.

Usefulness of SPT in Subsurface Investigation.

The primary advantage of using SPT in soil profiling investigations is that of sample recovery. Material is made available for grain-size analysis, soil classification, or other laboratory tests. Samples from SPT are generally undisturbed enough that sedimentary layering details may be observed and used for profiling the borehole over the 18-inch test interval. Layers

A. Borehole:	4 to 5-inch diameter rotary borhole with bentonite drilling mud for borehole stability.
B. Drill Bit:	Upward deflection of drilling mud (tricone or baffled drag bit).
C. Sampler:	O.D. = 2.00 inches I.D. = 1.38 inches, constant (i.e. no room for liners in barrel).
D. Drill Rods:	A or AW for depths less than 50 feet. N or NW for greater depths.
E. Energy Delivered to Sampler:	2520 inch-pounds (60% of theoretcial maximum).
F. Blowcount Rate:	30 to 40 blows per minute.
G. Penetration Resistance Count:	Measured over range of 6 to 18 inches of penetration into the ground.

Table 2. Recommended SPT Procedure for Use in Liquefacation Correlations (from Seed, et al., 1984).

and interbeds as thin as a fraction of an inch may be observed in the SPT sample, under ideal conditions.

Because SPT tests are usually spaced at intervals through the borehole, use of the technique for subsurface mapping is somewhat limited. In geotechnical investigations, it is common to specify SPT test spacings of five feet or more through the borehole. This allows only an intermittent delineation of sedimentary layers. Continuous or near-continuous sampling by SPT, though cumbersome, may be done to completely delineate the borehole profile. Another solution might be the drilling of two adjacent boreholes, with SPT samples taken at alternating intervals within each borehole. This would allow for an 18-inch advance of the hole between each tests and also ensure complete coverage of the soil profile.

Use of SPT in conjunction with another investigative method is perhaps the best solution to the problem of intermittent sampling. Cone Penetrometer Testing (CPT), which will be discussed subsequently, lends itself well to this approach. Preliminary CPT soundings may be used for judicious location of boreholes and SPT samples. Correlation between CPT and recovered samples will minimize the possibility of missing a soil layer critical to the investigation.

When using SPT in subsurface exploration, the engineer should be particularly alert for cohesionless soil layers with low penetration resistance. The areal extent and continuity of such materials must be accurately defined to assess the potential for damage during earthquake shaking. Standard Penetration Testing is useful in identifying and delineating liquefiable materials, especially when used in conjunction with a sounding technique such as the cone penetrometer.

Economics and Limitations.

The SPT is an economical and fairly simple method for profiling subsurface sediments and obtaining soil samples for laboratory index tests. In most locations, equipment for SPT is readily available, and mobilization costs are relatively low. The existence of methods for correlating SPT with liquefaction susceptibility and the relatively large amount of SPT data derived from field investigations of liquefaction sites are other advantages of using SPT.

Use of the SPT in conjunction with the Cone Penetrometer Test can greatly reduce the cost of drilling and SPT testing. By using CPT soundings, sampling sites may be selected which will produce the most information about the soil profile. Cone penetrometer data can also be used for correlation of subsurface stratigraphy between boreholes, thereby reducing the total number of SPT tests required.

The SPT can be conducted successfully in most soil types. Optimum drilling and sampling performance is achieved in sand, silts, and clays containing little or no gravel. When auger drilling is performed in loose cohesionless deposits, problems may be encountered with material flowing up the auger stem. Loss of sample from the split spoon upon withdrawal is another frequent problem, which may be remedied by use of a steel spring sample retainer.

The presence of large amounts of gravel or cobbles will result in slower drilling rates and increased equipment wear. Profiling by SPT in sediments containing large amounts of gravel has been accomplished in past investigations (i.e. Andrus et al., 1986), although extremely coarse material may be impenetrable by standard auger or rotary drilling techniques and large particles may impede or block penetration by the split-spoon sampler.

The effect of gravel on liquefaction susceptibility of a soil, or SPT blowcount, is not well understood. Recorded N-values in gravelly material may be artificially high as a result of the sampler shoe striking and pushing aside gravel particles. A possible alternative when exploring extremely coarse sediment is to drive a larger size sampling device, thereby reducing the relative effect of gravel particles in the soil (Committee on Earthquake Engineering, 1985). While improving sampling capability, such modifications make the procedure nonstandard and the results may not be readily comparable.

CONE PENETROMETER

General Description.

The Cone Penetrometer Test (CPT) was introduced about 50 years ago in Europe as a method for estimating pile bearing capacity. In recent years, it has emerged as an effective and economical technique for delineating subsurface stratigraphy. Data derived from the CPT can also be used in soil identification, estimation of strength parameters or density, and evaluation of liquefaction susceptibility of a soil mass. As with any sounding technique, the CPT has the disadvantage of no sample recovery.

The standard cone penetrometer has a 60^{o} apex angle, a 10 cm^2 tip area, and a sleeve area of 150 cm^2 (ASTM D-3441). The apparatus is advanced through the soil, by means of a hydraulic or mechanical jack, at 2 cm/s. During testing, the tip resistance (q_c) and sleeve friction (f_s) acting on the penetrometer are measured. The friction ratio ($FR = f_s/q_c$) is a useful parameter in correlating CPT data with soil type.

Figure 9 shows a simplified diagram of the mechanical-type cone penetrometer tip. This type of tip must be alternately advanced and collapsed

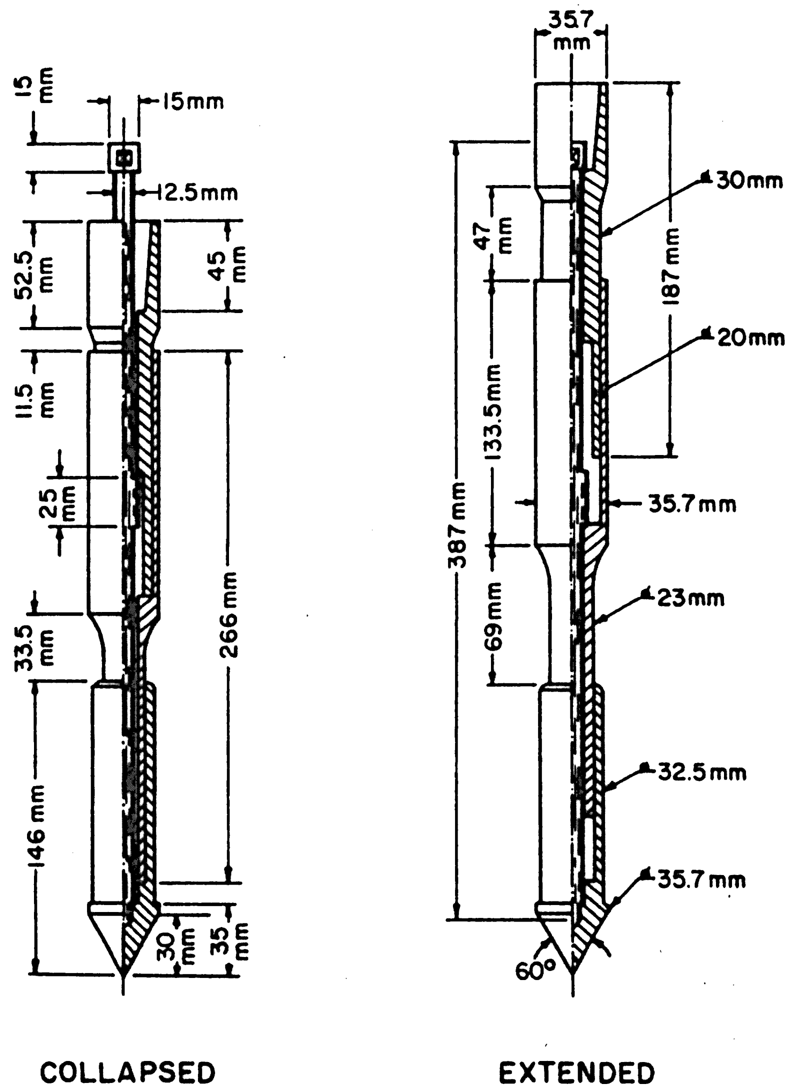


Figure 9. Example of a Mechanical Friction-Cone Penetrometer Tip (Begemann Friction Cone; from ASTM D-3441-85).

so the CPT data is not continuous through the profile. A Begemann friction-cone is shown here, which measures both tip and sleeve resistance. Some mechanical cones are capable of tip resistance measurements only. The introduction of the electric cone penetrometer, with transducers to measure driving forces (figure 10), allowed the continuous monitoring and recording of CPT data. The cone tip shown here is an ASTM standard cone, with tip area of 10 cm^2. The electric cone is generally quicker and easier to operate than the mechanical cone and provides more accurate profiling data. The mechanical cone has the advantage of lower initial cost.

Figure 11 shows photographs of an all-wheel-drive vehicle set up with Hogentogler CPT equipment. The truck provides a reaction force of about 20 tons and is equipped with a sophisticated computer and plotter for data reduction and output. These photographs were taken at a potential liquefaction site near Parkfield, California, which is currently under investigation (Holzer, et al., 1986).

One advantage of the cone is that additional sensors can be placed in the tip. One such sensor which can be incorporated into the cone tip is the temperature probe. Measurement of soil temperature can furnish information about environmental changes in the soil profile and can also be used in calibration of the cone (Campanella & Robertson, 1981). Tringale and Mitchell (1982) describe an acoustic penetrometer which might be useful in detailed soil profiling, particularly in granular to gravelly soils which emit recordable sounds during penetration. Miniature seismometers in the cone tip have been used which allow the cone penetrometer to perform a downhole seismic test at appropriate depths in the soil profile (Robertson & Campanella, 1984). A radioactive source and detector can be installed in the penetrometer tip, used to measure gamma backscatter (Ledoux et al., 1982). From this informa-

Figure 10. Photograph of Disassembled Electrical Cone Penetrometer Tip, Standard (10 cm^2) Size.

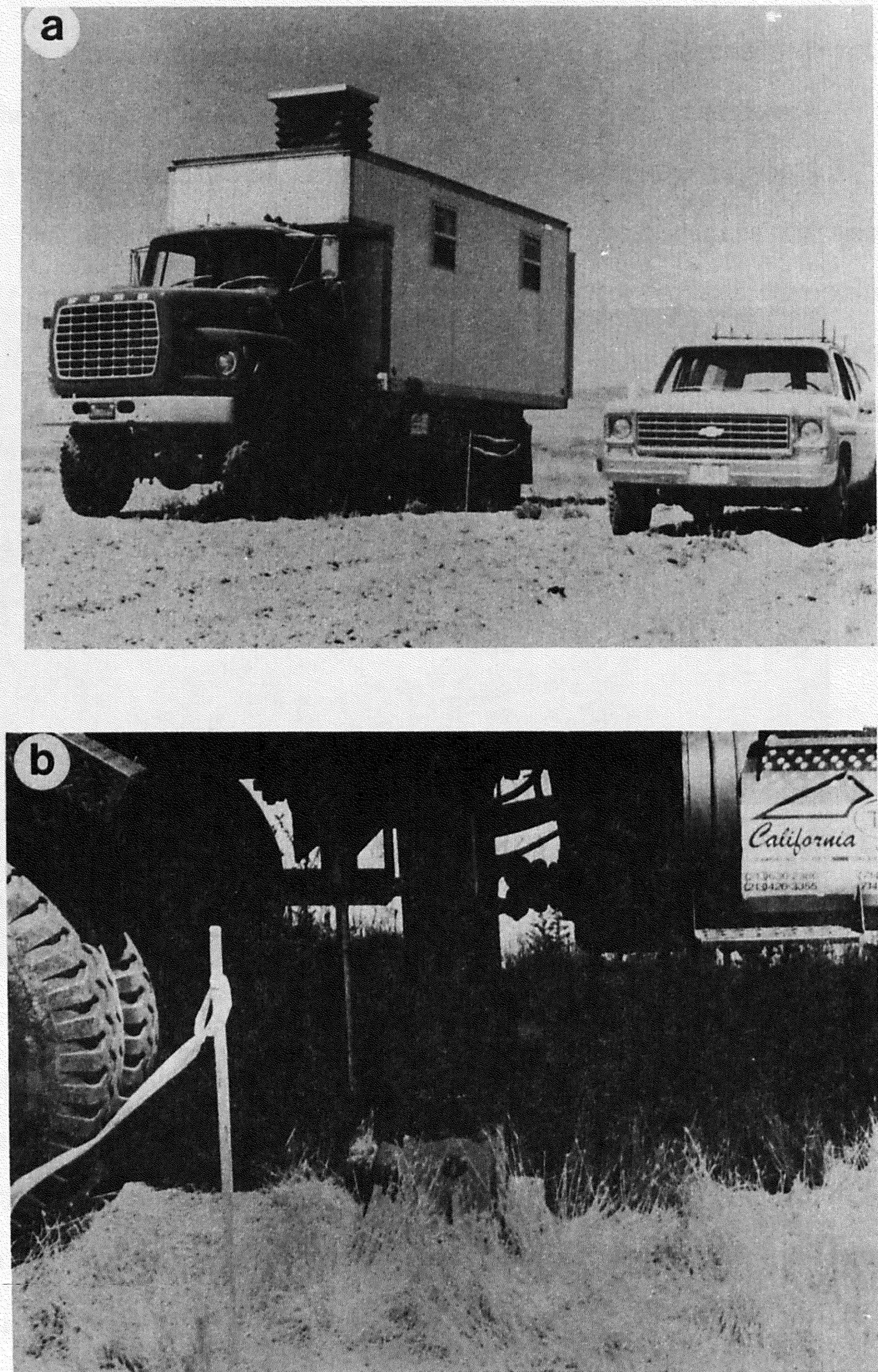


Figure 11. All-Terrain Vehicle Equipped for Cone Penetrometer Testing: (a) Performing CPT Soundings at Parkfield, California Site; and (b), Showing Cone Tip Extended below the Truck Bed.

tion, bulk density of the penetrated soil may be calculated. These additions are presently being researched and are not widely used in common practice. Use of a piezometer in the cone tip (CPTu) is discussed later in this report.

Usefulness of CPT in Subsurface Investigation.

A cross-section extrapolated from CPT data at the Heber Road Site, Imperial Valley, California (Youd and Bennett, 1983) is shown in figure 12. The different stratigraphic units can be readily identified, and an initial estimate of soil type in each layer can be made from q_c and FR versus depth curves. This type of information is useful in soil profiling and in planning additional investigations in a rational and more cost-effective manner. Further cone soundings can be specified as needed for a more detailed stratigraphic profile, and desired locations for soil samples may be intelligently selected.

Several methods for soil classification based on CPT data, most employing the friction ratio in some form, have been proposed. Figure 13 is a Soil Classification Chart developed by Robertson & Campanella (1984). Because an increase in clay content generally results in a relative increase in soil cohesion and adhesion, clay soils are generally characterized by high friction ratios. In sandy soils, the CPT records low friction ratio, and tip resistance increases with density of the material being penetrated. These properties are reflected in the classification chart shown. This chart is useful only in clays, silts, and sands, as the effect of gravel on CPT tip resistance and sleeve friction has not been quantified.

Liquefiable soils generally have low clay content and low density, and may be tentatively identified from figure 13. Zone A in the figure is a proposed boundary for soils with a potential for liquefaction.

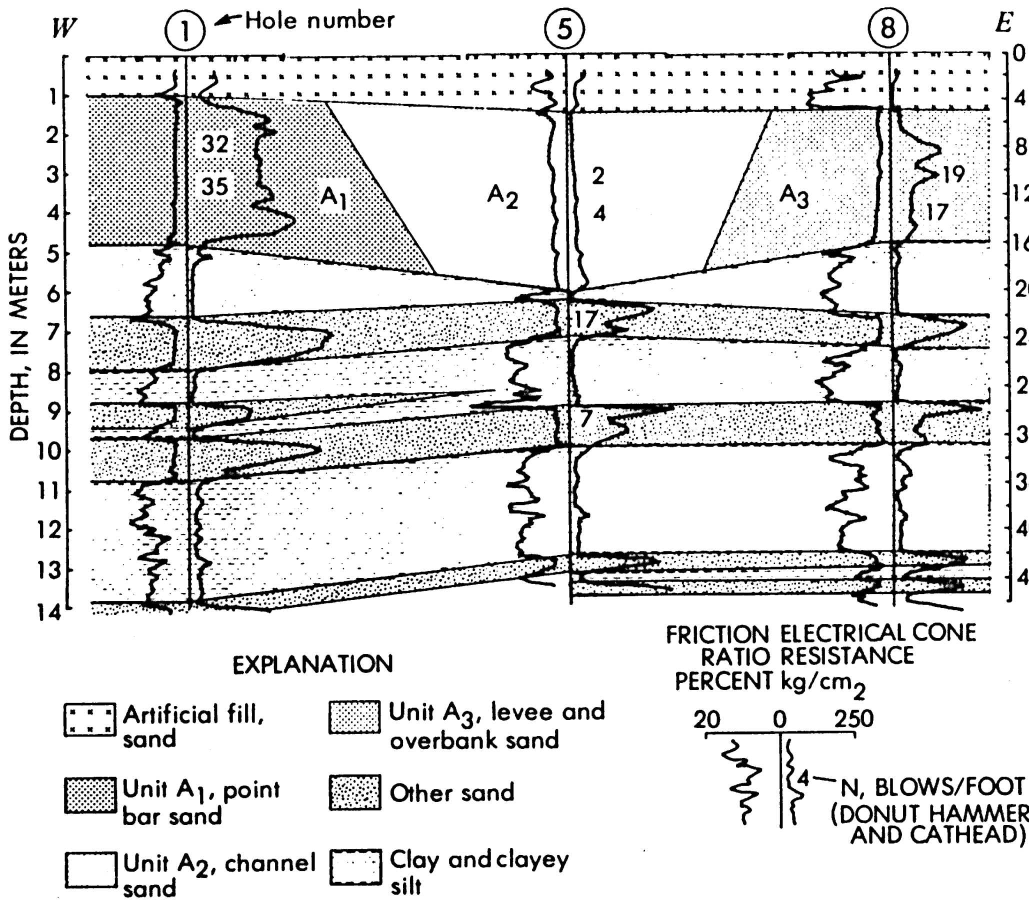


Figure 12. Cross Section of Sediments at Heber Road Site, Showing Electrical CPT Records and Donut-Hammer N-Values (from Youd and Bennett, 1983).

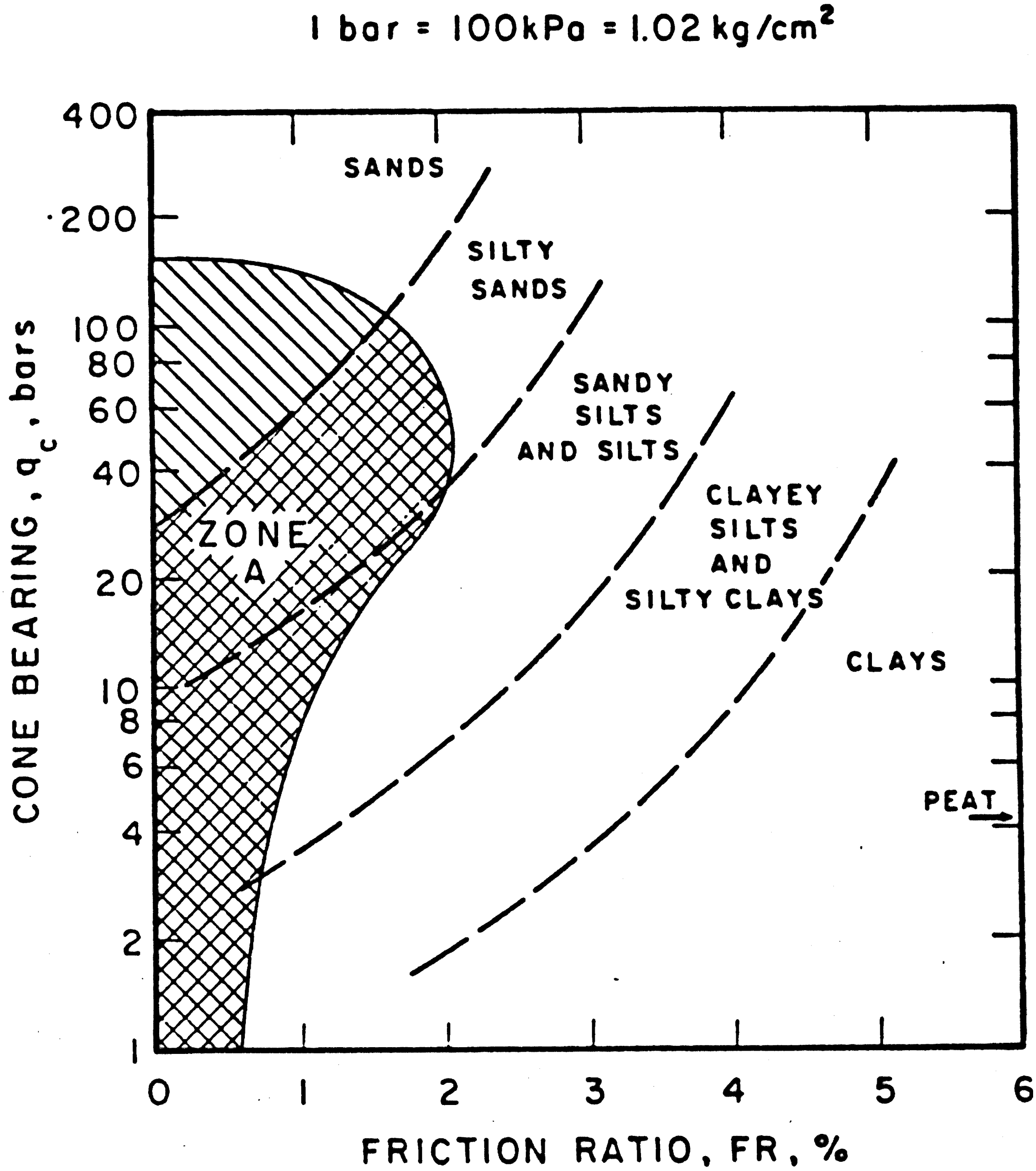


Figure 13. Soil Classification Chart for Electronic Cone Showing Proposed Zone of Liquefiable Soils (from Robertson and Campanella, 1984).

In classification schemes based on CPT soil parameters, the boundaries between different soil types are somewhat arbitrary and should be used with caution. Samples should be taken at appropriate locations in the profile to verify estimated soil types and further calibrate the cone for local soil conditions. As the body of CPT data for a given area accumulates, the need for verification by sampling may be reduced.

Piezocone (CPTu).

The piezocone was introduced about a decade ago (Janbu and Senneset, 1974) for the purpose of making pore pressure measurements during the cone penetrometer test (CPTu). The piezometer-equipped cone tip is sensitive to changes in soil conditions and provides increased accuracy in soil profiling and identification in some types of soil layers. Other applications of the CPTu in geotechnical engineering are tentative assessment of stress history or consolidation properties (cohesive deposits), assessment of ground water conditions, and as an indicator of liquefaction susceptibility of sand deposits (Jamiolkowski et al., 1985).

Figure 14, from Baligh et al. (1980), shows a typical pore pressure record for a CPT conducted in clay interbedded with sand layers. Small spikes in the record represent the pore pressure dissipation which takes place when testing is stopped to add a new section of rod. These breaks occur at approximate one-meter intervals, as that is the length of a standard rod section. The large spikes shown in the pore pressure record indicate large changes in drainage conditions during cone penetration. Such sudden decreases in pore pressure indicate regions of higher permeability than surrounding layers, probably sand seams. The ability of the CPTu to detect thin layers of

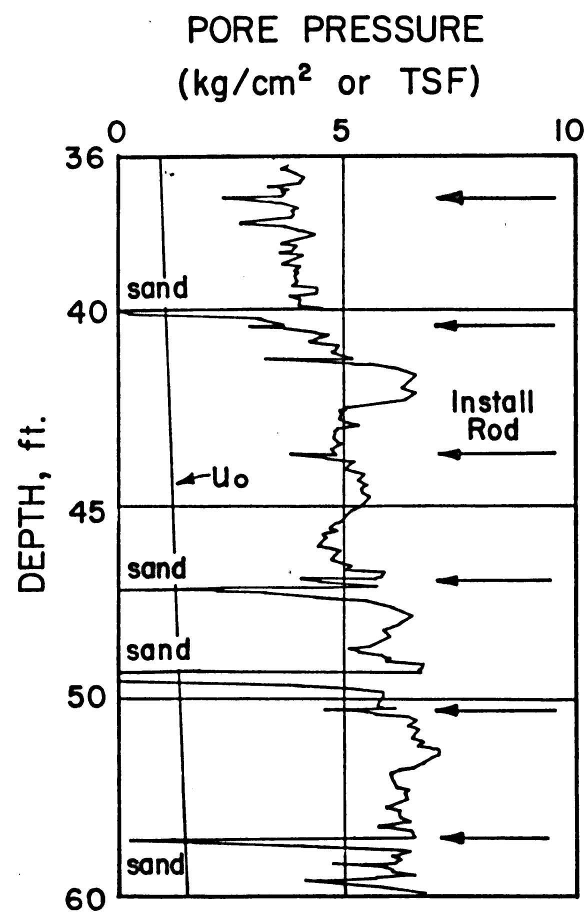


Figure 14. Pore Pressure Log from Piezocone (CPTu) Sounding (from Baligh, et al., 1980).

sand in a stratified deposit make the method particularly applicable to detection of liquefiable layers.

Pore pressure measurements taken by CPTu are highly sensitive to the rate at which the cone advances. The current standard of 2 cm/s was selected as a speed which produces nearly undrained conditions in homogenous cohesive deposits (Roy et al., 1982). Practically drained conditions may prevail in tests of clean sands (with < 10% passing ASTM sieve No. 200) at the standard penetration rate. Intermediate soil properties produce a condition somewhere between drained and undrained.

Stresses and strains which develop around the cone tip during soil penetration are highly complex. Because of this, the location and size of the piezometer element in the apparatus have a large effect on the recorded pore pressure. Currently, the two main locations for piezometer elements which are preferred for geotechnical applications are 1) on the cone face, and 2) immediately behind the cone tip.

Robertson and Campanella (1984) list the following advantages of having the pore pressure element located immediately behind the cone tip: 1) the porous stone is well-protected and less prone to damage; 2) it is generally easier to saturate; 3) it gives reasonably stable pore pressure response; 4) it gives a good range of dynamic pore pressures from negative to positive (therefore good for stratigraphic logging); and 5) it is the best location when applying pore pressure corrections to cone bearing and friction. Because of these advantages, location of the pore pressure element behind the cone tip is probably best when using CPTu to locate liquefiable soil layers in stratified deposits.

Advantages of locating the pore pressure unit on the cone face are listed by the above authors as: 1) it is the best location to record maximum pore pressure; 2) it is the best location for inclusion into solid body cone design; and 3) measured pore pressures are potentially more repeatable because of the lower stress gradients on the cone face. Because there is no single best location and size of piezometer element for all applications, there will not likely be a trend toward standardization in the near future. To avoid confusion, it is necessary to specify the piezometer type and location when reporting pore pressure data from CPTu.

Accurate pore pressure measurements can only be taken by CPTu when the porous element is completely saturated. In practice, this is frequently a problem, especially when the ground water elevation is more than a few feet below the ground surface. Currently, a solution to the problem is being researched. Proposals include saturation of the porous stone with glycerine, deairing in vacuum chambers, and airtight membranes which can be broken at an appropriate depth in the profile.

Many pore pressure ratios derived from CPTu data have been proposed, most based on some ratio of pore pressure and cone tip resistance. Use of the pore pressure ratio is helpful in accurate identification of soil layer interfaces and thicknesses. Because pore pressure ratios are sensitive to the nature of the penetrated soil, they are also an excellent tool for soil identification. A classification chart proposed by Senneset and Janbu (1984) is shown in figure 15. The pore pressure coefficient B_q is a function of penetration and hydrostatic pore pressures, and total overburden stress. The equation for B_q and a definition of terms are as follows:

$$B_q = (u_{max} - u_o)/(q_t - \sigma_{vo})$$

where
u_{max} = penetration pore pressure
u_o = hydrostatic pore pressure
σ_{vo} = total overburden stress
q_t = total cone resistance corrected for unequal end area effects.

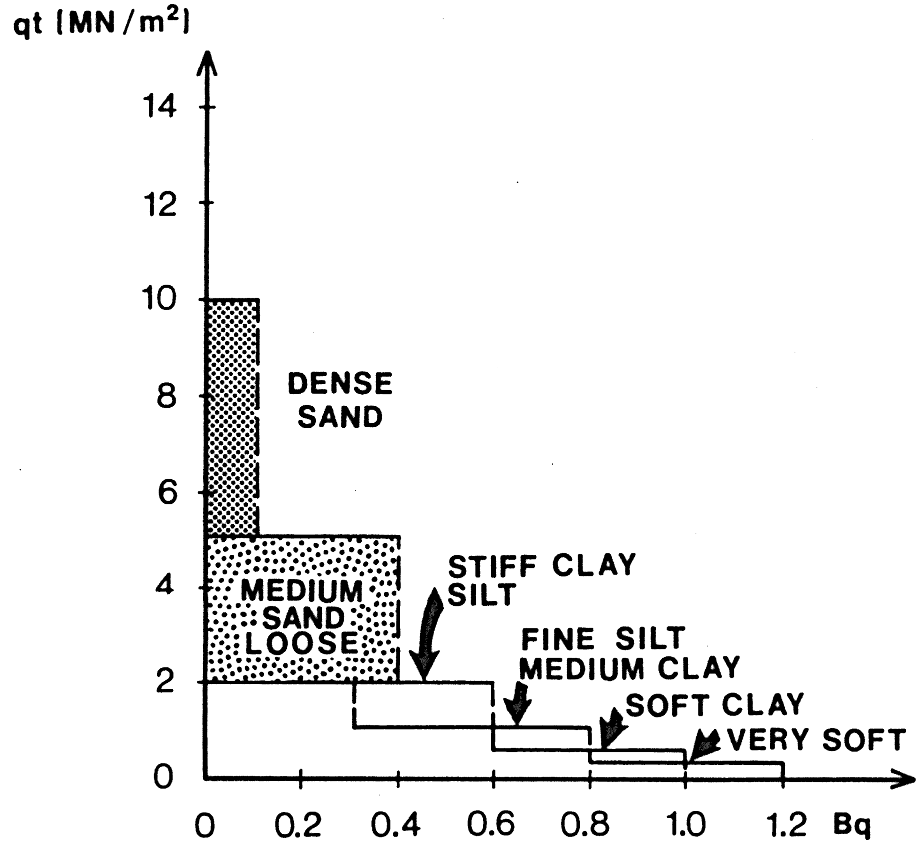


Figure 15. Tentative Classification Chart Based on q_c and B_q for Standard Electrical Friction Cone (adapted from Senneset and Janbu, 1984).

As with other CPT classification techniques, this chart should be used carefully and verified on a site-specific basis.

Economics and Limitations.

The CPT is a fast, economic logging test which provides continuous information on soil bearing, friction, and pore pressure. These data can be used to map the stratigraphy of a soil profile and identify potentially liquefiable soil layers. Generally, CPT can be done about four times as fast as conventional drilling and at about one-quarter of the cost, and much more information is obtained (Norton, 1983). Use of the CPT can also reduce costs of drilling and sampling in a liquefaction investigation by allowing the engineer to plan his sampling sites intelligently. Some limitations of the CPT are discussed below.

The thinnest layer which can be detected by CPT depends upon the stiffness of the material, since the tip resistance is influenced by the soil properties ahead of and behind the tip. The distance over which the cone tip 'senses' an interface increases with increasing soil stiffness. In order for the cone tip to achieve full q_c at mid-depth in the soil layer, the thickness of a stiff soil mass should exceed 30 inches. In a soft layer, this minimum thickness is reduced to about 12 inches (Jamiolkowski et al., 1985).

It should be noted that it is possible to detect layers which are considerably thinner than the constraints listed above with the CPT. Although q_c is not measured accurately in the thinner layers, stratigraphic sequences on the order of two or three inches may be identified from electrical CPT data. When the mechanical cone is used, values of tip and sleeve friction are averaged over the length of advance for a single push (about four inches).

Hence, under ideal conditions, a four-inch layer is the thinnest which is detectable by mechanical cone. Use of the piezocone may permit more accurate, positive identification of these soil layers, and location of the different layer interfaces. Use of a smaller diameter cone has also been used successfully instances for profiling soft soils where finer definition of stratigraphy is required (Oral Communication, Steve Brown, The Earth Technology Corporation). It should be possible to define layers as thin as an inch with a one-half to three-quarter inch diameter cone.

Worldwide experience has shown that the CPT can be used successfully in all types of soils, except extremely dense or highly cemented materials or those containing large amounts of gravel, cobbles, or boulders. The CPT is especially useful in mapping the stratigraphy of layered deposits of sands and clays. In layers of dense or cemented material, reaction force supplied by the truck or drill rig may establish a limit to the depth of penetration. Schmertmann (1978) suggests that, as a rough guide, 10-ton equipment can just penetrate a 5 foot layer of SPT N = 100 sand at a depth of 25 feet.

Soil with significant amounts of gravel and/or cobbles can produce erratic CPT data, damage equipment, or stop penetration altogether. In gravelly soils, cone tips of more rugged construction allow testing to greater depths than are possible with the standard 10 cm^2 cone. In a liquefaction study at Whiskey Springs, Idaho (Andrus, et al., 1986), a 15 cm^2 cone was used. Through use of the oversize cone, adequate penetration depths were achieved, and accurate information about the soil profile was obtained, despite gravel contents of 50 to 60 percent and particle sizes larger than several inches in penetrated layers.

From experience gained at Whiskey Springs and other sites, it is clear that use of the cone penetrometer need not be restricted to gravel-free materials. Penetration and testing of gravels may be accomplished, depending on shape, size, and angularity of gravel particles, as well as density and degree of cementation in the sediment layer. When penetrating highly-resistant layers, the cone rods may buckle unless adequate lateral support is present.

Proper calibration of equipment and standardized operation techniques are essential to ensure accuracy and repeatability of CPT results. For the electronic cone, the two main errors related to the design of the load cells are calibration error and zero load error. Schaap & Zuideberg (1982) give a detailed discussion of the accuracy and calibration of electronic cones.with the large body of SPT data.

BECKER HAMMER DRILL

General Description.

The Becker Hammer Drill (BHD) was developed in the late 1950's as a method for driving holes through deposits of gravel and cobbles. It has since been used for mineral prospecting in gravel and placer deposits, tailings or dump exploration, offshore exploration, and a number of other applications. Recently, the Becker Drill has been used in geotechnical investigations such as dam exploration, soil sampling, and gravel and borrow pit exploration.

The BHD is being evaluated for use in liquefaction field studies of gravelly deposits, where SPT, CPT, standard auger and rotary drilling are difficult to impossible. Because the Becker apparatus is capable of testing very coarse materials, it should be considered as a means for preliminary site exploration when large amounts of gravel or cobbles are present. An example is the Ririe Dam, Idaho investigation (Sykora et al., 1986), where very coarse alluvial materials were encountered in the dam shell and foundation. Despite difficult conditions, the Becker drill was used with some success for initial subsurface mapping of this site.

Sand, gravel, and cobbly soils can be penetrated rapidly and efficiently with the Becker drill. A double wall drive pipe is driven by a diesel-operated pile hammer, while air is forced down the outer drive pipe. The material cut by the drill bit is forced up the inside pipe by compressed air and can be collected in bags. These samples are very highly disturbed, but may be adequate for grain size analysis or similar tests.

The BHD diesel hammer delivers 95 blows per minute to the drill pipe assembly, at 8,000 ft-lb/blow. Only the outer pipe takes the impact during the hammering cycle as the inner pipe floats inside against a neoprene cushion. Standard casing has an outside diameter of 5.5 inches, and inside diameter of 3.25 inches, though drill pipe up to 9 in. outside diameter is available.

Figure 16 is a photograph of a Becker Drill in use on the Mackay Dam, Idaho. The standard truck-mounted drill is most commonly used in geotechnical investigations, though the Becker Drill has also been mounted on barges, ocean-going vessels, steel tracks, and soft tracks. Bits, other equipment, and up to 120 feet of drill pipe can be carried on the truck-mounted rig. A support vehicle is required when additional drill pipe is needed.

Use of the Becker Hammer Drill in geotechnical exploration is described by Beckwith and Bedenkop (1973), who made the following recommendations to the Arizona Highway Department:

> The Becker Hammer Drill is recommended as the primary method of subsurface exploration [in very coarse material]. Care should be exercised that the full hammer energy of 8,000 ft-lb/blow is delivered and that sufficient compressed air is provided to rapidly clean cuttings. Blow count should be kept in 6 inch increments to fully define the degree of stratification and the presence of boulders.

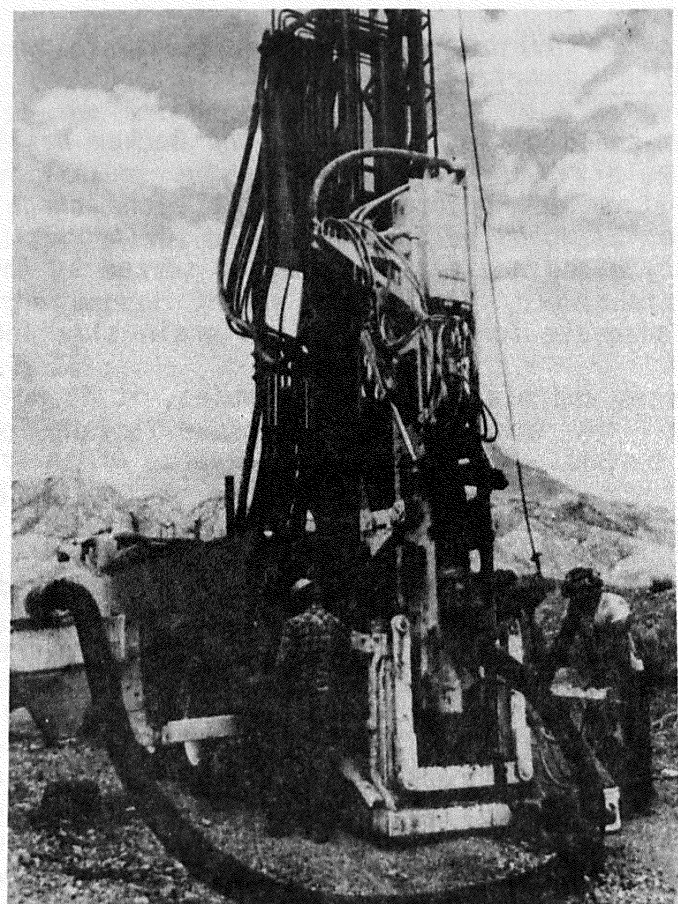


Figure 16. Photograph of Becker Hammer Drill in Use at Mackay Dam, Idaho.

A two-man field engineering crew should be used with one man directing operations and keeping blow count with a tally page and the other continuously observing cuttings recovery and taking samples. Reference samples of cuttings should be taken at 5 foot intervals or each soil change, whichever is less.

Where sand layers are encountered, standard penetration tests should be performed. Shelby tube or open-end drive samples should be taken of any clay or clayey sand layers encountered.

Usefulness of BHD in Subsurface Exploration.

When an open-ended bit is used with the Becker Drill, soil samples are generally taken at selected intervals or at each change in strata. These samples are highly mixed and may be somewhat sorted by the sampling process, but are usually adequate for such tests as grain size analysis. Because of rapid drill progress and mixing of soil samples, it is not possible to detect very thin layers by BHD. The continuous presence of an engineering geologist or soils engineer during testing is justified to ensure that representative samples are taken, and an accurate soil profile is obtained.

Material up to 3 inches in diameter can be recovered when borings are conducted with standard-size casing. In gravelly soils, samples taken by Becker Drill may be more representative of actual conditions than, say, the SPT sample, which is limited to a 1.5 inch maximum particle size. However, BHD samples may be adversely affected by the extreme dynamic pressures which are generated. These conditions may crush, compact, or force material around the drill bit rather than being forced into the drill pipe, resulting in non-representative soil samples. Some sorting may also occur as the sampled material is lifted up through the drill pipe by the flow of compressed air.

When using the Becker drill, the record of blowcount per increment of advancement provides continuous information regarding penetration resistance of the soil. This can be done with either open or closed casing. As with SPT, the record of Becker blowcount versus depth reflects differences in soil properties and soil types encountered and is useful in profiling subsurface sediments.

The Becker Density Test (BDT) is currently being developed as a method for assessing liquefaction potential of a soil (Harder, in press). When performing BDT, a closed-end bit is used, and the compressed air is shut off. The standard (5.5 inch diameter) drill pipe is driven as a large dynamic penetrometer. Because the Becker penetrometer is much larger than the SPT split spoon, local variations introduced by gravel particles may have less influence on BHD results. Also, blowcounts can be recorded continuously with the advancement of the Becker Drill, whereas SPT blowcounts are generally recorded intermittently.

When used in geotechnical exploration, the Becker Drill may be equipped for SPT, CPT, tube sampling, or hard-rock drilling, and used in conjunction with these methods. These tests may be performed through the Becker drill pipe, or at the bottom of the hole after the Becker bit is withdrawn. For example, a dense, cemented, or gravelly layer overlying a liquefiable soil layer could be penetrated by the Becker Drill, and CPT tests performed through the bottom of the hole. Such a procedure was employed by Andrus, et al. (1986) at the Whiskey Springs, Idaho liquefaction site.

Economics and Limitations.

A distinct advantage of the Becker Hammer Drill is that it is fast and in some instances may be more economical than other methods. In loose, saturated deposits of gravel and cobbles, penetration rates can average 50 ft/hr and depths beyond 300 feet can be obtained. Dense formations can slow the rate of penetration to about 10 ft/hr, with depths limited to perhaps 150 feet. The economic advantage of quick penetration is offset somewhat by high operation and mobilization costs.

The Becker Drill can penetrate most types of overburden and can even penetrate into some types of bedrock. Large boulders will stop penetration, but may be blasted when necessary. The BHD is economical to operate in very gravelly or cobbly sedimentary formations. However, other techniques such as SPT and CPT generally provide better stratigraphic information at less cost where gravelly materials do not prevent penetration or invalidate test results. Rapid penetration of the Becker drill usually inhibits definition of layers thinner than about one foot. Because of the high rate of penetration, thin layers of interbedded deposits may not be detected in the sampling process or by the blowcount log which is commonly listed in 6-inch or one-foot increments. Experience at Whiskey Springs, Idaho (Ron Andrus, oral communication) indicated that the minimum thickness of layers identifiable with the BHD in that study was on the order of one to two feet. Samples obtained by Becker drill at Whiskey Springs were highly disturbed and appeared to be less representative of the fine-gravel, sand and silt fractions than samples taken by SPT or the CME continuous sampler.

Because the Becker drill is a rather new technique, standardization of operation procedures and equipment has yet to be established. Use of BHD in liquefaction correlations initially will be hampered by the lack of case histories for comparison purposes.

TRENCHING AS AN EXPLORATORY METHOD

A method which is sometimes used to advantage in liquefaction investigations is trenching or similar excavation. By exposing a vertical profile at a site, a complete and detailed record of the subsurface stratigraphy is available for inspection. Block or other type samples may be obtained, and in-situ tests performed on sediments exposed in the excavation.

Many types of excavations open the earth to inspection by geologists and engineers. In construction of earth dams, cutoff trenches or abutment excavations provide opportunity for a detailed inspection of subsurface materials. Exposures of subsurface stratigraphy are also often provided by many other construction activities including highway and railway cuts, excavated water channels, exposed bridge abutments, large-diameter borings or shafts, tunnels, excavated basements, etc.

Trenching is perhaps the most definitive of all subsurface exploratory methods. It permits inspection of a continuous geologic section by geologists, engineers, and other interested parties. More importantly, it makes possible the preparation of a graphic log that delineates both obvious and subtle geologic features. Potentially liquefiable soil layers, or zones which have experienced liquefaction in the past, can be studied in considerable detail from a well documented trench log. For example a trench was used to document that Holocene sediment had not been disturbed by faulting or liquefaction in an investigation of earthquake hazards at the California School for the Blind in Fremont, California (Lockwood-Singh and Associates, 1984).

Techniques and Objectives of Exploratory Trenching.

Trench locations should be specified only after a sound understanding of the stratigraphic and structural framework of the site has been developed. The results of reconnaissance or preliminary investigations should be available before expensive accessible explorations are undertaken. These results are necessary not only to establish a need for trenching but also aid in determining the proper location, type, depth, dimensions, and methods of excavation. The long axis of the trench should be placed as nearly perpendicular as possible to the stratigraphic-structural feature to be examined.

Geomorphology is important in the selection of trench locations. A study of landforms, both in the field and on aerial photographs, is useful in distinguishing earth features which may be of interest to the study, such as faulting, landsliding, or soil and rock contacts. Examination of geomorphic features also aids in identifying the depositional environment of the site, a critical consideration for liquefaction studies.

Test pits and trenches can be constructed by bulldozers, backhoes, draglines, or ditching machines. The method of excavation least disturbing to the host soil will also likely prove to be the least expensive. Trenches of depths less than about 20 feet are usually excavated with backhoe-type equipment. The practical minimum bottom width of such trenches is two to five feet. Deeper excavations generally require slope-stabilized bench configurations and a width sufficient to allow working space for bulldozer-type equipment.

In all phases of the trenching operation, it is imperative that safety precautions be observed. Personnel entering the trench should be equipped with proper head and foot gear. A fence should be erected around the trench site to restrict entry by people or livestock. Shoring may be required when trench walls are vertical or nearly so. Trenches dug in questionable material should be sloped or benched in a manner that will minimize the hazard of caving. In some cases, a wide bulldozer trench can be deepened with a backhoe trench excavated less than four feet in depth as a means of eliminating shoring.

Because liquefaction sites are generally characterized by a high water table, provisions for trench dewatering are almost always required. Water flowing into the trench under even low head constitutes not only a nuisance but a direct threat to sidewall stability. Installation of wellpoints around the trench perimeter is a possible solution to this problem. Drilling and pumping from wellpoints is generally quite expensive. Portable sump pumps and over-the-bank collection systems may be sufficient and are less costly.

Survey requirements for trench logging are not exhaustive. It is generally sufficient to establish the desired end points of the trench location by staking, complete the excavation, and then return to stake the alignment of the trench along the crown of one wall. Narrow trenches require staking along one side only, preferably at 10-foot stations, with stakes placed far enough from the walls to be unaffected by minor sloughing. Vertical stationing is transferred into the trench from a benchmark of known elevation.

The primary objective of trench logging is the mapping of earth materials in detail. Raw field data is presented to scale, and lithologic units and geologic structures are delineated. Correlations of sedimentary layers between trenches or other soundings and the resulting geologic map will be correct and complete only if trench logging is properly performed by a competent geologist or engineering geologist. A comprehensive treatment of exploratory trenching and logging procedures is presented by Hatheway and Leighton (1979).

A permanent record of each trench should be obtained by documentary photography. The record should be panoramic and overlapping, with photographs taken from the trench floor if possible. These photographs may prove invaluable to future investigations or in clarifying details of the trench log. The trench should not be filled until photography has been completed and the film developed and found acceptable.

Usefulness, Economics, and Limitations.

The primary advantage of exploratory trenching is that it permits visual inspection of a continuous section of earth materials. In contrast, geophysical methods provide only indirect information, and boreholes lack horizontal continuity. Trenching has a decided advantage over these methods in places where sedimentary units are complicated, and where a detailed picture of the geologic section is necessary.

Many problems which might remain undetected or unrecognized can be recognized and interpreted by examination of trench exposures. These problems include the identification of fault traces, the identification of sedimentary structures caused by past episodes of liquefaction, and the tracing of complex interbeds and layers of liquefiable soil deposits. Several trenches have been excavated recently through materials that have experienced soil liquefaction during a past seismic events. Trenching was probably the only method of subsurface investigation that could yield sufficient information to identify many of these subtle features.

Trenches are commonly made for other purposes during the construction of an earth dam. Advantage should be taken of opportunities these trenches provide opportunities to examine and photograph sediments exposed in abutments, a cut-off trench, or other excavations. Important information pertaining to the extent of liquefiable soil layers, or soil conditions not identified by the initial foundation investigations, may be revealed in such exposures.

Exploratory excavations are useful in mapping practically all types of material and may in fact be the only recourse in strata containing large amounts of cobbles, boulders, or highly cemented material which is impenetrable by standard drilling methods. Trenching is also applicable in mapping deposits of loose, cohesionless material. Samples of this type of material, difficult to obtain by other means, may be taken with a minimum of disturbance by block sampling of sediments exposed during excavation.

Exploratory excavations are relatively expensive. This is primarily due to the labor-intensive nature of trench logging and the cost of dewatering where required. Excavation costs are generally not excessive as a bulldozer or backhoe can be used to produce a trench at considerably less cost than drilling a large number of boreholes. The cost of trench logging must be weighed against the return from the results obtained. The additional expense is warranted when details of a geologically complex profile are required or when other investigative techniques are not sufficient for the purpose at hand.

Excavation costs do become prohibitive with extremely deep excavations. Shoring and dewatering expenses also increase exponentially with increased trench depth. Because of the cost, exploratory trenches are usually limited to depths of about 30 feet or less. This is not a significant restriction in many liquefaction investigations as such studies are usually concerned with sediments in the upper portion of the soil profile.

SAMPLING OF COHESIONLESS SOILS

Samples from subsurface soil layers are usually required in evaluating liquefaction potential at sites where liquefaction hazard is an important consideration. These samples are required primarily for tests to evaluate susceptibility of subsurface sediment to liquefaction. They are also beneficial if not a necessary component for identifying and mapping the thickness and extent of the liquefiable layers. Techniques currently available for obtaining soil samples may be broken classed into four categories: tube sampling, auger sampling, block sampling, and in-situ freezing. No single device yields satisfactory results in every instance. The best choice is determined by such considerations as cost of sampling, soil type and relative consistency, and the objectives of the investigation.

Tube Sampling.--As noted above, samples collected by tube sampling are obtained as part of most liquefaction investigations. These tests may include standard penetration tests with samples recovered from the split-spoon sampler, or thin-wall tube samples for laboratory strength testing. Although such samples are generally used for susceptibility evaluation purposes, the samples also provide an opportunity to visually identify soil materials and large- and small-scale sedimentary structures. Standard index tests yield data for positive identification and classification of soil types. These samples are very beneficial identifying and mapping soil layers, and in many instances the samples obtained for susceptibility evaluation may be adequate for profiling and mapping purposes. However. where very fine definition of soil layers in the soil profile is required, such as for some critical structures, more detailed profiles of subsurface sediments may be necessary. One technique for obtaining such profiles is through a program of continuous tube sampling.

Where samples are required for evaluation of cyclic strength parameters or in-situ density, the internal structure of the material must be preserved and techniques that minimize disturbance must be employed (Marcuson et al., 1980). Though retrieval of a completely 'undisturbed' sample is not possible, disturbance can be minimized through careful use of a thin-walled tube sampling method. A plethora of tube samplers are available for taking samples of sands, silts, and clays with relatively little disturbance. However, block sampling or in-situ freezing are practically the only means of obtaining sufficiently undisturbed samples of sands of soils containing large amounts of gravel or stones for dynamic soil testing.

Tube Samplers.

Tube samples may be obtained relatively quickly and inexpensively using a variety of drilling equipment. Many different tube samplers of varying complexity are available for sampling clays, silts, and sands. Tube sampling has been a standard procedure for many years, and most drilling contractors have the equipment and capabilities required. A classic work describing methods and guidelines for tube sampling is Hvorslev (1949).

Tube samples may be taken continuously in a borehole for the purpose of mapping sedimentary units. Provided full sample recovery is accomplished, this method allows identification of layers a fraction of an inch in thickness. Continuous tube sampling through a borehole generally proceeds slowly and is relatively expensive. Tube samplers of varying complexity and cost are available. Use of such samplers is generally restricted to uncemented sands, silts, and clays as highly cemented materials or soils with large amounts of gravel usually cannot be penetrated without damage to the sampler or severe disturbance to the sample.

Continuous samplers.--Recent progress in development of soil sampling devices has resulted in new techniques for continuous sampling. Three such methods are the Delft Soil Mechanics Laboratory continuous sampler for taking undisturbed samples, the Central Mine Equipment device which samples soil continuously during hollow-stem auger drilling, and the vibration-drills. Each of these methods extract a continuous column of material from the ground, allowing rather detailed logging of sedimentary layers and structures. Stratigraphically in place, but somewhat disturbed soil is also recovered for laboratory analysis and classification.

Delft Sampler.--The Delft sampler, developed in Holland, is capable of taking a continuous and undisturbed soil sample up to 20 meters in length. It is available in 29 and 66 mm diameters. In principle, this device is similar to the Kalstenius piston sampler in which steel foil is used to reduce friction between the soil sample and sampler tube. Rather than foil, the Delft device uses a nylon stocking to encase the sample and prevent sample disturbance from friction as the tube is advanced.

The bottom section of the Delft sampler consists of three tubes centered on one another (figure 17). The outside thick-walled tube (1) bears the compressive force used to push the apparatus through the ground. Immediately inside this tube is a very thin-walled tube (2) over which about 20 meters of watertight precoated nylon stocking (4) can be slid in folds (stocking magazine). The inner plastic tube (3) extends the entire length of the sampler in 1-meter sections. This tube has somewhat smaller diameter than the stocking tube and is slightly larger than the diameter of sample taken. As the sampler advances, nylon stocking is drawn from an opening in the stocking magazine near the cutting shoe, surrounding the soil sample. A lubricant is injected through small holes at the sampler tip (A), eliminating friction between the stocking and plastic tube 3. When the desired sampling depth is reached and the apparatus withdrawn, a specially-designed valve (B) automatically closes the inner plastic sampling tube. This creates a vacuum,

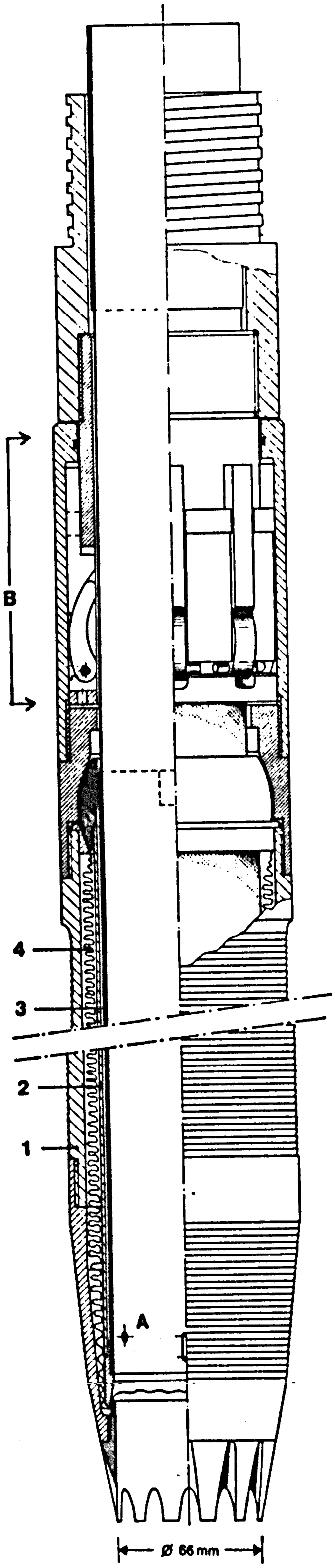


Figure 17. Cross Section of Delft Soil Mechanics Laboratory Continuous Sampler (from Kruizinga, et al., 1974).

preventing sample fall-out and preserving the undisturbed nature of the soil sample (Kruizinga, et al., 1974).

This sampling method provides the investigator with a complete and detailed soil profile. One-meter sections of the soil sample may be examined for stratigraphic profiling. Details of soil layering a fraction of an inch thick are discernible from these high-quality samples.

The reaction force driving the sampler is provided by the same hydraulic equipment used for the static cone penetrometer test. Maximum pressure provided by this equipment is 20 tons, but the practical limit to driving force will be weight of the sampling rig, or the reaction force provided when screw-type anchors are used. Depth of penetration with the Delft sampler may be limited in dense or gravelly soils. Sampling is not possible in very gravelly or cobbly material.

The main limitation to use of this sampler in the United States is availability. Equipment is available only through the Delft Soil Mechanics Laboratory, and it may be that one may only obtain use of this equipment through contract with Delft. Though this continuous sampler shows much promise in subsurface investigation, it is not widely-used outside of Europe because of the expense involved.

A continuous sample tube system manufactured by Central Mine Equipment (CME) provides a method for sampling soil and weak rock continuously during hollow auger drilling. The system has been used successfully in many different materials such as glacial drift, hard clays and shales, peat, mine tailings and loess. When full sample recovery is achieved, this method allows logging of subsurface sediments in detail.

Figure 18 shows the continuous sampler in operation. A 5 foot sample barrel fits within the lead section of a hollow auger column, allowing

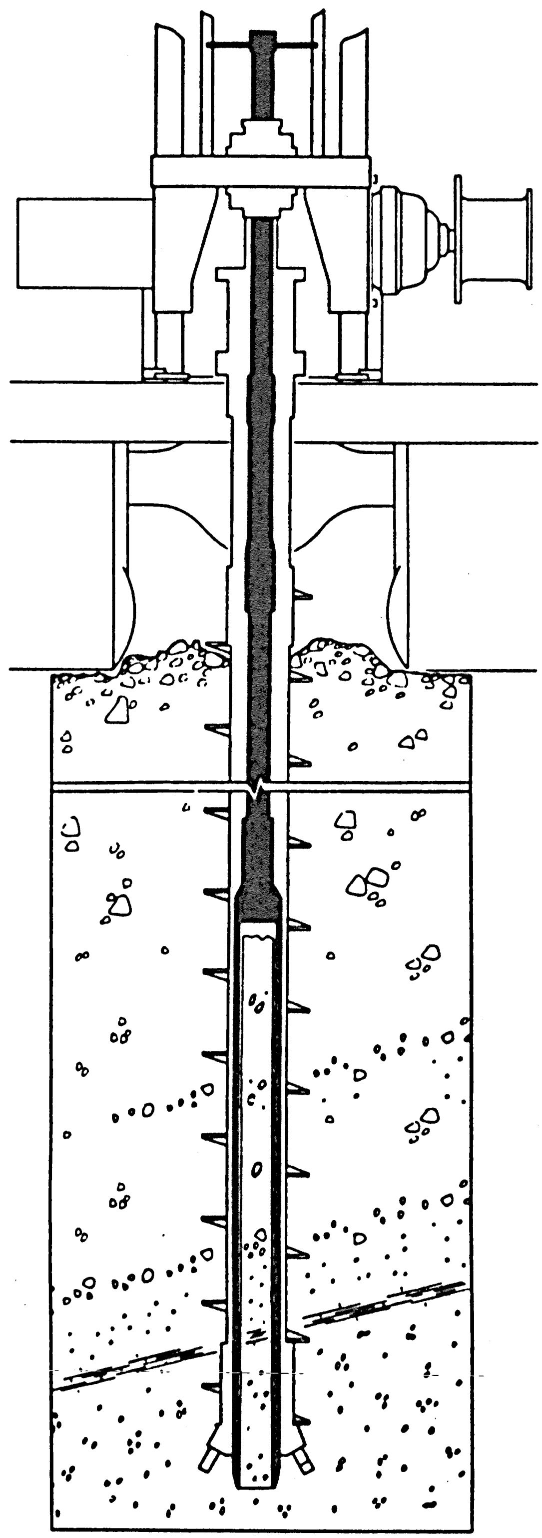


Figure 18. Central Mine Equipment (CME) Continuous Sample Tube System.

sampling to occur with advance of the augers. The sample tube is fixed relative to the auger sections by an inner string of drill rods and does not rotate. The sample barrel can be either 'solid' or 'split', can be used with or without liners, and is available in 3-1/2, 5, or 7-inch outside diameters.

A specialized adjustment rod allows a length adjustment for setting the desired protrusion of the sample tube shoe relative to the hollow auger head and bit teeth. In softer soils, the sample tube may be extended several inches ahead of the auger head to help minimize sample disturbance. For stiffer materials the sample tube shoe may be adjusted close to or within the throat of the hollow auger head so that the auger teeth assist in carving the core sample.

Use of the CME continuous sampler is limited to soils which can be penetrated by conventional auger drilling. In soils with little or no gravel, samples obtained by this method are usually sufficiently undisturbed to preserve stratigraphic details such as complex interbedding. Sample disturbance increases when large particles are encountered. Gravel or cobbles rotate as they enter the tube barrel, mixing the surrounding material. A thin sand layer sandwiched between gravelly deposits would likely be obliterated by the sampling process. This problem is accentuated when materials are loose and saturated.

Another problem encountered when using the continuous sampler is sample retention. Lowest recovery ratios generally occur in materials of greatest interest to a liquefaction investigation, i.e. loose deposits of cohesionless materials. Steel spring core catchers can be used to increase recovery ratios, but these also cause an increase in sample disturbance.

The CME continuous sampler may be used economically to profile the stratigraphy of augerable materials. Under ideal conditions, layers a fraction of an inch thick may be identified by this method. Resolution increases to perhaps a foot or so in soft soils with high percentages of gravel or cobbles. Only materials smaller than the sampler diameter will be sampled, and drilling will be hampered or stopped when large cobbles or boulders are encountered. A further limitation to subsurface mapping will occur when poor recovery ratios are obtained.

Vibratory drills.--Vibratory drills have been developed to drive tube samplers into sediments with aid of vibrations. This method greatly reduces the reactive forces that are required to drive the sampler compared to conventional single-push drive samplers, but the vibration also increases the amount of disturbance to the sample. In recent years, vibratory drills have been used most primarily for offshore sampling of soils. There use has been very limited on land or on shallow water because conventional rotary or auger drills have been preferred for drilling programs where the latter equipment can be deployed at reasonable cost. Tirey (1972, p. 48-49) gives the following summary on vibratory drilling.

> Russian engineers began soil sampling by vibration drilling methods about 1950. Results of work by Gumenskii and Komarov [cited in Tirey, 1979] showed that much faster sampling rates could be achieved with vibro-drilling. The long cores taken in a single drive into an area where there has been fine bedding exhibit evidence of disturbance; namely a downward curl of the fine bedding at the edge of the core, which results from forcing the sample over a great length up the core tube. Their work indicates that the core is disturbed a minimum amount if taken in 3-ft sections and if the driving rate of the sampler is faster than 1 ft per minute of vibration. The recommended maximum section length for a core is 8 ft. Other than pointing out that a higher frequency, for the same amplitude of oscillation, usually produced better results than lower frequencies, they reached no definite conclusions concerning penetration rates in various materials relative to amplitude and frequency of vibration (or oscillation). It is evident that as the frequency increases more power must be put into the system to maintain constant amplitude.
>
> Vibration drills are designed such that the direction of oscillation or vibration is limited to the vertical or to the direction of drilling. Vibrators are primarily of two types: (a) combinations of

eccentric weights in pairs, which are driven in opposite directions in a manner such that their forces add in the vertical direction and cancel in the horizontal direction; (b) vibrating pistons (or hammers), which produce vibrations in only one direction.

Tirey further states (p. 51-52):

> In attempting to take long continuous cores the core tube at times will meet a point of refusal in unconsolidated sediments or soils ad, at other time, the core cutter will clog and the sampler will drive as would a stake. This limitation has been overcome by developing a technique of combining vibration sampling and jetting. The procedure is to take a Vibracore sample in the normal method, bring the sampler back aboard the support vessel, extract the sample and replace the liner, lower the sampler to the seabed, jet the core tube into the seabed by pumping water through the core pipe until it is at a depth approximately 1 to 2 ft above where refusal or recovery was obtained, stop pumping of the water, and the vibrate the sampler to obtain another section of sample. The maximum number of sections needed to acquire 40 ft of sample has been found to be three.
>
> The types of soils that have proved to be the most difficult for vibratory samplers are fine sands with a small silt content and extremely stiff clays. With these materials it is possible to obtain only 5 to 10 ft of sample before the coring tube stakes up. The materials that present least resistance in vibratory sampling are soft silts and soft clays. Cores 40 ft in length have been taken in this type of soil with approximately 100 percent recovery; the core length was limited by the length of the core pipe. Coarse sand and gravel core easily by the vibratory method although penetration rates are not as fast as for soft silts and clays. Cores up to 35 ft in length have been taken in this material. Coarse gravel layers which are completely void of sand present some problem for the vibratory sampler, but layers as thick as 18 in have been successfully cored.

Although the vibratory drill has had limited use on land, there are two possible advantages this equipment could have for soil profiling over conventional drilling methods: (1) The vibratory equipment is capable of extracting long (up to 40 ft long) continuous cores of subsurface materials. This complete profile of sediment layers would have great value for detecting thin sand layers, low permeability layers, and other soil conditions of importance to liquefaction investigations. (2) The vibratory equipment could be more cost effective in some situations. Drilling rates can be much faster than with conventional equipment, and the equipment can be lighter weight and more easily deployable than the conventional drilling equipment. The principal disadvantage to the vibratory equipment is that the retrieved samples of cohesionless soils are generally too disturbed for sensitive dynamic testing or determination of insitu density.

Penetration rates are routinely recorded during vibratory drilling. These rates give some indication of the stiffness or density of the soils being penetrated. Some attempts have been made to correlate these rates with other penetration values, such as standard penetration resistance. Correlative trends have been shown to exist but no definitive correlations between these penetration values have yet been developed (Babcock and Miller, 1979). Because cone penetration tests are also routinely conducted offshore, there has been more emphasis recently on correlations between CPT resistance and liquefaction susceptibility than on correlations with vibrational drilling rates.

In-Situ Freezing Methods.

In the search for a more complete method of undisturbed sampling, a freezing technique has been developed. Yoshimi et al. (1978) describe a method where a mixture of ethanol and crushed ice is circulated through an open pipe inserted vertically into the ground. After the soil surrounding this pipe has been frozen, the steel pipe and frozen column of sand are pulled out of the ground by means of a crane.

By this method, sedimentary structures are preserved and may be logged exactly as they occur insitu. Very high-quality samples may be obtained from that portion of the soil column undisturbed by driving of the steel pipe and expansion of water as it freezes. Samples may be transported with little concern for sample disturbance, and thawed prior to testing in the laboratory.

For extracting of undisturbed samples, use of the in-situ freezing technique is applicable only in a limited range of soil and groundwater conditions. Permeability of the sampled material must be sufficient to permit free drainage as the freezing front advances. This enables the increased volume of frozen water to migrate radially, thus producing a minimum of volume change and disturbance to the soil skeleton. If permeability of the soil is too low, complete drainage does not occur during freezing and volume change occurs disrupting the structure of the soil. Where conditions of high permeability and rapid groundwater flow exist in an underground soil layer or unit, the equipment may not be capable of freezing the soil in that zone. Complete freezing of the soil was used by the Corps of Engineers (1938) in sampling complex, post-failure foundation conditions of the Fort Peck Dam. A large frozen column was created by circulating a freezing mixture through seven pipes driven in a circle. A 36-inch calyx core barrel was used to cut and retrieve samples of the frozen material. The samples were sawed lengthwise, revealing well preserved soil and rock structures, stratifications, and planes or zones of failure where shear had occurred during the embankment failure that had occurred at that site. Ice lenses formed in shale and clayey materials but were not observed in samples of clean sands and gravels.

Sampling of soils by freezing is limited to very shallow depths (less than about 30 feet) and is quite expensive. Because of the high cost, use of freezing for the sole purpose of stratigraphic mapping is seldom warranted. Where an investigation requires frozen samples for laboratory testing, however, and excellent provile of the subsurface sediment profile is also obtained. There could also be instances where extraction of a frozen profile could be economically advantages for profiling a site.

GEOPHYSICAL FIELD METHODS

The science of geophysics was originally concerned with the study of the shape and structure of the earth as a whole, but from about 1920 onwards geophysical techniques have been increasingly used on a smaller scale in the search for minerals, particularly petroleum, in the upper few miles of the earth's crust. More recently, geophysical methods have been applied to the problem of shallow geological structure in which depths are measured in tens rather than hundreds or thousands of feet. Geophysics is emerging as a tool which can give some information about a site as effectively and more cheaply than other techniques, although some boreholes are always necessary before a geophysical survey can be uniquely interpreted (Griffiths and King, 1975).

A few seismic techniques have been developed for use in mapping subsurface stratigraphic units and can provide some useful information for liquefaction investigations. Measurement of compressional and shear wave velocities through the soil profile allows identification of changes in soil type, groundwater elevation, and so forth. The number of test borings, which are usually quite expensive, may be reduced when supplemented by information from seismic tests.

There are several seismic techniques which are capable of measuring shear wave velocity, V_s. Each of these methods has concomitant advantages and limitations. Methods which the authors consider to be most useful in liquefaction studies are seismic refraction surveys (SRS) and a new technique for spectral analysis of surface waves (SASW). These seismic techniques will be discussed in the following paragraphs.

Of secondary importance to this study is the use of seismic techniques to evaluate shear modulus in the initial portion of the stress-strain curve (G_{max}). Measurement of this property, which is useful in modeling soil properties for dynamic response analyses and in making liquefaction correlations (Dobry et al., 1981), is commonly done by seismic crosshole or downhole survey. However, because little stratigraphic information is added by these methods to that obtained from careful logging of the boreholes, their use in subsurface mapping is quite limited.

In recent years, geophysical well logging techniques have been modified for use in shallow geotechnical exploration. Of the many available methods, natural-gamma and gamma-gamma logging are most useful in mapping sedimentary layers, as in earth dam foundation explorations. Surveys conducted from the surface using ground-penetrating radar may also be applicable for such investigations. These methods will be discussed below.

Several other geophysical techniques are in development, but are not sufficiently refined to be of practical use in subsurface mapping. An example is the electric resistivity survey, which shows promise in evaluating soil properties. At present, however, this technique is most applicable in locating buried metallic structures, depth to bedrock or water table, and making crude estimates of rock strength (Keller, 1979).

Seismic Refraction Survey.

The seismic refraction survey (SRS) is performed using an impact or explosive energy source with a single or multiple array of transducers, as shown in figure 19. Apparent travel time versus distance curves can be derived from recorded data, which can then be used to determine layering of the soil system, elastic wave velocities, thickness, and dip of each layer.

Several factors are important in conducting the SRS, such as topography, distance, geophone spacing, and depth of the seismic source. Traverses should be selected where no abrupt changes in topography occur. Distances between

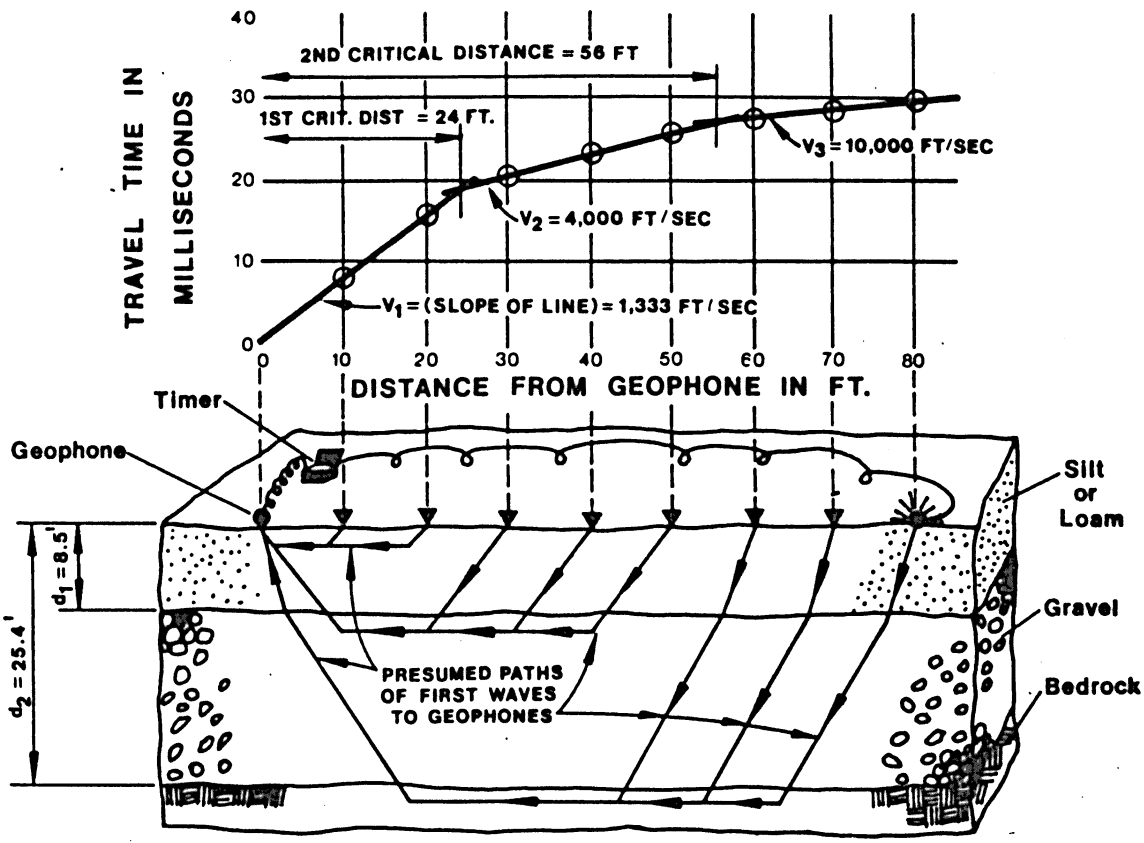


Figure 19. Seismic Refraction Survey (from Woods, 1978).

the source and receivers must be accurately surveyed to ensure correct velocity calculations. The geophone spacing and overall length of the seismic traverse are dependent on the amount of detail and depth requirements of the survey. In all cases, velocities of the near-surface materials must be determined, and initial geophone spacings may be on the order of two to five feet apart. Spacing of geophones is then increased, and geophones for shallow surveys may be placed as far as 50 feet apart on an exceptionally long traverse (Ballard and McLean, 1975).

When using seismic refraction techniques, it is almost always desirable to run both forward and reverse traverses along the seismic line. Velocities measured from a single point are influenced by the dip of the soil layers, and only apparent time - distance curves may be obtained. These errors can be detected and accounted for when both forward and reverse traverses are performed, and calculation of a true velocity profile is possible.

The SRS is particularly useful as a rapid means for preliminary investigation of a site. In this capacity, seismic refraction is economical and provides a high cost-return ratio. General features of the subsurface, velocities, and depth to water table and bedrock can be obtained quickly and relatively cheaply. Being performed entirely from the ground, the SRS has the additional benefit of requiring little or no drilling.

This method is not very useful where details of the soil stratigraphy are required. Layers thinner than two or three feet are not easily detected, and a low-velocity layer sandwiched between layers of higher velocity will not be identified. In a liquefaction investigation, where detection of a thin, loose zone might be of critical importance, use of the SRS would not be recommended, except as a means of preliminary investigation.

A recent field investigation was conducted near Parkfield, California to locate a liquefaction-susceptible layer for instrumentation (Holzer et al., 1986). Seismic refraction was used in the initial reconnaissance of prospective sites. In this case, a large hammer was used for generating impact energy. Though a detailed mapping of sedimentary layers was not obtained by SRS at the Parkfield site, some layers down to about 15 feet were detected. Water table elevations were also detected and were ascertained to be quite accurate by subsequent auger borings. The sand layer to be instrumented was at a depth greater than 15 feet and was not detected by the seismic refraction technique either because of its depth or low velocity or perhaps both.

Spectral Analysis of Surface Waves.

The Spectral Analysis of Surface Waves (SASW) is a relatively new seismic method for in situ measurement of shear wave velocities in soil deposits and for detecting the thicknesses of soil layers. It is being developed principally by researchers at the University of Texas, Austin, and is based upon the generation and measurement of Rayleigh surface waves (Stokoe and Nazarian, 1985). The method has some technical advantages over other seismic methods for measuring shear wave velocities, but has the disadvantage of few case histories. As with other newly-developed methods, this disadvantage can only be remedied by further research, and an accumulation of site-specific studies.

Figure 20 is an idealized diagram of the SASW apparatus. The common receivers midpoint geometry (CRMP) described by Nazarian et al. (1983) is also shown, in which two geophones are placed equal distances from an imaginary centerline. Surface waves are generated by a hammer impact or other

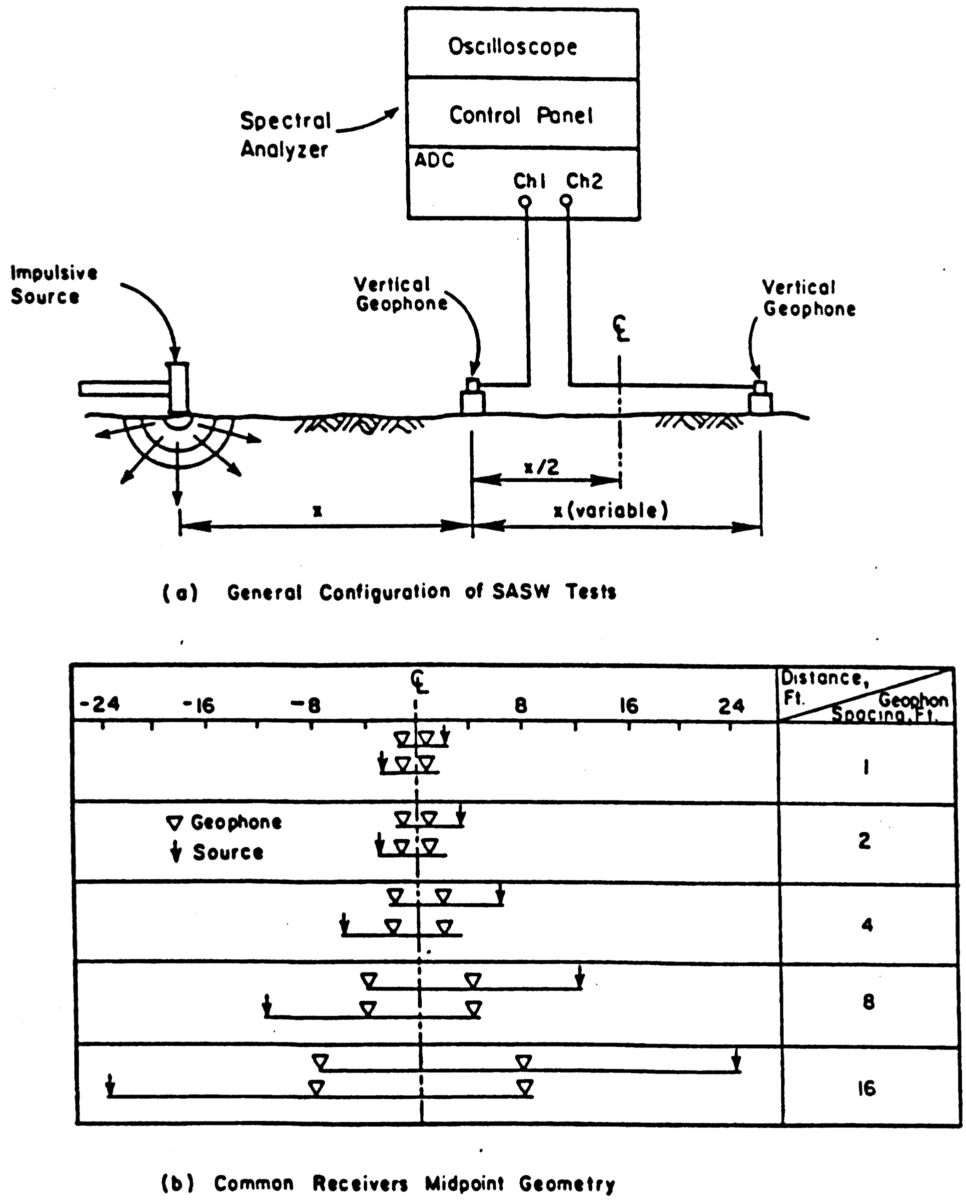


Figure 20. Schematic of Experimental Arrangement for SASW Tests (from Stokoe and Nazarian, 1985).

mechanism, striking a point such that the distance between the source and the near receiver is equal to the distance between the receivers. Wave trains arriving at the geophones are recorded, digitized, and saved for further analysis. Receiver spacing is then increased, and the process repeated. Distances between the geophones of 2, 4, 8, 16, 24, 32, 48, and 64 feet are generally used (Stokoe and Nazarian, 1984). To generate measurable surface waves, hammer size must be increased with increased geophone spacing. As with the crosshole method, forward and reverse tests are performed and the results averaged to eliminate errors. The amount of impact energy transferred to the soil and the frequency content of the generated surface waves control the depth to which shear wave velocity profiles can be delineated by this technique.

In the SASW technique, Rayleigh wave velocity, shear wave velocity, and shear modulus are determined by analyzing the phase information for each frequency of waves detected by the two receivers. If the stiffness of a soil site varies with depth, then the velocity of the Rayleigh wave will vary with frequency. To quantify this variation of R-wave velocity with frequency, time-domain data recorded by the geophones must be transformed into the frequency domain. Several wave form analyzers, such as the Hewlett-Packard 5423A, are capable of performing this transformation rapidly and efficiently. Output from this analysis, the dispersion curve, is a plot of frequency (wavelength) versus Rayleigh wave velocity. A typical dispersion curve, which may be considered the 'raw data' from SASW testing, is shown in figure 21.

Measurement of velocities of underlying soil layers is affected by the existence of a layer with high or low velocity at the surface of the medium. Because of this, Rayleigh wave velocities shown in a dispersion curve are not actual velocities of the layers but are apparent R-wave velocities. Actual

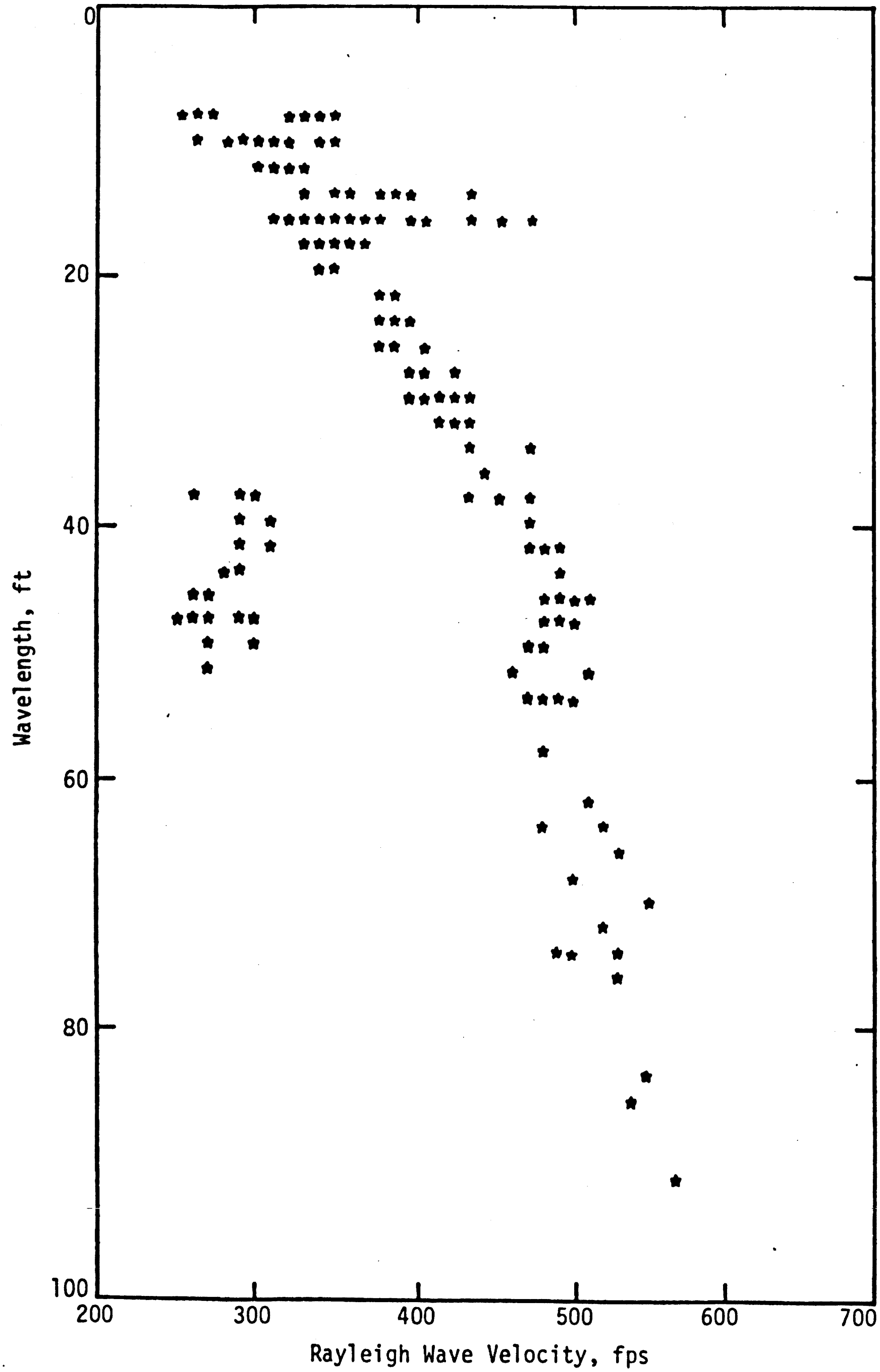


Figure 21. Dispersion Curve Determined from SASW Tests at Spirit 1 Site, Near Spirit Lake, Washington (from Stokoe and Nazarian, 1984).

propagation velocities at different depths are determined by 'inversion' of the dispersion curve. Inversion consists of determination of the depth and actual surface wave velocity of each layer from the apparent R-wave velocity versus wavelength information. Data analysis methods employed in SASW testing and assumptions implicit in the calculations are described in detail by Stokoe and Nazarian (1985).

Spectral Analysis of Surface Waves is somewhat useful in determining the subsurface stratigraphy of a site, but the definition of soil layers generally is not as precise as by other profiling methods such as the CPT. Figure 22, derived from the dispersion curve shown in the previous diagram, shows a typical plot of variation of shear wave velocity with depth. Though the computed shear wave velocities are relatively sensitive to changes in soil properties with depth, very thin layers in highly-stratified deposits might remain undetected. Layers as thin as a foot or so are detectable very near the surface by SASW, but the thickness of detectable layers increases with depth. In delineating soil layers, the SASW technique is somewhat more sensitive than seismic refraction and can also detect low-velocity layers beneath high-velocity layers.

Testing by SASW is nondestructive, is performed from the ground surface, and requires no boreholes. The method is more complex and expensive than seismic refraction but provides for more detailed mapping of stratigraphy. Though sophisticated electronic equipment is needed, the SASW has the potential for full automation. Such automation would make the SASW more economical and reduce the need for highly-trained personnel in the field.

Another advantage associated with the SASW is that testing can be performed in gravelly or cobbly deposits where exploration by other methods might not be feasible. Initial investigations conducted by SASW are useful in

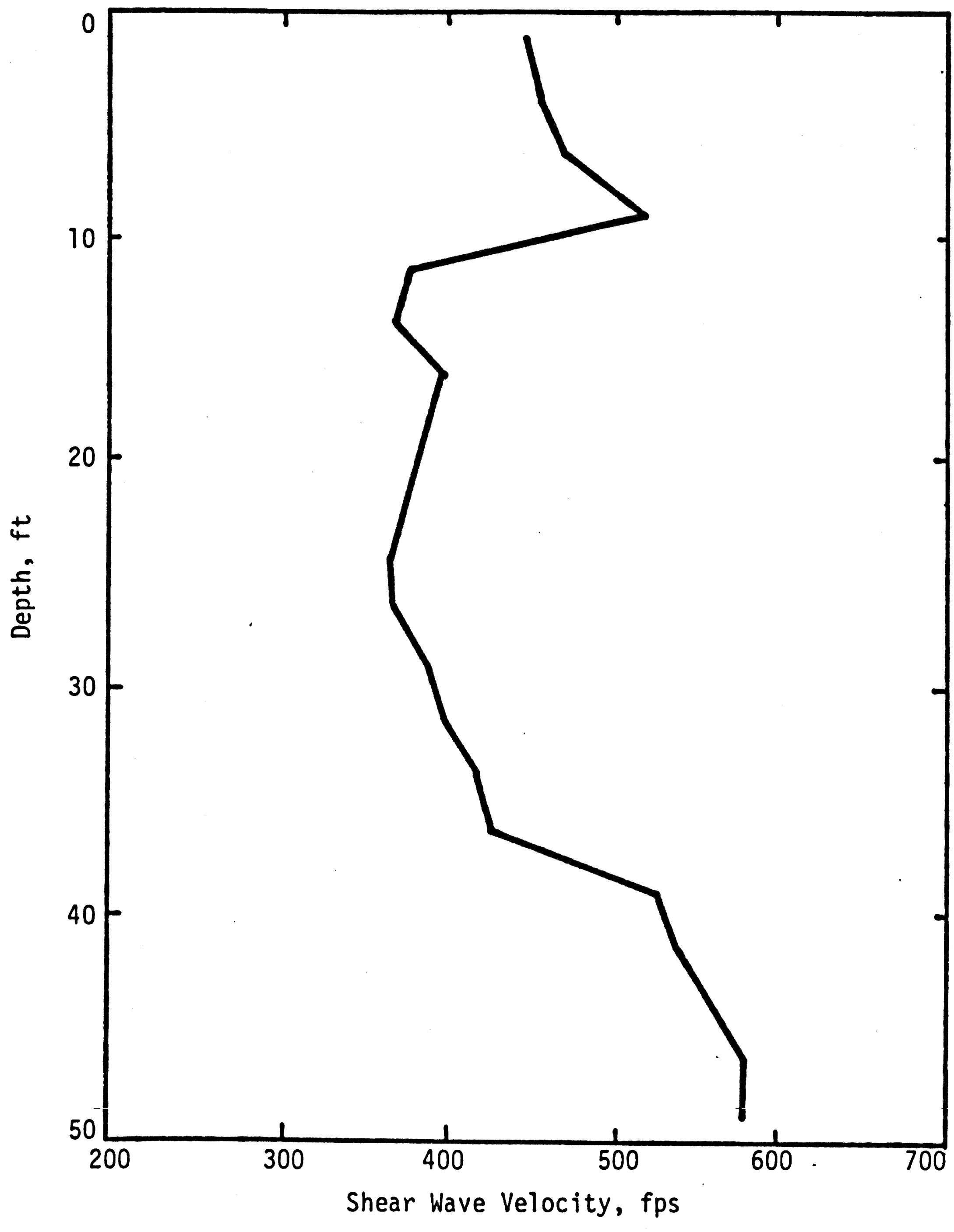


Figure 22. Shear Wave Velocity Profile Determined by SASW Tests at Spirit 1 Site, Near Spirit Lake, Washington (from Stokoe and Nazarian, 1984).

profiling very coarse materials, locating bedrock surfaces, and detecting interfaces between soil units with large velocity contrasts. Also, low-velocity layers suspected of having liquefaction potential may be identified by SASW and slated for more detailed investigation by other techniques.

A disadvantage of the SASW technique is that no sample is recovered, and identification of soil type is not possible from shear wave data alone. Therefore, for soil profiling, some drilling and sampling is required in addition to SASW. The method was developed primarily for assessing stiffness profiles for soils and may be appropriately used in liquefaction investigations when measurement of soil modulus is needed. In these instances, some stratigraphic information is gained as a side-benefit. Some preliminary correlations between shear wave velocity measured with SASW and liquefaction occurrence have been made and give some promise for using velocity information for assessing liquefaction susceptibility (Stokoe and Nazarian, 1985).

Ground Penetrating Radar.

Ground Penetrating Radar (GPR) uses high electromagnetic frequencies (100 to 1000 MHz) which are transmitted from a radar antenna coupled to the ground's surface. The transmitted radar signals are reflected from various lithologic interfaces within the ground, such as soil/rock interfaces or different soil layers, and picked up by the radar receiver. Details of equipment and physical theory for this method are discussed by Morey et al. (1978).

For presentation of data, GPR signals are processed and displayed by a graphic recorder. As the antenna is moved along the ground surface, a graphic record providing a continuous profile along the traverse is developed. The antenna may be towed up to 5 mph for rapid exploratory work. Detailed studies can be performed by hand towing the antenna at very slow speeds (1/4 mph) or placing the antenna in a static mode at specific locations (Benson and Glaccum, 1979).

Under some conditions, high-resolution mapping of sedimentary units is possible with GPR. Figure 23 is a radar profile of stratified gravel and sand deposits. In these types of materials, penetration depths of 30 to 50 feet are commonly achieved. Even deeper explorations can be performed with GPR by utilization of borehole antennas.

A severe limitation of the radar survey is that significant penetration may not occur in certain soil types and groundwater conditions. In fine-grained materials (silts or clays), the radar pulse is attenuated, and penetration may not exceed three feet or so. Radar is similarly ineffective in salt-contaminated soils or where ground water increases in electrical conductivity. Use of the radar method in identifying soil layers is also restricted because no sample is obtained, and there is no direct way to identify soil types.

Though the GPR is a shallow, site-specific tool, in some cases the technique offers high resolution subsurface information which provides a rapid, cost-effective means of making site assessments. The minimum thickness of layer detectable by the radar method depends on the geologic environment ~~and may range from an inch or less to a foot. Radar surveys~~ must be verified by borehole or other technique, and in some cases may be an economical technique for delineating the soil cross section between boreholes.

Successful use of the GPR is reported by Dr. J.P. Singh (verbal communication, May 1986), who investigated a site in Charleston, South Carolina known to have liquefied during the 1886 earthquake. Past lique-

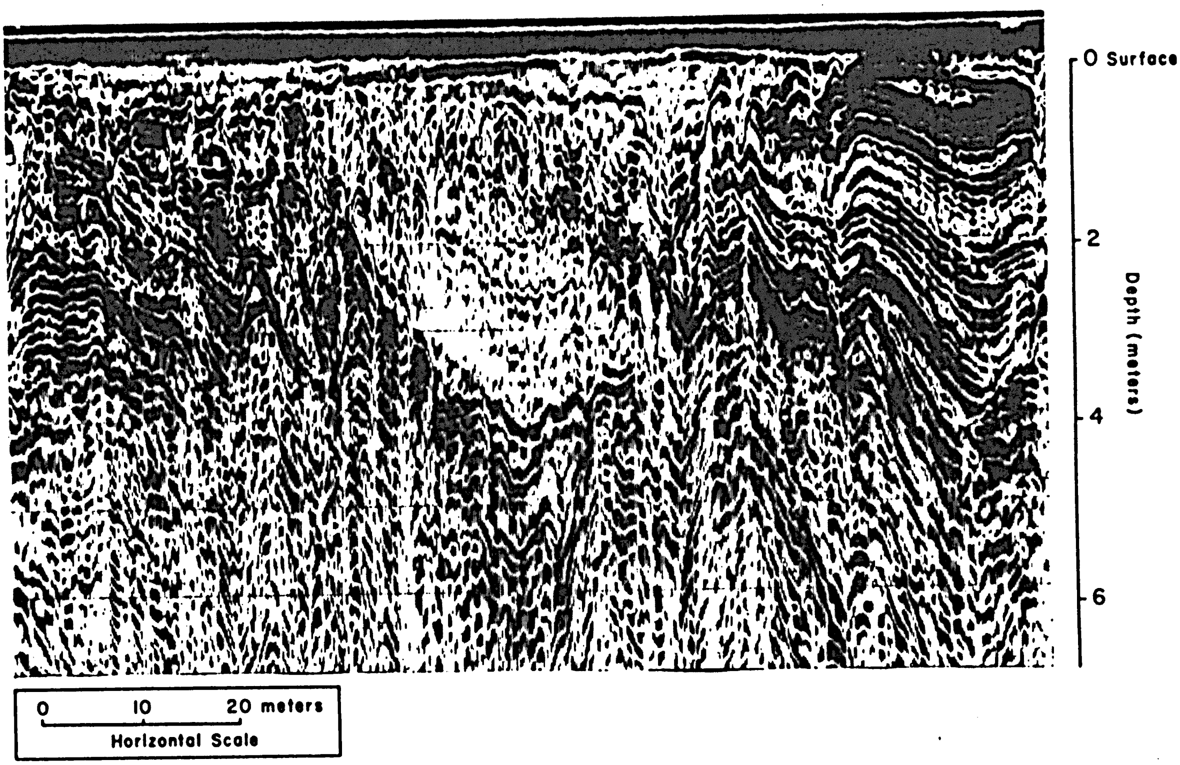


Figure 23. Radar Profile of Stratified Deposits, Consisting of Alternating Beds of Gravel with Coarse and Fine Sands (from Benson and Glaccum, 1979).

faction effects such as fissures and sand intrusions were identified by radar surveys. Sand layers as thin as 1/4 to 1/2 inch at shallow depths were located during the initial investigation. Existence of these layers was later verified by exploratory trenching.

Geophysical Borehole Logging.

Geophysical logging of boreholes was developed primarily by the oil industry for large-scale lithologic mapping. During the last two decades, these techniques have become available for geotechnical exploration on a much smaller scale. Though not widely used, borehole geophysics is emerging as an economical method for stratigraphic characterization and correlation. Modifications to borehole logging techniques and equipment have been made recently which make the technique more applicable to shallow subsurface investigations.

The primary logging system component is a bullet-shaped device called the sonde, which contains sensors and/or a excitation source. Individual sondes are positioned in the borehole by means of a shielded cable, which also provides a path for signal transmission uphole and for power to the sonde. Depth control is accomplished by a mechanical depth meter, and signals are analyzed and recorded by proper processing assemblies (Wright and Harrell, 1979). With proper site-specific calibrations, data derived from geophysical logs can be used with a limited amount of laboratory testing to provide a picture of subsurface conditions.

Of the various sondes which are available for borehole logging, natural gamma is most applicable to lithologic identification. The natural gamma log records the gamma photon emission of materials being investigated. The device used for this purpose responds to potassium 40 (K-40) found in most soils and rock, but can also be influenced by small amounts of uranium and thorium. Due to high K-40 content of clays, in most environments the log shows a positive response to clay content in the formation. Clean quartz sands generally produce low count rates on the natural gamma log. Because of such an intimate relationship with soil chemistry, the log is useful in delineating sedimentary layering through the borehole.

The gamma-gamma log records the backscatter gamma radiation induced by a cobalt-60 source and detected by receivers in the sonde. The principle of operation is similar to nuclear density devices used for earthwork inspection, with the number of backscattered electrons being proportional to density of the material. When properly calibrated, the gamma-gamma log can be used to derive bulk density.

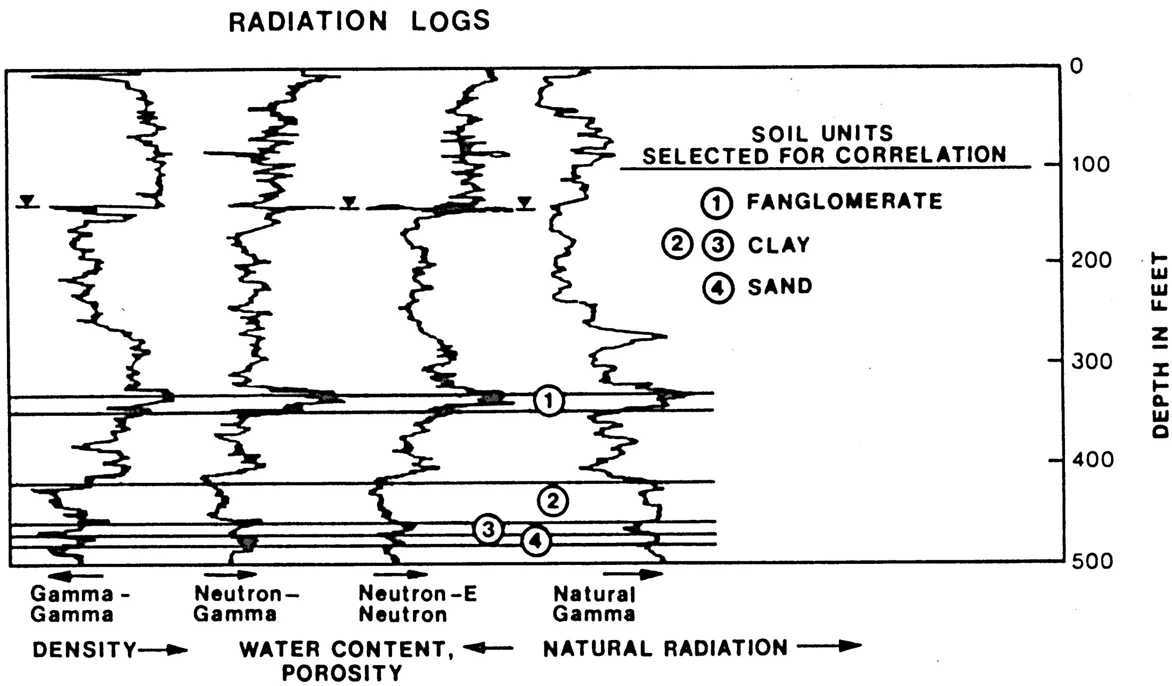
Neutron-gamma and neutron-epithermal logs, which provide a measure of moisture content, are also useful in subsurface mapping. Of the two, neutron-epithermal is most reliable and is correlatable with porosity of the material being tested. Other logging techniques, such as sonic or electrical resistivity, are also being developed for use in geotechnical work.

For accurate determination of soil layering or other physical properties, geophysical techniques require careful on-site calibration for borehole effects, decay characteristics of the radioactive sources, and chemical characteristics of the formations investigated. Size of the borehole is a most important factor in interpreting borehole measurements, and each borehole should be logged by caliper sonde or similar device. The caliper log may also provide some useful stratigraphic information. For example, zones of loose or weathered material may be indicated by a localized increase in borehole diameter.

Identification of sedimentary units in the subsurface is the most reliable when positive indicators are determined from more than one logging technique. Figure 24 shows typical logs obtained from use of the radiation sources and receivers discussed above. Layering in the profile is evident, as is location of the ground water table. Resolution of sedimentary structures is rather coarse in these logs, as layers two to three feet are the thinnest layers detectable on these logs. On logs at a larger scale, layers as thin as six inches may be detected.

Geophysical borehole logging is a fairly reliable technique for differentiating between soil types, such as sands and clays. Geophysical borehole logging is a useful tool for stratigraphic characterization and correlation, particularly over large distances. At present, the method is still of limited value for small-scale geotechnical investigations, consisting of a few, shallow borings. However, as the number and depth of borings is increased, savings can be realized by reducing the sampling program and substituting geophysical exploration (Crosby et al., 1979).

Borehole logging has been used to advantage in some foundation investigations for dams. In investigations at the Twin Lakes and Jackson Lake dams (to be discussed subsequently), zones of dense gravel or cobbles were identified from geophysical logs which aided engineers and geologists in selecting sampling depths. Sample depths were selected from the borehole logs, so that interference of the sampling process by coarse materials was minimized.



NOTE: Arrows point in the direction of increasing values.

Figure 24. Geophysical Borehole Logs, Sundesert Boring B-DH-35 (from Crosby, et al., 1979).

IV. CASE STUDIES

JACKSON LAKE DAM, WYOMING

Description of Project.

Jackson Lake Dam is located on the Snake River in northwestern Wyoming, within the boundaries of the Grand Teton National Park. The dam was constructed in stages over an 11- year period, from 1906 to 1917. The 1906 temporary dam consisted of a rock- and gravel-filled timber crib dam flanked on either side by earthen embankments. A low dike about 1500 feet in length was constructed to the north. In 1911 a permanent structure was built with a short southern rockfill embankment, a concrete section 200 feet in length, and an enlarged northern dike. A final enlargement of the northern dike to its present dimensions was completed in 1917.

Figure 25 shows the extent of the 1906 and 1911 construction phases, the completed Jackson Lake Dam, and locations of several of the foundation investigation programs which have been conducted. The north dike currently has a crest length of about 4,500 feet, a crest width of 20 feet, and varies in height from 45 to 4 feet. It is constructed of sand and gravel placed by hydraulic fill methods.

Foundation conditions beneath Jackson Lake Dam are extremely variable. Figure 26 is a simplified cross-section beneath the dam showing predominant geologic units. The southern dike and concrete section are founded entirely on a Pleistocene volcanic tuff. This material has high strength, low permeability and low deformability, and is considered a competent foundation material.

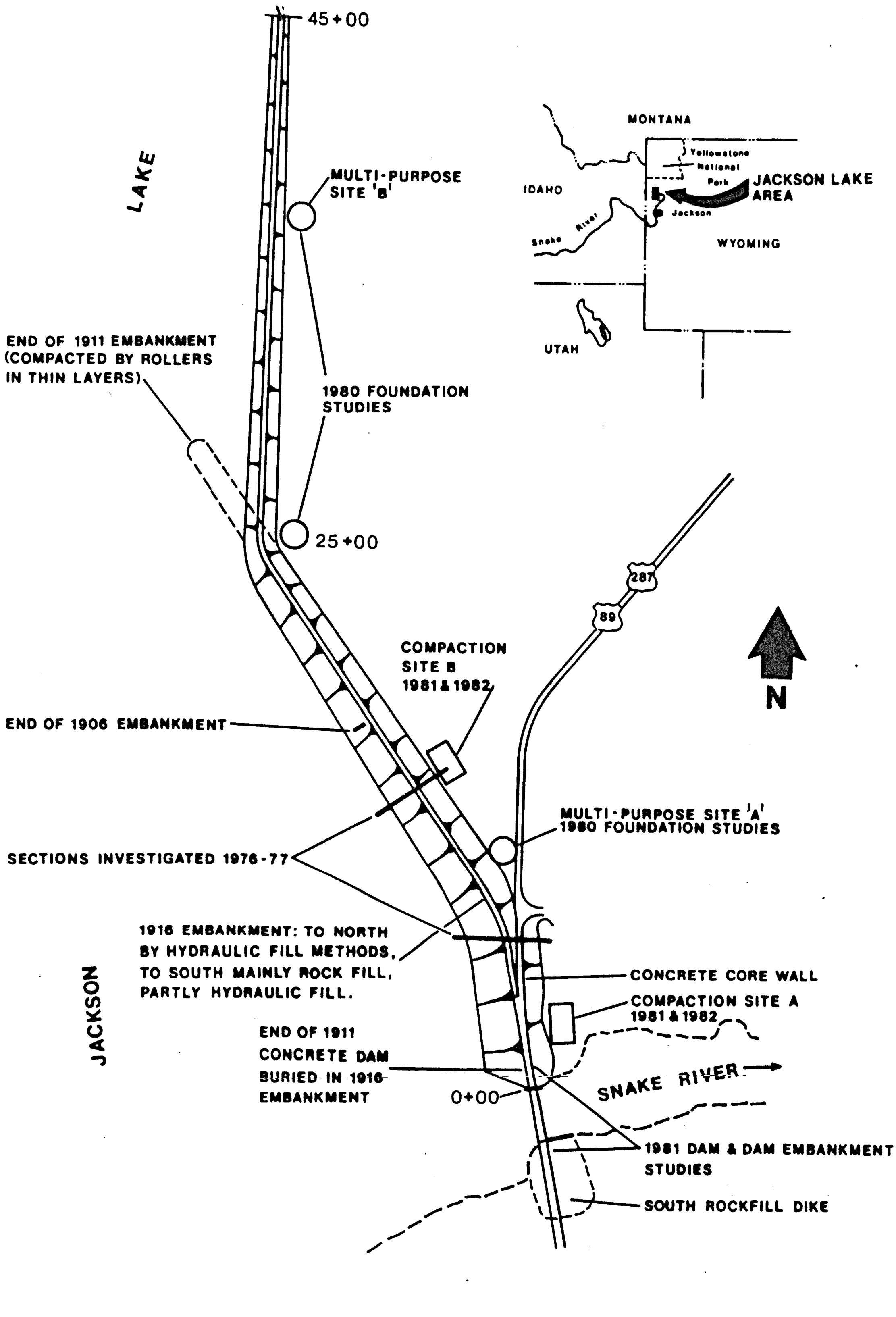


Figure 25. Drawing of Jackson Lake Dam, Showing Extent of 1906 and 1911 Construction Phases, and Locations of Recent Foundation Investigations (adapted from Lockhart and Link, 1981).

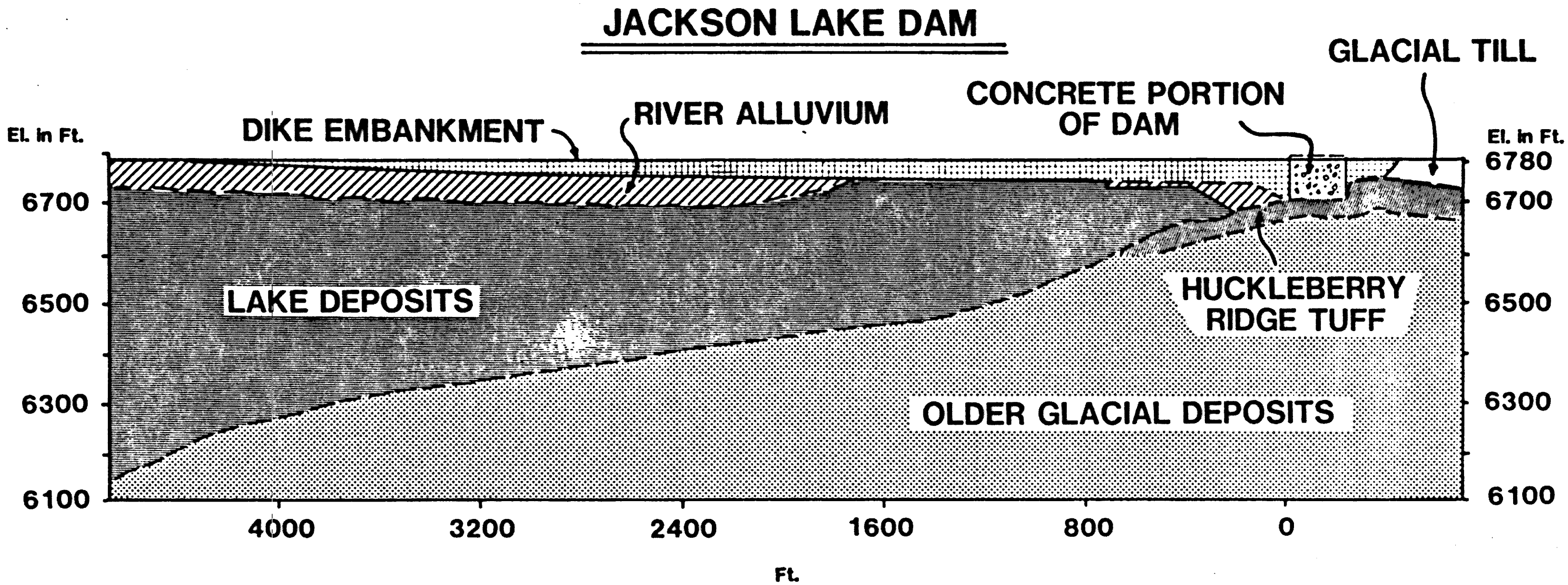


Figure 26. Longitudinal Cross Section of Foundation Materials at Jackson Lake Dam, Wyoming (from U.S.B.R., 1984).

The northern dike is founded on fluviolacustrine deposits, ranging from 8 to 270 feet thick. The foundation subunits consist of fine-grained lacustrine sediments (sands, silts and clays) deposited by the ancestral Jackson Lake overlain by coarse-grained alluvial fan and stream channel sediments (cobbles, gravel, and sands with some silt and clay) deposited by Pilgrim Creek and the Snake River. The underlying lacustrine sediments form a thick and laterally extensive geologic subunit of fairly uniform consistency. The upper alluvial fan/delta sequence of sediments is highly variable and extremely complex, with many lenses and discontinuous layers. These materials often have a low density and are generally loose and saturated, indicating potential vulnerability to liquefaction under seismic loading.

Jackson Lake Dam is located in an area with potential for strong earthquake shaking. The major concern to seismic stability of the dam is unconsolidated, recently deposited sediments beneath the northern embankment. An accurate map of the foundation materials beneath this section of Jackson Lake Dam was compiled by the U.S. Bureau of Reclamation for evaluation of the seriousness of this problem.

Intensive subsurface investigations have been conducted from 1975 to the present at Jackson Lake Dam. Various drilling, sampling, and in-situ testing methods have been instituted to map and obtain a correlatable relationship between the subsurface stratigraphic units. Correlation of SPT testing, undisturbed sampling, and cone penetrometer soundings proved to be the most adequate and complete means of evaluating the thickness, areal extent, and engineering properties of the unconsolidated foundation units. Other investigative techniques employed were geophysical borehole logging, shear wave velocity measurements, and exploratory shaft excavations. A summary of the subsurface investigations which have been completed are given in the Geologic Report for Final Design (U.S.B.R., 1985).

Mapping of Subsurface Stratigraphy

Subsurface explorations and field investigations programs were conducted intermittently at Jackson Lake Dam from the early 1900's to 1984. A total of 224 drill holes were logged during this period. The early investigations (1913 through 1981) were conducted principally to determine the geologic conditions and stratigraphy of the foundation materials and the engineering properties of the embankment materials. Later investigations (1982 through 1984) were conducted to expand the stratigraphy and to evaluate the potential for liquefaction at the site. The geologic exploration programs conducted over the years were distributed upstream, downstream, and through the entire dam to define the location and engineering properties of the various materials in the foundation and embankments.

Penetration resistance tests (SPT) were conducted on 2.5-, 3- and 5-foot intervals through the embankment and foundation materials. Procedures essentially in accordance with ASTM standards discussed previously in this report were used. For each sample taken, blowcount was recorded, a moisture sample taken, and the remaining sample was saved for laboratory analysis. No penetration resistance tests were conducted in intervals in which the action of the drill and the cuttings return indicated the presence of considerable coarse gravel and/or cobbles. Some difficulties involving artesian pressures and resultant flowing sand were encountered during testing. Zones of heaving or flowing material almost always involved fine-grained black sands. Intervals in which considerable gravel material, flowing sand, or associated phenomena occurred were recorded in the drill logs.

The mechanical cone penetrometer test (MCPT) was used extensively in the 1980 field program to provide stratigraphic correlations with the penetration resistance tests. Procedures outlined by ASTM were used, and standard cone rods one meter in length were advanced through the subsurface to refusal. Where refusal occurred above the required depth of the hole, the cone was retrieved and the hole then drilled out and cased to a depth beyond the impenetrable material. Cone testing was then resumed until the design depth was attained. The geologic interpretation of the material type and stratigraphy encountered in the MCPT was made on the basis of the cone resistance q_c, as detailed below.

Geologic Unit	Predominant Soil Type	Cone Resistance (Kg/cm^2)
Qfc	Fines	$q_c < 50$
Qfs	Sand	$50 < q_c < 110$
Qfg	Gravel	$q_c > 110$

Correlation with known geologic units was made on the basis of depth, thickness, and stratigraphic position.

The electronic cone penetrometer test (ECPT) was used extensively in the north dike and a compaction pile test site investigations conducted as part of the 1984 exploration program. This test was used to expand the stratigraphy developed from the earlier investigations and, in particular, to define the continuity of the units present in the foundation. Tests were conducted according to ASTM specifications, using either a 10 or 15 cm^2 cone, with data stored electronically. Where refusal occurred before the desired sounding depth, drilling procedures described previously were used to continue the test.

The geologic interpretation of the ECPT data was based on a point-by-point analysis of the cone resistance (q_c) and friction ratio (FR) data using the simplified classification chart (figure 13) developed by Robertson and Campanella (1984). Based on the large amount of cone data accumulated at Jackson Lake, site-specific modifications were made to this classification scheme. The following criteria were used for identification of various soil types based on ECPT friction ratio:

Soil Classification	Friction Ratio (FR)
Sandy Gravel	FR < 0.75
Sands	0.75 < FR < 2.0
Silty Sands to Sandy Silts	2.0 < FR < 5.0
Clays	FR > 5.0

The detailed testing and sampling of the north dike foundation materials also included the collection of undisturbed samples. Both 3- and 5-inch diameter samples were obtained using hydraulic Osterberg piston samplers. Standard U.S.B.R. field procedures were used, which included measuring, trimming, and weighing each sample to determine wet and dry field densities. Selected 5-inch long samples were sealed in their tubes, frozen, and transported to Denver for laboratory testing. Where gravels did not prevent driving of the tube sampler, undisturbed samples provided excellent delineation of foundation sediments over the intervals sampled..

Correlation of SPT and CPT data with information obtained through undisturbed sampling proved to be the most effective and economical method of detailing the subsurface stratigraphy at Jackson Lake Dam. Use of the electronic cone penetrometer allowed investigators to delineate in some detail the layering of foundation sediments, and accurately identify zones susceptible to soil liquefaction. Tube samples and SPT were used to verify soil types and add detail to developed profiles.

Explorations at Jackson Lake Dam also included evaluation of the condition of embankment materials. Due to large amounts of gravel and cobbles in the hydraulic-fill section of the dam, drilling and CPT were not used for this investigation. Rather, eight 48-inch shafts were excavated with a clamshell bucket. This allowed close inspection, and provided an opportunity for block-sampling within the embankment. Excavation of these shafts was expensive, but was a successful method for exploring the coarse sediments used in construction of the dam.

Multi-Purpose Testing Sites.

In order to provide site-specific correlation of the various methods used in Jackson Lake Dam explorations, several 'multi-purpose' sites were established and investigated during the 1979 and 1980 drilling seasons. At each of these sites, detailed profiles of subsurface sediments were developed by utilizing several different investigative techniques. The effectiveness of each technique in delineating subsurface sediments at Jackson Lake may be evaluated through a comparison of the test results at these multipurpose sites.

Two of the multipurpose sites, designated 'A' and 'B' for the purposes of this study, are shown in figure 25. Sediments at site B were coarse, having as much as 75% gravel. The maximum amount of gravel in SPT samples obtained at site A was around 20%. These two sites provided an opportunity to compare test results from various techniques in both coarse soils and those having a low gravel content.

Figure 27 shows SPT blowcounts, CPT tip resistances, and classification of soils from SPT samples at multipurpose site A. Also shown are neutron and gamma-gamma logs from nearby boreholes. Cone tip resistances greater than 150 kg/cm^2 are not plotted.

At this site, the CPT sounding gives the best delineation of sedimentary lithology. More stratigraphic detail was detected by this method than the others, and a quantitative 'picture' of soil layering is shown. The friction ratio plot (not shown), allows tentative identification of soil types in each layer. The CPT logs as published in the USBR reports provide sufficient detail to identify layers as thin as about 4 to 6 in. Finer definition might be provided by logs digitized at a closer intervals and plotted at a larger scale. The cone penetrometer was also the quickest and most economical method used at Jackson Lake Dam.

The plot of SPT N-values versus depth was less effective in delineating changes in soil conditions. Because of the 1.5-foot sampling length and intermittent test intervals, layers thinner than two feet were not discernible. Soil classifications from SPT samples taken at 3-foot intervals give added clarity to the profile developed from this method.

Because materials at site A were not too gravelly, continuous undisturbed sampling was done with a 3-inch Osterberg sampler. These samples provided a most accurate profile of sediments, useful in correlating results of the other test methods. Five-inch Osterberg samples were taken at selected intervals, frozen, and shipped to Denver for laboratory analysis.

Identification of soil layers at site A would be difficult with the geophysical logs alone. Some bed definitions were picked up in the gamma-gamma log, but these are clear only when the geophysical log is compared with CPT or blowcount records.

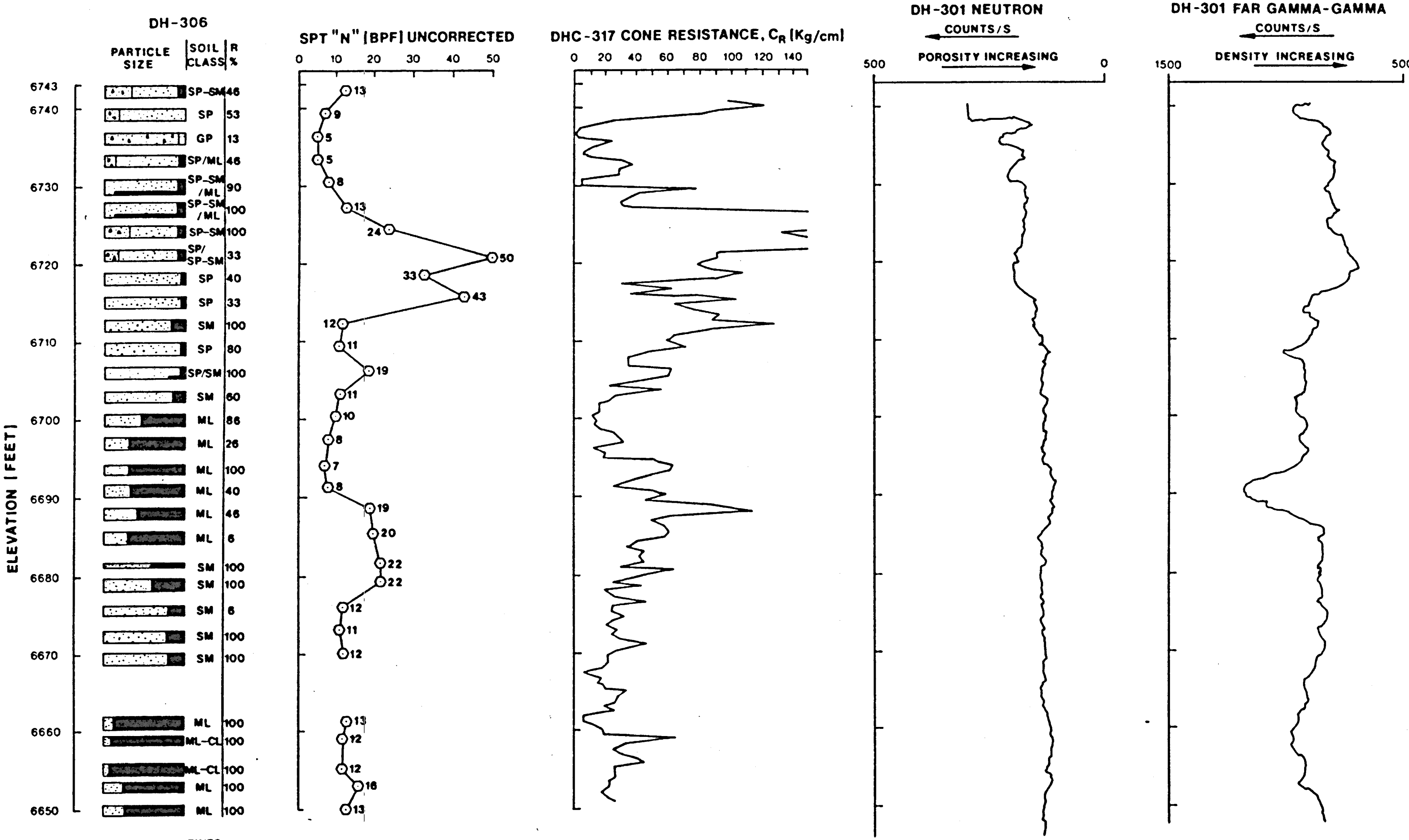


Figure 27. Comparison of Test Results at Multipurpose Site A, Jackson Lake Dam (from Drawing 17-D-1683, Lockhart and Link, 1981).

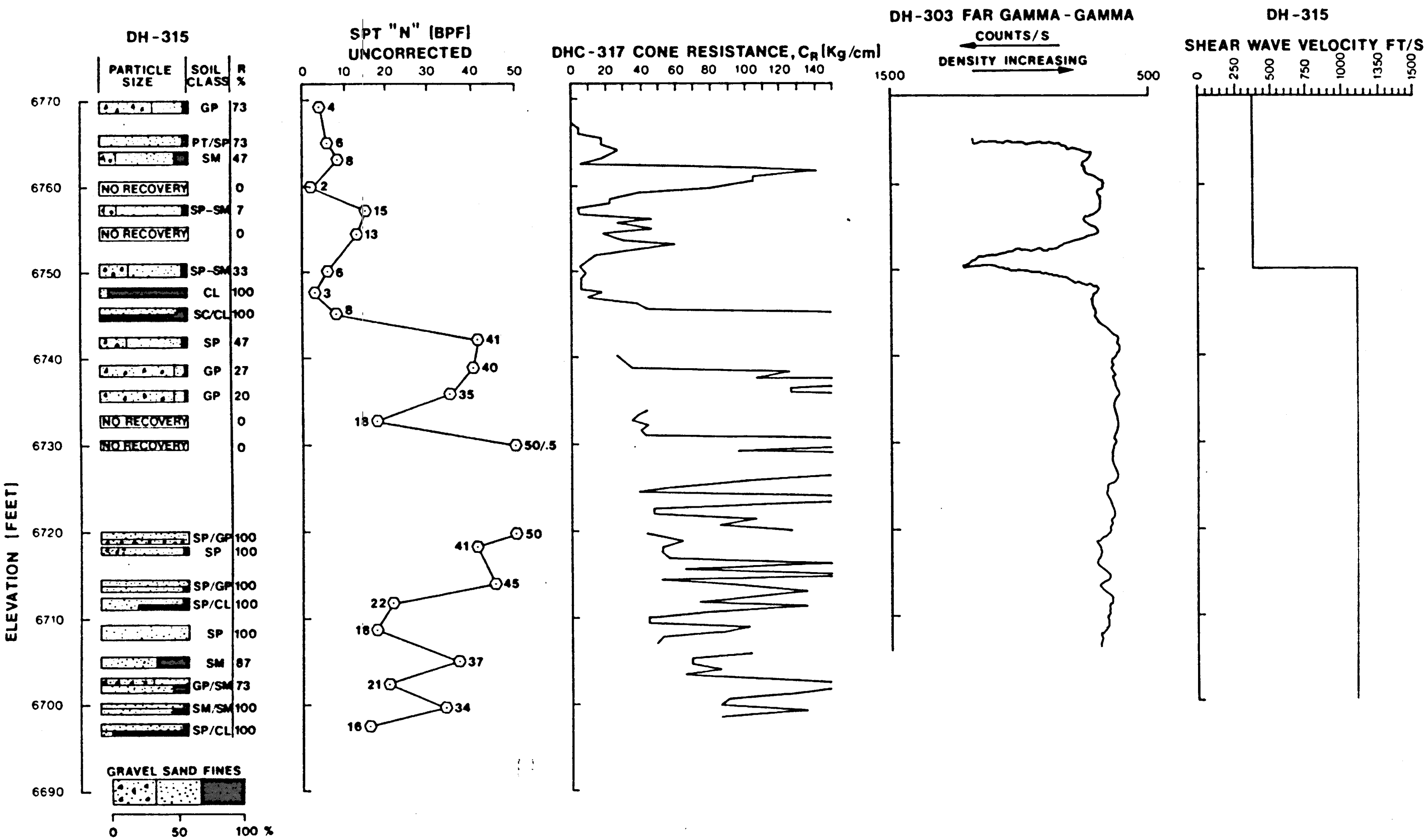


Figure 28. Comparison of Test Results at Multipurpose Site B, Jackson Lake Dam (from Drawing 17-D-1685, Lockhart and Link, 1981).

The gravelly conditions at multipurpose site B are very evident in the CPT sounding of figure 28. Again, values greater than 150 kg/cm^2 are not shown. Some gaps in this log appear where the cone met refusal, and drilling was performed over a short interval until the cone sounding could be continued. Though tip resistance values are erratic, the CPT was able to penetrate and give a fairly detailed profile in most of the coarse deposits.

The densest material in the gravel layer at site B was too hard to sample by SPT. Other gravelly sections of the profile were tested and sampled, however. Comparison of SPT and CPT logs in figure 28 shows that the SPT definition of lithology in these gravelly sediments is again not as good as that provided by CPT.

Tube sampling at site B was not possible throughout much of the profile. Only a few samples were taken in the finer materials shown at the top and bottom of the SPT borehole. Tube sampling was not possible in materials containing more than about 20 to 30% gravel, depending on the size of gravel particles, density of surrounding material, and so forth.

The gamma-gamma log was more useful in subsurface mapping here than at site A. Changes in density between the sand, clay, and gravel layers are reflected in the geophysical sounding. Resolution of layers from radiation logs was still very coarse, and four to five-foot layers were the thinnest discernible in the record. Shear wave velocities recorded from downhole seismic testing were practically useless in identifying layering at this site (figure 28).

Conclusions.

Standard Penetration Testing and the cone penetrometer were used extensively in investigations of the Jackson Lake Dam foundation sediments. An evaluation and comparison of the two methods was included in the Geologic Report for Final Design (U.S.B.R., 1985). The U.S.B.R. evaluation of the usefulness of SPT and CPT in liquefaction investigations is especially germane to the purpose of this report:

> The geologic investigations of the Stage II reach of the north dike conducted in 1979, 1980, and 1984 involved detailed testing of the embankment and foundation materials using the penetration resistance test (PR or SPT) and both the mechanical (MCPT) and electronic (ECPT) cone penetrometer tests. Comparison of the results from these tests has been an integral part of the examination of the stratigraphy and liquefaction potential of Jackson Lake Dam's north dike and foundation. The purpose of this section is to evaluate the utility of the SPT-CPT correlation for the Stage II reach of the dam and discuss conditions requiring special consideration. . .
>
> The ECPT is probably the most effective method of determining the detailed stratigraphy in a profile. The classification chart shown [see figure 13] provides a satisfactory geologic interpretation of the ECPT data, but may not be sufficiently detailed for use in complex analyses performed by an engineer. The major drawback to this chart is its failure to address gravel. The 1984 ECPT interpretations denote gravel only where its presence was indicated by the gravel's resistance to penetration by the cone, resulting in strong vibrations to the testing apparatus and in loud, grinding noises observed at the drill's head. The particle size at which the resistance to penetration becomes noticeable is not known, but may occur at the transition from fine to coarse gravel, where the particles are large enough to resist being pushed aside as the cone passes through. . . . One solution to this problem may be to arbitrarily select a cone or tip resistance value as the break between sand and gravel. The field experience gained in the 1984 ECPT program suggests that this arbitrary break may occur at tip resistances of 250 to 300 Tsf.
>
> The correlation of SPT and CPT data provides one of the most effective means of evaluating the stratigraphic sequence and engineering properties of materials in a given section. The apparent differences between the two types of tests, as discussed above, can be satisfactorily resolved with careful examination of the data and application of one's knowledge of the conditions and relationships present in the field.

Experience gained from the Jackson Lake Dam investigation demonstrates the value and usefulness of Standard Penetration Testing conducted in conjunction with electronic cone penetrometer soundings. Investigations conducted in this manner are generally economical, and details of complex stratigraphy are identified. Limitations of SPT and CPT are associated primarily with the occurrence of large amounts of gravel or cobbles in the subsurface. Additional research, field experience, and possibly the development of new techniques will make exploration of these types of materials more practical in the future.

Undisturbed tube sampling allowed highly-detailed mapping of sediments at Jackson Lake. Continuous sampling by this method was expensive and was only possible in materials with little or no gravel. Tube samples gave an accurate log at one point in the soil profile, which was then extrapolated laterally on the basis of cone results. This correlation with CPT results made use of these techniques even more economical.

Geophysical and seismic techniques provided some additional stratigraphic information at Jackson Lake Dam, particularly in gravelly deposits. However, resolution of these methods in mapping foundation sediments was limited. Correlation with CPT or SPT logs was necessary for a positive identification of sedimentary units.

TWIN LAKES DAM, COLORADO

Description of Project.

Twin Lakes Dam is located adjacent to State Highway No. 82, approximately 12 miles southwest of Leadville, Colorado. The embankment is a roller-compacted zoned earthfill structure, constructed as part of the U.S. Bureau of Reclamation Fryingpan-Arkansas project. The dam has a maximum height of 55 feet above stream bed and is 3100 feet long. For seepage control, a cutoff trench was excavated 25 to 50 feet below the original ground surface and backfilled with cohesive material.

The Twin Lakes Dam is founded on glacial sediment deposited by multiple advances and retreats of Pleistocene glaciers. This depositional process left highly variable soil conditions at the site. Terminal and recessional moraines, glacial till, and outwash material form complex systems of interbedded silts, sands, gravels, and combinations of these materials in the dam foundation.

Because of potential seismicity in the area and the unconsolidated condition of cohesionless deposits beneath Twin Lakes Dam, an evaluation of the liquefaction potential of these materials was conducted. This study (Kiene, 1983) employed Standard Penetration Testing and examination of construction records of the cutoff trench as the primary investigative procedures. Results of this investigation are summarized below.

SPT Testing and Results.

Prior to beginning a testing program at Twin Lakes Dam, SPT data from the original foundation investigation were evaluated to obtain a preliminary indication of liquefaction potential. These data were from two holes drilled in 1966, and it is not known if the SPT procedures employed meet current standards. Also, grain size information from the material tested were not available. The information from the prior investigation was found to be inadequate data to assess liquefaction hazard at the dam.

Because seismotectonic studies and evaluation of 1966 SPT data indicated a possibility for liquefaction problems, a testing program was initiated at Twin Lakes in the fall of 1981. The testing program included SPT data from six boreholes (TW-1 through TW-3, TW-9, TW-10, and TW-13), which were spaced 450 to 600 feet apart along the dam axis. The holes were drilled with a Failing 1500S drill rig. Standard Penetration Tests were conducted in accordance with ASTM recommendations, including the use of a split liner in the sampler barrel.

The general guidelines for the investigation initially required SPT tests to be taken at 3-foot intervals in the boreholes. During testing in TW-1, gravel layers interfered with test results and caused damage to the cutting shoe. To avoid further problems with such gravel layers, a pilot hole was drilled near each of the remaining boreholes. The pilot holes were logged by geophysical techniques consisting of gamma-gamma, natural gamma, neutron, caliper, and single point resistivity logs. Standard Penetration tests were then performed in the intervals indicated by the geophysical logging to be sandy or fine-grained soils.

Despite this precaution, gravel and cobble-sized material still presented a problem to the investigation. Of 146 tests conducted, 45 had to be terminated before the full 18-inch drive was completed. To evaluate the influence of gravel particles on the remaining tests, depth of penetration versus cumulative blowcount plots were constructed for each test. The slope of the curve representing the final 12 inches of penetration were then examined. A curve with a relatively uniform or gradually decreasing slope was interpreted as meaning that gravels had no influence on the measured blowcount. Figure 29 is an example of a plot where no adjustment to N value was made. Sudden decreases in slope, such as that shown in figure 30, were taken to indicate the presence of gravels. The measured N value was adjusted by shifting the curve to attain a relatively uniform slope based on the steeper portions of the plot. The adjustment to blowcount is illustrated in figure 30.

To evaluate the liquefaction susceptibility of Twin Lakes Dam foundation material, the Seed procedure (Seed and Idriss, 1982; Seed et al., 1984) was

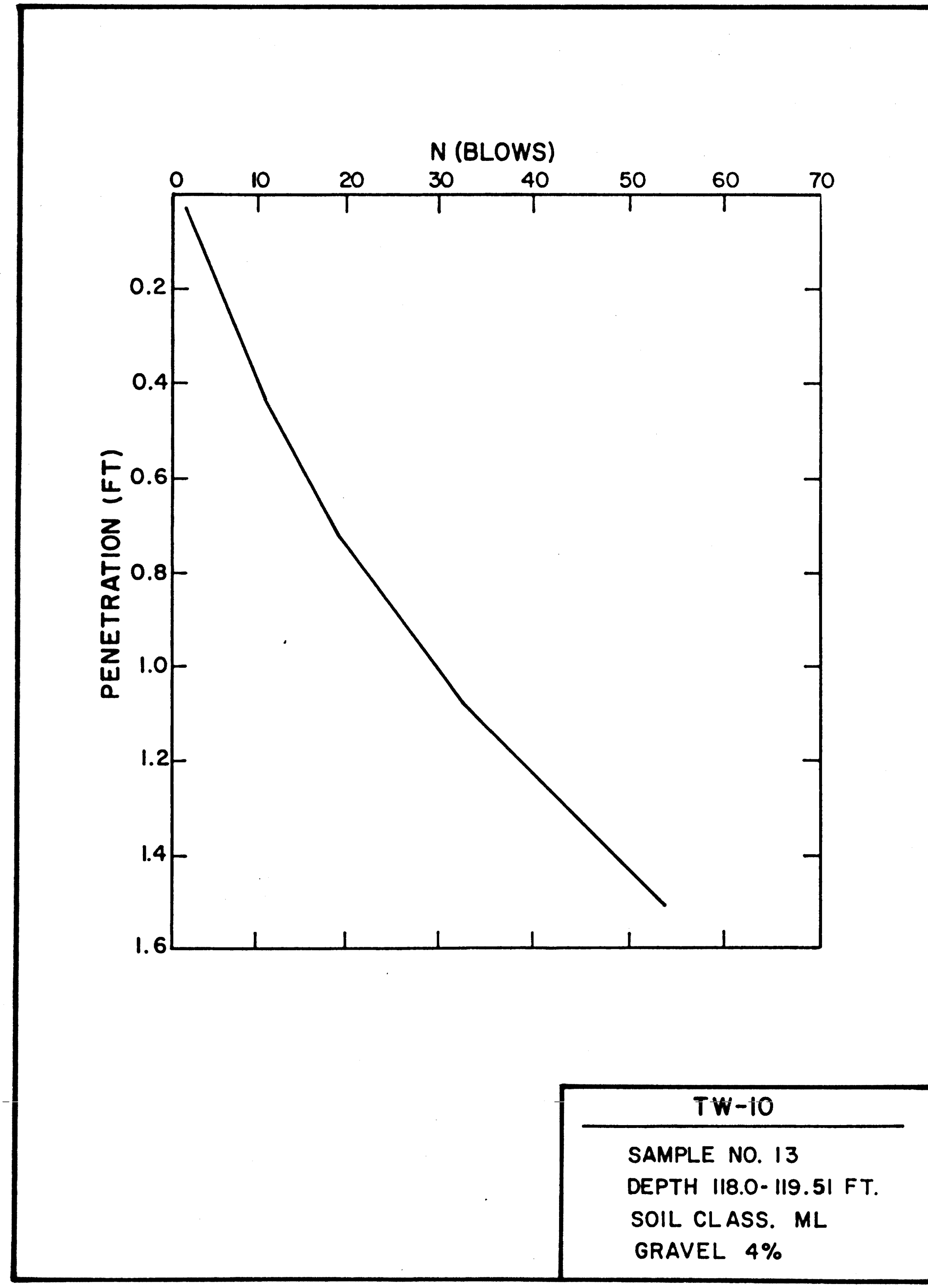


Figure 29. Standard Penetration Test at Twin Lakes Dam, Colorado, Requiring No Correction for the Effect of Gravel (from Kiene, 1983).

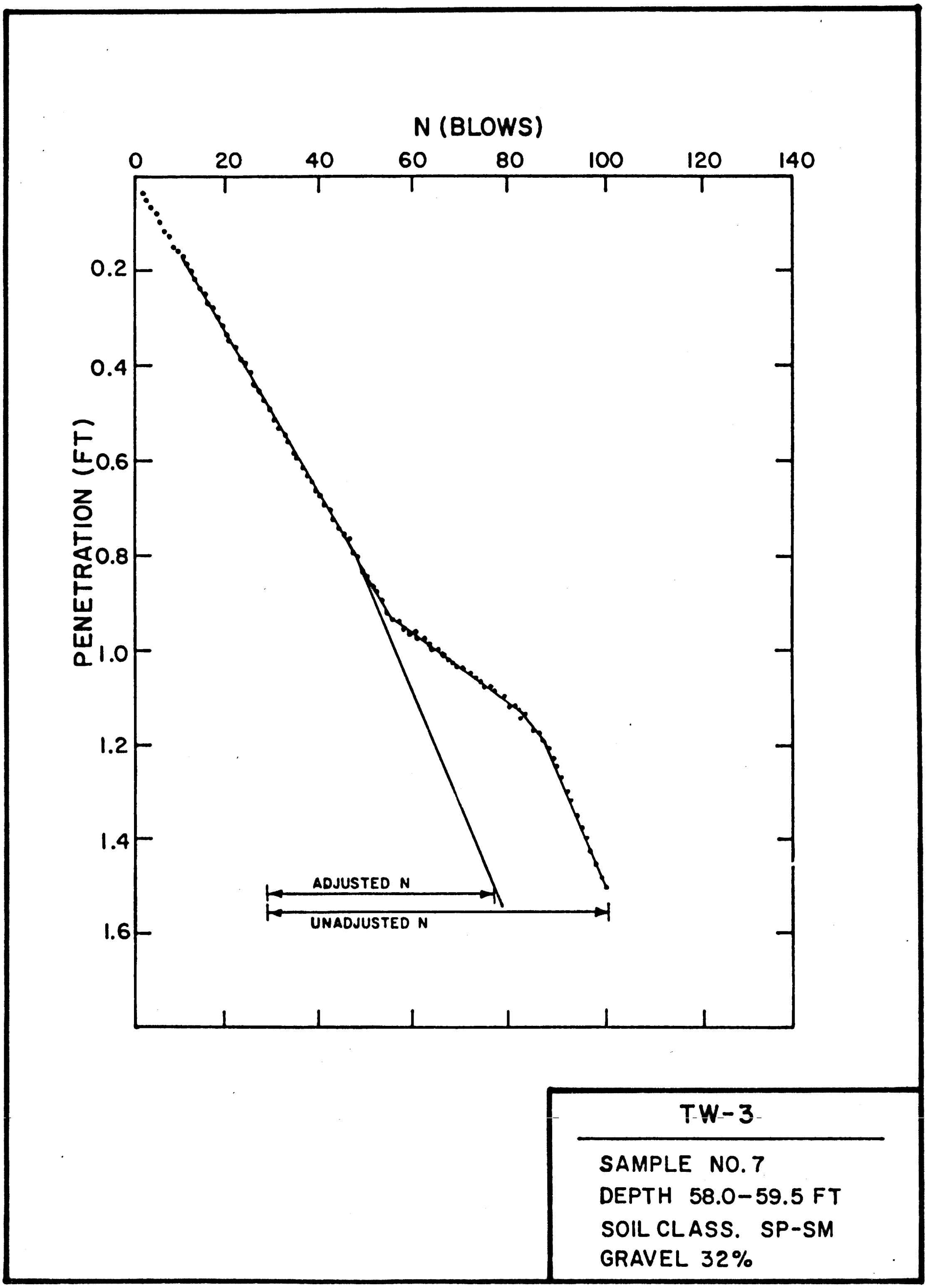


Figure 30. Standard Penetration Test at Twin Lakes Dam, Colorado, Corrected for Gravel Effects (from Kiene, 1983).

used. Calculations were made for assumed earthquake conditions of $M_L = 6.0$ and 6.75 and maximum accelerations of 0.21 and 0.35 g, respectively. Results indicated that 24 of the samples were potentially liquefiable during an earthquake with magnitude 6.75. Of these, three samples were found to be susceptible during the assumed magnitude 6.0 event.

The results of this subsurface exploration program demonstrated the highly variable nature of sediments beneath the Twin Lakes Dam. Few soil layers were identified as continuous between boreholes, with none of these being material with suspected liquefaction potential. Because of this lack of continuity between boreholes, the potential for liquefaction was judged to be confined to discontinuous layers with lengths less than 300 ft which could exist at several different depths.

Interpretation of Cutoff Trench Log.

Experience from the Twin Lakes Dam investigation demonstrates the importance of examining sediments exposed in dam construction excavations. During construction of Twin Lakes Dam, there was no particular concern with potential liquefaction problems. However, a photographic log was kept of the cutoff trench as construction of the dam progressed. In the subsequent liquefaction investigation, these records were referred to and provided invaluable information about the subsurface stratigraphy of the site.

Mapping of the cutoff trench from photographs was not as detailed as could have been done from direct observation. However, sedimentary units as thin as six inches or so were observable from the photographic log, and layers sufficiently thick to be a potential dam stability problem were easily identified.

An attempt was made to correlate zones of potential liquefaction identified from SPT testing with geologic units shown in the trench log photographs. Most of the susceptible units occurred below the level of the cutoff trench. However, SPT holes TW-1, TW-2, and TW-3 had intervals in common with the cutoff trench, allowing some comparison. Based on the limited amount of information available, two sand or silty sand layers were identified in the trench logs which corresponded to layers penetrated and sampled with the SPT. From the photographs, it was seen that neither of these layers was continuous, or extended for a significant length along the axis of the dam. This lack of length and continuity was used to argue that the liquefiable soil layers are isolated lenses of material in the foundation which do not have sufficient areal extent to cause a dam failure in the event of strong ground shaking.

During the initial foundation investigation for Twin Lakes Dam, pump tests were performed in the various geologic units to determine the permeabilities of the materials. Permeabilities from 0 to 800,000 ft/yr were measured, a further indication of the highly variable foundation conditions. An examination of the photographic trench log showed that zones identified as liquefiable were surrounded by materials with permeabilities greater than 10,000 ft/yr. These surrounding deposits have sufficient drainage to preclude the buildup of pore pressures during earthquake shaking. Thus, they may provide a drained condition for other materials indicated to have a potential for liquefaction under undrained conditions.

Conclusions.

The Twin Lakes Dam liquefaction investigation illustrates problems encountered during exploration of certain glacial deposits. The coarse, highly-stratified material was not generally penetrable by SPT or similar sampling devices. Cone penetrometer testing was not attempted, but would likely have met with little success. Although large amounts of cobbles and boulders rendered such 'standard' techniques practically useless, it was still necessary to delineate the subsurface stratigraphy and evaluate liquefaction potential at the site.

Investigations to depths as great as 170 feet were performed by rotary drilling, sampling of cuttings, and geophysical logging techniques. The logs provided a rough profile for each borehole, and coarse identification of loose sandy layers was possible. Zones in which SPT sampling could possibly be performed were selected from the geophysical logs. These tests provided some SPT data and samples for identification of soil type.

Photographic logs from construction of the cutoff trench provided the most detailed delineation of subsurface sediment. This excavation was relatively shallow, with a maximum depth of 50 feet. Comparison of borehole logs, samples, and the trench log enabled correlation of units and extrapolation of subsurface mapping to greater depths and provided sufficient information for the liquefaction study.

Results of subsurface investigation at Twin Lakes led to the conclusion that only scattered discontinuous lenses of material in the foundation are potentially liquefiable and these lenses are generally sandwiched between very permeable layers. Because of the apparent limited amount of liquefiable material in the dam foundation and the occurrence of an earthquake with sufficient magnitude to cause liquefaction is extremely remote, the investigators recommended that further investigation and/or remedial action at Twin Lakes Dam was not warranted.

LIQUEFACTION SITES, IMPERIAL VALLEY, CALIFORNIA

Description and Scope of Investigations.

An area which has a history of major seismic events is the Imperial Valley in Southern California. Notable recent seismicity in the region were the El Centro (1940), Imperial Valley (1979) and Westmorland (1981) earthquakes. Each of these earthquakes produced widespread liquefaction effects such as sand boils, ground cracks, and lateral spreads. Nearly all of this liquefaction occurred in gravel-free sands and silts. Damage caused by liquefaction during the 1979 event included differential settlements and fissures, fractured tile drains in fields, disrupted pavement on roads, and fractured linings and slumped banks along canals and drains (Youd and Wieczorek, 1982).

A map of the area affected by the October 15, 1979 earthquake (M_L = 6.6), with pertinent geographic and geomorphic features, is shown in figure 31. Two sites where liquefaction effects were particularly pronounced were selected for subsurface investigation by the U.S. Geological Survey. At the first site, a lateral spread disrupted Heber Road, adjacent fields, and a canal (Heber Road site); at the second site, hundreds of sand boils erupted on a park and rodeo ground (River Park site).

Investigations conducted by the U.S.G.S. during December 1979, January 1981, and May 1982, are described in Youd and Bennett (1983) and Bennett et al. (1981). The primary means of profiling the sediment section and liquefiable sediments at these sites was CPT, supplemented with standard penetration tests. Piezocone soundings were taken at the River Park and Heber Road sites by the Corps of Engineers (Norton, 1983). In addition to these tests, cross-hole and seismic refraction tests were conducted at the Heber Road Site

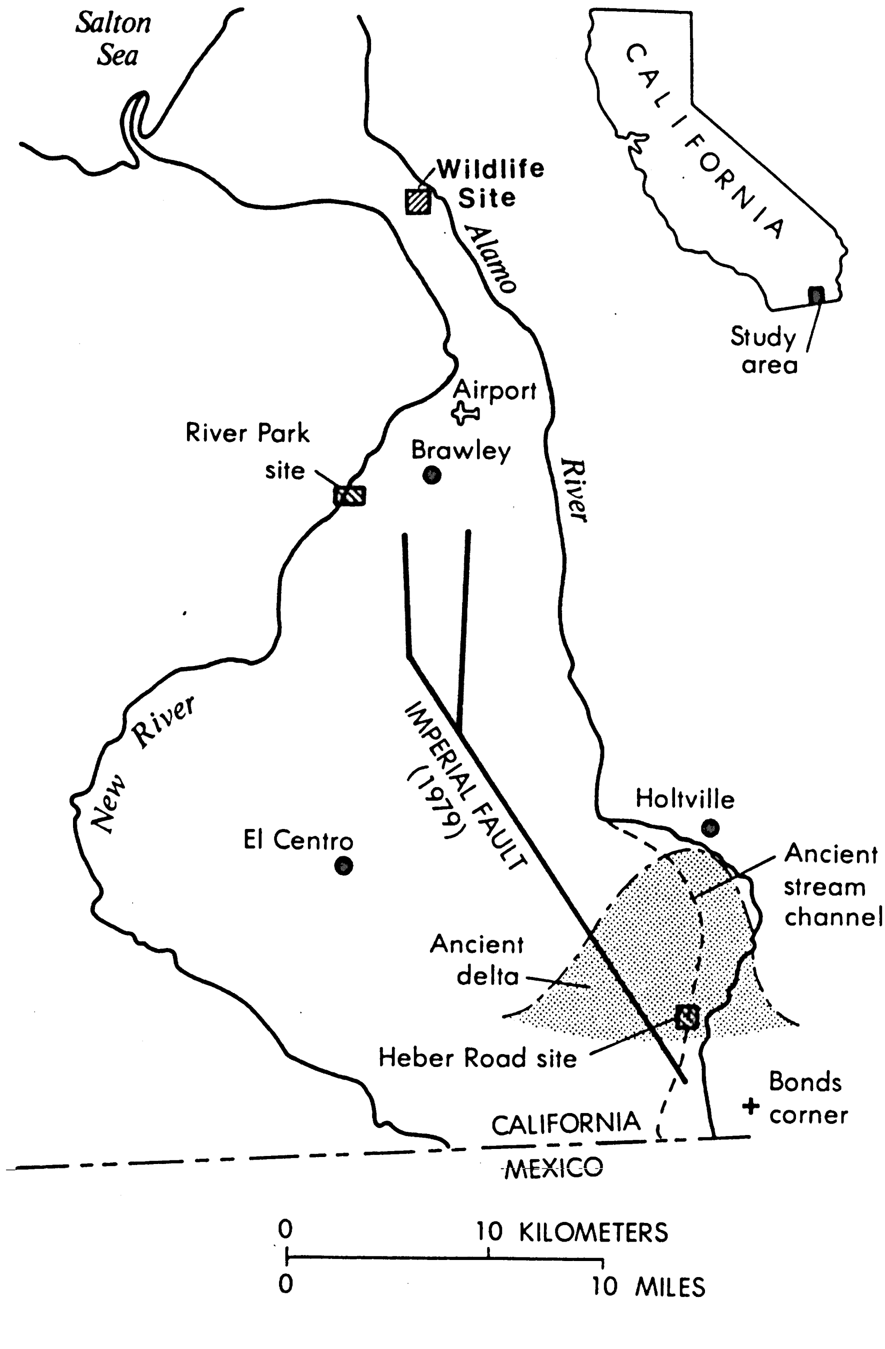


Figure 31. Map of Southern Imperial Valley Showing Pertinent Geographic and Geomorphic Features and Locations of Heber Road, River Park, and Wildlife Sites (from Youd and Bennett, 1983).

by University of Texas, Austin, personnel. Results of this seismic testing program are summarized by Kuo and Stokoe (1982).

On April 26, 1981, an earthquake of magnitude 6.0 occurred near Westmorland, California. Though no surface faulting was evident, the earthquake damaged buildings and canals and generated several ground failures. Four sites (Wildlife, Vail Canal, Kornbloom Road, and Radio Tower) of many which experienced liquefaction during the 1981 Westmorland earthquake have been the subject of extensive subsurface investigations in the past few years (Bennett et al. 1984). In anticipation of future seismic events, the Wildlife site (figure 31) has been instrumented with seismometers, and piezometers installed in the liquefiable sediments.

In May of 1982, U.S.G.S. personnel conducted investigations of the above sites using SPT and cone penetrometer soundings. Results of these tests are summarized by Bennett et al. (1984). Seismic cross-hole and SASW testing were conducted by the University of Texas, Austin (Bierschwale and Stokoe, 1984). Purdue and Stanford Universities and the University of British Columbia have also conducted subsurface investigations of these liquefaction sites.

The importance of the Imperial Valley investigations to this study is the variety of testing techniques which were employed in the field. Comparison of information obtained by the different methods provides an indication of the relative usefulness of each and their applicability in liquefaction site studies.

River Park Site.

The River Park site lies in the floodplain of the New River (figure 31), and the sediments were deposited by a range of fluvial regimes. The stratigraphy beneath the site was readily determined from a line of CPT soundings, shown in figure 32. Mechanical cone tip resistances are presented here. Soil type and additional penetration data were obtained from SPT. The lowermost unit (C) is a medium dense fine sand typical of a point bar deposit. The cone logs revealed that a 1- to 3-ft thick layer of loose sand lies at the top of unit C. This layer was identified as one that liquefied during the 1979 earthquake (Youd and Bennett, 1983). The loose layer most likely resulted from liquefaction and resedimentation actions caused by the 1979 and prior earthquakes in the region (Youd, 1984). The middle unit (B) is an unliquefiable silty clay typical of a backswamp deposit, and the uppermost layer (A) is a very loose floodplain deposit of silty sand. Parts of layer A also liquefied during the 1979 earthquake.

The mechanical CPT was able to detect all soil layers of importance to the liquefaction investigation at River Park. However, demarcation of the interface between the clay unit (B) and the zone of loose sand (C), can only be with precision of about 8 in to one ft using the MCPT logs (figure 32). Use of the piezocone (PQS, according to Norton's terminology) soundings aids in accurate location of the clay/sand interface. Figure 33 is a PQS log, taken near test site 4 of figure 32. The sharp break in the u/q log (figure 33) gives the best indication of the change in materials, where u is the pore water pressure measured during penetration of the cone and q is the point resistance. By plotting the digitized data points from the PQS logs on a larger scale, the depth of the interface at this location could be discerned to within one to two inches.

Piezocone data added some precision but little new information for delineation of soil stratigraphy or assessment of liquefaction susceptibility at River Park compared to ECPT logs alone. For example, Norton was unable to discriminate between loose and dense sands on the basis of dilative versus

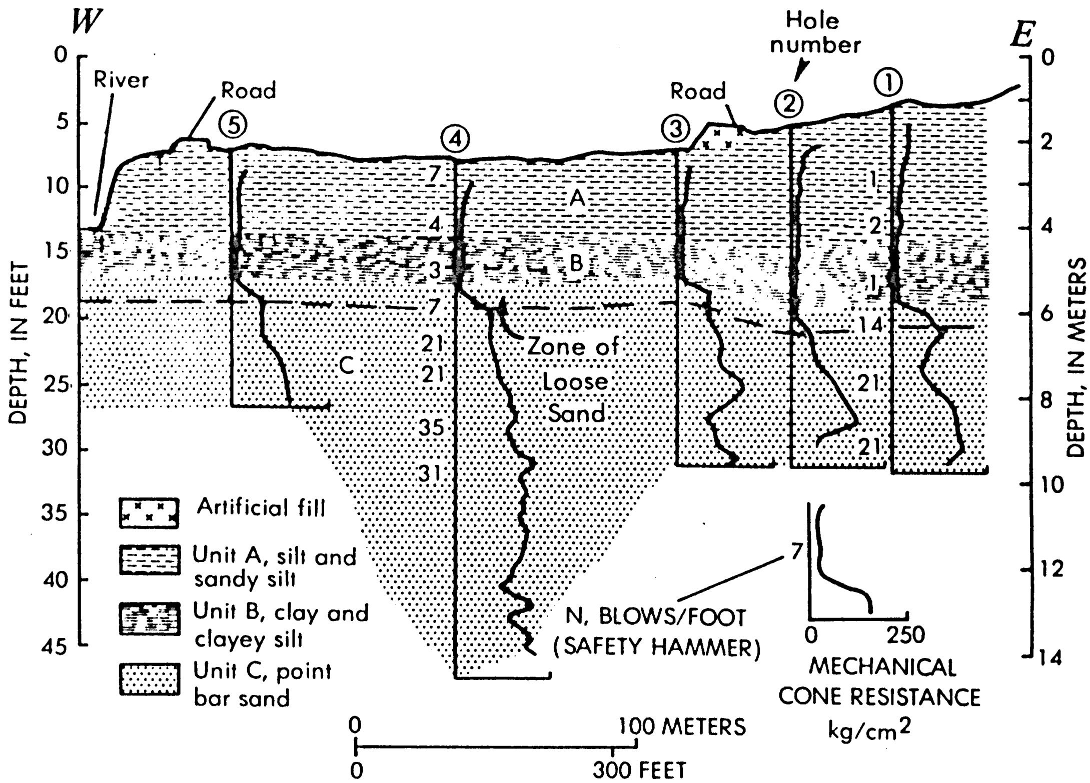


Figure 32. **Cross Section of Sediments beneath Flood Plain at River Park Site, California (from Youd and Bennett, 1983).**

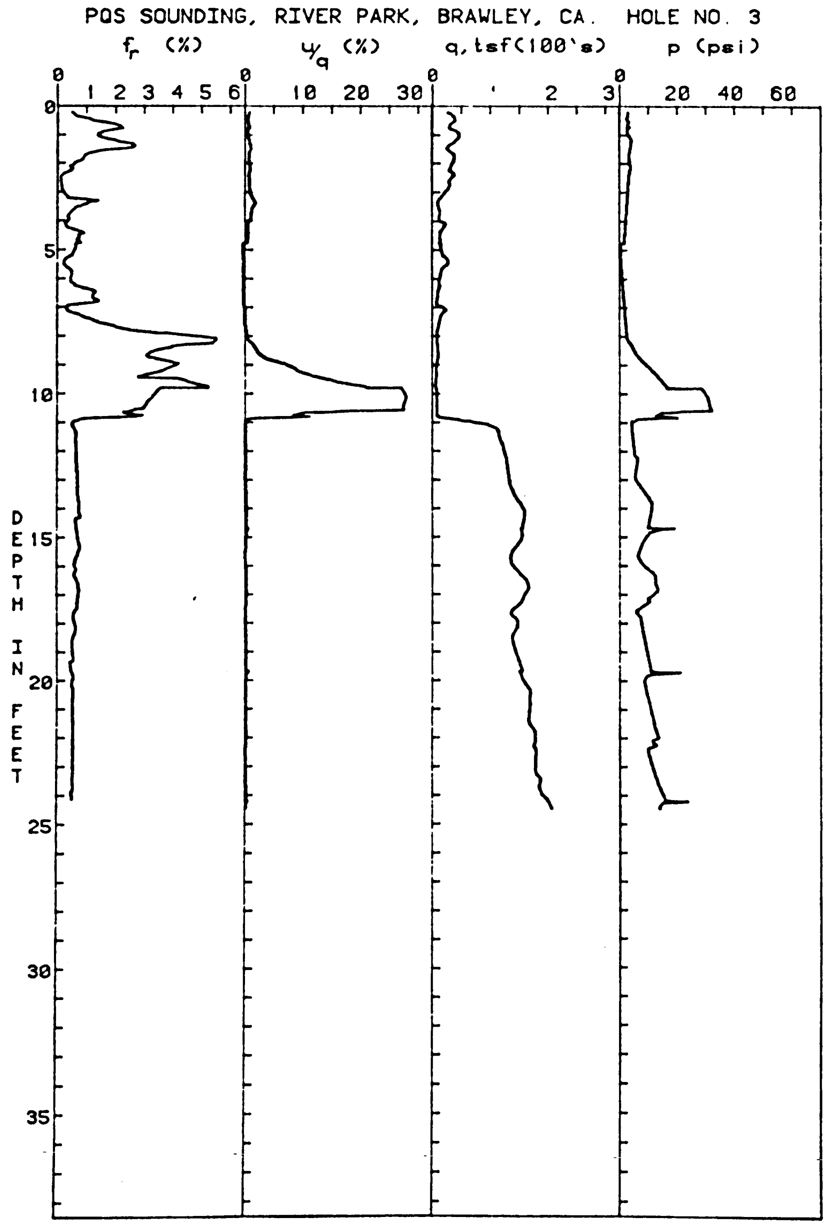


Figure 33. Piezocone (PQS) Sounding from Corps of Engineers Investigation at River Park Site (from Norton, 1983).

contractive pore pressure response. Pore water pressures greater than hydrostatic were measured in both loose and dense sands during penetration with the PQS probe. This positive pore pressure response was apparently caused by a compressive state of stress in the soil at the cone tip such that even dense sands exhibited contractive behavior during probe advancement (Norton, 1983).

Heber Road Site.

Figure 34 shows the Heber Road area and surficial evidence of liquefaction at the site. Liquefaction was confined to sediments deposited by a prehistoric stream, which formerly coursed through this area. From time to time in recent geologic history, the area has been flooded by a large lake. Because of this sporadic inundation, the fluvial sediments of the Heber Road site are interbedded with lacustrine clays and silts.

A line of electronic CPT soundings was taken along the south edge of Heber Road. Figure 12 shows three electrical CPT logs used in delineating Heber Road stratigraphy, with accompanying SPT blowcounts. A point bar deposit (A1), a loose channel fill deposit (A2), and a levee or overbank unit (A3) were identified from the penetration resistances.

In addition to the ECPT soundings, eight mechanical CPT tests were performed at Heber Road. A comparison of profiles determined by mechanical (figure 35) and electronic (figure 12) cone penetrometer shows the superiority of ECPT in delineating subsurface stratigraphy. Though tip and friction resistances recorded on MCPT logs allow identification of different sedimentary units, the ECPT provides more detailed stratigraphic information and a better indication of soil type. Resolution of thin layers and layer boundaries is also more precise with the ECPT. Layers as thin as six inches

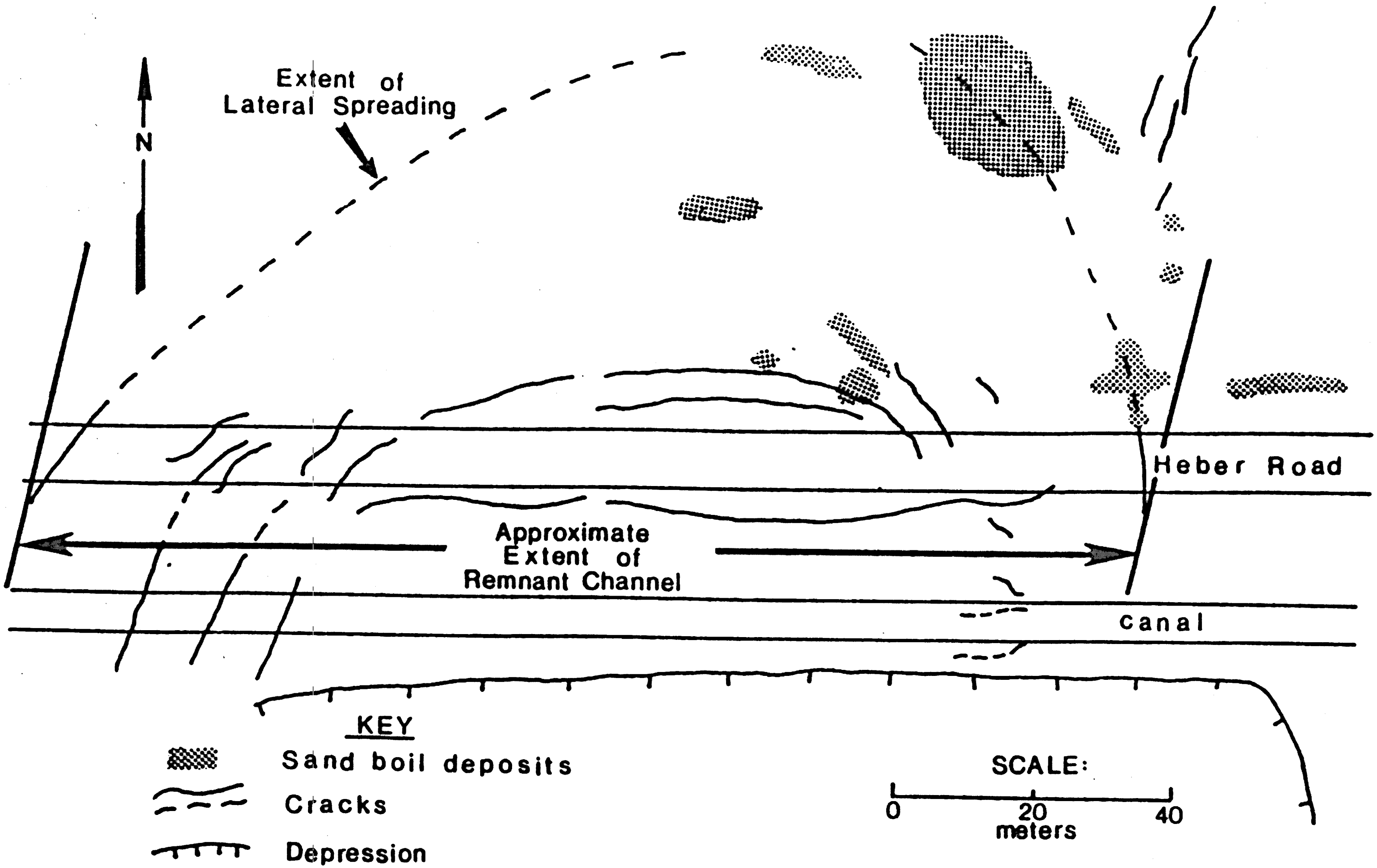


Figure 34. Location of Various Earthquake-Induced Features along Heber Road, North of Heber Dunes State Park, California (from Youd and Wieczorek, 1982).

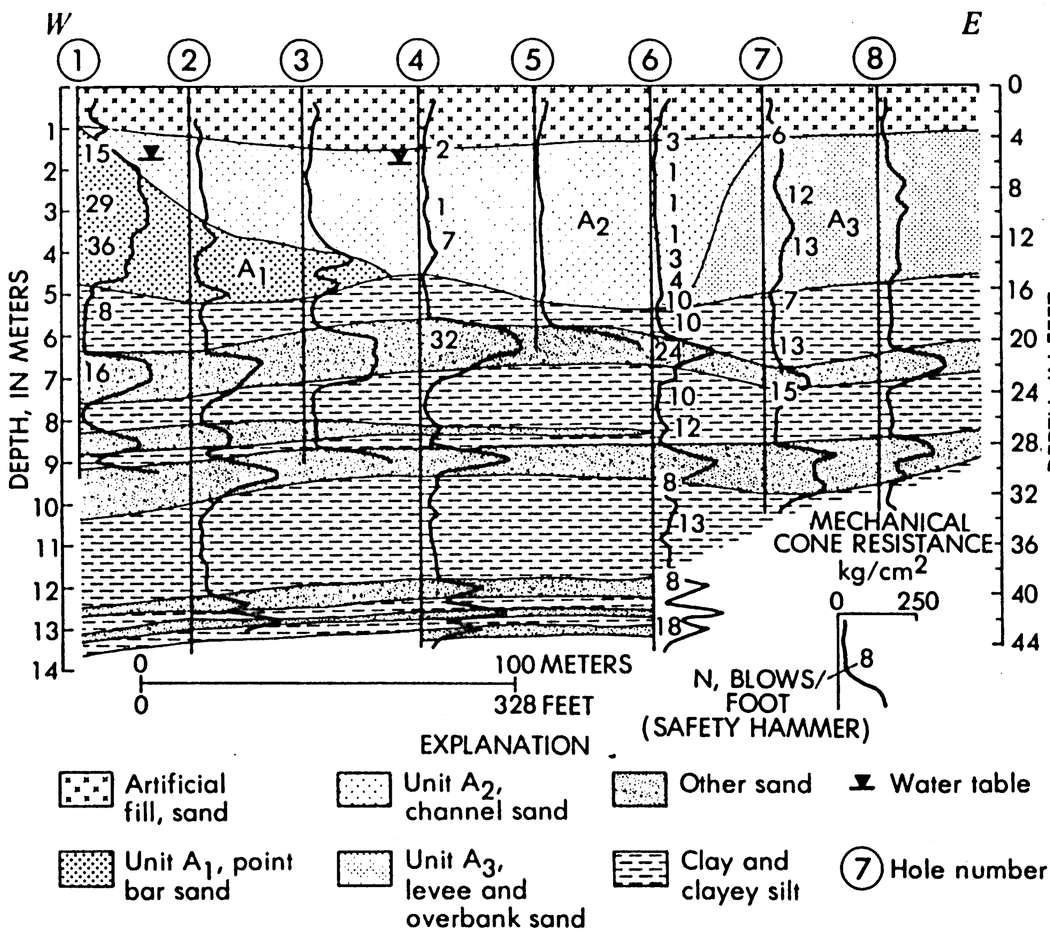


Figure 35. Logs from Heber Road Site Showing Mechanical CPT Records and Safety-Hammer N-Values (from Youd and Bennett, 1983).

are identifiable from the electric cone soundings shown here, and resolution to about 3 or 4 inches could be possible if the data were plotted at a larger scale. One-foot is about the minimum thickness of layer identifiable from the mechanical cone logs.

University of Texas researchers conducted shear wave velocity tests at three localities at the Heber Road site, one each on the point bar, channel fill and overbank deposits, respectively. Both cross-hole and spectral analysis of surface wave (SASW) tests were used. Composite profiles of the point bar, channel fill and overbank sites are given in Figures 36, 37 and 38, respectively, along with the measured shear-wave velocity data. For each site, the shear wave velocity profile has roughly the same shape as the cone penetrometer tip resistance profile, but the layers are much less distinct.

The data from Heber Road show that the subsurface profile delineated by CPT is much more detailed than that obtained by other methods. Six-inch layers (and possibly thinner) are definable by CPT. Standard penetration tests were conducted intermittently, but even if they had been taken continuously, it is doubtful that a layer less than about two feet thick would have been discernible from SPT without examining the samples. Changes in soil conditions were detected by the seismic techniques, but the layers are not well delineated. Layers less than about three feet thick are not readily definable in the shear-wave velocity profiles.

In the Heber Road investigation, locations of SPT samples were selected from preliminary CPT logs, thus optimizing the information obtained from the sampling program. This optimization allowed characterization of the standard penetration of the sediment units with a minimal number of tests. Conversely, these few SPT tests greatly improved the cross-section determined from the CPT profiles by providing samples from which sediment types could be positively

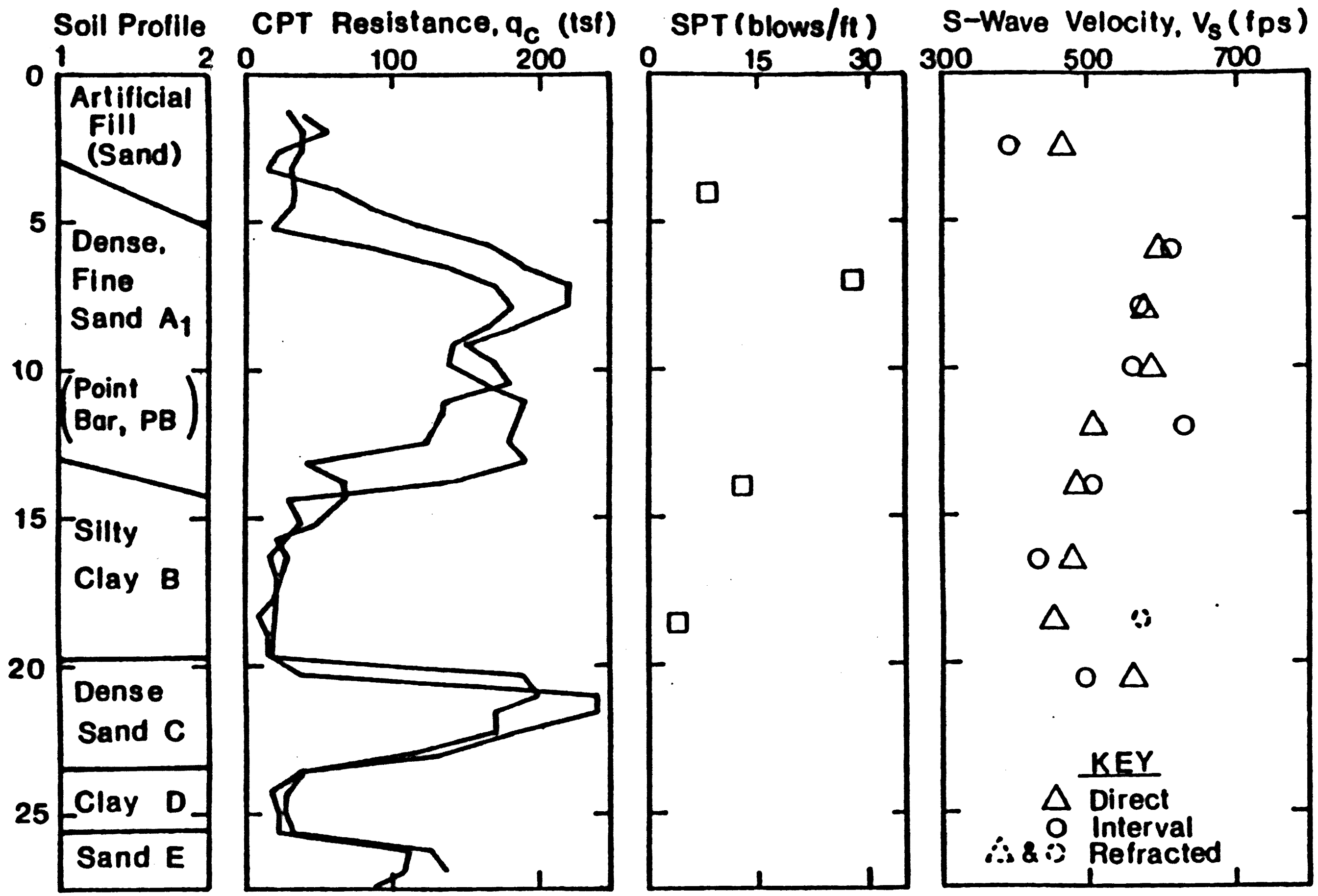


Figure 36. Composite Profile at Point Bar Site (from Sykora and Stokoe, 1982).

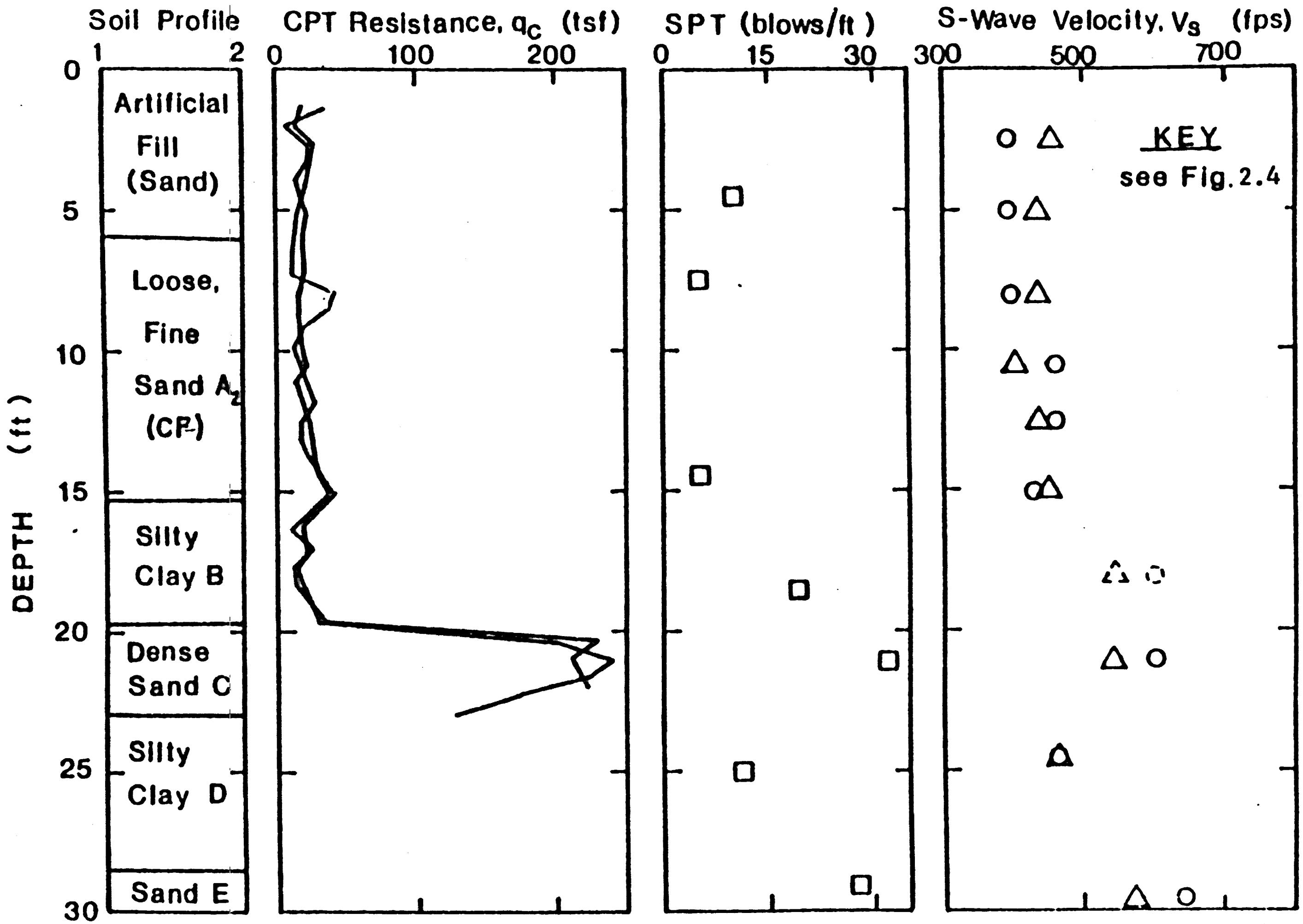


Figure 37. Composite Profile of Channel Fill Site (from Sykora and Stokoe, 1982).

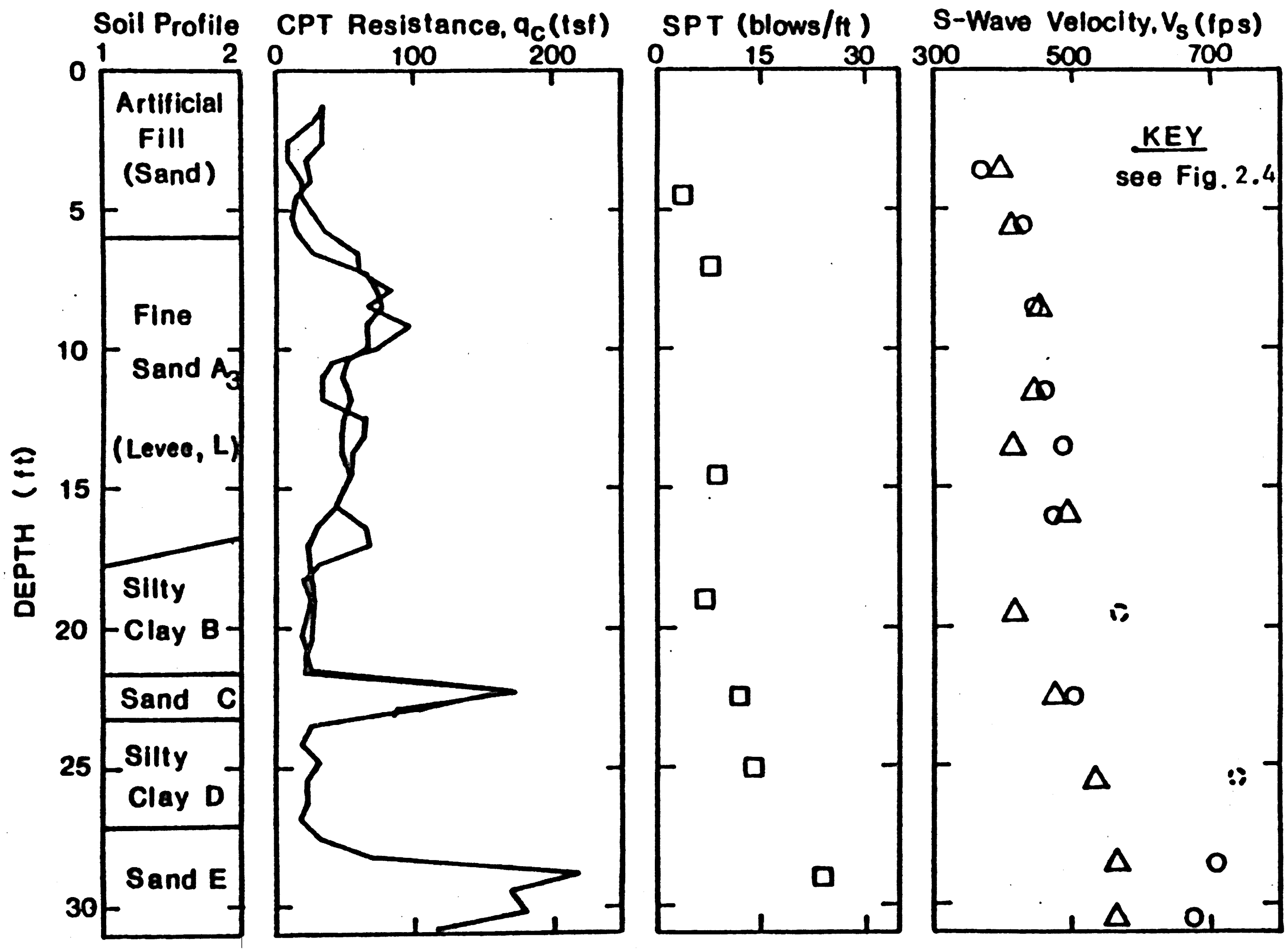


Figure 38. Composite Profile of Levee Site (from Sykora and Stokoe, 1982).

identified. The CPT profiles also greatly aided the interpretation of the shear wave velocity data by delineating soil layers which then could be characterized in terms of shear-wave velocities measured in those layers.

It should be noted that subsurface exploration of liquefaction sites may be performed for the determination of soil layering or for quantifying soil properties. The cone penetrometer in this instance is best in delineation of soil layers. Soil type and N-values are better-defined by SPT. Shear-wave techniques are very useful in measuring soil modulus. All of these tests may be needed at a site, but for different purposes.

Wildlife Site.

In the Imperial Wildlife Management Area (Wildlife site, figure 39) numerous sand boils erupted and several stream banks slumped along the Alamo River as a result of the 1981 earthquake. Sand boils and fissures extended as far as 300 ft from the river but were more concentrated near the stream. Segments of the bank as wide as 16 ft collapsed into the river at several places (Youd and Wieczorek, 1984). In addition to the field testing, this site has been instrumented with strong ground motion sensors and piezometers to measure ground response during future earthquakes.

The geology at Wildlife is dominated by recent fluvial processes, as the site lies on the incised Alamo River flood plain. Figure 39 shows a cross-section of the upper sediments, determined from CPT and continuous auger samples. Location and types of instrumentation at the site are also shown in the drawing. The stratigraphic units were delineated by Bennett et al. (1984) are as follows:

From the ground surface to a depth of 9 ft the sediment consists of interbedded sandy silt, silt, and clayey silt. Cone penetrometer and SPT data

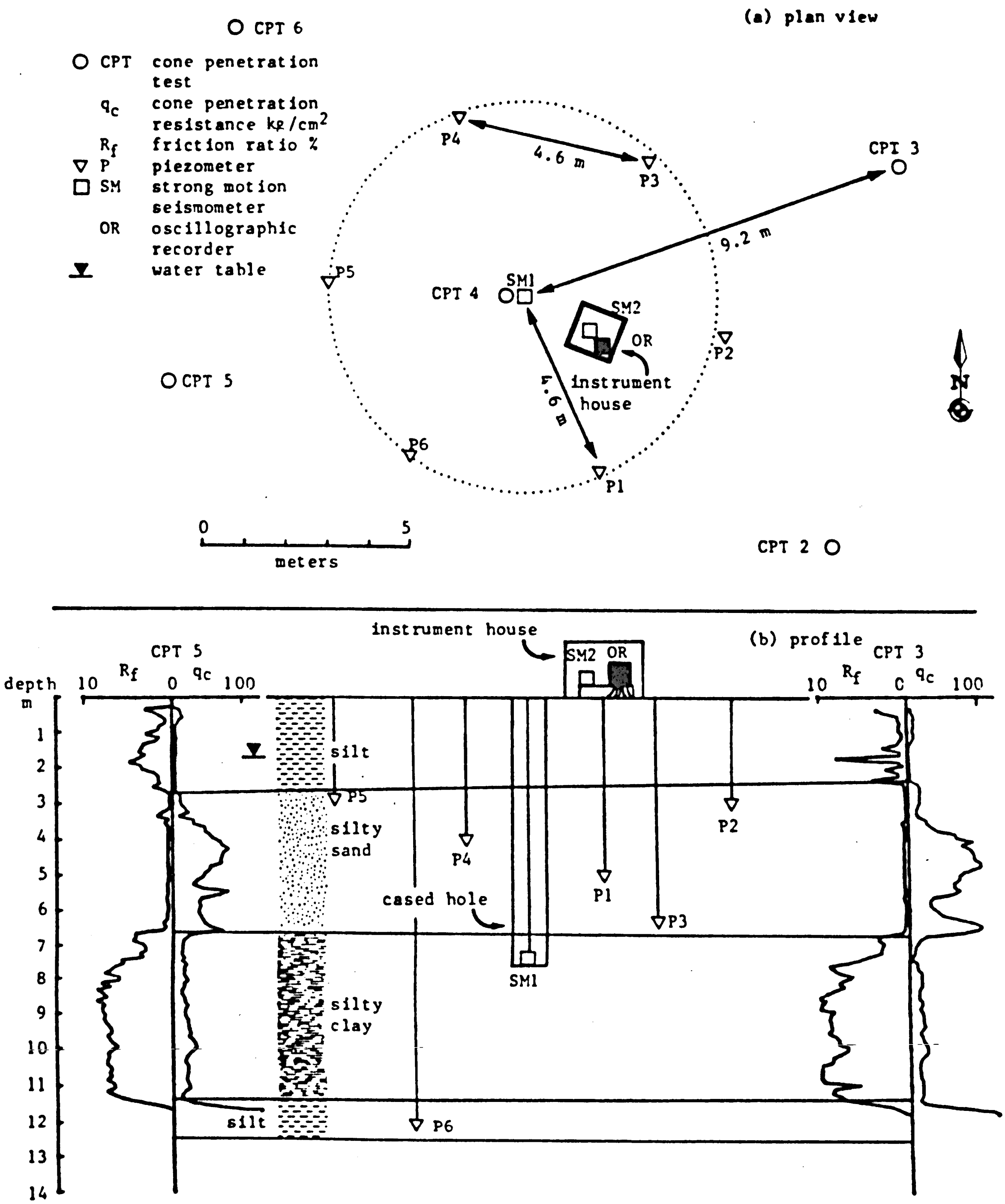


Figure 39. Cross Section of Wildlife Site Showing Electrical CPT Logs and Location of Piezometers (from Bennett et al., 1984).

indicate that material is in a very loose or soft condition. This unit is a flood-plain deposit, resulting from over-bank deposition. The coarsest sediment, sandy silt, lies near the bottom of the unit. The materials generally become finer with distance from the river.

From 9 to 23 ft, the sediment consists of very loose to medium dense sands and silty sands. The upper part of this unit is a moderately sorted sandy silt. Gradation becomes coarser with depth, with the coarsest sediment (a medium sand) occurring at the base of the unit as a lag deposit. Bennett et al. (1984) identified this material as a point-bar deposit. Based on the looseness of sediment, the saturated condition, and similarity to sand boil material, the upper part of the cohesionless unit was identified as the layer that liquefied during the 1981 earthquake.

A silty clay layer lies beneath the sand in the depth interval from 6.8 to 12 m. Bennett et al. identified the origin of that layer as lacustrine. Beneath the silty clay lies a unit of layered silt which is too dense to liquefy.

Tests conducted at the Wildlife site include CPT, SPT, and down-hole, cross-hole, and SASW shear-wave velocity measurements. In addition, a continuous soil profile was extracted by auger sampling. To extract the latter samples, solid stem augers were screwed into the ground at a penetration and rotation rate to minimize displacement of soil up the auger flight. The auger was then lifted to the surface without rotation to retrieve the continuous sample. The samples were extracted in either 5 ft or 10 ft increments. Youd and Bennett (1983) have further described this technique. They also conclude that most nongravelly materials can be continuously sampled with this technique, but poor recovery has been attained in some loose silty sands, materials of most importance to many liquefaction investigations. Youd and Bennett also conclude that the larger layers were adequately preserved to identify most soil layers more than a few inches thick, but that sample disturbance was generally too severe for identification of small scale stratigraphic structures in cohesionless sediment.

A composite profile of the Wildlife site, showing CPT tip resistance and friction ratio, uncorrected SPT blowcounts, and Shear Wave Velocity from SASW, is shown in figure 40. As at other sites, details of the subsurface stratigraphy are more evident on the cone penetration logs. Again, layers of six inches or thinner were detected by the cone. The interbedded nature of the upper unit and the gradually increasing density and grain size of the point bar unit with depth are also reflected in the CPT log. These features are evident to a lesser degree in SPT and SASW profiles.

A comparison of the Wildlife site profile with figure 41 shows that SASW results gave greater detail of sedimentary structure than did shear wave velocity profiles from either downhole or cross hole techniques. Downhole and SASW profiles gave some indication of the loose sandy layer at about 10 foot depth, but do not clearly define the location or thickness of the layer. Cross-hole tests did not detect this loose sand layer.

Conclusions.

Depositional environments of Imperial Valley sediments were mostly low-energy fluvial or lacustrine, resulting in beds of sand, silt, and clay (or mixtures), with little gravel, conditions that were ideal for investigation by CPT and SPT, which were performed rapidly and economically.

Of the various techniques used in these investigations, CPT proved to be the most useful in defining the geometry of sediment units. The profiles developed from CPT data were completed in a much shorter time than could have

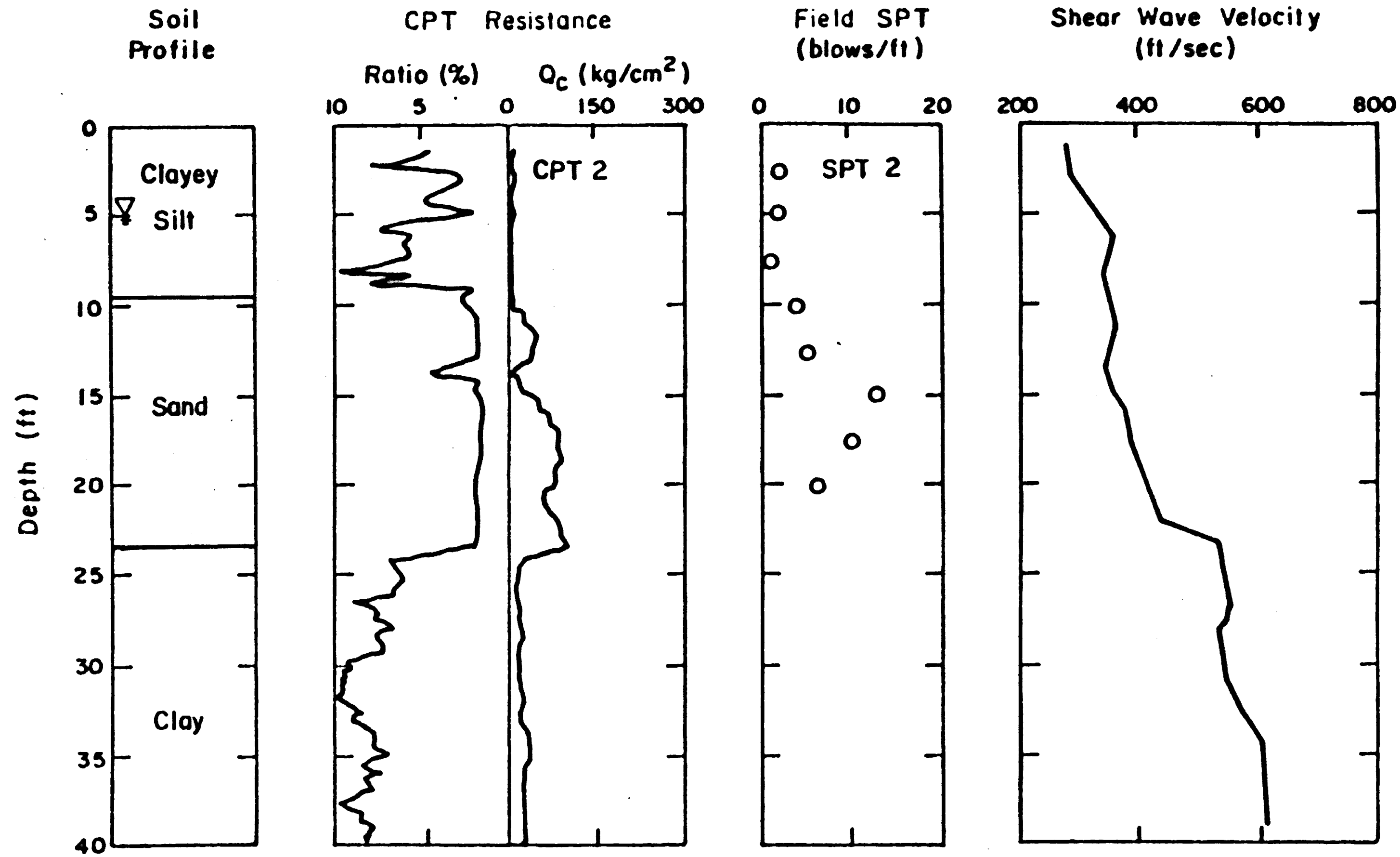


Figure 40. Composite Profile of Wildlife Site (from Bierschwale and Stokoc, 1984).

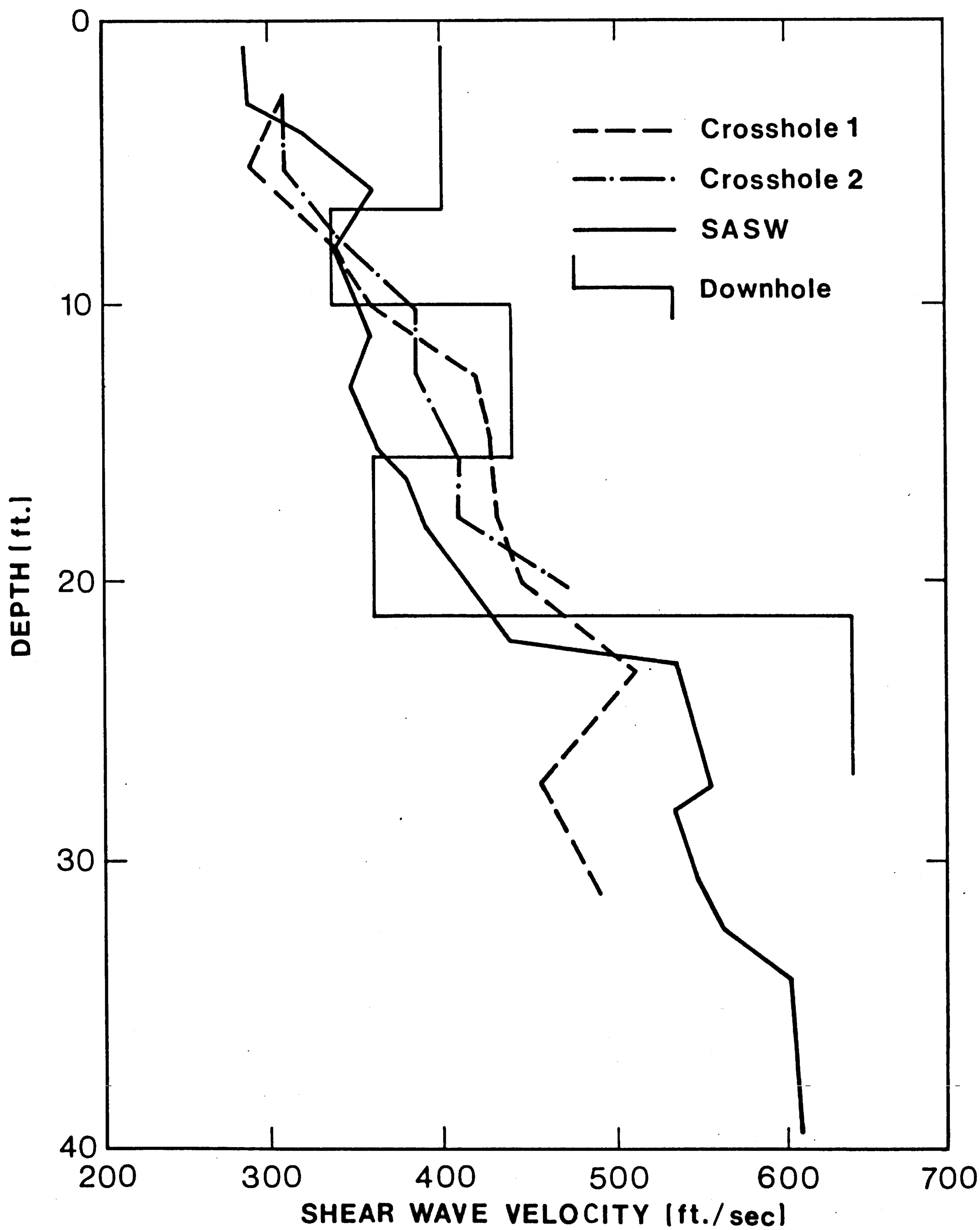


Figure 41. Comparison of Shear Wave Velocity Profiles from Seismic Crosshole, Downhole, and SASW Testing at the Wildlife Site (adapted from Stokoe and Nazarian, 1985).

been done by conventional drilling and sampling methods. Piston samples gave better resolution of sedimentary structure than the cone; 1/8-inch microstructures could be seen in the samples. The CPT probably left no layers thinner than six inches undetected, providing sufficient stratigraphic information for the liquefaction studies in Imperial Valley.

The disadvantage of no sample recovery with CPT was remedied by selective SPT, auger, and piston sampling, which aided in correlation of sedimentary units between CPT soundings. Blowcounts recorded from SPT provided additional stratigraphic and liquefaction susceptibility information.

The worth of seismic techniques such as cross-hole, refraction, and SASW was demonstrated by the Imperial Valley investigations. Fairly detailed stiffness information and some indication of soil liquefiability was obtained from these methods. However, soil layering was better-defined by SPT and the cone penetrometer. Some stratigraphic features which were plainly evident in CPT logs were not identifiable from shear-wave velocity plots.

CALIFORNIA SCHOOL FOR THE BLIND, FREMONT, CALIFORNIA

Description of Investigation.

The California School for the Blind (CSB) occupies a site of approximately 40 acres at the southeast corner of Walnut Avenue and Gallaudet Drive in Fremont, California. The site is located close to the foothills of the Diablo Range on an alluvial plain between the Diablo Range and San Francisco Bay. It was improved in the late 1970's by construction of the CSB campus, several one-story classroom, recreational, and residential structures. The property was previously unimproved and was in agricultural use.

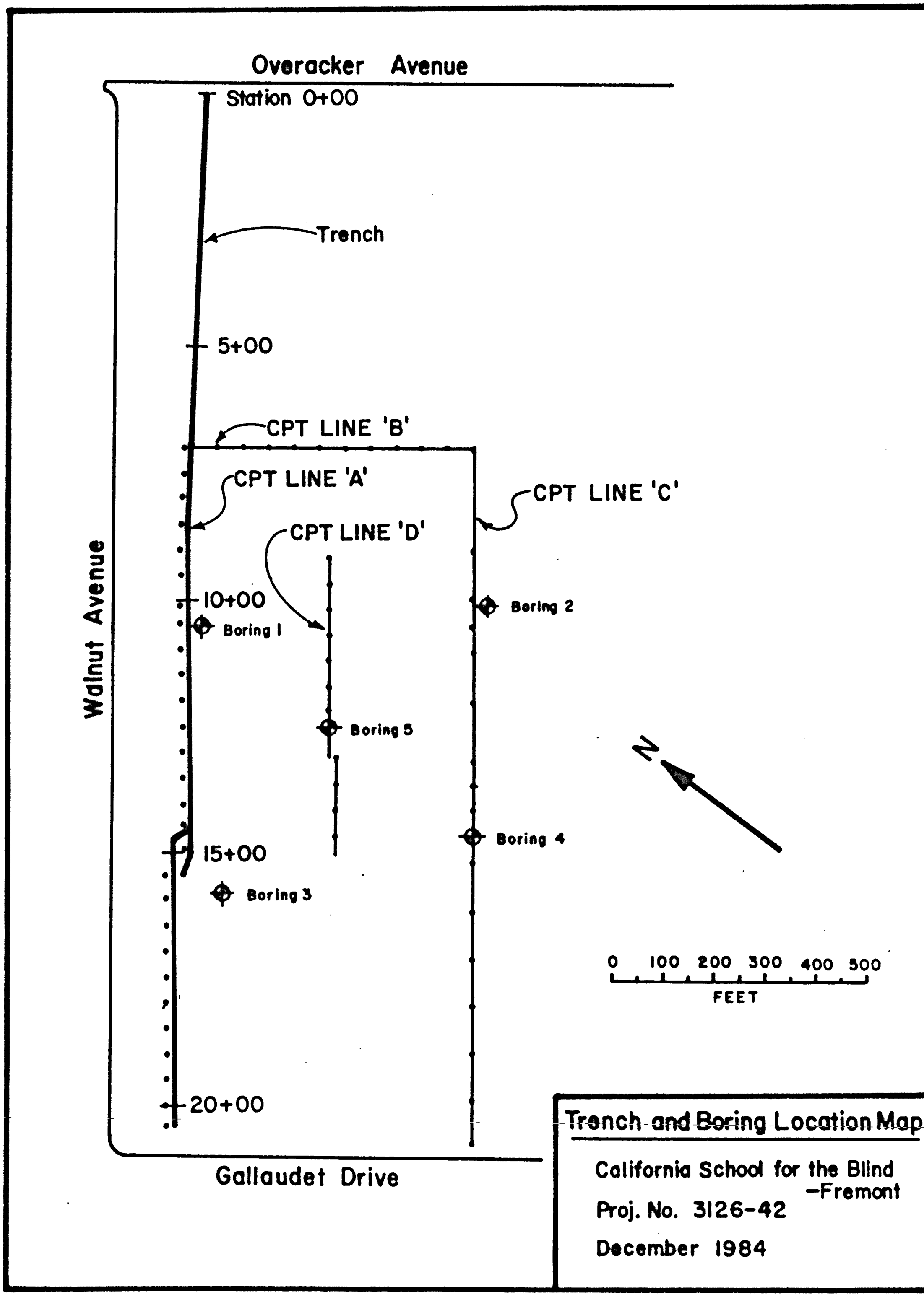


Figure 42. Map of California School for the Blind, Fremont, Showing Location of Trench, Borings, and CPT Soundings (adapted from Lockwood-Singh and Associates, 1984).

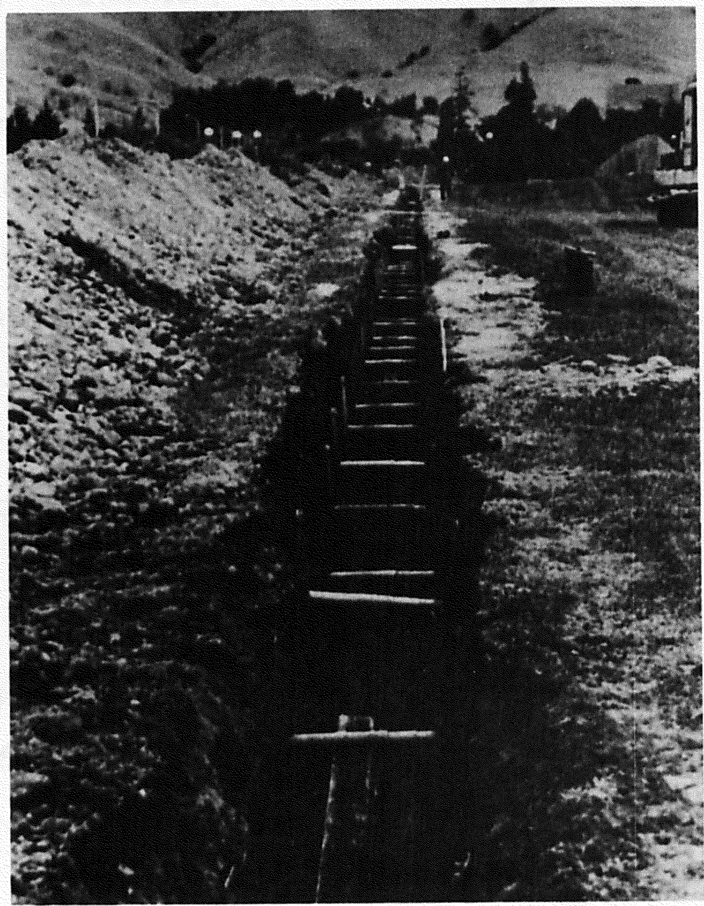


Figure 43. Typical View of Exploratory Trench at California School for the Blind, with Shoring and Board Walkway in place. Note Large Cobbles in Spoil Pile from Bottom of Trench.

Because of the possibility of damage to the school from soil liquefaction or fault rupture during an earthquake, the California State Architect commissioned a study of the problem in 1984. Lockwood-Singh & Associates conducted the geotechnical site investigation and reported their findings in an unpublished report (December, 1984). Figure 42 shows locations of the various tests conducted. A trench approximately 18 ft deep and 2000 feet long was excavated and logged to search for evidence of fault rupture or disturbance by liquefaction effects. The trench log was also used to locate zones of potentially liquefiable material. In addition to the trench, five wash borings and 75 CPT soundings were conducted at the CSB. Results from these tests were used to prepare a detailed delineation of the subsurface stratigraphy at the site. Conjunctive use of trenching, conventional drilling with SPT, and CPT proved an economical and satisfactory means of mapping these soil deposits and estimating their stability during earthquake shaking.

Fault Investigation by Trenching.

The primary objective of the exploratory trenching investigation was to determine whether an inferred "Harding fault" or a branch or splay of the active Hayward fault crosses the California School for the Blind. Also, the trench log was to help delineate potentially liquefiable materials underlying the site and to evaluate any remnant effects of past liquefaction, such as sand boils or dikes, that might be encountered. An additional objective of the trenching program was to correlate sediments exposed in the trench with cone penetrometer logs in order to fully delineate key sedimentary strata underlying the site.

Figure 43 shows a typical section of the shored trench, with a plank walkway in place eight feet below ground surface. From the trenching effort, three primary sediment units of concern to the CSB study were identified. These sediments are overbank deposits, stream-channel sands, and alluvial or channel gravels. The overbank deposits, composed generally of silty sand with 9 to 17% clay, form a mantle of sediment to a depth of 14 - 18 feet. Within the overbank deposits there are lenses of sand or silty sand with some gravel particles but no appreciable fines content. These lenses are alluvial deposits and generally lie near the base of the overbank deposit. Alluvial gravels of late Pleistocene age underlie the overbank deposits to depths greater than 50 feet.

Very accurate profiling of sediments was accomplished by trench logging. Sediment layers as thin as a fraction of an inch and relative displacements along lithologic facies of an inch or less were detectable. The trench log was used to prove that no disruption to the bedding exposed in the trench had occurred since deposition either as a result of faulting or liquefaction. The thickness and extent of all cohesionless sand silt layers more than a few inches thick were also accurately delineated on the trench log and samples of the material taken for soil classification tests and density determination.

Investigations by CPT and Rotary Drilling.

In addition to the exploratory trench, five rotary wash borings and 75 CPT soundings were performed at the CSB site. The purpose of these investigations was to further delineate and characterize the sedimentary units beneath the property. In particular, it was desired to identify the areal extent and thickness of zones susceptible to liquefaction and to evaluate the probable consequences of liquefaction in these sediments. Sixty-six SPT samples were taken at various elevations in the rotary borings, and recorded blowcounts were applied to an empirical evaluation of liquefaction potential.

A total of four linear arrays of CPT soundings were laid out in the field (figure 42), with test spacing selected on the basis of depositional environment. Cone soundings were conducted 50 feet apart across the direction of fluvial deposition. It was reasoned that no stream channel deposit of significant thickness would remain undetected with this spacing.

In conducting the CPT's, procedures and equipment consistent with the requirements of ASTM D-3441, except for the cone size, were employed. In order to penetrate the dense gravels underlying the site, the more rugged cone tip having a cross-sectional area of 15 cm^2 was used. As a direct benefit of using the larger cone, test penetrations in excess of 30 feet were achieved at most locations. Approximately 20 CPT's were advanced close to the maximum desired test penetration of 50 feet.

The detailed, continuous stratigraphic information obtained from the cone penetrometer allowed a direct correlation between CPT soundings and trench logs. To facilitate this correlation, a series of CPT's were conducted along the trench between stations 7+00 and 20+45. These soundings were conducted before excavation began, and the trench was deliberately located to permit exposure of a number of CPT holes in the trench wall. Excellent correlation was found between the CPT and trench logs. Where the CPT holes passed from slightly clayey soils to cleaner sands and gravels, it was observed that a thin clay film was carried several inches into the coarser sediments by the cone apparatus. This downward transfer of clay appeared to cause the CPT to record a less distinct contact than that which was observed in the trench. Otherwise, resolution of the CPT data was within a few inches.

A photograph of CPT 24, exposed in the trench wall at station 18+50, is shown in figure 44. The top contact of the alluvial gravel unit, about 13 feet below ground level, is directly beneath the stationing sign. Although no

Figure 44. Cone Penetrometer Test Hole (CPT 24) Exposed in Sidewall of Trench at Station 18+50.

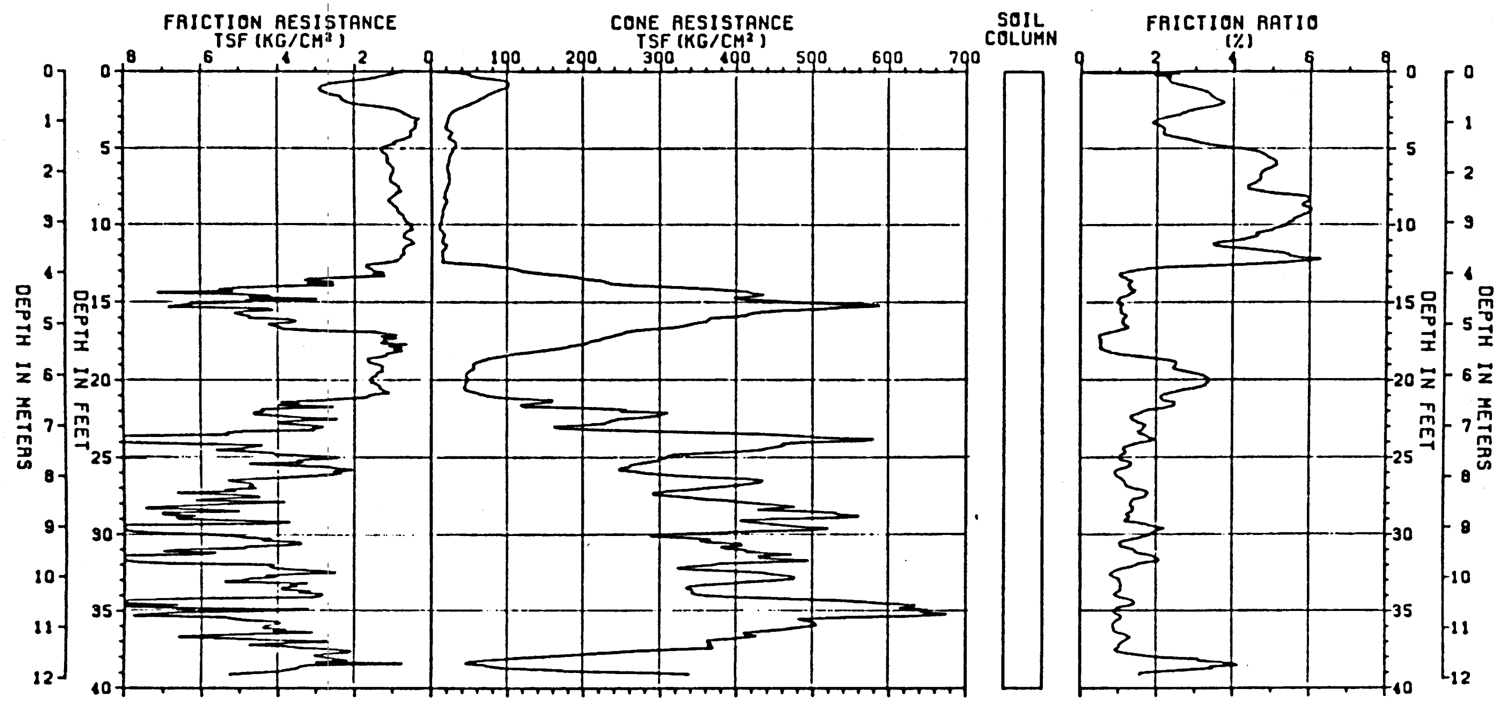
grain-size analysis was performed on the gravel material, it is estimated that this unit contains about 50% gravel and cobbles. Use of the 15 cm^2 cone permitted penetration of this dense, coarse material. Close agreement of CPT data and trench log was evident from a comparison of this CPT sounding (figure 45) and the trench log at the same location.

Location of the five rotary-wash borings conducted at the CSB site are shown in figure 42. Each boring was advanced to a depth of 70 feet. SPT's were taken at regular intervals and in all of the potentially liquefaction-susceptible deposits interpreted from adjacent CPT logs. Procedures outlined in ASTM D-1586 were used to conduct the SPT's. All five boreholes were cased with PVC pipe, and elevation of the groundwater was monitored. Water table depth at the time of investigation was found to be approximately 50 feet below ground surface.

Conclusions.

The investigative techniques selected for use at the School for the Blind site provided the information necessary for a reliable assessment of subsurface stratigraphy. Exploratory trenching was expensive but was the best available means for performing the fault investigation. Sedimentary details a fraction of an inch thick were observable during trench logging.

A large number of cone penetrometer soundings were conducted rapidly and economically at the CSB site. The CPT was not as precise in defining soil layering as trench logging but was much less expensive. Layers as thin as six inches were identified by CPT, and cone soundings were in very close agreement with trench logs. Use of the oversize cone tip facilitated in-situ testing of dense gravel deposits and those sediments beneath the gravel layers. With a CPT spacing of 50 feet, it is very unlikely that a sand layer greater than one



PROJECT: CAL SCHOOL FOR BLIND-FREMONT
PROJECT NUMBER: 85-140-01
INSTRUMENT NUMBER: F15CKE075
DATE: 09/05/84

CONE PENETROMETER TEST
PROBE: CPT-24

FIGURE 28

Figure 45. Cone Penetrometer Test Results (CPT 24) at Trench Station 18+50 (from Lockwood, Singh and Assoc., 1984).

foot that crosses the cone line went undetected (T.L. Youd, oral communication).

Correlation of the highly-accurate trench logs with CPT and SPT data enabled investigators to extend the soil profile below the depth of the trench with confidence. Cone penetrometer soundings were also useful in placing SPT tests in effective locations and in delineating the extent of potentially liquefiable sediments. Standard penetration tests were not very useful in subsurface mapping, but provided valuable soil type and liquefaction susceptibility information. At the CSB, use of these techniques in concert proved to be an effective and economical method for delineation of soil layering beneath the site.

WHISKEY SPRINGS SITE, THOUSAND SPRINGS VALLEY, IDAHO

Description of Investigation.

The Borah Peak earthquake (M_S = 7.3) shook central Idaho on October 28, 1983. Up to 11 feet of vertical displacement occurred along a section of the Lost River fault 23 miles long. The geomorphology of this region consists of parallel mountain ranges separated by alluvium-filled valleys and is largely a result of tectonic activity. Recurrent normal faulting has formed the basin-and-range structure of central Idaho.

Landslides, disruptions of ground water, and numerous liquefaction effects in the Thousand Springs Valley were described in a post-earthquake field reconnaissance of the epicentral area (Youd et al., 1985; Keefer, et al., 1985). Of particular interest was a zone of lateral spreading caused by liquefaction at the distal end of an alluvial fan. This zone, 1.3 miles long and 250 feet wide, was located between Whiskey Springs and Trail Creek Road

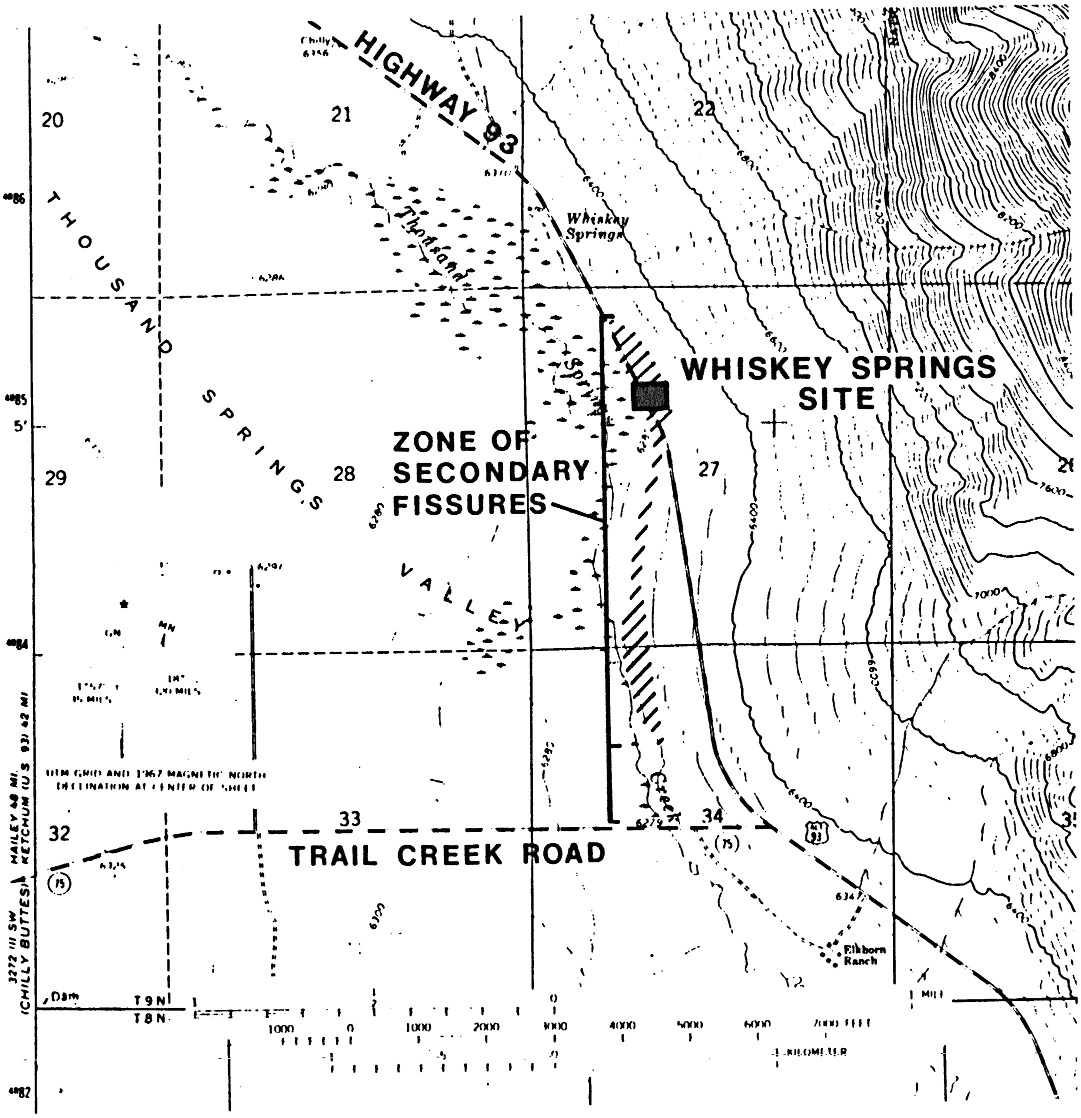


Figure 46. Topographic Map Showing Zone of Fissures Generated by Lateral Spreading of Distal Ends of Alluvial Fans and Location of the Whiskey Springs Investigation Site (modified from Youd et al., 1985).

(figure 46). Large subparallel fissures and cracks were concentrated at the head of the lateral spread, with smaller fissures occurring throughout the area. Compression of the soil at the toe of the spread caused the ground to buckle, forming ridges as high as four feet.

A section of this lateral spread (designated the Whiskey Springs site) was investigated in July, 1985 (Andrus, 1986). Various techniques were employed at this site for the purpose of delineating sediment layers beneath the lateral spread and for evaluating soil properties and liquefaction susceptibility of the sediment encountered. Exploratory trenches, auger boreholes, and SPT's were conducted by researchers from Brigham Young University (Provo, Utah). Cone penetrometer testing was provided by the Bureau of Reclamation (Denver, Colorado). Tests were also performed with a Becker Hammer Drill on contract to the University of California, Berkeley. Seismic velocity studies were conducted by the University of Texas at Austin, including cross-hole and Spectral Analysis of Surface Waves (SASW).

The importance of the Whiskey Springs site investigation is the variety of methods employed, which provides an opportunity to compare and contrast results obtained by the different investigative techniques in coarse alluvial sediments. Liquefaction occurred in an alluvial fan deposit of Pleistocene age, which would be expected to have a low susceptibility to liquefaction (Youd and Hoose, 1977). Also, lateral spreading was caused by liquefaction of gravelly material, for which no proven method of evaluating liquefaction potential has been developed. Results of this study have been reported by Andrus (1986). Principal liquefaction features and locations of tests conducted at the Whiskey Springs site are shown in figure 47. Large fissures containing downdropped blocks of material occurred at the head of the lateral spread, about 250 feet upslope on the alluvial fan. Lateral movement of 2.6

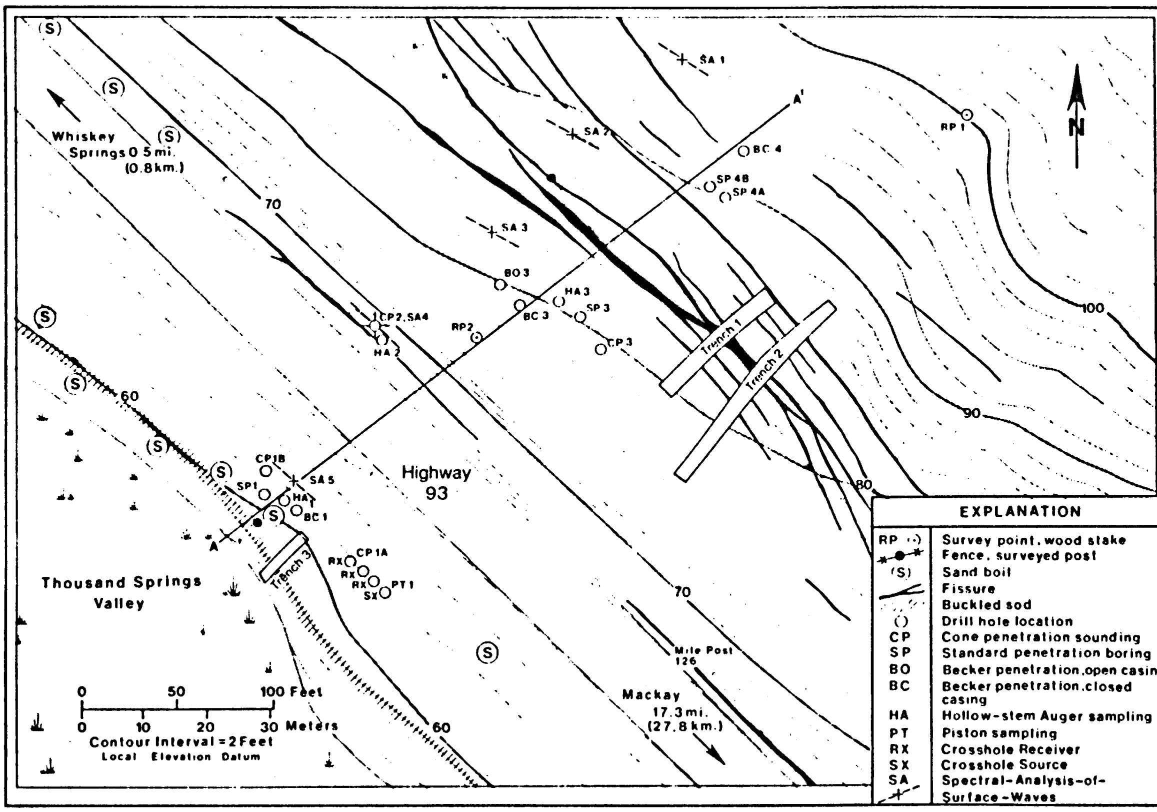


Figure 47. Map of the Whiskey Springs Site Showing Liquefaction Effects and Sites of Testing Conducted in July 1985 (from Andrus, et al., 1986).

to 3.3 feet on the spread caused compression and buckling of sod at the interface of marsh and alluvial fan sediments. This buckled zone, as well as locations of sand boils and minor fissuring, are also shown on the map.

Four test sites were aligned parallel to the direction of lateral displacement and perpendicular to the fan fronts. Becker Drill testing and SPT's were conducted at sites 1, 3, and 4. Cone penetrometer tests were completed at sites 1, 2, and 3. Continuous hollow-stem auger samples were taken from sites 1 and 2. Seismic velocity tests were performed at various locations along the cross-section.

<u>Site Stratigraphy</u>.

Figure 48 shows a cross-section (section A - A', figure 47) of sediment layers beneath the Whiskey Springs site that was developed from results of the various testing methods. Logs from BDT, SPT, and CPT are superimposed on the profile at appropriate locations. Separate sediment units were delineated on the basis of penetration resistance and sample classification. Nearly all of the sediment is poorly sorted, gap-graded, subangular, matrix-supported gravel with cobbles. Soil classification of each unit is shown on the figure.

The area is capped by a zone of dense to very dense silty gravel (units A and B). These units thin downslope, and are characterized by high blowcounts and penetration resistances. ~~Unit C has been subdivided into three subunits.~~ The lowest penetration resistances recorded were in the loose, saturated silty gravel of unit C1, with N-values ranging between 5 and 14, and cone tip resistances of 1 to 120 kg/cm^2. Unit C2 exhibited similar resistance values, but had a much higher clay content than C1. Unit C3 was not detected at test site 1, but is more than five feet thick at sites 2 and 3. Standard Penetration blowcounts of 11 to 14, and cone tip resistances of 100 to 175

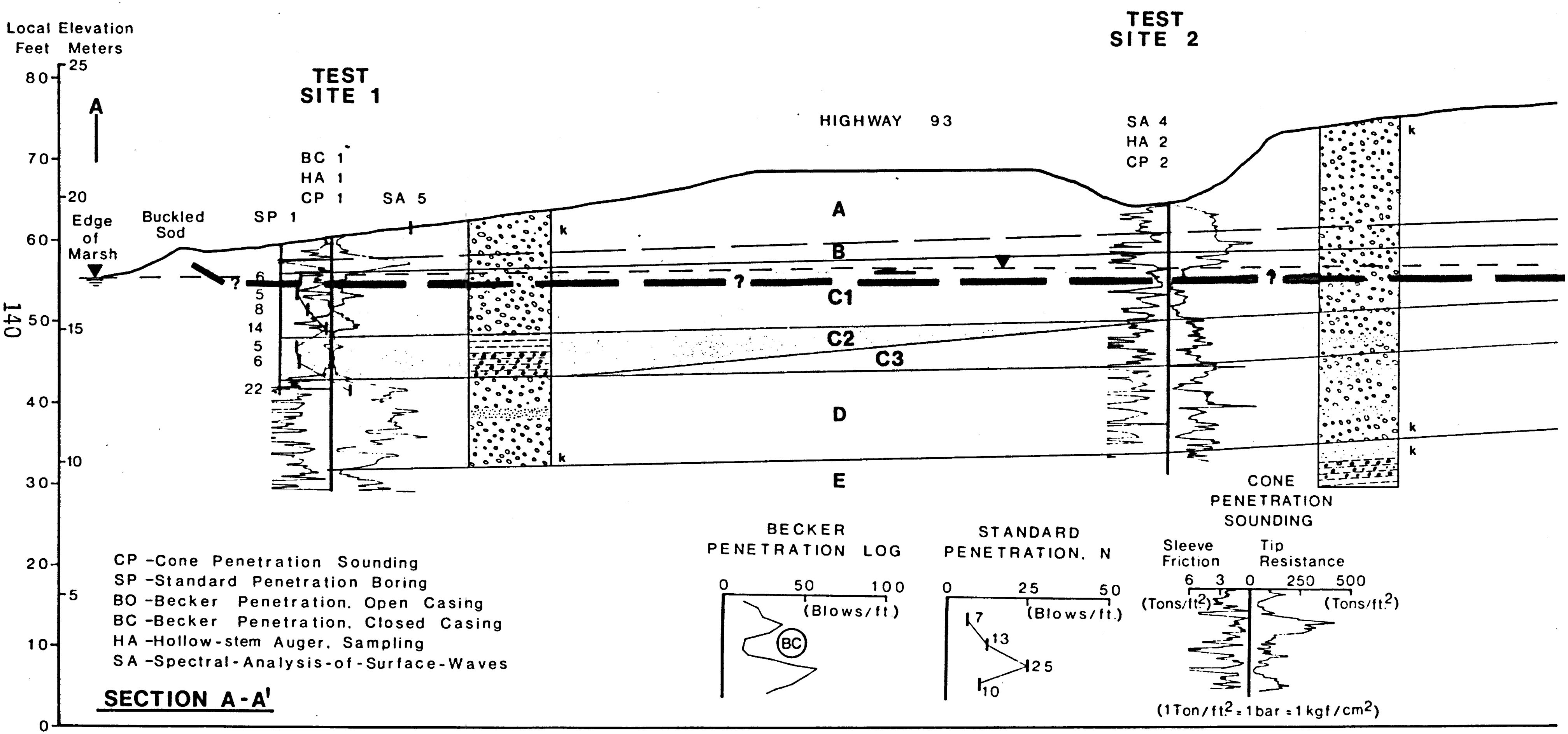


Figure 48. Cross Section of the Lateral Spread Near Whiskey Springs Showing SPT, CPT, BDT Penetration Data, and Stratigraphic Logs (from Andrus, in press).

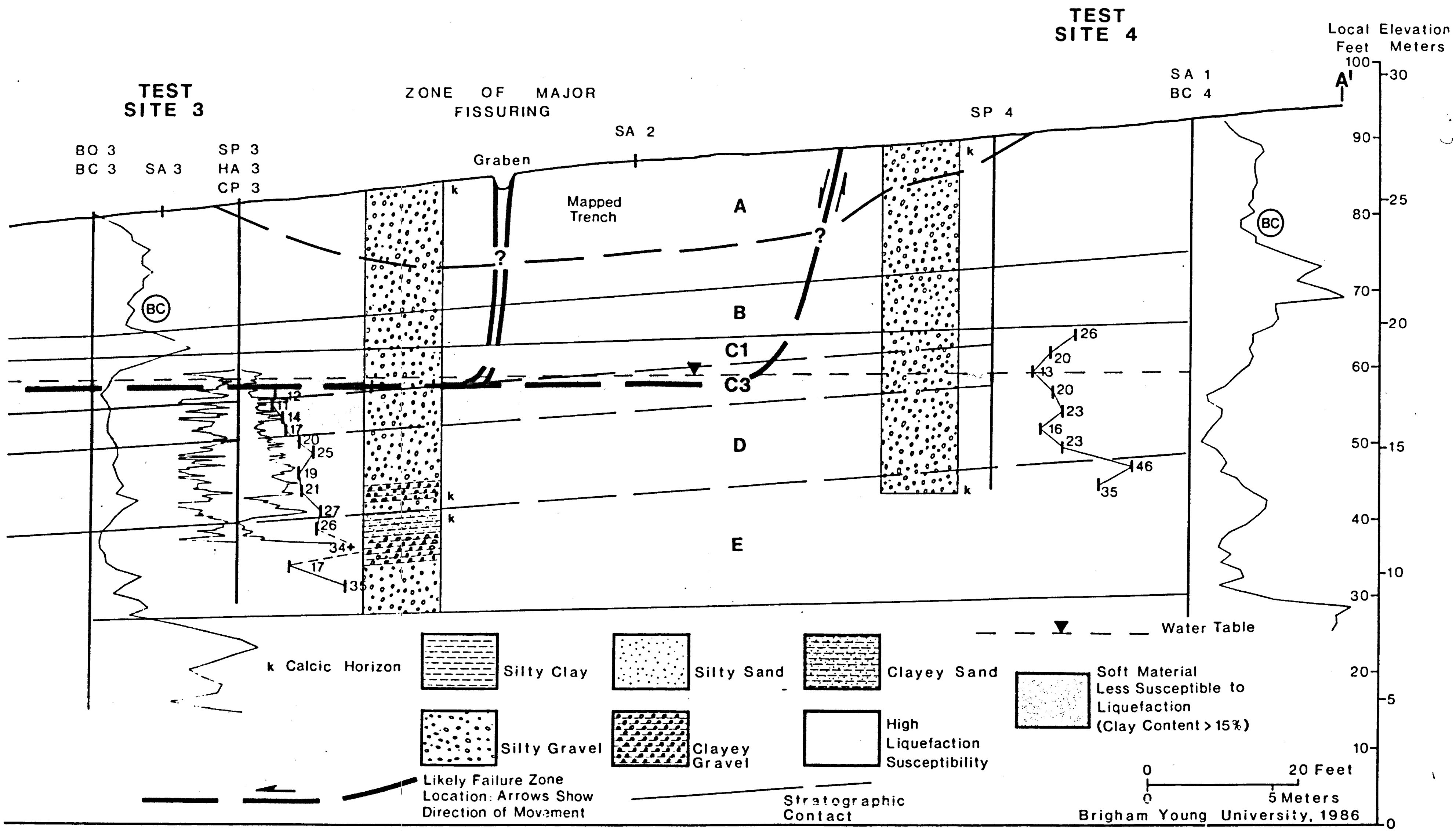


Figure 48, Continued.

kg/cm^2 were measured in this unit. Underlying units are silty gravels interbedded with silty sands. These materials have sufficiently high penetration resistance to preclude the development of soil liquefaction.

From results of subsurface investigation at Whiskey Springs, it was possible to identify zones which experienced significant strength loss during the 1983 earthquake. Analysis by the Seed procedure (Seed and Idriss, 1982; Seed et al., 1984), with assumptions to account for the high gravel content, led to the conclusion that liquefaction occurred in units C1 and C3. Unit C1 has the highest susceptibility to liquefaction and also provides the most favorable path for the shear failure zone. The assumed failure surface along the upper part of unit C1 is shown in figure 48. This surface is parallel to, and a few feet below, the water table elevation (Andrus, 1986).

Comparison of Investigative Techniques.

Procedures outlined in ASTM D-1586 with holes drilled with hollow-stem augers were used in Standard Penetration Testing. Depth of penetration for each blow, as well as the blowcount per six-inch interval, were recorded during testing. Blowcount values may have been affected by gravel or cobbles in the coarser units in the profile. Such an effect was not apparent, however, in plots of penetration versus number of hammer blows. These plots ~~showed essentially uniform penetration per blow for any given~~ test.

Even though large amounts of coarse material were present at Whiskey Springs, auger drilling and SPT were difficult but possible. The blowcount versus depth logs were roughly similar in profile shape to CPT tip resistances and gave a fairly good indication of layering beneath the site. Samples obtained by SPT were very useful in soil classification and correlation of the various soil units.

Other than cone size, the specifications of ASTM D-3441 were followed during cone penetrometer testing. A 15 cm^2 cone tip was used to allow penetration of dense or gravelly deposits. The penetrometer was advanced until excessive tip resistance or penetrometer inclination threatening damage to the equipment was encountered. Because of an impenetrable cobbly surface layer, holes for the CPT and SPT tests were predrilled by the Becker drill at site 3. CPT and SPT tests were then successfully performed to a depths of 35 and 25 feet below the bottom of the Becker hole, respectively. The materials beneath site 4 were too coarse and dense to permit CPT penetration at that locality. With considerable effort one hollow-stem auger hole was drilled at that site for SPT testing, but an attempt to drill a second hole for continuous sampling met with refusal when a large stone was encountered.

Of the many techniques used, cone penetrometer soundings gave the best delineation of subsurface layering at the Whiskey Springs site (figure 48). Even so, CPT alone would not have been adequate for the purposes of this investigation. Because of the high gravel content of penetrated soils, tip resistance and friction ratio logs were erratic, and probably not very reliable (Andrus, 1986). Also there was one site that was impenetrable with the CPT. Consequently, identification of soil types in the different sedimentary units would have been difficult without the complementary SPT, Becker and continuous sampling program.

The Becker apparatus was able to penetrate the gravelly alluvium much more easily and quickly than the other techniques. Drill bits of 6-5/8 inch diameter were used, and casing was advanced in 10 foot sections with blowcount recorded for each foot of penetration. Both open- and closed-end bits were used. Penetrations with a closed-end bit (BDT) are shown at sites 3 and 4. Becker and SPT penetration resistances recorded on these logs are generally consistent with each other. Details of soil layering are not very evident from the Becker soundings, and the best resolution of changes in soil conditions is on the order of one to two feet. However, given the large particle sizes encountered in many of these layers, one to two foot accuracy may be adequate for most practical purposes in these layers. Blowcounts recorded from BDT were approximately 2/3 of those measured with the conventional split-spoon. From his past studies, Harder (in press) suggests that the ratio of N-values Becker penetrations are too high at Whiskey Springs. He further suggests that this high ration was caused by cobbles increasing the SPT resistance.

At test site 3, one hole was driven using an open bit to collect disturbed samples by cycling the return air fluid. Because of rapid bit penetration and sample mixing during transport up the drill stem, Andrus (1986) could not delineate sediment units and boundaries between units very precisely from the Becker samples. Andrus also concluded that the Becker samples were not as representative of the fine grained fraction as samples obtained by continuous sampler, or SPT. This inferior quality was apparently caused by sorting and mixing in the sampling process. All of the sampling methods suffered from the inability to sample the larger particles in the coarser layers. The larger the diameter the sampler, the more representative the sample in this instance. The continuous hollow-stem sampler with a 5-in inside diameter provided the "best" samples of the coarser materials in the Whiskey Springs investigation.

The CME continuous hollow-stem sampler and was able to sample cobbles up to 4-1/2 inches in size. Approximately 70% recovery was achieved with this device. Some difficulty was experienced with sample retention in soft cohesionless materials, and poorest recovery generally occurred in the silty gravels suspected of liquefying during the 1983 earthquake. Considerable difficulty in penetrating the coarser layers also inhibited use of this device. Penetration of gravelly and cobbly sediments by the auger bit also caused significant sample disturbance and mixing. Nevertheless, some stratigraphic detail was observable, and representative samples of each soil unit penetrated were obtained for laboratory analysis. It is possible that thin layers of sand may have been mixed with gravelly sediments in the sampling process. However, in the coarser material most layers as thin as one foot were sufficiently undisturbed by the CME sampling process to be identified. In the finer grained sediment where disturbance and mixing were minimal, layers as fine as a fraction of an inch could be delineated.

Trenches as deep as 18 feet were excavated for the purpose of determining the depositional environment of the alluvial fan material and identification of 1983 and any previous liquefaction effects. Very detailed stratigraphic mapping was done by trench logging (Andrus, 1986) although neither of the upper trench excavations extended into the liquefiable material. Extension of the subsurface profile was possible through correlation of trench logs and data from drilling and testing at the adjacent test sites.

Shear wave velocity profiles were not as useful as other methods used in delineating Whiskey Springs sediments. Figure 49 is a SASW profile derived from testing at site 2. Sedimentary layers are not very evident from this profile unless compared with other soundings taken at the same site. The lowest shear wave velocities (750 fps) were recorded in unit C3, which was determined by other methods to be less liquefiable than unit C1. The loose, soft nature of C1 material is not characterized very well by the SASW record.

The coarse, dense material of units D, E, and the underlying sediments was roughly delineated by shear wave velocity plots. Because the

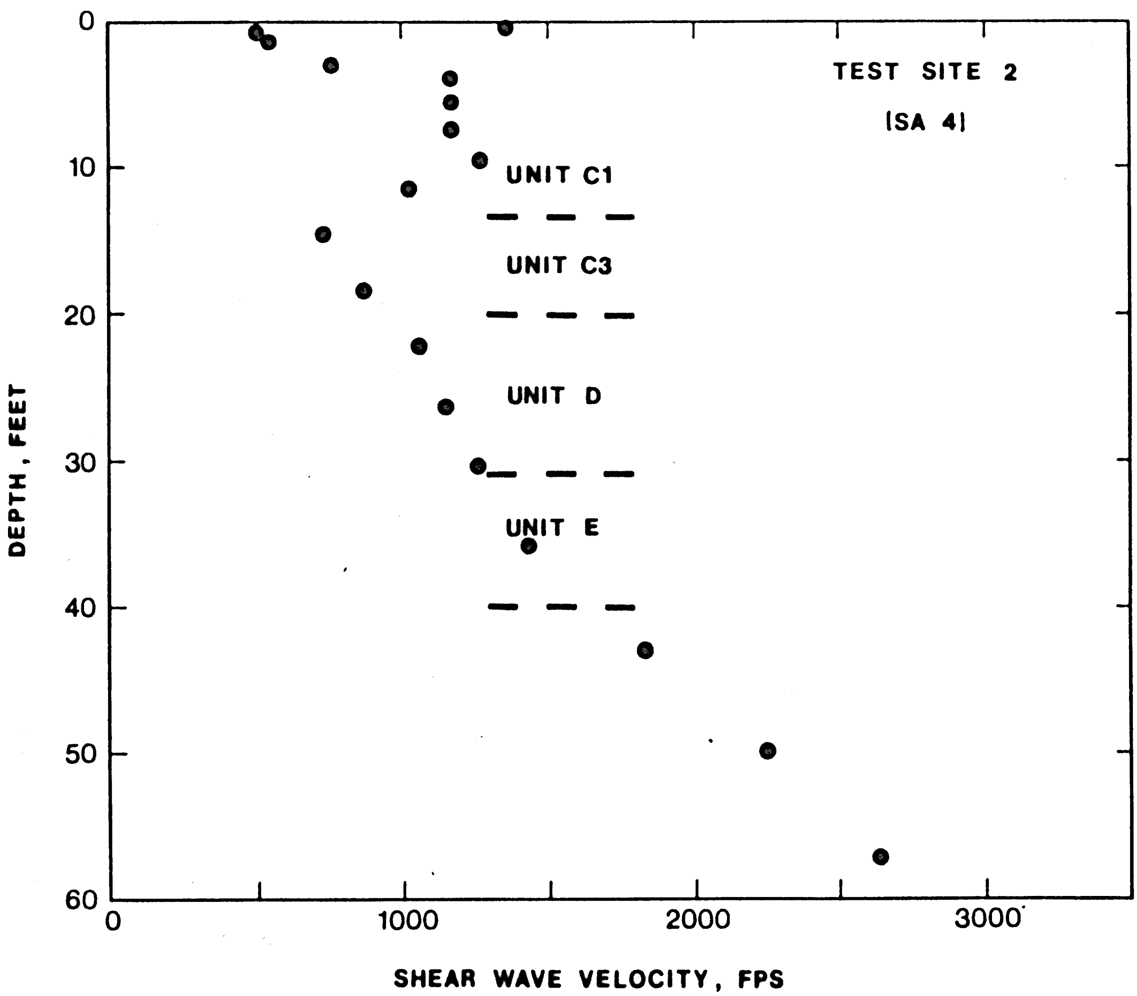


Figure 49. Profile of Sediments at Whiskey Springs Site, Test Site 2, from SASW Testing (from Andrus, in press).

higher-velocity layers are also more difficult to penetrate, a comparison of the SASW logs with CPT soundings is worthwhile. The highest-velocity material at Whiskey Springs penetrated by the oversize cone was characterized by V_s of 1500 to 2000 ft per sec. Penetration was difficult, but generally possible, in materials with measured shear wave velocities between 1000 and 1500 ft per sec. Penetration and sampling was nearly always possible in sediment with shear wave velocities less than 1000 ft per sec.

Conclusions.

At the Whiskey Springs site, the cone penetrometer performed beyond expectations. Use of the oversize cone allowed penetration of material suspected of liquefying, and through an underlying layer of dense gravel and cobbles (unit D). Continuous samples in the penetrated material had gravel contents of 50 to 60 (and as high as 72) percent. Though this unit was tested by CPT, some damage and excessive wear to the equipment resulted. Refusal was met on several occasions during testing. Cone penetrometer logs provided better depiction of soil layers than the other exploratory methods used. The disadvantage of no sample recovered by CPT was overcome with SPT and continuous sampling.

The Becker Hammer Drill penetrated the subsurface materials quickly and easily and was the most economical (in terms of time) method used at Whiskey Springs. A rough depiction of subsurface conditions would have been possible with Becker logs only, but stratigraphic detail from BDT was not as clear as that obtained from SPT or CPT. Samples from the open-casing Becker hole were not very useful in mapping Whiskey Springs sediments.

Auger drilling and SPT sampling proved to be possible, but proceeded slowly in the gravel/cobble layers. Stratigraphic units were delineated quite well with SPT. Soil samples and N-values were useful in soil classification, correlation of CPT soundings, and evaluation of liquefaction susceptibility. Results obtained in coarse sediments by SPT may have been adversely affected by the gravel particles.

Of the many exploratory methods employed at Whiskey Springs, trenching was the most time-consuming and expensive, but was the only technique available for evaluating past liquefaction effects. Trenching provided the best resolution of stratigraphic detail and aided in correlating soil units between soundings and boreholes.

It is evident that no single method of geotechnical investigation would have sufficed at the Whiskey Springs site. Use of the methods available and correlation of results provided sufficient information for the study. Comparison of logs and soundings enabled the researchers to compile a cross-section of the site showing the principal layers and evaluate the probable cause of failure with some degree of confidence. However, because of the coarseness of the gravelly sediment and testing and sampling difficulties in that sediment, many sublayers could have escaped detection, possibly layers as thick as one or two feet.

V. CONCLUSIONS

From the review of techniques available for delineating liquefiable soil layers and comparison of their performance in site investigations, it is evident that no single exploratory method works best in all instances. The following conclusions place emphasis on the material types most conducive to performance of each method, and the resolution of sedimentary layering each is able to provide. Guidelines for the use of these investigative techniques in subsurface mapping are also suggested.

1. In materials that can be penetrated, the cone penetrometer was found to be the most economical and useful technique in mapping the thickness and extent of liquefiable soil layers. Some investigators feel that CPT can be performed at 1/4 the cost of standard drilling and provides more information about stratigraphic changes. Cone logs allow delineation of layers in the profile down to a few inches thick, and use of the friction ratio makes preliminary identification of soil types possible.

    a. Field experience with the CPT indicates that, if it is possible to push the cone at a site, then that method will generally be the most economical technique to use, and will also give adequate stratigraphic resolution for most liquefaction studies. The best resolution of CPT is obtained with the electrical probe and continuous recording of data, in fine-grained materials. Under these conditions, it is possible to detect layers as thin as two inches, and practical to delineate four-inch and thicker layers. In gravelly material, where cone data may be erratic, layers as thin as six inches are detectable.

    b. Use of the piezocone (CPTu) has been proposed as a means for improving stratigraphic resolution of the cone penetrometer. From field experience cited in this study, the CPTu did not significantly improve the detection of soil layers, but did increase accuracy in boundary determination. Sharp lithologic contacts, such as sand/clay interfaces, are readily identifiable from CPTu logs, and may be located in the profile within an inch or two by that method.

    c. The cone penetrometer works best and most economically in sands, silts and clays with little gravel material. However, use of CPT is by no means restricted to gravel-free soils. Penetration and testing of gravels may be accomplished, depending on shape, size, and angularity of gravel particles, as well as density and degree of cementation in the sediment layer. Use of the oversize (15 cm^2 tip area) cone at Jackson Lake Dam, California School for the Blind, and Whiskey Springs, Idaho allowed penetration of loose to moderately dense gravels with 50 to 60% material coarser than the No. 4 sieve.

    d. When the cone penetrometer meets refusal before required depth of sounding is reached, it may be advantageous to use standard drilling methods to extend the hole through the coarse and/or dense material, and then continue cone penetration. A similar technique is appropriate at sites capped by a layer of very coarse sediments (as at Whiskey Springs). In this case, holes may be pre-drilled by Becker or other drilling apparatus, and CPT soundings performed from the borehole bottoms.

e. A distinct advantage of the CPT is that large numbers of soundings may be performed quickly and economically. In highly-stratified materials, tests may be conducted at very close intervals if needed. When weighed against the large amount of stratigraphic information produced by CPT, the cone is a very economical exploratory technique, even when such close test intervals are specified.

f. A disadvantage of the cone penetrometer is that no sample is recovered. Because of this, the CPT cannot be used in isolation. Verification of soil types by SPT or other sampling method is required.

2. Where materials are not penetrable by CPT, the Becker Hammer Drill (BHD) may be a useful and economical reconnaissance tool. Though a rather new exploratory technique, the BHD shows promise in profiling sediments where gravel and cobbles occur in abundance.

a. Because of rapid penetration rates and mixing of soil samples, it is not generally possible to detect very thin layers by BHD. One or two feet is a practical limitation to resolution of soil layers with the Becker drill. A thin sand layer sandwiched between gravel units will likely be missed. However, in very coarse materials, profiling may not need to be as refined. Results obtained by the Becker drill may be detailed enough for many liquefaction investigations.

b. Sample material is provided when the BHD is used with open casing. Because these samples are forced up the drill pipe by compressed air, they are mixed, making it difficult to distinguish thin soil units. Also, extreme dynamic pressures generated during testing may crush, compact, or force material around the drill bit rather than being forced into the drill pipe, resulting in non-representative soil samples.

3. The Standard Penetration Test has been widely-used in the past to investigate liquefiable soils. It provides penetration resistance and a sample for profiling. Equipment for the SPT is readily available and

routinely used in most areas of the United States, and the test is relatively economical to perform.

a. Because a sample is retrieved by SPT, it is a good technique to use in conjunction with CPT. Sample locations may be pre-located from preliminary CPT logs to optimize sediment information recovery. Soil types determined from SPT samples may then be correlated laterally between adjacent CPT soundings. Use of these techniques in tandem was highly-recommended by the Bureau of Reclamation from experience in Jackson Lake Dam investigations (U.S.B.R., 1985): "The correlation of SPT and CPT data provides one of the most effective means of evaluating the stratigraphic sequence and engineering properties of materials in a given section."

b. The SPT may be performed in a range of materials from clay to gravel. Large gravel particles may halt penetration, or adversely affect recorded blowcounts and sample recovery. Nevertheless, it is still a rugged and relatively economical way to retrieve samples. Practically any material which is augerable may be sampled by SPT.

c. Where the cone penetrometer is not available, standard drilling and SPT may be the most accurate and economical technique for sedimentary profiling. If SPT samples are undisturbed by gravel, soil layers as thin as 1/8 inch are identifiable. When SPT is used without the cone penetrometer, the completeness of profiling depends on test intervals. Complete coverage may be attained by continuous SPT sampling through a single borehole, though this may lead to excessive sample disturbance and questionable penetration values. An alternative is to drill parallel holes, and stagger test locations through each borehole. Costs for such drilling and sampling programs will be significantly higher than cone penetrometer soundings.

4. From an examination of various sampling techniques, we conclude that those most useful in stratigraphic logging are the thin-walled piston tube and the CME continuous sample tube system. In-situ freezing or other techniques are expensive and/or exotic, and are not generally applicable to subsurface mapping. Vibratory drilling has been used to advantage offshore, but its use onshore has been limited. This technique is capable of retrieving continuous samples as long several tens of feet with acceptable quality for soil profiling purposes. Auger sampling using solid stem augers has also been used for soil profiling purposes,

but the level of disturbance is sufficiently great that considerable stratigraphic detail is usually lost.

a. Continuous sampling by thin-walled piston tube is expensive, but provides complete definition of the soil profile. Microstructures a fraction of an inch thick may be observed in these undisturbed samples. The presence of significant amounts of gravel will restrict or prohibit use of the piston sampler.

b. The Central Mine Equipment (CME) continuous sample tube system has been used economically in profiling subsurface sediments. The CME sampler may be used in any material which is augerable, including some gravel/cobble sequences. Materials retrieved by this apparatus are more disturbed than thin-walled tube samples, but are usually high-quality enough to permit detailed profiling of sediments. Layers an inch or less in thickness are generally detectable in soils with less than about 20% gravel. As the gravel content increases, sample disturbance does also. Rather severe mixing may occur in loose sediment with a large fraction of gravel. As with the Becker drill, a thin sand layer occurring between gravel sequences might be obliterated by the sampling process.

5. Exploratory trenching is useful and definitive in mapping soil layers and may be conducted in practically any soil type. Because it is also expensive, this method may be limited to investigations for expensive critical structures and sites where excavations are required for other aspects of dam construction, such as cutoff trenches. The costs and difficulties of dewatering of trenches may place a practical limit on the depth to which trenches can be excavated, usually about 30 ft below water table.

a. Because sediment exposed in the trench sidewall are available for direct observation, this method is usually the most accurate and definitive in subsurface mapping. Sedimentary structures a fraction of an inch thick may be identified and logged two-dimensionally. Because trench excavation, logging, and dewatering are generally expensive, and trenching is impractical at great depths, this method has not been extensively used for liquefaction investigations.

b. Wherever excavations are required for dam construction, it is good practice to examine, log, and photograph exposed sediments. Usefulness of this practice was demonstrated by the Twin Lakes Dam liquefaction investigation. A cutoff trench was excavated and photographed before a liquefaction problem at the site was recognized. These photographs and other records proved to be invaluable for delineation of sedimentary layers and relating them to layers and properties determined from other investigative techniques.

6. The geophysical techniques reviewed in this paper were not generally found to be sufficiently accurate methods for mapping liquefiable soil layers, although they are useful for delineating major structural contacts such as bedrock boundaries and for evaluating modulus properties of soils. As a category, geophysical methods provide only rough delineation of soil thicknesses or layering. To make these techniques more applicable to mapping sedimentary strata, further research and field experience are needed.

a. Seismic velocity measurements are economically performed from the ground surface, with little to no drilling required. Currently, Spectral Analysis of Surface Waves (SASW) appears to be the best seismic technique for delineation of soil layers. Use of SASW is presently restricted to depths of about 50 feet or less. Some lithologic interfaces, depth to shallow bedrock, and layers thicker than about two feet are identifiable from SASW shear wave velocity plots. Seismic refraction gives a less well-defined profile, and liquefiable layers in gravel sequences will likely be invisible with SRS.

b. An advantage of the SASW and SRS techniques is that they can be used in gravelly or cobbly materials. In addition, shear wave velocities in these materials may provide a guide to soil penetrability by CPT. Sediments characterized by V_s of 1500 to 2000 fps were just penetrable with the oversize cone at the Whiskey Springs, Idaho site. Verification is needed, however, before this criterion can be recommended as a general guideline.

c. Geophysical borehole logging is currently applicable to shallow geotechnical investigation, but delineation of stratigraphy by these methods is coarse (layers thinner than a foot or two are not generally distinguishable). Of the various radiation logs, natural gamma and gamma-gamma appear to be the most useful in stratigraphic mapping.

d. Radar surveys have some potential for identifying thin liquefiable soil layers near the surface. At present, little field experience has accumulated, but there is some tendency toward increased use of the radar method. In some silts or clays, and under conditions of salt-contaminated soils or high groundwater (a condition common to many liquefiable soils), depth of penetration by radar is severely restricted.

7. Determination of the minimum thickness of a layer with the potential for causing slope instability in and embankment dam was an objective of this study. The minimum layer thickness is need for selection of investigative methods and for planning and effective and economical subsurface investigation program. Several experts in earthquake engineering were asked their opinion concerning the thinnest liquefiable layer which should be of concern in a dam foundation. The following conclusions are drawn from the responses obtained

a. Many factors influence the minimum thickness of a liquefiable layer that could be critical to the stability of an embankment dam. These factors include 1) the continuity of the layer, 2) the depth of liquefiable layer; 3) the orientation of the layer with respect to a potential failure surface; 4) the looseness of materials in the layer; 5) the permeability of surrounding materials; 6) the geologic origin of the soils; 7) the embankment configuration; and 8) the mechanism of failure.

b. The experts generally agreed that a continuous contractive liquefiable layer which is as thin as a few inches and which is aligned with a potential failure plane could lead to embankment instability during strong seismic shaking. If the layer were not continuous, were dilative, or were not aligned with the potential failure plane, the influence of such a layer on slope instability would be greatly reduced. In most geologic environments, the average continuous length of a granular layer increases with layer thickness. In alluvial environments, layers less than one foot thick are seldom continuous for more than 50 to 100 times their maximum thickness. Thus, the continuity of alluvial granular layers a few inches thick rarely exceed a few tens of feet. Such a layer may not be critical to a large embankment dam. Conversely, in lacustrine environments, layers a few inches thick may be continuous for considerable distances and could be critical to an embankment structure. If the layer were dilative under the given static loading condition, then postearthquake shear deformation would likely be arrested preventing massive slope instability. If the liquefiable layer were oriented such that only a small part of the potential failure zone passes through the layer, then liquefaction of that layer would cause only a small reduction in the static factor of safety which might not be critical to the structure.

8. In actual field investigations, initial spacings of boreholes and/or soundings will be determined by a number of factors including economics, site geology, and geometric configuration of the embankment. Selection of initial test spacing is crucial, however, since this will have a large bearing on total cost and adequacy of the subsurface investigation:

   a. Some general guidance on hole spacing can be gained from an analysis of the stability of the embankment. Such analyses show that the minimum length of a horizontal liquefiable layer that is critical to the stability of an embankment generally increases with depth of the layer beneath the base of the embankment (Figs. 5, 6, and 7). Thus, for nearly horizontal soil layers, which includes most sedimentary regimes, the required spacing between exploratory holes or probes to detect critical liquefiable layers increases with depth of the layer. Figures 5, 6, and 7 can be used as approximate guides for selecting hole spacings. For example, for an embankment 100 ft high with 2 to 1 side slopes built on foundation materials with a strength ratio of 0.5, the lengths of critical layers at depths of 20, 50, and 100 ft, are about 60, 130, and 260 ft, respectively. Exploratory holes or probes could be placed conservatively to intersect all layers of these lengths at these depths. An adequate spacing might be half to a third of the critical layer length for the given depth. Interestingly, applying this guideline to this example yields hole spacings and exploratory depths that are approximately equal to the depth of the critical layer. For critical structures, stability analyses could be made that incorporate presumed unfavorably oriented liquefied layers. Such analyses could provide more specific guidance for selecting hole spacings.

   b. Selection of test locations and spacings should also consider local geologic conditions. Because of the complexity and variability of depositional processes, it is not possible to establish a single general guideline applicable to all geologic settings. Local experience and preliminary geologic site studies provide the best geologic guidance for selection of test spacings.

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